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IMPLEMENTATION OF CENTRIFUGE TESTING OF EXPANSIVE SOILS FOR PAVEMENT DESIGN

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16. Abstract

The novel centrifuge-based method for testing of expansive soils from project 5-6048-01 was implemented into use for the determination of the Potential Vertical Rise (PVR) of roadways that sit on expansive subgrades. The centrifuge method was modified to allow for testing of both undisturbed and reconstituted specimens as well as to match the boundary conditions of the state of practice laboratory method, ASTM D4546. The test was used to expand the expansive soils database through a number of sites east of the Balcones Fault Zone in Central Texas in order to give designers a better understanding of the characteristics of soils in the region and to compare against results from Tex-124-E. The test was further used to test field specimens from various sites to illustrate the need for either uncontaminated bulk specimens or undisturbed specimens and to develop a new method for sites with limited testing. Instrumentation and monitoring of the heave at field sites were used to validate the testing method by showing similar moisture content changes in the field section as in laboratory specimens as well as showing a field site in which Tex-124-E misidentifies a site that sits on a non-expansive subgrade. The project provided a new method to calculate the PVR of a site using real data, expanded the database of soils to include new soils and look at heterogeneity between soil horizons, and analyzed field results to validate the results from laboratory experiments.

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Products

Appendix A presents 5-6048-03-P1, *Spreadsheet with Data of Swelling Curves for Clays in TxDOT Austin District*.

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Chapter 1. Introduction

Expansive soils are a commonly encountered soil in the Central Texas region which undergo significant volumetric changes under moisture fluctuations. To meet the needs of the communities in the region, many roadways and other low-rise infrastructure are built over these subgrades, which leads to significant environmental cracking. As such, the need to understand their expansion characteristics is vital to the design, construction and maintenance of roadway infrastructure in order to prevent or delay failure from environmental conditions. The need to design and construct roadways on highly plastic clays is common in central and eastern Texas, where expansive clays are prevalent. A previous implementation project, 5-6048-01, developed a centrifuge-based method for the testing of these expansive soils using an in-flight data acquisition system to reduce testing time, as compared to the standard laboratory method. This report consists of the implementation of centrifuge-based testing into various sampling locations with known expansive soils, as well as a field component to understand the moisture fluctuations beneath roadways in Central Texas. Laboratory testing and field data were combined to create a new method of calculating the Potential Vertical Rise (PVR) of a site in an expedited manner to assist pavement engineers at TxDOT with the design of roadways over expansive subgrades.

Chapter 2 focuses on the experimental methodology, with emphasis on upgrades to the centrifuge experimental methods and new calculation method for Potential Vertical Rise. Chapter 3 focuses on the implementation of the new PVR method, including the rationale behind the initial conditions and an analysis of the various ways to use the new method. Chapter 4 focuses on the laboratory testing component of the project, including the testing of numerous field sites to expand the expansive soil database. Chapter 5 focuses on field projects, including sites that are instrumented with moisture and suction sensors, sites that are monitored with total station readings, and a look at the heterogeneity of a soil deposit sampled using Shelby tube specimens. Chapter 6 contains conclusions from the project and recommendations for further work in this area to look at remediation strategies in conjunction with the newly designed method.

Chapter 2. Experimental Methodology

Over the course of the project, the centrifuge technology was upgraded using a new permeameter cup with an increased capacity in testing. This chapter covers those changes, validates the procedure by comparing the results to state of practice results, and shows the calculation for the curve fitting method, which is similar to that used for unsaturated soils and their hydraulic properties.

2.1 Advances and Upgrades in Centrifuge Technology

At the end of Project No. 5-6048-01 in February 2013, a centrifuge-based testing procedure had been developed and verified for the characterization of expansive soils, namely the direct measurements of vertical swell in a reconstituted soil at a given effective stress. The premise of this test is to subject a soil sample to vertical water infiltration during a small testing period, typically 24 to 36 hours, in which the height of the soil is measured via an in-flight Data Acquisition System (DAS), to determine the swell and swelling characteristics of a given soil. These tests proved to be more efficient than the typical ASTM D4546 test, as a swell versus stress curve could be generated more rapidly with a higher degree of repeatability due to each test containing up to 4 samples in a centrifuge environment. However, the test was unable to accommodate undisturbed soil specimens due to the original permeameter cup's design, which required the soil be compacted within the cup. In response to this issue, a new centrifuge cup has been developed that allows for testing on undisturbed specimens taken from the field via Shelby tube samples.

In order to accommodate testing of field specimens from push samplers, the boundary conditions and permeameter cup design was changed to incorporate a cutting ring. The change in the boundary conditions and comparison between the original centrifuge permeameter cup design and the double infiltration cup design is shown in Figure 2.1. The components of the new double infiltration permeameter cup are shown in Figure 2.2, with the completed assembly shown in Figure 2.3.

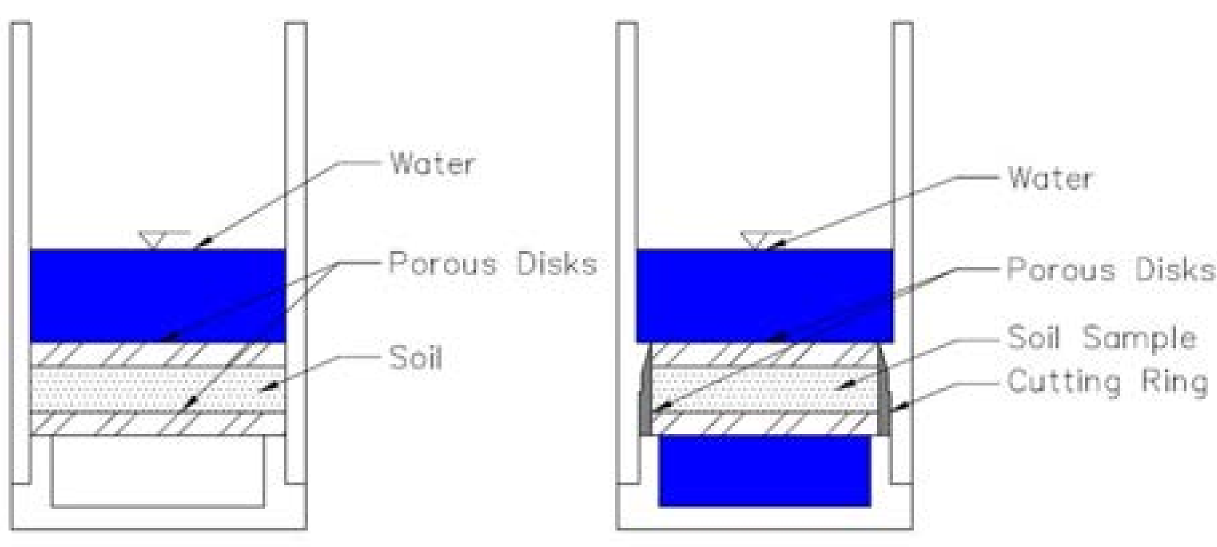


Figure 2.1: Single Infiltration and Double Infiltration Permeameter Cups

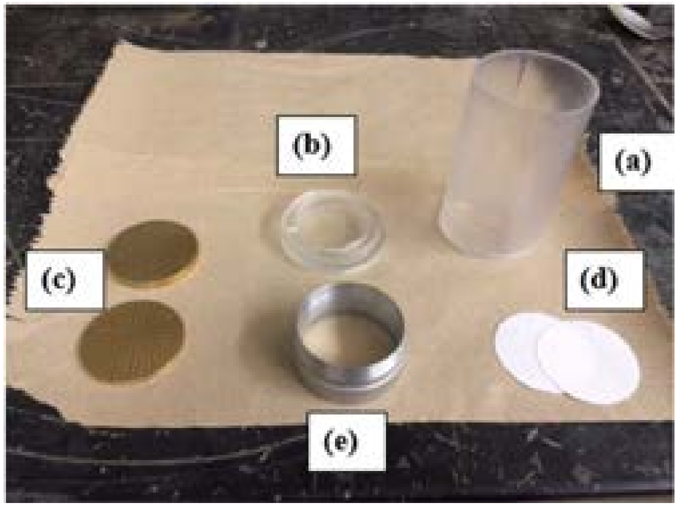


Figure 2.2: Double Infiltration Permeameter Cup Components

Figure 2.3: Double Infiltration Assembly

The new boundary conditions match the boundary conditions from the traditional experimental method for characterization of expansive soils, ASTM D4546. Due to this change in conditions, the stress applied to the specimen is no longer as dependent on the head of water above the specimen, as the permeameter cup allows for a single, linked reservoir between the top and bottom of the expansive clay specimen. As such, the consistency between tests is much higher, as the stress applied to the specimen is consistent between tests and not as dependent on voids within the compacted soil. Other improvements include the application of the overburden stress being solely attributed to the porous disk and centrifugation, as opposed to the metal washers, increasing testing workability.

In order to test soils taken from the field, a method was developed to dry the specimens. The specimens were originally dried in the laboratory environment due to the relatively consistent climatic conditions, with a temperature between 70 and 75 ºF and a relative humidity between 40 and 55%. However, results from testing indicated that the rate of drying was too rapid and tensile, and shrinkage cracks formed in the specimens themselves (Armstrong 2014), as shown in Figure 2.4.

Figure 2.4: Shrinkage Cracks formed from rapid drying.

To retard the rate of drying, an environmental chamber with a relatively constant, high relative humidity was assembled using a glove box and saturated salt solution. The weights of the specimens were measured using a scale with an accuracy of ±0.001 g with a Fisher Scientific temperature and humidity monitor. The environmental chamber is shown in Figure 2.5.

Figure 2.5: Environmental Chamber

The time to reach the targeted moisture content, 3 moisture percentage points dry of optimum, as prescribed by a NAVFAC correlation with the optimum moisture content and the soil's liquid limit, increased from approximately 12 hours to 36–72 hours using six specimens in the environmental chamber, but the increase in time was met with a decrease in the shrinkage cracking of specimens. Testing of these specimens is further explored in Section 3.1.

2.2 Validation of Improvements to Centrifugation of Expansive Soils

Testing was done to validate the new testing methodology via long-term testing of reconstituted specimens of Eagle Ford Shale and Cook Mountain Clay (Behring series), and testing on undisturbed specimens taken from the Shelby tube samples of Branyon Clay from FM487 west of Bartlett, TX.

2.2.1 Validation of Experimental Methodology using Reconstituted Specimens

The first validation testing regime's goal was to analyze specimens under long-term testing conditions in order to verify the time difference between testing methods to reach ultimate swelling, especially for very high plasticity clays. Secondary swelling in expansive soils can be fairly significant, especially for soils with liquid limits above 80. Thus, ultimate swelling may take as much as a month for these soils in traditional ASTM D4546 tests. Eagle Ford Shale and the Cook Mountain Clay were the soils selected for this testing regime. Both of these soils have been heavily researched by the University of Texas at Austin laboratory, and their geotechnical index properties are shown below in Table 2.1.

Table 2.1: Index Properties for Eagle Ford Shale and Cook Mountain Clay

Soil	Soil Location	USDA Soil Survey	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Soil Classification (USCS)	Clay Content (%)	Specific Gravity (G_s)	ω_{opt} (%)	$\gamma_{d,max}$ (kN/m^3)
EF	I-35	-	88	39	49	CH	64	2.74	24.3	15.25
CM	SH-21	BeC2/BeB	58	17	41	CH	40	2.784	20	15.42

Three separate tests are shown, as follows. The Cook Mountain test focused on testing carried out at the optimum condition as prescribed by standard proctor tests, while the Eagle Ford Shale tests were both done at the dry of optimum condition.

The results from the long-term Cook Mountain tests are shown below in Figure 2.6 and Table 2.2.

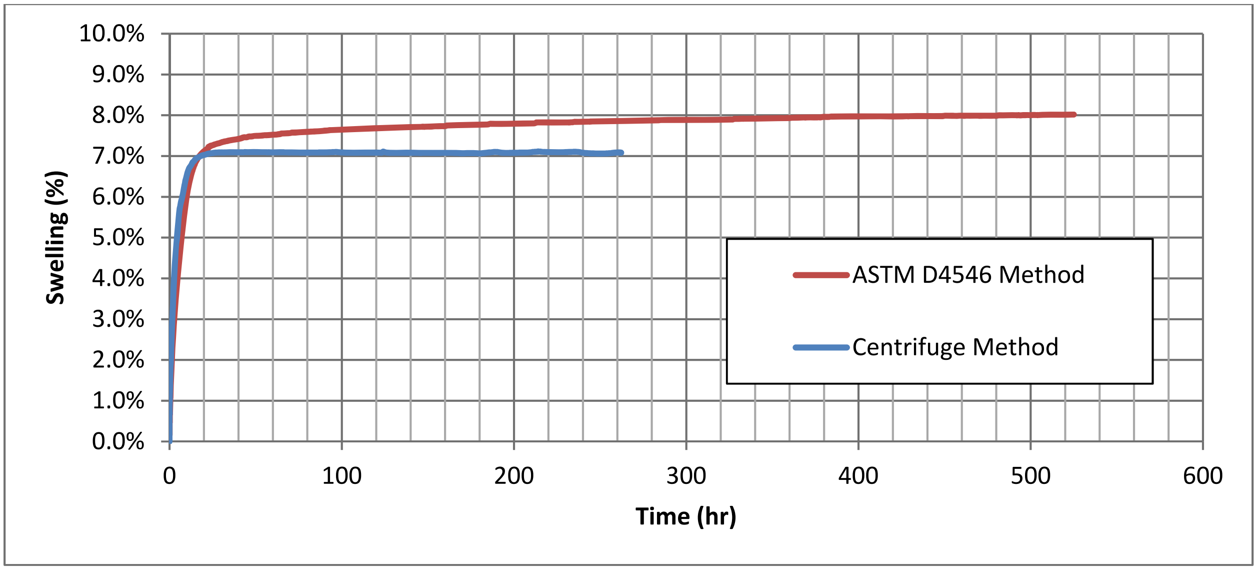


Figure 2.6: Results from Long-Term Tests on Cook Mountain Clay

Table 2.2: Overview of Conditions for Long-term Tests on Cook Mountain Clay

Method	ω_i (%)	γ_d (kN/m^3)	θ_i	ω_f (%)	Primary Swell (%)	Time to Primary Swell (hr)	Max Swell (%)	Time to Max Swell (hr)	Overburden Stress (psf)
ASTM D4546	19.3%	15.93	0.313	34.6%	7.1%	18.50	8.0%	524	125
Centrifuge	19.6%	15.67	0.313	32.3%	7.0%	16.61	7.1%	43.2	133

From the results, the tests came to a similar primary swelling value with a slight difference in ultimate swelling. The convergence to a similar primary swelling value is to be expected, as both specimens started at the same initial volumetric moisture content which would give a very similar initial suction. Since the primary swelling is dependent on the gradient in the suction, this initial start would lead to both specimens coming together for primary swelling. The differences in ultimate swelling can be explained by the slight differences in effective stress and initial dry densities, which led to a specimen that would have a smaller ultimate swelling. Overall, however, the trend is that the centrifuge test comes to an ultimate swell at a much quicker time than the ASTM D4546 tests, reducing total testing time by a factor of almost ten.

The results from the long-term tests on the Eagle Ford Shale at an effective overburden stress of 500 psf are shown below in Figure 2.7 and Table 2.3.

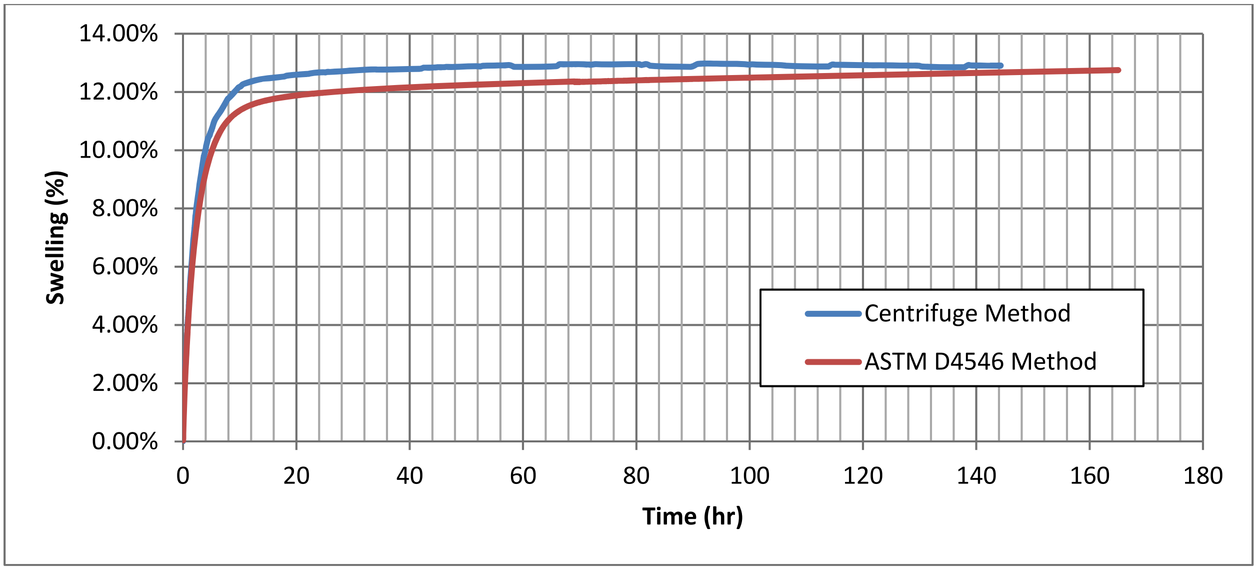


Figure 2.7: Results from Long-Term Tests on Eagle Ford Shale at 500 psf

Table 2.3: Overview of Conditions for Long-term Tests on Eagle Ford Shale at 500 psf

Method	ω_i (%)	γ_d (kN/m^3)	θ_i	ω_f (%)	Primary Swell (%)	Time to Primary Swell (hr)	Max Swell (%)	Time to Max Swell (hr)	Overburden Stress (psf)
ASTM D4546	21.3%	15.80	0.343	41.9%	11.4%	10.17	12.8%	165.04	500
Centrifuge	20.7%	16.10	0.341	40.6%	12.1%	10.15	12.9%	52.83	495

From the results, similar trends in Cook Mountain Clay are seen in the Eagle Ford Shale comparison. Starting from similar initial volumetric moisture contents, the tests came to a similar primary swelling value over a similar timeframe. However, ultimate swelling is slightly different again due to a difference in the initial dry density. Note that the trend is reversed in this case, i.e. the ASTM D4546 test has a lower dry density, as compared to Cook Mountain Clay, but a slight effect in ultimate swelling was seen. The differences between the time to ultimate swelling were not as great in this situation, but again, the result indicated that centrifuge testing has a significant impact on the time to primary swelling. Note that while the stresses showed a similar amount of difference as compared to the Cook Mountain test, the difference in the effect of stress was quite different due to the semi-log-linear effect of stress on the swelling of an expansive soil. Thus, a difference of 8 psf closer to 100 psf has a higher effect than that at a stress of 500 psf.

The results from the long-term tests on the Eagle Ford Shale at an effective overburden stress of 250 psf are shown below in Figure 2.8 and Table 2.4.

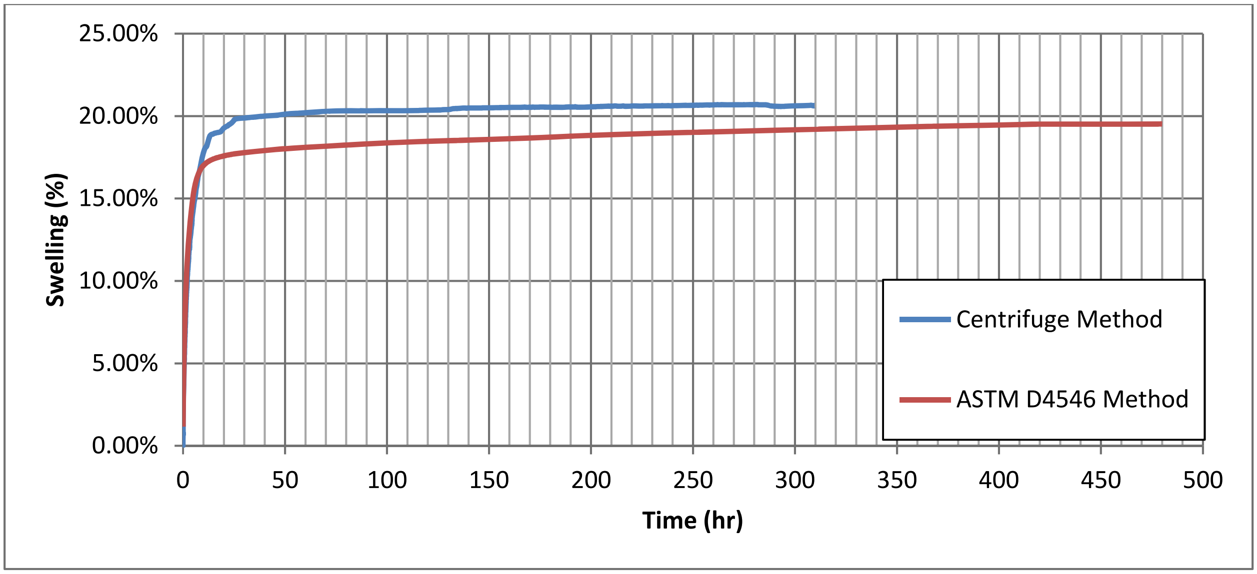


Figure 2.8: Results from Long-term Tests on Eagle Ford Shale at 250 psf

Table 2.4: Overview of Conditions for Long-term Tests on Eagle Ford Shale at 250 psf

Method	ω_i (%)	γ_d (kN/m³)	θ_i	ω_f (%)	Primary Swell (%)	Time to Primary Swell (hr)	Max Swell (%)	Time to Max Swell (hr)	Overburden Stress (psf)
ASTM D4546	20.0%	15.81	0.315	42.8%	19.5%	7.67	19.5%	480	250
Centrifuge	19.7%	16.01	0.316	42.6%	20.7%	12.73	20.7%	265.68	229

Here, too, the results show a similar trend. The primary swelling values were similar due to a similar initial volumetric moisture content and suction. However, the timeframe to reach the end of ultimate swelling was drastically decreased for the centrifuge test. The centrifuge test did swell slightly more, but the results come from a higher initial dry density as well as a lower effective stress.

Overall, the results from reconstituted specimens and a comparison between methodologies indicated that while the tests came to similar primary swelling values in a similar timeframe, the time to ultimate swelling was drastically quicker in the centrifuge, as compared to ASTM D4546. This difference stemmed from the increased gravitational gradient dominant in the secondary swelling portion of the swelling test, as opposed to the suction gradient for the primary swelling portion of the test. In the secondary swelling portion of the test, the flow of water into the microvoids was very slow due to low hydraulic conductivity. Thus, the increased gravitational gradient increased the flow of water through the specimen to saturate the specimen. Note that the specimens came to a similar end of test volumetric moisture content, indicating the specimens had been fully saturated by the end of the test. Thus, the centrifuge method provides similar results to the traditional method over a smaller time frame.

2.2.2 Validation of Experimental Methodology using Undisturbed Specimens

In winter 2014, soil samples were collected from both the stockpile of soils used for the roadway expansion of FM487 west of Barlett, TX, as well as undisturbed specimens collected via Shelby tubes on the east side of the section of FM487 and FM301. The collected soil samples included a large amount of fines, and belong to the Taylor-Navarro Formation, according to geologic mapping of the area. The roadway that was being expanded was known to sit on an expansive subgrade from previous experience, according to TxDOT personnel familiar with the project. The location of the project is shown in Figure 2.9, and the sampling operation is shown in Figure 2.10.

Figure 2.9: Project Location

Figure 2.10: Soil Stratigraphy at the Excavation Pit (left, a) and Shelby Tube Sampling at Intersection (right, b)

The results from swelling tests on Branyon Clay samples at their in-situ condition are shown below in Figure 2.11 and Table 2.5. The samples were taken from a depth of 4 ft. below the ground surface and tested at an equivalent stress to that which would be experienced in the field.

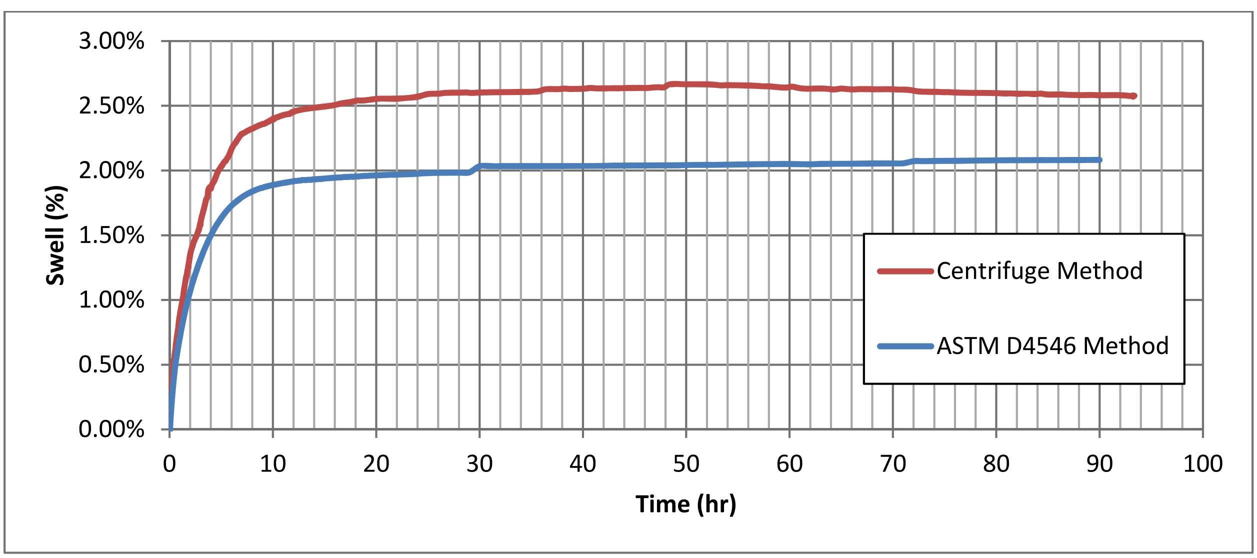


Figure 2.11: Results from Branyon Clay Specimens at their In-Situ Condition at a Depth of 4 ft.

Table 2.5: Overview of Conditions for Branyon Clay Specimens at their In-Situ Condition at a Depth of 4 ft

Method	ω_i (%)	γ_d (kN/m³)	θ_i	ω_f (%)	Primary Swell (%)	Time to Primary Swell (hr)	Max Swell (%)	Time to Max Swell (hr)	Overburden Stress (psf)
ASTM D4546	24.4%	15.30	0.381	31.8%	1.86%	8.50	2.08%	90	500
Centrifuge	23.8%	15.30	0.371	29.3%	2.25%	8.28	2.67%	48.64	477

The results from testing are slightly different primarily due to variations and natural heterogeneity in the soil formation. Even when specimens are taken from the same boring, there is local variability in both grain size distribution and soil structure, such as slickensides and vugs, near the surface, which can change the dry density and moisture at depth. Note that even with these differences, the swelling values were reasonably similar, with the specimen tested using the traditional method being slightly less expansive due to a higher moisture content. However, the trend remains of the centrifuge specimen reaching ultimate swelling faster than that of the ASTM D4546 method.

The results from swelling tests on Branyon Clay samples at their moisture condition are shown below in Figure 2.12 and Table 2.6. The samples were taken from a depth of 4 ft. below the ground surface and tested at an equivalent stress to that which would be experienced in the field.

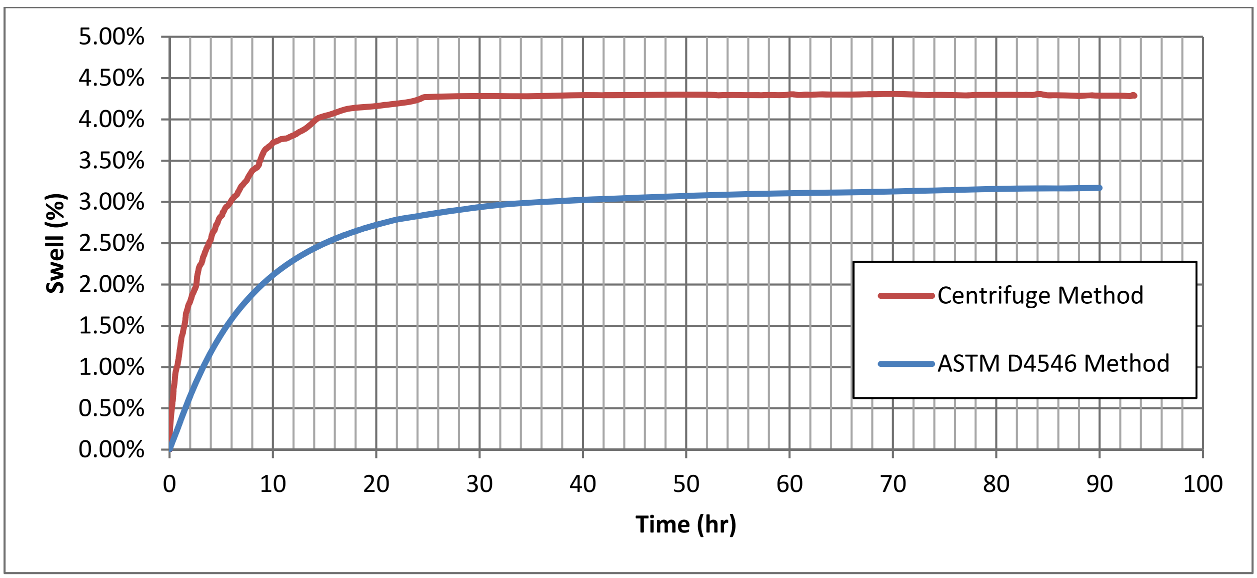


Figure 2.12: Results from Branyon Clay Specimens at their Moisture Adjusted Condition at a Depth of 4 ft.

Table 2.6: Overview of Conditions for Branyon Clay Specimens at their Moisture Adjusted Condition at a Depth of 4 ft.

Method	ω_i (%)	γ_d (kN/m^3)	θ_i	ω_f (%)	Primary Swell (%)	Time to Primary Swell (hr)	Max Swell (%)	Time to Max Swell (hr)	Overburden Stress (psf)
ASTM D4546	20.8%	16.25	0.368	26.7%	2.87%	26.00	3.17%	90	500
Centrifuge	22.7%	15.90	0.345	30.1%	4.15%	20.99	4.31%	26.08	477

The results indicate a trend similar to other tests, in that ultimate swelling was reached quicker in the centrifuge method as opposed to the traditional method. However, due to the moisture conditioning technique, there was inherently more variability in both the moisture content and dry density in the moisture adjusted specimens.

Using samples taken from the field, the trends seen from reconstituted specimens can also be seen in undisturbed specimens. Centrifuge testing will lead to faster ultimate swelling, which allows for full testing of specimens as opposed to traditional tests, which capture a smaller portion of the swelling curve.

2.3 Curve Fitting of Experimental Results

The shape of a stress-swell curve is similar to that of a soil-water retention curve, representing the relationship between volumetric soil moisture, θ, and suction, ψ. Van Genuchten (1980) developed a relatively simple and continuous analytical expression to describe the soil-water retention curve, shown in Equation 2.1

$$\theta(\psi) = \theta_r + \frac{\theta_s - \theta_r}{[1+(\alpha|\psi|)^n]^m} \tag{2.1}$$

Where θ_r and θ_s are the residual and saturated volumetric water contents, respectively, and represent the asymptotic minimum and maximum values, α is related to the inverse of the air-entry pressure, the suction at which inflection occurs in the soil-water retention curve, and n and m control the log-linear slope; m is commonly defined by Equation 2.2.

$$m = 1 - \frac{1}{n} \tag{2.2}$$

Van Genuchten's equation was rewritten to represent the stress-swell behavior of expansive soils by replacing α, n, and m with B/σ_{atm}, A, and C, respectively, and assuming θ_r is equal to zero. While n may be any value greater than 1, the equation was rewritten so that A may be any value greater than 0. The saturated volumetric water content, θ_s, was replaced with ε_0, and rearranged so that ε_0 represents the ASTM D4546 "free swell." This finalized equation, presented in Equation 2.3, is intended to be used to characterize the swell at stresses below the stress at which swelling no longer occurs and consolidation begins. Similar to the relationship between n and m, C may be assumed to be the function of A shown in Equation 2.4.

$$\varepsilon(\sigma') = \varepsilon_0 \left[\frac{1+\left(\frac{B\times 20.89\text{ psf}}{\sigma_{atm}}\right)^{A+1}}{1+\left(\frac{B\times\sigma'}{\sigma_{atm}}\right)^{A+1}} \right]^C \tag{2.3}$$

$$C = \frac{A}{A+1} \tag{2.4}$$

When fitting any combination of centrifuge and oedometer test results with the 4V-SLL model, ε_0 and A should always be optimized to minimize the objective function describing the error between the measured and modeled results. Setting B equal to 100 results in an inflection point located at approximately 1 kPa (σ_{atm}/1 kPa ≈ 100), and is recommended for most fits unless inflection is observed to occur at a larger stress. Equation 2.6 may be assumed to describe C in most cases. It is recommended to optimize the B and/or C parameters in addition to the ε_0 and A parameters if an unsatisfactory fit is produced from optimizing only ε_0 and A. An example of a set of centrifuge data fit with the 4V-SLL model is shown in Figure 2.13.

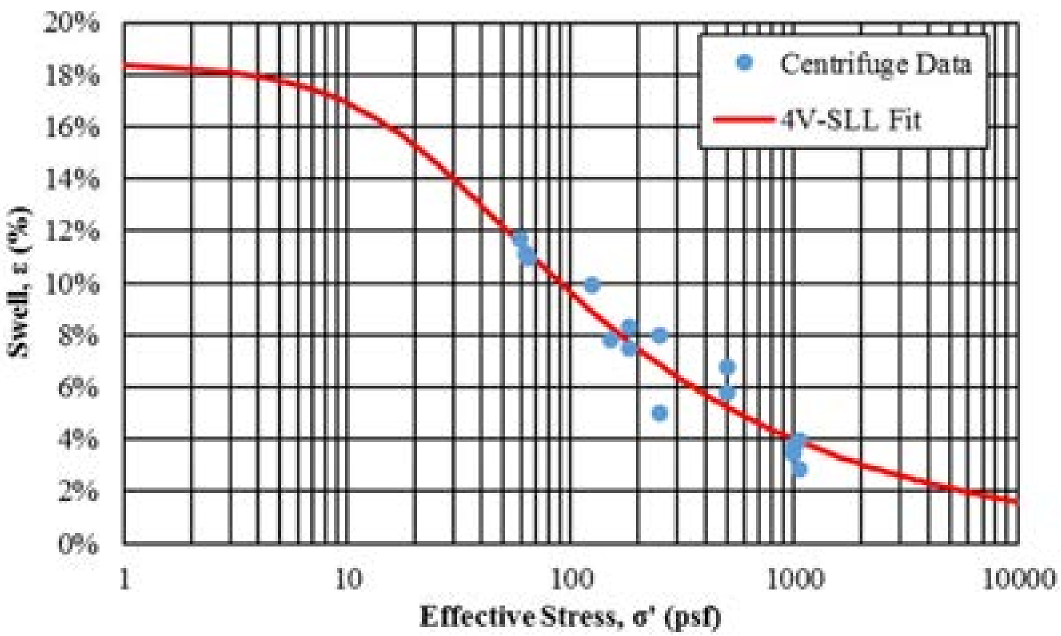


Figure 2.13: Example of the 4V-SLL model fit to the stress-swell behavior of a Tan Taylor clay (ε_0 = 15.2%, A = 0.396, B = 100, and C = A/(A+1); RMSE = 8.11 × 10^{-3}; r^2 = 0.922).

The derivative of the 4V-SLL model and its slope when plotted on a base-10 logarithmic σ'-axis are presented in Equation 2.5 and Equation 2.6, respectively. The integral of the 4V-SLL model evaluated between σ'_1 and σ'_2 is shown in Equation 2.7.

$$\frac{d}{d\sigma'}\left(\varepsilon(\sigma')\right) = \frac{-\varepsilon_0(A+1)BC\left(\frac{B\times 20.89\text{ psf}}{\sigma_{atm}}\right)^{A}\left(1+\left(\frac{B\times 20.89\text{ psf}}{\sigma_{atm}}\right)^{A+1}\right)^{C}}{\sigma_{atm}\left(1+\left(\frac{B\times\sigma'}{\sigma_{atm}}\right)^{A+1}\right)^{C+1}} \tag{2.5}$$

$$\log_{10}\text{-linear slope} = \sigma' \ln(10)\ \frac{d}{d\sigma'}\big(\varepsilon(\sigma')\big) \quad (2.6)$$

$$\int_{\sigma_1'}^{\sigma_2'} \varepsilon(\sigma')\, d\sigma' = \varepsilon_0 \sigma' \left(1 + \left(\frac{B\times 20.89\text{ psf}}{\sigma_{atm}}\right)^{A+1}\right)^{C} \times \mathrm{F}\left(\frac{1}{A+1}, C;\ \frac{A+2}{A+1};\ -\left(\frac{B\times\sigma'}{\sigma_{atm}}\right)^{A+1}\right)\Bigg]_{\sigma_1'}^{\sigma_2'} \quad (2.7)$$

Where F(a, b; c; z) is the hypergeometric function, defined in Equation 2.8 and $(q)_n$ is notation for the rising Pochhammer function, defined in Equation 2.9.

$$\mathrm{F}(a, b; c; z) = \sum_{n=0}^{\infty} \frac{(a)_n\,(b)_n\, z^n}{(c)_n\, n!} \quad (2.8)$$

$$(q)_n = \begin{cases} 1 & n = 0 \\ \prod_{i=1}^{n} q + i - 1 & n > 0 \end{cases} \quad (2.9)$$

Alternatively, the integral may be evaluated numerically by the midpoint rectangular method, shown in Equation 2.10, to calculate an approximation of the definite integral by finding the sum of the areas of rectangles with heights equal to the stress-swell function evaluated at the geometric mean of the stresses at the corners of each rectangle. This approximation becomes more accurate as the width of these rectangles, *Δσ'*, decrease and the number of these rectangles, *n*, increases.

$$\int_{\sigma_1'}^{\sigma_2'} \varepsilon(\sigma')\, d\sigma' = \lim_{\Delta\sigma' \to 0} \sum_{n=1}^{\frac{\sigma_2'-\sigma_1'}{\Delta\sigma'}} \Delta\sigma' \times \varepsilon\left(\sqrt{(\sigma_1' + (n-1)\Delta\sigma') \times (\sigma_1' + n\Delta\sigma')}\right) \quad (2.10)$$

The rectangular approximation of the integral may be easier to evaluate when solving by spreadsheet, because of difficulty with solving the hypergeometric function. Figure 2.14 shows that *Δσ'* of approximately 1 psf or less provides an acceptable approximation when compared to the analytical solution.

For a soil profile composed of *N* total distinct expansive soil units, each with a distinct stress-swell behavior, $\varepsilon_n(\sigma')$, the PVR is calculated by Equation 2.11, in which t_n represents the thickness of each soil unit, and $\sigma'_{1,n}$ and $\sigma'_{2,n}$ are the stresses at the top and bottom of each soil unit. Each integral may be evaluated analytically or numerically. If evaluated numerically, each soil unit should be divided into a minimum of 100 subsections per 1 ft. of soil so *Δσ'* approximates 1 psf or less.

$$\mathrm{PVR} = \sum_{n=1}^{N} \left[t_n \times \int_{\sigma_{1,n}'}^{\sigma_{2,n}'} \varepsilon_n(\sigma')\, d\sigma'\right] \quad (2.11)$$

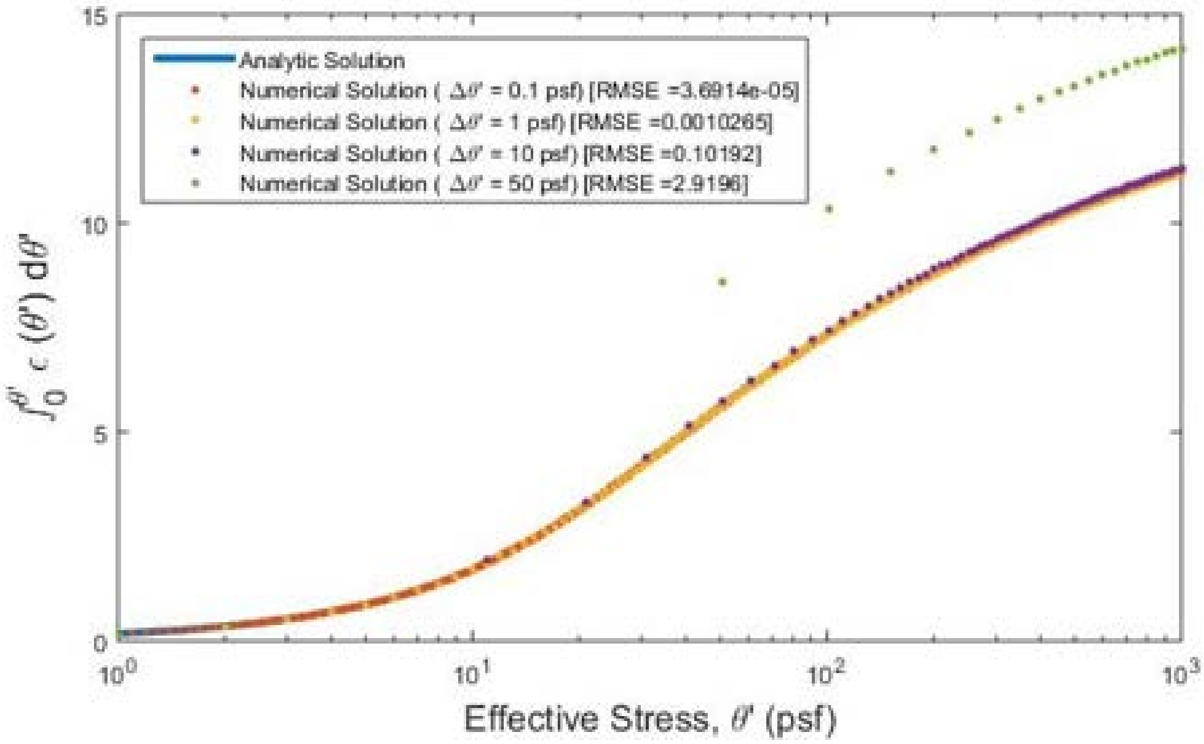


Figure 2.14: Comparison between the analytic and numerical solutions of the integral of the 4V-SLL model evaluated between 0 and σ' for varying Δσ' (ε_0 = 12.0%, A = 1.3, B = 100, and C = A/(A+1)).

2.4 Conclusions from Improvements to Centrifuge Environment

At the start of the project, improvements to the centrifuge allowed for an increased capacity for testing as well as a quicker, more reliable method due to decreased dependence on the height of water. The change in the permeameter cup allowed for the testing of both undisturbed and reconstituted specimens, as well as new boundary conditions that allowed for infiltration from both sides, matching the state of practice boundaries. Changes in the data acquisition system increased capacity from 4 cups to 6 cups per test. Testing was done to validate the new method, which was shown to be consistent with state of practice methods. Finally, the newly developed curve fitting allowed for curve fitting of results to be similar to the fit for unsaturated soils.

Chapter 3. Revised Potential Vertical Rise Methodology

Over the course of project 5-6048, a new revision to the TxDOT method for the characterization of expansive soils, the Potential Vertical Rise (PVR), was developed due to issues with Tex-124-E. Few studies have confirmed that Tex-124-E reasonably predicts PVR values that match ground movements observed in the field (Zornberg et al., 2008). The studies that have compared Tex-124-E predictions to field measurements have consistently shown that Tex-124-E poorly predicts ground heave, shown in Figure 3.1. When PVR predictions are small, field measurements are consistently larger, and conversely, when PVR predictions are large, field measurements are consistently smaller, shown in Figure 3.2 (Allen & Gilbert, 2006). This limitation may be caused by the limited number of tested soil specimens and poor curve fitting. McDowell tested relatively few specimens with low plasticity indices, and in the 1999 TxDOT modification, the correlation between volumetric change and plasticity index were extrapolated above the range of measured data without further testing of soils with plasticity indices greater than those that McDowell had tested. McDowell defined the soil binder as the amount of soil less than the No. 40 sieve; however, swelling is dominated by the effects of clay mineralogy and not by the presence of silts. Swelling would have been better estimated if the percentage of soil passing the No. 200 sieve had been included. The scope of soils tested by McDowell were limited to Guadalupe County, a small geographic region which does not include some of the more prominent geologic formations of Texas. Consequently, the PVR method is apt to perform poorly for soils derived from other formations, especially for soils outside of Texas, though the method is used nationally. Additionally, the method limits the influence of the soil's initial moisture content by dividing soil moisture into only three groups, while previous research has shown that a ± 3% difference in initial moisture content can significantly affect the magnitude of swelling (Walker, 2012).

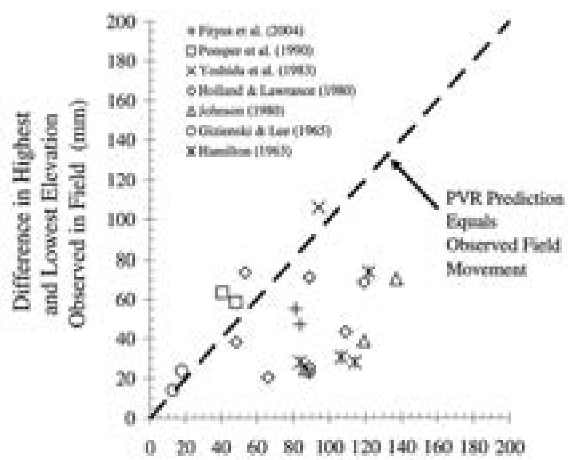


Figure 3.1: Measure ground movements compared to Tex-124-E predictions from twenty-two cases found in literature (Allen & Gilbert, 2006).

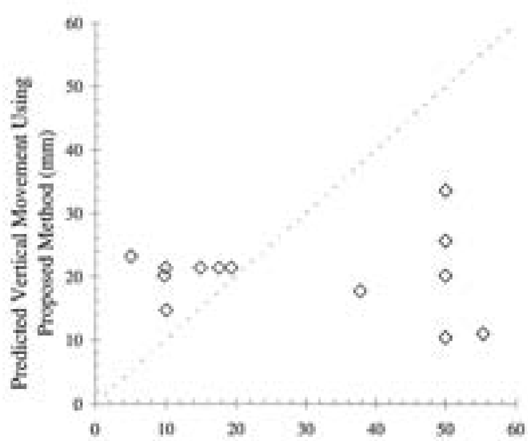


Figure 3.2: Accelerated shrink-swell oedometer test measurements compared to Tex-124-E predictions (Allen & Gilbert, 2006)

Due to these limitations, a new method was developed. The following chapter covers the calculation for the use of experimental data to calculate the potential vertical rise at a site, as well as the rationale for the initial test conditions.

3.1 Revised PVR Method

3.1.1 Developing a Stress-Swell Curve

To calculate the PVR of a given location, the soil profile must be divided into distinct soil units, each with their own stress-swell behavior, $\varepsilon(\sigma')$. Each soil's expansive behavior is described by a stress-swell curve. These curves are generated by plotting laboratory-measured welling against the corresponding stresses to which the specimens were tested and fitting the data with a curve, such as the 4V-SLL model. In the absence of available soil or time necessary for complete testing of a soil layer or layers, access to a database containing stress-swell information of previously tested soils may compensate for the lack of complete measurements.

Three methods are described in this section to develop stress-swell curves. Method A is preferred, but requires the largest number of tests. Method B involves using one test per soil strata and fitting the resulting data with "free swell" expected for the strata, predicted by the soil's liquid limit. Method C relies exclusively on stress-swell data from a database and involves no direct measurement.

3.1.2 Developing a Stress-Swell Curve: Method A

In Method A, each distinct soil unit of a soil profile is tested under a range stresses comparable to the stresses expected on site to develop a stress-swell curve for each unit. Generally, a minimum of three tests are performed for each soil unit, however, more tests are preferred, especially for thicker units subjected to large variation in stress with depth. The stresses at which these tests should be performed correspond to the total stresses at the top of the soil layer, the bottom of the soil layer, and the geometric mean. Consequently, the stresses at which a soil layer is tested are dependent on the thicknesses and unit weights of that layer and shallower layers above, as well as any surcharge loading. Similarly, a minimum of two tests may suffice for particularly thin layers, such as those with a thickness less than 3 ft. It is highly recommended that tests should be performed at stresses corresponding to depths within the active zone in 2 ft. increments (2, 4, 6, 8, 10 ft., etc.) on the appropriate soils for those depths. The swelling of a minimum of two soil specimens should be measured per centrifuge test. An example of a set of measured and fitted stress-swell curves for a soil profile is shown in Figure 3.3.

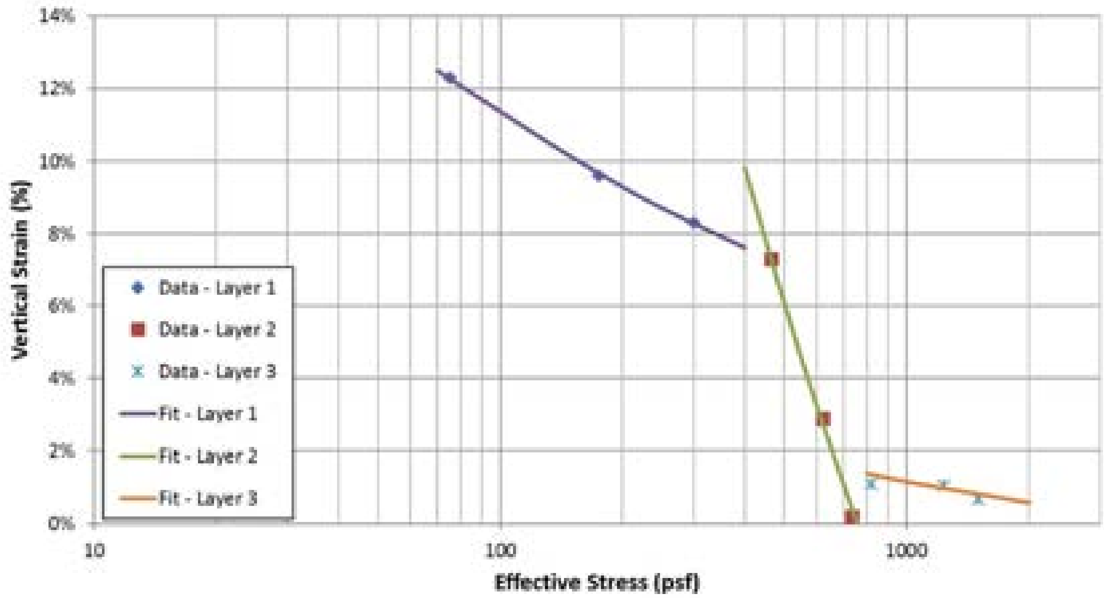


Figure 3.3: An example of a series of stress-swell curves for a profile of three distinct expansive soil units, plotted within the range of stresses expected for each layer.

Once a soil profile has been divided into distinct soil units within the active zone, the swelling behavior of each soil unit should be measured and modeled to calculate PVR. The testing procedure for a single soil layer and subsequent stress-swell fitting for Method A is described in the following steps:

1. Determine the moisture content of the soil layer (ASTM D2216-10). Note that the moisture content may vary with depth within a soil layer and may require measurement at multiple depths.

2. Determine the optimum moisture content and maximum dry unit weight of the soil by standard proctor (ASTM D698-12). Alternatively, the Atterberg Limits test (ASTM D4318-10) may be performed, and the optimum moisture content and maximum dry unit weight, in kN/m^3, may be predicted from the soil's Liquid Limit and Plasticity Index using the NAVFAC correlations (1962), reprinted below in Equation 3.1 and Equation 3.2.

$$\omega_{opt} = 6.77 + 0.43 \times LL - 0.21 \times PI \quad (3.1)$$

$$\gamma_{d,max} = 20.48 - 0.13 \times LL + 0.05 \times PI \quad (3.2)$$

3. Compare the soil's moisture content to 3% below optimum. If wetter than 3% below optimum, moisture-condition the soils specimens until the specimens dry to 3% below optimum. Moisture-conditioning of soil specimens should be performed in an environmental chamber or similar environment to ensure a relatively constant temperature and a humidity greater than 65%, preventing the formation of desiccation cracks caused by rapid moisture-loss, shown in Figure 3.4.

Figure 3.4: Examples of desiccation cracks in moisture conditioned soil specimens formed by excessive moisture-loss and caused by unstable humidity and temperatures during drying (Armstrong, 2014).

4. Calculate the total unit weight of the soil at optimum conditions using Equation 3.3.

$$\gamma = \gamma_{d,max}(1 + \omega_{opt}) \tag{3.3}$$

5. Calculate the effective stresses at the top and bottom of the soil layer at the end of swelling and the geometric mean using Equation 3.4, Equation 3.5, and Equation 3.6. Total unit weights may be used to calculate the effective stresses at the top and bottom of the layers because the pore-pressure in fully-swollen soil above the water table equals zero.

$$\sigma'_{top} = \int_0^{z_{top}} \gamma(z)\, dz + q \tag{3.4}$$

$$\sigma'_{bottom} = \int_0^{z_{bottom}} \gamma(z)\, dz + q \tag{3.5}$$

$$\sigma'_{midpoint} = \sqrt{\sigma'_{top} \times \sigma'_{bottom}} \tag{3.6}$$

Where z is depth, z_{top} is the depth to the top of the soil layer, z_{bottom} is the depth to the bottom of the soil layer, $\gamma(z)$ represents the total unit weight of the soil profile changing with depth, and q is any surcharge loading.

6. Calculate the artificial gravities which the accelerometer inside the centrifuge should measure so the soil specimens are subjected to the intended equivalent stresses of σ'_{top}, σ'_{bottom}, $\sigma'_{midpoint}$, and any other stresses at which testing will be performed. The final equivalent stress applied to a soil specimen is a function of the final soil and water

masses and heights. However, the artificial gravity may be approximated by Equation 3.7 using the initial heights and masses of the soil specimen and ponded water.

7.

$$Ng_a \cong \frac{\sigma' A_s\, e\, r_a}{r_{ob}\, m_{ob,s}\left(1+\frac{\frac{1}{2}m_s H_s - m_w\left(H_s+\frac{1}{2}H_w\right)}{r_{ob} m_{ob,s}}\right)^{\left(1+\frac{r_{ob} m_{ob,s}}{\frac{1}{2}m_s H_s - m_w\left(H_s+\frac{1}{2}H_w\right)}\right)}} \tag{3.7}$$

Where Ng_a is the artificial gravity measured at the location of the accelerometer, σ' is the intended equivalent stress, A_s is the cross-sectional area of the soil specimen, r_a and r_{ob} are the radial distances between the center of the centrifuge and the accelerometer and the top of the specimen, respectively, $m_{ob,s}$ is the submerged mass of the overburden weight, m_s and m_w are the initial masses of the soil specimen and ponded water, respectively, and H_s and H_w are the initial height of the soil specimen and the initial ponding height of the water, respectively. This equation has been derived by substituting and simplifying equations described by Plaisted (2015). For a complete derivation of the stresses in the specimen, see Plaisted (2009), Zornberg et al. (2013) and Armstrong (2014).

8. Perform centrifuge testing on the soil at the artificial gravities calculated in the previous step following the procedure outlined by Zornberg and Armstrong (2016). The swelling of a minimum of two specimens of the soil layer should be measured per centrifuge test. Note that over large ranges of stress, such as those larger than one log-cycle, the stress-swell curve may not be log-linear, and consequently, more tests are recommended to capture the full shape of the stress-swell curve for soil layers with large ranges of stress with depth. Similarly, fewer tests may be warranted if the range of stresses is small, such as for shallow soil layers with thickness less than 3 ft.

9. Calculate the equivalent effective stress at the end of testing for each specimen using Equation 3.8.

$$\sigma' = \frac{Ng_a\, r_{ob}\, m_{ob,s}\left(1+\frac{m_s(r_b-r_{cs})-m_w(r_b-r_{cw})}{r_{ob} m_{ob,s}}\right)^{\left(1+\frac{r_{ob} m_{ob,s}}{m_s(r_b-r_{cs})-m_w(r_b-r_{cw})}\right)}}{A_s\, e\, r_a} \tag{3.8}$$

This equation is rearranged from the equation presented in Step 6. The new variables, r_b, r_{cs}, and r_{cw} represent the radial distances between the center of the centrifuge and the base of the specimen, the center of mass of the soil specimen, and the center of mass of the ponded water above the specimen at the end of the test, respectively. Note that the value of r_{ob} has changed since being used in Step 6 due to swelling, and Ng_a represents the average artificial gravity between adding water to the sample and the end of the test.

10. Plot the measured swell as vertical strains against effective stresses on a semi-logarithmic plot.

11. Predict the resulting strain for an assumed set of fitting parameters for the stresses at which centrifuge tests were performed using the 4V-SLL stress-swell model and its assumption for *C*, reprinted in Equation 3.9 and Equation 3.10, respectively.

$$\varepsilon_{modeled}(\sigma') = \varepsilon_0 \left[\frac{1+\left(\frac{B\times 20.89\ \text{psf}}{\sigma_{atm}}\right)^{A+1}}{1+\left(\frac{B\times\sigma'}{\sigma_{atm}}\right)^{A+1}} \right]^{C} \tag{3.9}$$

$$C = \frac{A}{A+1} \tag{3.10}$$

Where $\varepsilon_{modeled}$ is the modeled strain, σ' is the effective stress in units of psf, σ_{atm} is standard atmospheric pressure, 2116.2168 psf, and ε_0, *A*, *B,* and *C* are fitting parameters. Note that all parameters may be any value greater than zero. Setting *B* equal to 100 may be a reasonable value to assume when fitting a limited amount of data or when a log-linear curve is desired.

12. Calculate the root-mean-square error (RMSE) between the measured strains, $\varepsilon_{measured}$, and the modeled strains, $\varepsilon_{modeled}$, for the *n* sets of data. RMSE is defined in Equation 3.11.

$$\text{RMSE} = \sqrt{\frac{\sum_{i=1}^{n}(\varepsilon_{measured,i} - \varepsilon_{modeled,i})^2}{n}} \tag{3.11}$$

13. Optimize the ε_0 and *A* parameters of the 4V-SSL stress-swell model to the measured data to minimize the resulting RMSE using Solver in Microsoft Excel, MATLAB's Optimization Toolbox, or any similar optimization algorithm.

14. Plot both the modeled and measured results on the same semi-logarithmic stress-swell plot to visually ensure the quality of the fit of the optimized parameters. If an unsatisfactory fit was produced, attempt fitting the data again. A better fit may be produced by removing outlier data or optimizing an additional parameter or parameters, i.e. *B* and/or *C* if *B* equal to 100 or *C* defined by Equation 2.23 produced a poor fit. Recommendations for fitting data are described below and should be followed in the order listed:

    a. First fit centrifuge data by optimizing the ε_0 and *A* parameters with *B* set to 100 and *C* defined by Equation 2.23.

    b. If the best optimized ε_0 parameter appears unrealistically large, ε_0 may be set equal to the log-linear extrapolated value; in Microsoft Excel, this formula for ε_0 may be expressed as *=forecast(log10(20.89),*" Array of Swell Values"*, log10(*"Array of Stress Values"*))*. Optimize *A* parameters with *B* set to 100 and *C* defined by Equation 2.23.

    c. If an unsatisfactory fit if produced from Steps a and/or b, attempt fitting the centrifuge data by optimizing *B* in addition to the parameters previously optimized (i.e. ε_0 and A, or just A if ε_0 has been set equal to the log-linear extrapolation in Step b) and with C defined by Equation 2.23.

d. If an unsatisfactory fit if produced from Steps a - c, attempt fitting the centrifuge data by optimizing C in addition to the parameters previously optimized, except for B, which should be set equal to 100.

e. Lastly, if an unsatisfactory fit if produced from Steps a - d, fit the centrifuge data by optimizing B and C in addition to the parameters previously optimized.

3.1.3 Developing a Stress-Swell Curve: Method B

Method B, commonly referred to as the "single-point" method, requires direct measurement swelling at only one stress, the geometric mean of the maximum and minimum stresses expected of the soil unit. The "free swell", or swell at 1 kPa, is predicted from available liquid limit data, and a stress-swell curve is fitted through the measured and predicted data.
The testing procedure for a single soil layer and subsequent stress-swell fitting for Method B is described in the following steps:

1. Perform the Atterberg Limits test (ASTM D4318-10) to determine the liquid limit of the soil unit.

2. Perform Steps 1 through 6 of the Method A procedure. Procedure for determining optimum conditions, soil preparation, and predicting the effective stresses expected of the soil layer are identical. In Step 6, only the centrifugal artificial gravity corresponding to $\sigma'_{midpoint}$ needs to be calculated.

3. Perform centrifuge testing on the soil at the artificial gravity corresponding to a stress equal to $\sigma'_{midpoint}$ following the procedure outlined by Zornberg and Armstrong (2016). The swelling of a minimum of two specimens of the soil layer should be measured in the centrifuge test.

4. Perform Step 8 of the Method A procedure to calculate the equivalent effective stresses felt by specimens at the end of testing.

5. Estimate the "free swell," ε_0, the swell at 1 kPa (20.89 psf), from a geologically similar soil using the NRCS soil survey. Note that the map of sites used for project 5-6048-03 is shown in Figure 3.5.

    a. Note that there is uncertainty in the equation above and assumed to be cubic in relation to the liquid limit. The data is taken from 41 separate soils, relating their liquid limit to the "free swell" condition, and is shown below with the 99% confidence intervals shown assuming non-constant variance.

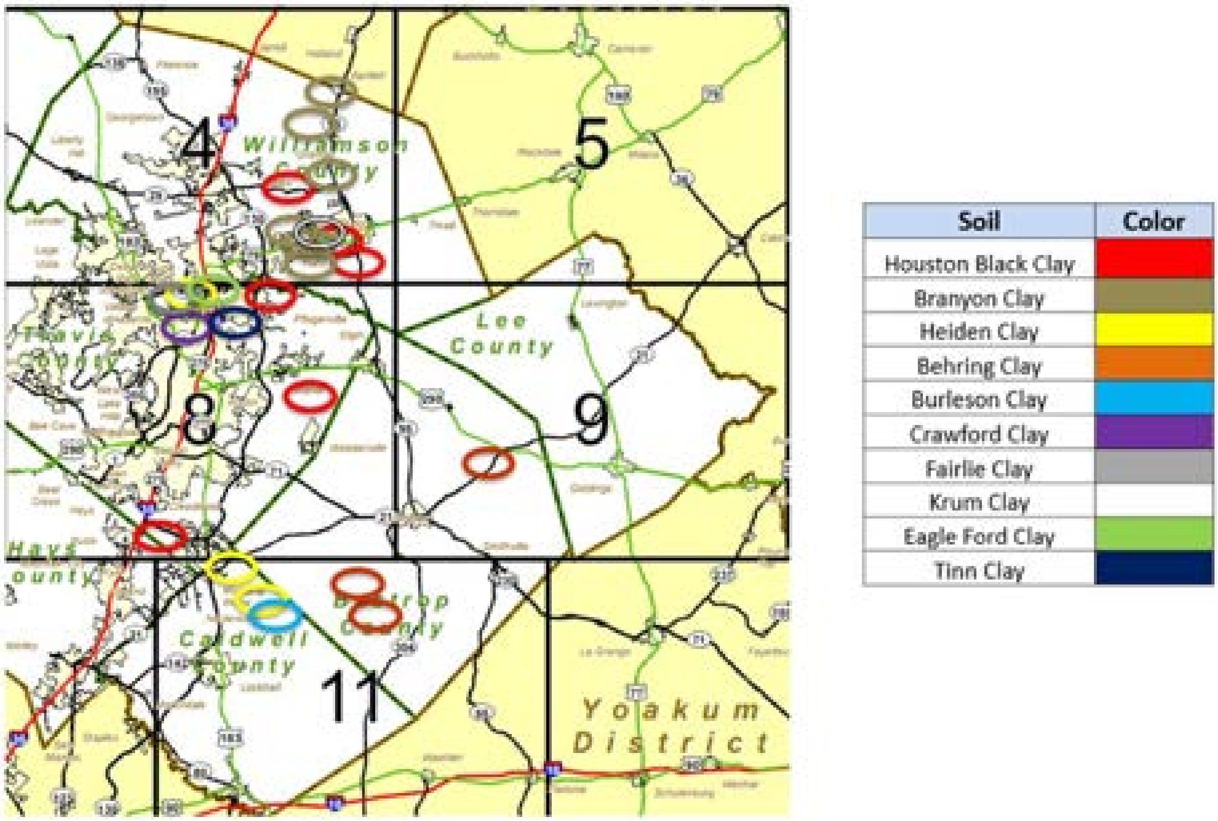


Figure 3.5: Sites and Soils within the Database

6. Perform Steps 9 through 13 of the Method A procedure. The stress-swell curve fitting procedure is similar to that described in Method A, but only involves fitting a curve through the predicted ε_0 and the swell measured at $\sigma'_{midpoint}$. When parameter-optimizing, set ε_0 to the value predicted to in Step 5 and optimize the *A* parameter. The *B* and/or *C* parameters may be optimized if optimizing only *A* results in an unsatisfactory fit.

3.1.4 Developing a Stress-Swell Curve: Method C

Method C involves choosing stress-swell curve fitting parameters appropriate for the soil from a soil-information data base to develop a stress-swell curve without any direct measurements. Method C is not advised for developing a PVR value to be used in detailed design but may be reasonable for use in preliminary design of projects on expansive soils.

A soil's swelling behavior is based on geologic and depositional history, grain size analyses, and minerology. Likewise, the measured stress-swell behavior of a soil is based on the conditions under which the soil has been tested, since any stress-swell curve and its parameters may only be appropriate for a given range of stresses or initial testing conditions. Consequently, the following information is recommended to be included in a database if Method C were used:

- Location of sampling
- USDA soil classification and taxonomy

- Atterberg Limits
- Fines content
- Initial water content and compaction unit weight of tested soils
- 4V-SLL model fitting parameters: ε_0, A, B, and C
- Fitting parameters of any other stress-swell model (ASTM D45446, 2V-NLL, etc.)
- The range of stresses between which the soil has been tested and between which those fitting parameters are appropriate
- Root-mean-square error (RMSE) corresponding to each set of fitting parameters as a measure of assurance of fit

The procedure for generating a stress-swell curve of a single soil unit by Method C is described in the following steps:

1. Use the information from the United States Department of Agriculture (USDA)'s Natural Resources Conservation Service (NRCS) tool, the web soil survey, to identify the soil and horizon at the location. Utilize the results from the database to estimate the "free swell," ε_0, condition and A parameter based upon the best fit from geotechnical properties or the horizon and liquid limit. Note that geologically similar soils from the same horizon may have significantly different swell-stress properties, thus some estimation of the grain size distribution and liquid limit are preferred.

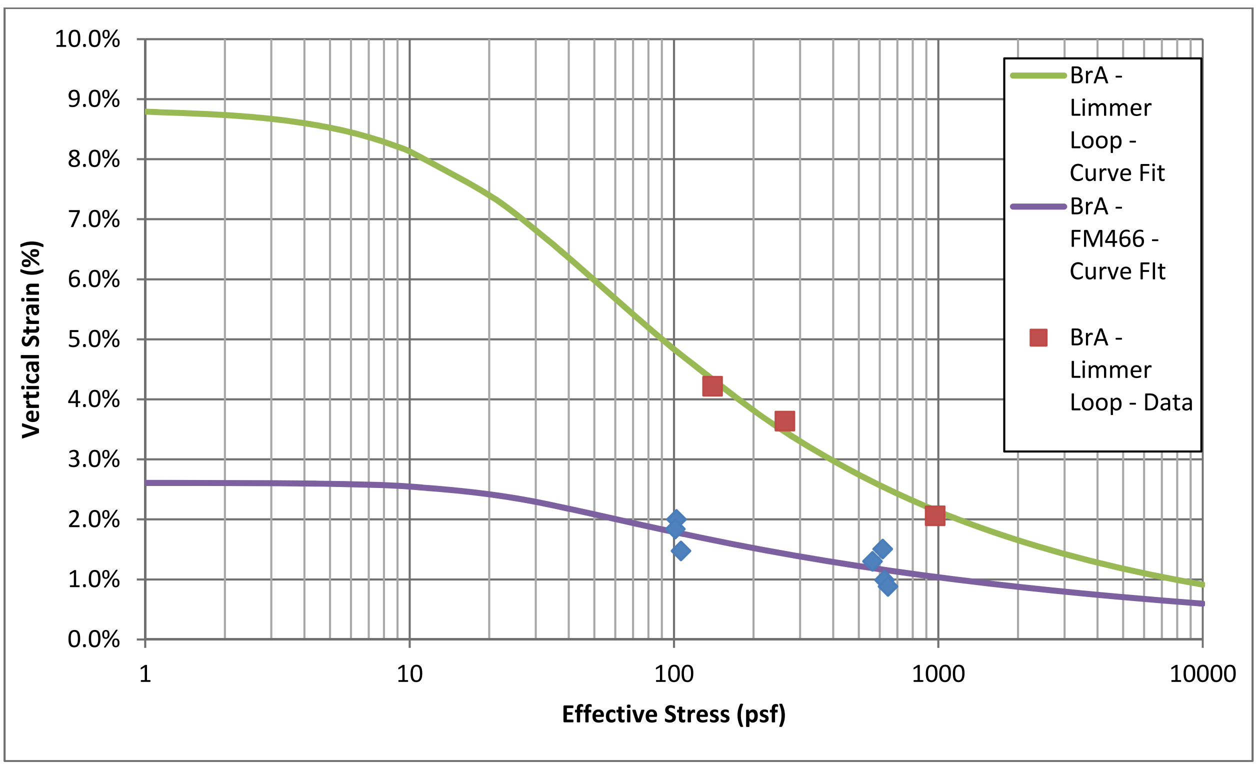


Figure 3.6: Difference in Geologically Similar Soils

3.1.5 Calculating PVR from the Stress-Swell Behavior of Expansive Soils

PVR of an individual soil unit is calculated by integrating the soil's stress-swell curve between the range of stresses felt by that layer. The total PVR for a soil profile equals the sum of each soil unit's PVR. For the active zone of a soil profile composed of *N* total distinct expansive soil units, each with a distinct stress-swell behavior, $\varepsilon_n(\sigma')$, the PVR is calculated by Equation 3.12.

$$\mathrm{PVR} = \sum_{n=1}^{N} \left[t_n \times \int_{\sigma'_{1,n}}^{\sigma'_{2,n}} \varepsilon_n(\sigma')\, d\sigma' \right] \tag{3.12}$$

Where t_n represents the thickness of each soil unit, $\varepsilon_n(\sigma')$ represents the stress-swell curve of each soil unit, and $\sigma'_{1,n}$ and $\sigma'_{2,n}$ are the stresses at the top and bottom of each soil unit. For spreadsheet calculation of PVR, the integral of the stress-swell curve may be evaluated numerically using the rectangular method. Alternatively, this integral may be evaluated analytically; see Section 2.3 for full description of the integral of the 4V-SLL model.

The procedure for numerically calculating the PVR of a soil profile with *N* distinct expansive soil units within the active zone is described in the following steps:

1. Determine the thicknesses, t_n, of each soil unit.

2. Divide each soil unit into M_n subsections with equal thicknesses, Δt_n. For each 1 ft. section of soil, a minimum of 100 subsections should be created.

3. Determine the stresses at the top and bottom of each soil subsection, $\sigma'_{top,n,m}$ and $\sigma'_{bottom,n,m}$. See Step 5 of the Method A procedure for stress-swell curve generation in Section 3.1.2 for further information on determining these stresses.

4. For each subsection, determine the geometric mean of the top and bottom stresses, $\sigma'_{midpoint,n,m}$, using Equation 3.13.

$$\sigma'_{midpoint,n,m} = \sqrt{\sigma'_{top,n,m} \times \sigma'_{bottom,n,m}} \tag{3.13}$$

5. For each subsection, calculate the swelling in units of length due to $\sigma'_{midpoint,n,m}$, using the appropriate stress-swell curve, $\varepsilon_n(\sigma')$, and it's appropriate 4V-SLL parameters, $\varepsilon_{0,n}$, A_n, B_n, and C_n and Equation 3.14 which has been simplified for the 4V-SLL model in Equation 3.15.

$$\mathrm{PVR}_{n,m} = \Delta t_n \times \varepsilon_n\left(\sigma'_{midpoint,n,m}\right) \tag{3.14}$$

$$\mathrm{PVR}_{n,m} = \Delta t_n \times \varepsilon_{0,n} \left[\frac{1+\left(\frac{B_n \times 20.89\ \mathrm{psf}}{\sigma_{atm}}\right)^{A_n+1}}{1+\left(\frac{B_n \times \sigma'_{midpoint,n,m}}{\sigma_{atm}}\right)^{A_n+1}} \right]^{C_n} \tag{3.15}$$

6. Calculate the total PVR of the entire active zone of the soil profile using Equation 3.16.

$$\mathrm{PVR} = \sum_1^N \sum_1^{M_n} \mathrm{PVR}_{n,m} \tag{3.16}$$

3.2 Estimation of Initial Conditions

Various correlations have been developed to predict optimum water content and maximum dry unit weight from soils' Atterberg Limits. Snyder (2015) performed standard proctor tests (ASTM D698-12) on nineteen soils from San Antonio, Texas, and compared the measured optimum conditions to the values predicted by several of correlations, including McDowell's dry condition (1956), NAVFAC (1962), Al-Khafaji (1993), and USACOE (Meyer, 1968). In this study, the results of Snyder's tests were reanalyzed, and the proctor-measured optimum conditions of an additional 21 soils were compared to their predicted values. A summary of these correlations is presented in Table 3.1.

The results of Snyder's tests and of the more recent series of Atterberg Limits and proctor compaction tests and the corresponding predicted optimum values are shown in Figure 3.5 and Figure 3.6 for the various correlations presented in Table 3.1. The statistics of how well each correlation predicts the optimum conditions of the soils in from this study is shown in Table 3.2 and Table 3.3 for all soils and Texas's two most predominant expansive soils, Houston Black and Branyon soils. The raw data and resulting predicted values analyzed here are shown in Table 3.4 and Table 3.5.

Table 3.1: Correlations to Predict Optimum Conditions

Source	Optimum Water Content, ω_{opt}	Maximum Dry Unit Weight, $\gamma_{d,max}$ (kN/m^3)
McDowell* (1956)	$\omega_d + 3\% = 0.2 \times LL + 12\%$	-
NAVFAC (1962)	$\omega_{opt} = 6.77 + 0.43 \times LL - 0.21 \times PI$	$\gamma_{d,max} = 20.48 - 0.13 \times LL + 0.05 \times PI$
Al-Khafaji (1993)	$\omega_{opt} = 0.14 \times LL + 0.54 \times PL$	$\gamma_{d,max} = 9.81 \times (2.27 - 0.019 \times PL - 0.003 \times LL)$
USACOE (Meyer, 1968)	$\omega_{opt} = 1.74(PL)^{0.82}$	-

*Snyder (2015) compared ω_{opt} – 3% to McDowell's dry condition as part of a study to best predict the water content at which to prepare soil specimens for direct swell testing. Swell tests are commonly performed on samples prepared at ω_{opt} – 3%.

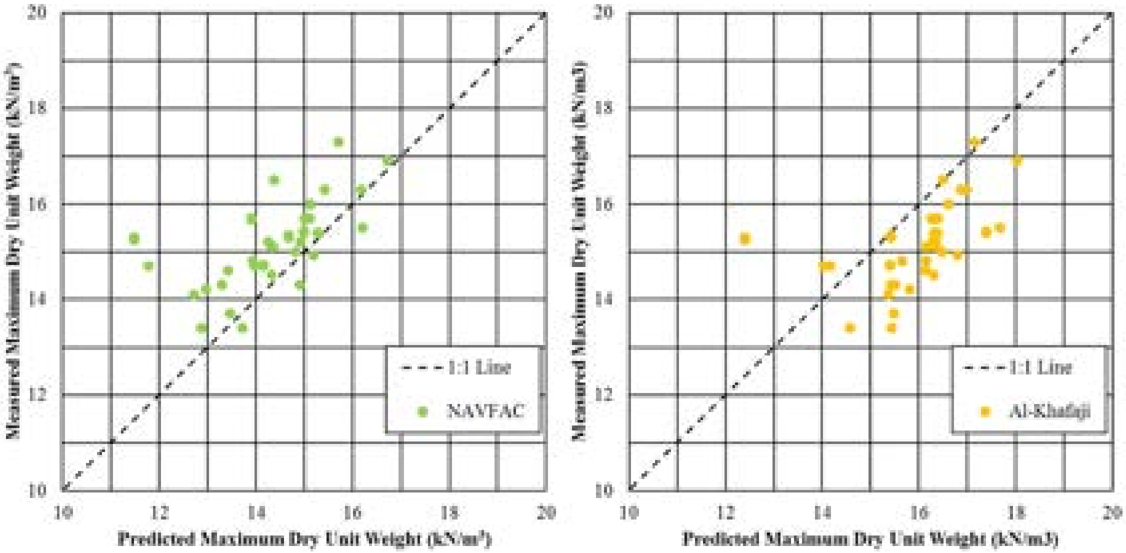


Figure 3.7: Comparison of measured and predicted maximum dry unit weight.

Table 3.2: Statistics of Correlative Methods' ability to Predict Maximum Dry Unit Weight

		NAVFAC	Al-Khafaji
		γ_d [kN/m^3]	γ_d [kN/m^3]
All Soils	**Root-Mean-Square Error:**	1.283	1.338
	Coefficient of Determination, r^2:	0.334	0.216
	Variance, σ^2:	1.231	1.206
Houston Black soils	**Root-Mean-Square Error:**	0.436	0.678
	Coefficient of Determination, r^2:	0.715	0.224
	Variance, σ^2:	0.598	0.618
Branyon soils	**Root-Mean-Square Error:**	0.297	0.510
	Coefficient of Determination, r^2:	0.423	0.482
	Variance, σ^2:	0.633	1.008

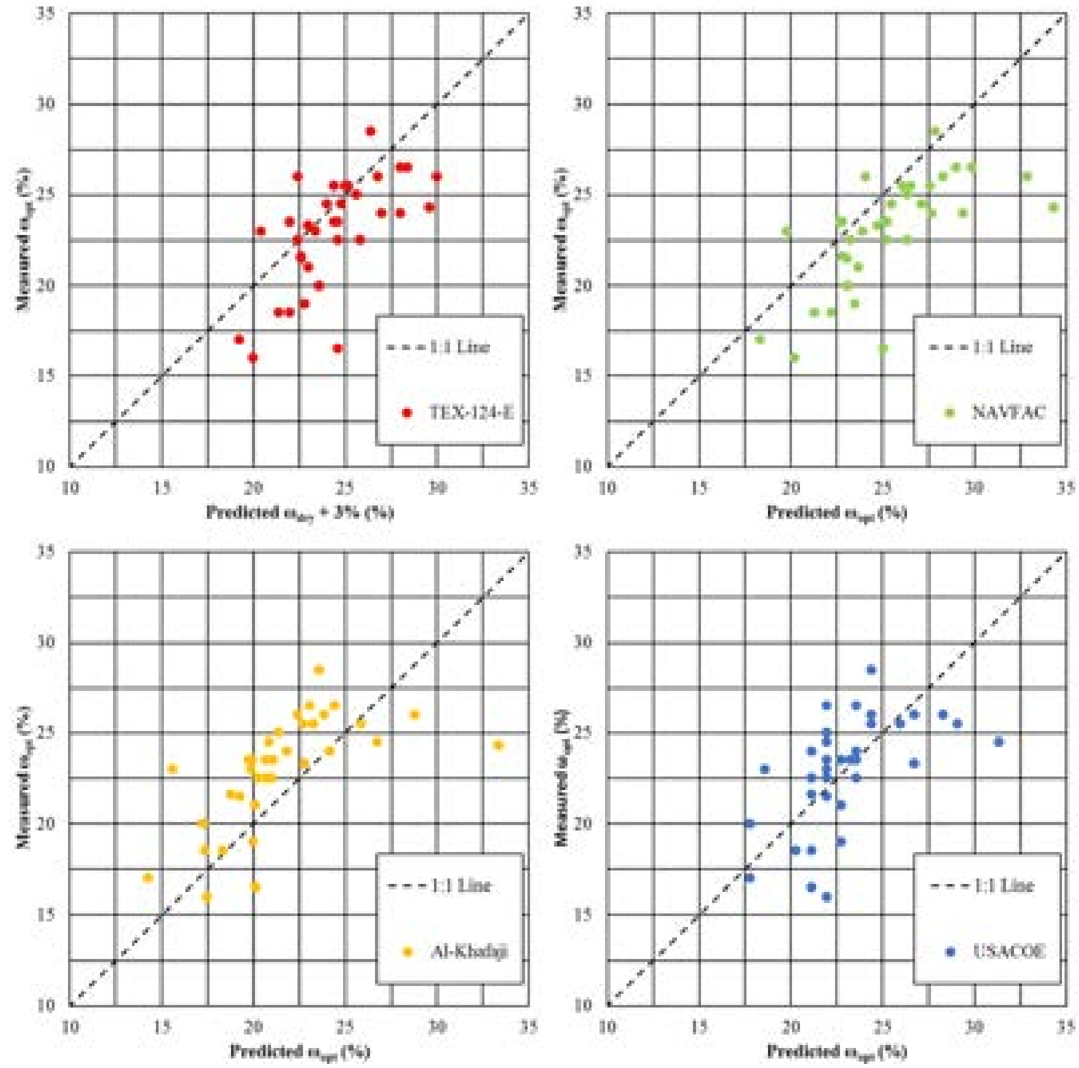


Figure 3.8: Comparison of measured and predicted optimum water content values.

Table 3.3: Statistics of Correlative Methods' ability to Predict Optimum Water Content

		TEX-124-E	NAVFAC	Al-Khafaji	USACOE
		$\omega_{dry} + 3\%$	ω_{opt}	ω_{opt}	ω_{opt}
All Soils	**Root-Mean-Square Error:**	2.821	3.735	3.377	3.609
	Coefficient of Determination, r^2:	0.375	0.413	0.374	0.243
	Variance, σ^2:	8.088	11.965	12.399	12.141
Houston Black soils	**Root-Mean-Square Error:**	1.544	2.078	2.863	2.849
	Coefficient of Determination, r^2:	0.552	0.660	0.636	0.010
	Variance, σ^2:	4.177	4.950	4.896	4.130
Branyon soils	**Root-Mean-Square Error:**	1.779	2.131	3.589	3.458
	Coefficient of Determination, r^2:	0.526	0.542	0.402	0.218
	Variance, σ^2:	2.858	4.489	7.866	8.194

Root-mean-square error (RMSE) is a measure of the difference between the measured and predicted values and equals the sample standard deviation of the square errors. The coefficient of determination, r^2, is another measure of a model's ability to predict and represents the percentage of variation from the model's mean prediction can be explained by the model. Variance equals the average of the square errors of the measured values and the mean. An identical fit between measured and predicted values is represented by an RMSE and variance of 0 and an r^2 of 1. The best correlation for soil optimum conditions is the one resulting in the lowest RMSE and variance and an r^2 nearest to 1. Based on these analyses, the NAVFAC correlations should be used to predict both optimum water content and maximum dry density of Texas soils when proctor compaction data is unavailable.

The NAVFAC correlation better predicted the maximum dry unit weight than Al-Khafaji's correlation for the Texas soils in this study. For all the soils in this study, both correlations had comparable RMSE and variance, but NAVFAC produced a larger r^2. For the Houston Black soils, NAVFAC produced a smaller RMSE, far larger r^2, and comparable variance compared to Al-Khafaji's model. And for the Branyon soils, NAVFAC produced a far smaller RMSE and variance and similar r^2 as Al-Khafaji's correlation.

The NAVFAC correlation also better predicted optimum moisture content than Al-Khafaji's correlation and the USACOE correlation. For all the soils in this study, NAVFAC produced comparable RMSE and variance to the Al-Khafaji and USACOE correlations, but the largest r^2. For the Houston Black soils, NAVFAC produced the smaller RMSE, larger r^2, and comparable variance. And for the Branyon soils, NAVFAC produced smaller RMSE and variance and larger r^2.

Though the water content predicted by the TEX-124-E dry condition plus an additional 3% consistently better predicted optimum water content than the NAVFAC correlation for these Texas soils, this correlation is nonetheless not advised to be used. The dry condition represents the water content at which McDowell's samples typically began to swell and has no intended relationship to optimum water content. Furthermore, the 3% added to the water content predicted by the dry condition is a somewhat arbitrary number. It is only the product of the previous study in which Snyder (2015) analyzed these correlations to study which method best predicted the water content at which samples should be prepared from swell testing, i.e. 3% below the optimum. In addition, the values predicted by the dry condition +3% and NAVFAC are remarkably similar for most soils. The average difference between NAVAFC and TEX-124-E predicted optimum water content values is 1.2%, and their predictions differed by more than 2% only for five of the forty soils, none of which were any of the Houston Black soils, the predominant expansive soil in Texas.

3.3 Conclusions from Revision to the Potential Vertical Rise Method

From this project, changes to the ways in which the PVR for a site was calculated were developed. The development came from the implementation of experimental results into the generation of stress-swell curves which in turn can be used to calculate PVR. The new method is an improvement on the previous method, Tex-124-E, as the results come from results of either geologically similar soils or from site-specific soils. The initial conditions for testing are very important, and thus the analysis of correlations for the optimum moisture content and density was performed to show that those from NAVFAC were the closest to those from experimental compaction curves. Overall, the section provides the new method to calculate PVR.

Chapter 4. Laboratory Characterization of Field Sites

During the summer of 2015, sites from Travis, Williamson, Bastrop and Caldwell Counties were sampled for characterization and testing of expansive subgrades. An additional site in Hutto, TX was sampled in December 2015 during the installation of volumetric moisture and suction sensors detailed in Chapter 5. These sites were selected because they exhibited previous failures during the Central Texas drought in the early part of the decade, and they served as a way to measure the results of centrifuge testing against field performance of pavements over potentially expansive subgrades. An overview of all sites is provided in Section 4.1, while a description of the sampling protocol is discussed in Section 4.2. More detailed descriptions of site locations, identification of soil types and classification of sampled soil are provided on a per-site basis in Sections 4.3.1 to 4.3.17. Additionally, as a part of the project, additional sites were sampled and tested from previous sampling works and are documented in sections 4.1.18 and 4.1.19.

4.1 Geotechnical Characterization of Sampling Locations

The following section covers the general geotechnical characterization at each site. Note that the sites typically only had Atterberg Limits test run on them for the determination of the liquid limit and plasticity index for use in Equations 2.1 and 2.2. Wet sieve analysis was used to determine the percentage of soil passing the No. 40 sieve and percentage of soils considered to be fines. The soil was prepared by air-drying the soil specimens on serving trays using forced-air ventilation. The specimens were then processed through a soil crusher twice. These crushed samples were used in their entirety for the sieving process and sieved through the No. 40 sieve for the Atterberg Limits and swelling tests.

4.1.1 Site 1: Yett Creek Neighborhood Park [Crawford Clay, CR]

This section summarizes the findings at Site 1. In August 2015, soil samples were collected from the Yett Creek Neighborhood Park north of Austin, TX in Travis County. The collected soil samples included a large amount of fines, and belong to the Edwards Formation, according to geologic mapping of the area. A roadway that leads into the south end of the park had a very significant amount of distress and cracking; previous studies from government agencies also indicate that the underlying subgrade may be expansive. The collected soil samples were extensively tested to identify soil characteristics and swelling properties.

<u>Location and Identification of Soil Samples</u>

The location of Site 1 corresponds to the south end of the Yett Creek Neighborhood Park in the northwest portion of Austin, TX, as shown in Figure 4.1. The soil samples were collected using shovels to remove the topsoil and sample the soil at depth. A layer of dark red to purplish soil was encountered and sampled to a depth of a foot below the ground surface. The GPS coordinates of the borehole locations at Site 1 were marked for soil identification purposes.

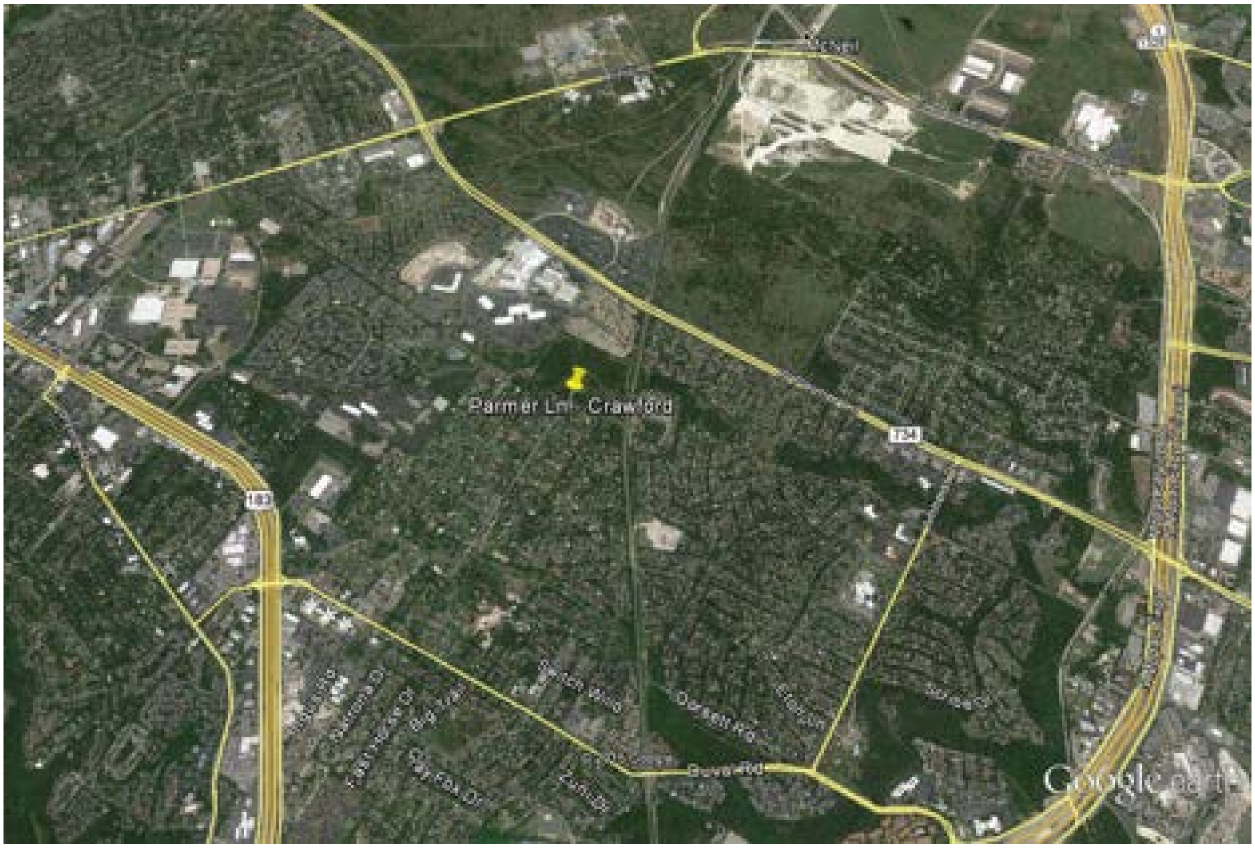
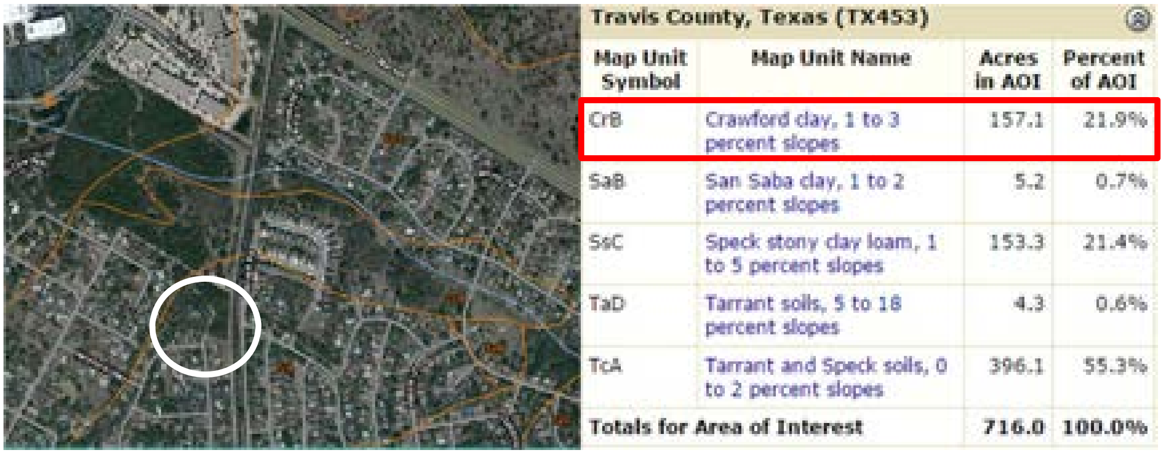


Figure 4.1: Map of Site 1 Location near Yett Creek Neighborhood Park (Google 2014)

The GPS coordinates were input into Google Earth (the resulting image is shown in Figure 4.1), and the U.S. Geological Survey (USGS) geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Fredericksburg Group of the Edwards Formation. To complement this information, an interactive map from the United States Department of Agriculture (USDA) was used to identify the soil found at the ground surface of Site 1. From this point of the study, the USDA interactive map was relied upon to identify the collected soil samples from each site, even prior to the sampling process. The information from the USDA soil survey indicates that the soil retrieved from the site is the Crawford Clay, as shown in Figure 4.2.



Travis County, Texas (TX453)			
Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
CrB	Crawford clay, 1 to 3 percent slopes	157.1	21.9%
SaB	San Saba clay, 1 to 2 percent slopes	5.2	0.7%
SsC	Speck stony clay loam, 1 to 5 percent slopes	153.3	21.4%
TaD	Tarrant soils, 5 to 18 percent slopes	4.3	0.6%
TcA	Tarrant and Speck soils, 0 to 2 percent slopes	396.1	55.3%
Totals for Area of Interest		716.0	100.0%

Figure 4.2: Soil Survey Map and Table at Site 1 (USDA 2013)

Characterization of Crawford Clay Samples [CR]

The Crawford Clay soil was air dried and processed for the soil characterization and centrifuge tests. Atterberg Limits tests, shown in Table 4.1, determined an average liquid limit of 71%, and an average plastic limit of 28%. These results defined the plasticity index as 43%. The grain size distribution (GSD) curve produced from the Wet Sieve test is shown in Figure 4.3. The results of the wet sieve analysis showed that the soil was composed of about 50% sand-sized particles and 50% fine-sized particles. The Standard Proctor curve was defined from six compaction specimens (Figure 4.4). From these test results, the optimum moisture content can be defined as 27%; the maximum dry unit weight was 14.4 kN/m^3 (91.7 pcf).

Table 4.1: Results from Atterberg Limit Tests on Crawford Clay Samples from Site 1

Test #	**1**	**2**
Predicted Liquid Limit, LL	72%	69%
Selected Liquid Limit, LL	72%	70%
Plastic Limit, PL	28%	28%
Plasticity Index, PI	44%	42%
Averaged Liquid Limit, LL_{avg}	**71%**	
Averaged Plastic Limit, PL_{avg}	**28%**	
Averaged Plasticity Index, PI_{avg}	**43%**	

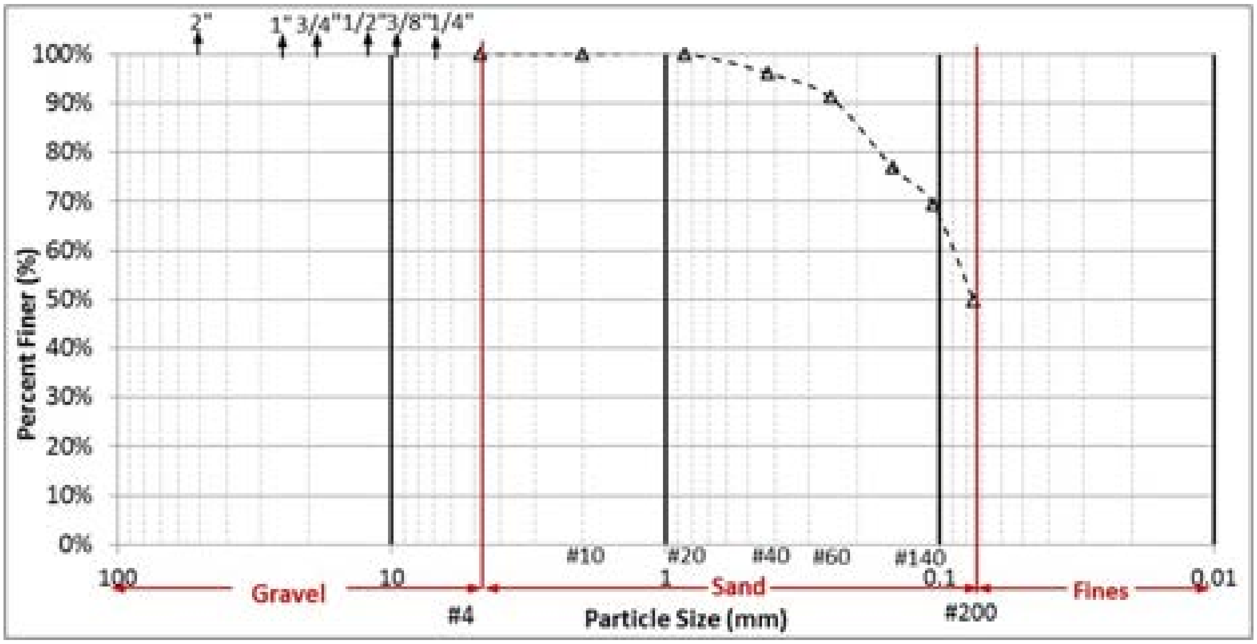


Figure 4.3: Grain Size Distribution Curve for Crawford Clay at Site 1

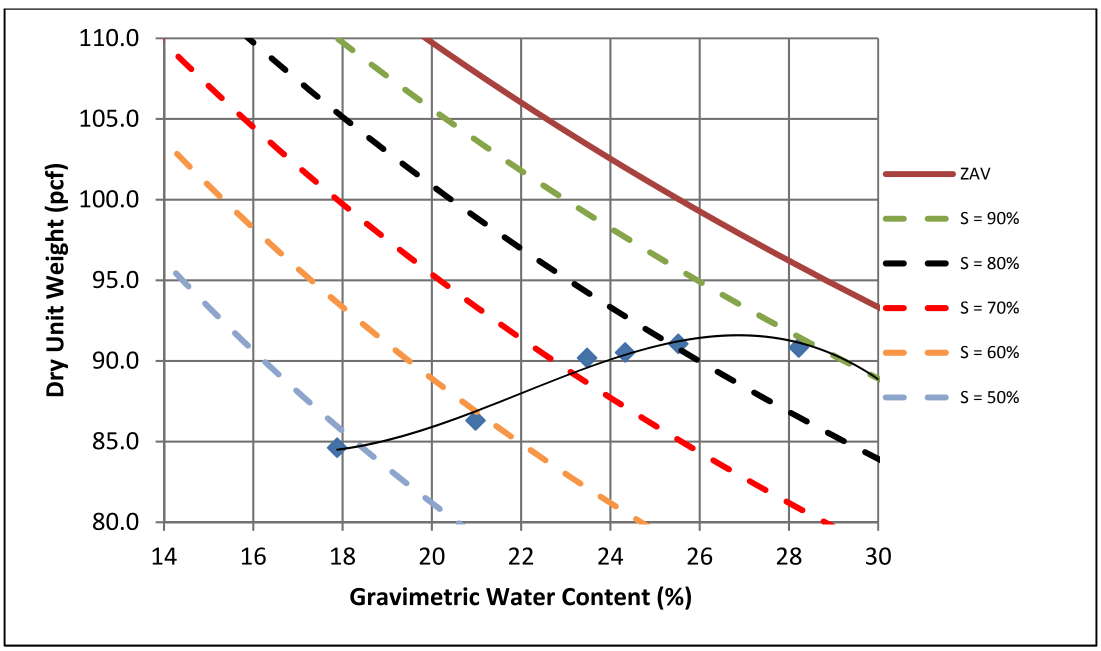


Figure 4.4: Results from Standard Proctor Compaction Tests on Crawford Clay Sample

4.1.2 Site 2: Greenlawn Boulevard and IH 35 [Tinn Clay, TN]

This section summarizes the findings at Site 2. On July 16, 2015, soil samples were collected from the median of Greenlawn Blvd just east of IH 35 in the northern portion of Austin, TX and Travis County. The collected soil samples included a large amount of fines, and belong to the Eagle Ford Formation according to geologic mapping of the area. The portion of Greenlawn Blvd in the area showed slight signs of distress on the edges of the roadway, indicating longitudinal cracking that is traditionally found on roadways built on expansive subgrades. The collected soil samples, shown in Figure 4.5, were extensively tested to identify soil characteristics and swelling properties.

Figure 4.5: Sample of Tinn Clay from Field Sampling

Location and Identification of Soil Samples

The location of Site 2 corresponds to a portion of Greenlawn Blvd just east of IH 35 in north Austin, TX. The soil samples were collected using shovels to remove the topsoil and sample the soil at depth. A soil layer of dark brown to black soil was encountered and sampled to a depth of 6 inches below the ground surface. The GPS coordinates of the borehole locations at Site 2 were marked for soil identification purposes.

The GPS coordinates were input into Google Earth (the resulting image is shown in Figure 4.6), and the USGS geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Eagle Ford Formation. To complement this information, an interactive map from the USDA was used to identify the soil found at the ground surface of Site 2. The information from the USDA soil survey indicates that the soil retrieved from the site is the Tinn Clay, as shown in Figure 4.7.

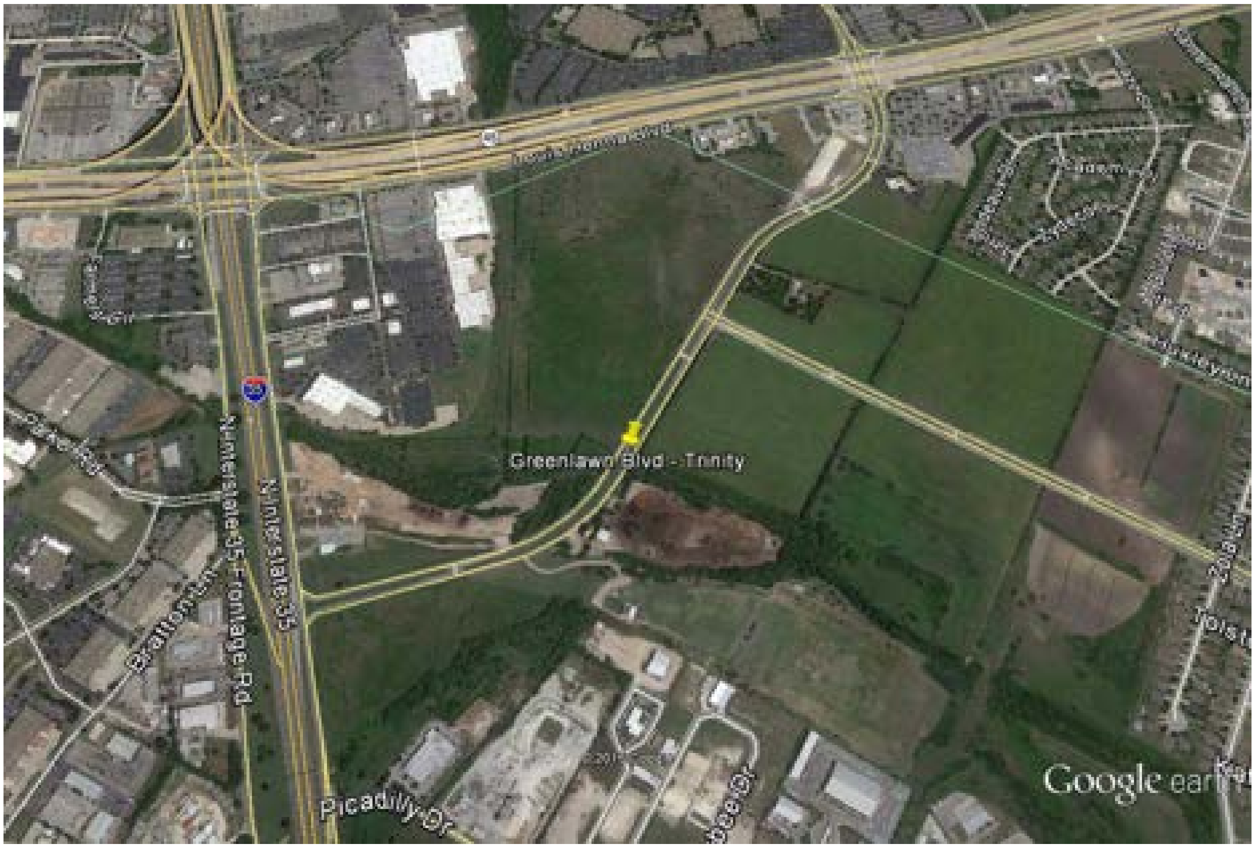


Figure 4.6: Map of Site 2 Location on Greenlawn Blvd (Google 2014)

HnA	Houston Black clay, 0 to 1 percent slopes	1.9	0.3%
HnB	Houston Black clay, 1 to 3 percent slopes	183.2	29.6%
Tw	Tinn clay, 0 to 1 percent slopes, frequently flooded	22.6	3.7%

Figure 4.7: Soil Survey Map and Table at Site 2 (USDA 2013)

<u>Characterization of Tinn Clay Samples [TN]</u>

The Tinn Clay soil was air dried and processed for the soil characterization and centrifuge tests. Atterberg Limits tests (Table 4.2) determined an average liquid limit of 67%, and an average plastic limit of 30%. These results defined the plasticity index as 47%. The GSD curve produced from the Wet Sieve test and Hydrometer is shown in Figure 4.8. The results of the wet sieve analysis showed that the soil was composed of about 4% sand-sized particles; the other 96% was fine-sized particles. The optimum conditions given by the Standard Proctor compaction test were defined by the NAVDAC equations, yielding an optimum moisture content of 24.9% and a maximum dry unit weight of 13.53 kN/m^3 (86 pcf).

Table 4.2: Results from Atterberg Limit Tests on Tinn Clay Samples from Site 2

Test #	1	2	3	4
Predicted Liquid Limit, LL	69%	66%	69%	65%
Selected Liquid Limit, LL	69%	64%	68%	66%
Plastic Limit, PL	24%	33%	28%	35%
Plasticity Index, PI	45%	31%	40%	31%
Averaged Liquid Limit, LL_{avg}	**67%**			
Averaged Plastic Limit, PL_{avg}	**30%**			
Averaged Plasticity Index, PI_{avg}	**37%**			

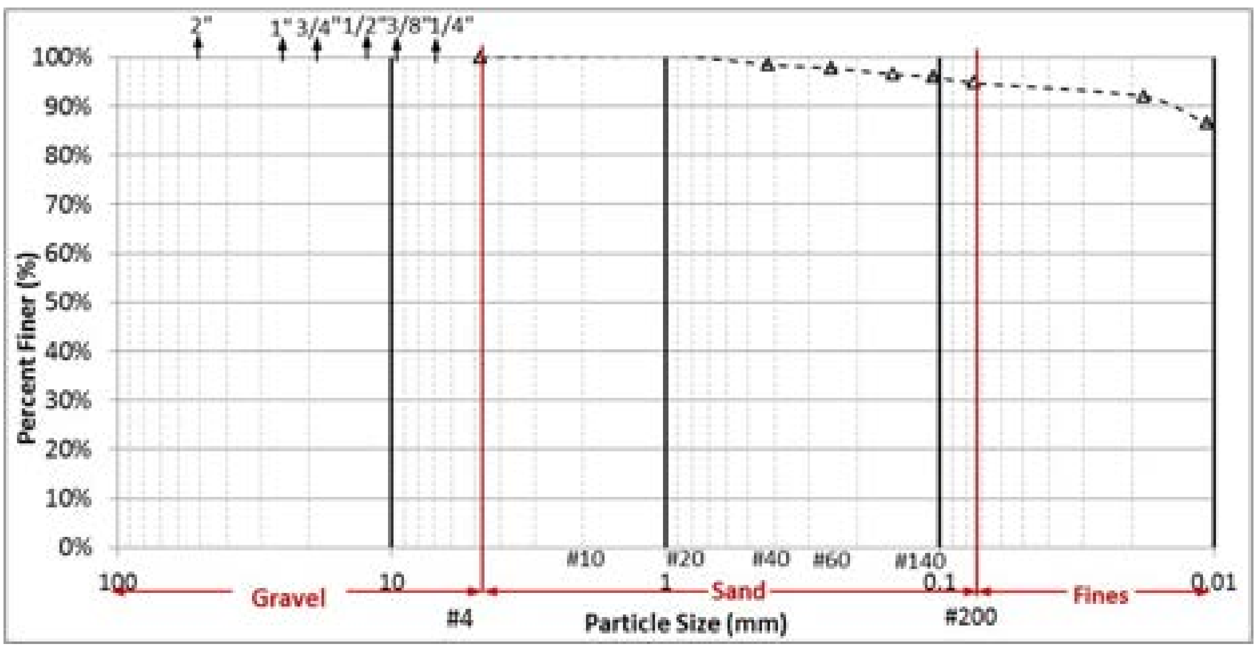


Figure 4.8: Grain Size Distribution Curve for Tinn Clay at Site 2

4.1.3 Sites 3 and 4: Manor Retaining Wall Site [Houston Black Clay, HB – M36 and HB – M127]

This section summarizes the findings at Sites 3 and 4. On August 12, 2015, soil samples were collected from the remnants of the Manor retaining wall site near Manor, TX in Travis County (Figure 4.9). The collected soils consisted of a significant amount of fines and belong to the Navarro and Taylor group. The site was sitting on a known expansive deposit studied previously by CTR (Brown 2012). The collected soil samples were extensively tested to identify soil characteristics and swelling properties. Figure 4.9 depicts the site location and the slope from which the soil was sampled.

Figure 4.9: Manor Retaining Wall Site in August 2015

<u>Location and Identification of Soil Samples</u>

The location of Sites 3 and 4 corresponds to a location near the Manor retaining wall site in Hutto, TX. A soil layer of grayish black, compacted, fat clay with a slight amount of gravel was encountered below the topsoil, with a lighter black soil found further down the slope of the wall. The soil was sampled at two depths, using a shovel to collect samples of the soil due to the inability of the Simco drill to reach the side slope. The site was sampled at two depths; one was 36 inches (3 ft) below the top of the ground surface at the top of the slope, and the other was at 127 inches (approximately 11 ft) below the ground surface at the top of the slope. The GPS coordinates of the borehole locations at Site 7 were marked for soil identification purposes.

The GPS coordinates were input into Google Earth (the resulting image is shown in Figure 4.10), and the USGS geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Taylor and Navarro group. To complement this information, an interactive map from the USDA was used to identify the soil found at the ground surface of Sites 3 and 4. The information from the USDA soil survey indicates that the soil retrieved from the site is the Houston Black Clay, as shown in Figure 4.11, though the characterization of the soil may vary at depth.

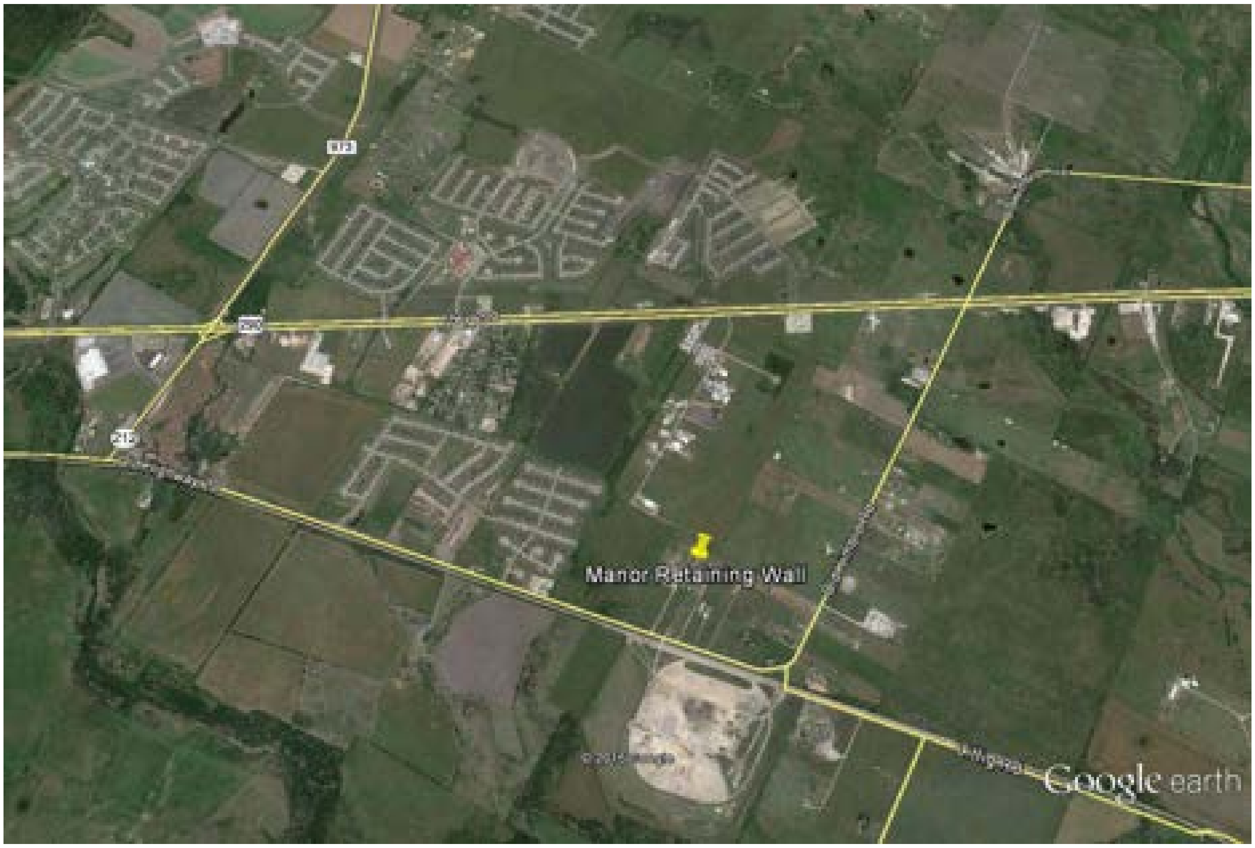


Figure 4.10: Map of Sites 3 and 4 Locations at the Manor Retaining Wall Site (Google 2014)

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
FhF3	Ferris-Heiden complex, 8 to 20 percent slopes, severely eroded	102.3	4.7%
HeC2	Heiden clay, 3 to 5 percent slopes, eroded	51.5	2.4%
HeD2	Heiden clay, 5 to 8 percent slopes, eroded	29.1	1.3%
HnA	Houston Black clay, 0 to 1 percent slopes	370.8	16.9%
HnB	Houston Black clay, 1 to 3 percent slopes	1,354.1	61.8%

Figure 4.11: Soil Survey Map and Table at Sites 3 and 4 (USDA 2013)

<u>Characterization of Houston Black Clay Samples [HM-M36]</u>

The Houston Black clay was air dried and processed for the soil characterization and centrifuge tests. Atterberg Limits tests (Table 4.3) determined an average liquid limit of 52%, and an average plastic limit of 24%. These results defined the plasticity index as 28%. The GSD curve produced from the Wet Sieve and Hydrometer tests is shown in Figure 4.12. The results of the wet sieve analysis showed that the soil was composed of about 32% sand-sized particles; the other 68% was fine-sized particles. The optimum conditions given by the Standard Proctor compaction test were defined by the NAVDAC equations and gave an optimum moisture content of 23.3% and a maximum dry unit weight of 15.09 kN/m^3 (96 pcf).

Table 4.3: Results from Atterberg Limit Tests on Houston Black Sample from Site 3

Test #	1	2	3
Predicted Liquid Limit, LL	50%	53%	54%
Selected Liquid Limit, LL	50%	53%	54%
Plastic Limit, PL	23%	25%	25%
Plasticity Index, PI	27%	28%	29%
Averaged Liquid Limit, LL_{avg}	52%		
Averaged Plastic Limit, PL_{avg}	24%		
Averaged Plasticity Index, PI_{avg}	28%		

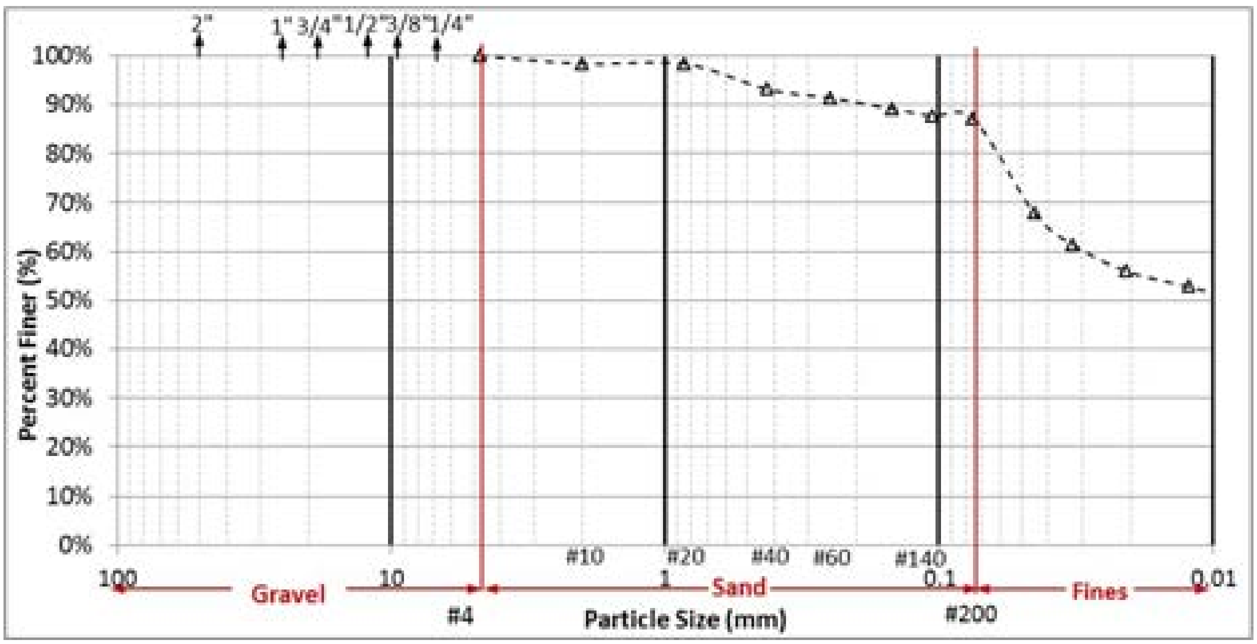


Figure 4.12: Grain Size Distribution Curve for Houston Black Clay at Site 3

<u>Characterization of Houston Black Clay Samples [HM-M127]</u>

The Houston Black clay was air dried and processed for the soil characterization and centrifuge tests. Atterberg Limits tests (Table 4.4) determined an average liquid limit of 55%, and an average plastic limit of 23%. These results defined the plasticity index as 32%. The GSD curve produced from the Wet Sieve test is shown in Figure 4.13. The results of the wet sieve analysis showed that the soil was composed of about 18% sand-sized particles, and the other 82% was fine-sized particles. These results indicate that, while the Atterberg Limits are similar at depth, the soil becomes finer at depth. This result is consistent with the depositional environment, having more fines near the surface due to weathering that would have occurred in the Quaternary Period. The optimum conditions given by the Standard Proctor compaction test were defined by the NAVDAC equations and gave an optimum moisture content of 23.7% and a maximum dry unit weight of 14.89 kN/m^3 (95 pcf).

Table 4.4: Results from Atterberg Limit Tests on Houston Black Sample from Site 4

Test #	1	2	3	4
Predicted Liquid Limit, LL	50%	53%	56%	58%
Selected Liquid Limit, LL	50%	53%	57%	58%
Plastic Limit, PL	23%	23%	23%	23%
Plasticity Index, PI	27%	30%	34%	35%
Averaged Liquid Limit, LL_{avg}	**55%**			
Averaged Plastic Limit, PL_{avg}	**23%**			
Averaged Plasticity Index, PI_{avg}	**32%**			

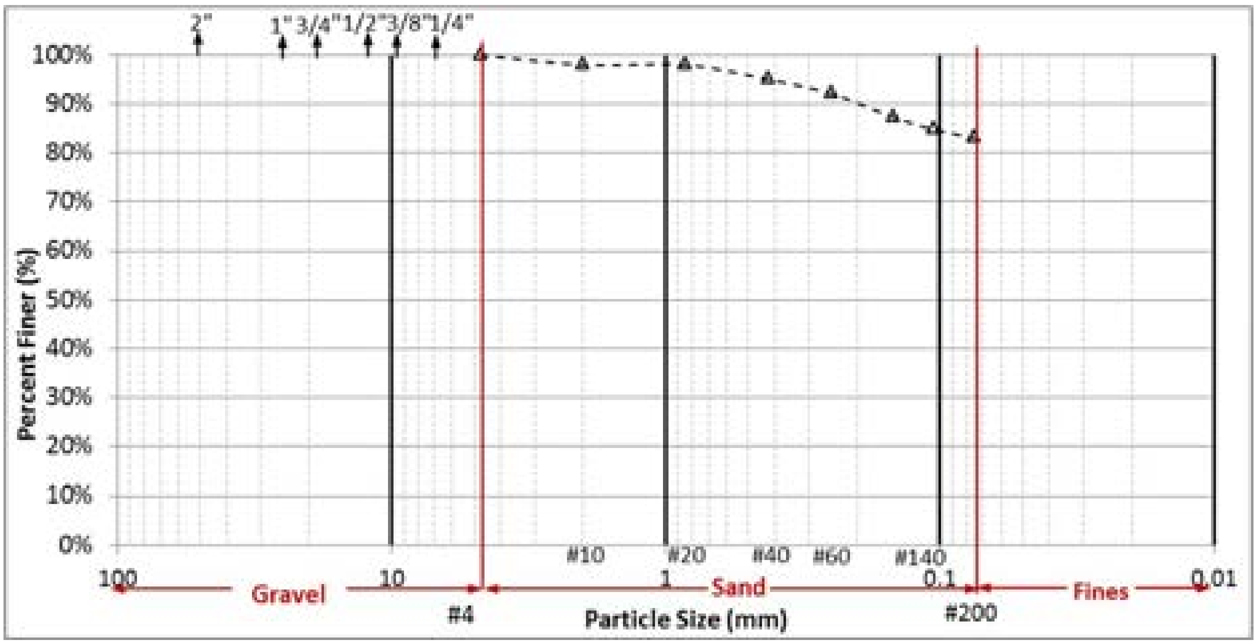


Figure 4.13: Grain Size Distribution Curve for Houston Black Clay at Site 4

4.1.4 Site 5: SH 45 and MoPac Interchange [Fairlie Clay, FR]

This section summarizes the findings at Site 5. On July 16, 2015, soil samples were collected from a strip of the frontage road south of SH 45 near Round Rock, TX in southern Williamson County (Figure 4.14). The collected soil samples included a large amount of fines, and belong to the Del Rio Formation, according to geologic mapping of the area. The portion of frontage road near the site did not indicate significant distress, but the site was previously identified by geologic and agricultural maps, which indicated possible expansive subgrade deposits. The collected soil samples were extensively tested to identify soil characteristics and swelling properties.

Figure 4.14: Sample of Fairlie Clay from Field Sampling

<u>Location and Identification of Soil Samples</u>

The location of Site 5 corresponds to a portion of the frontage road at the intersection of MoPac and SH 45 in southern Williamson County. The soil samples were collected using shovels to remove the topsoil and sample the soil at depth. A soil layer of dark brown to black soil was encountered and sampled to a depth of 6 inches below the ground surface. The GPS coordinates of the borehole locations at Site 5 were marked for soil identification purposes.

The GPS coordinates were input into Google Earth (the resulting image is shown in Figure 4.15), and the USGS geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Del Rio clay formation. To complement this information, an interactive map from the USDA was used to identify the soil found at the ground surface of Site 5. The information from the USDA soil survey indicates that the soil retrieved from the site is the Fairlie Clay, as shown in Figure 4.16.

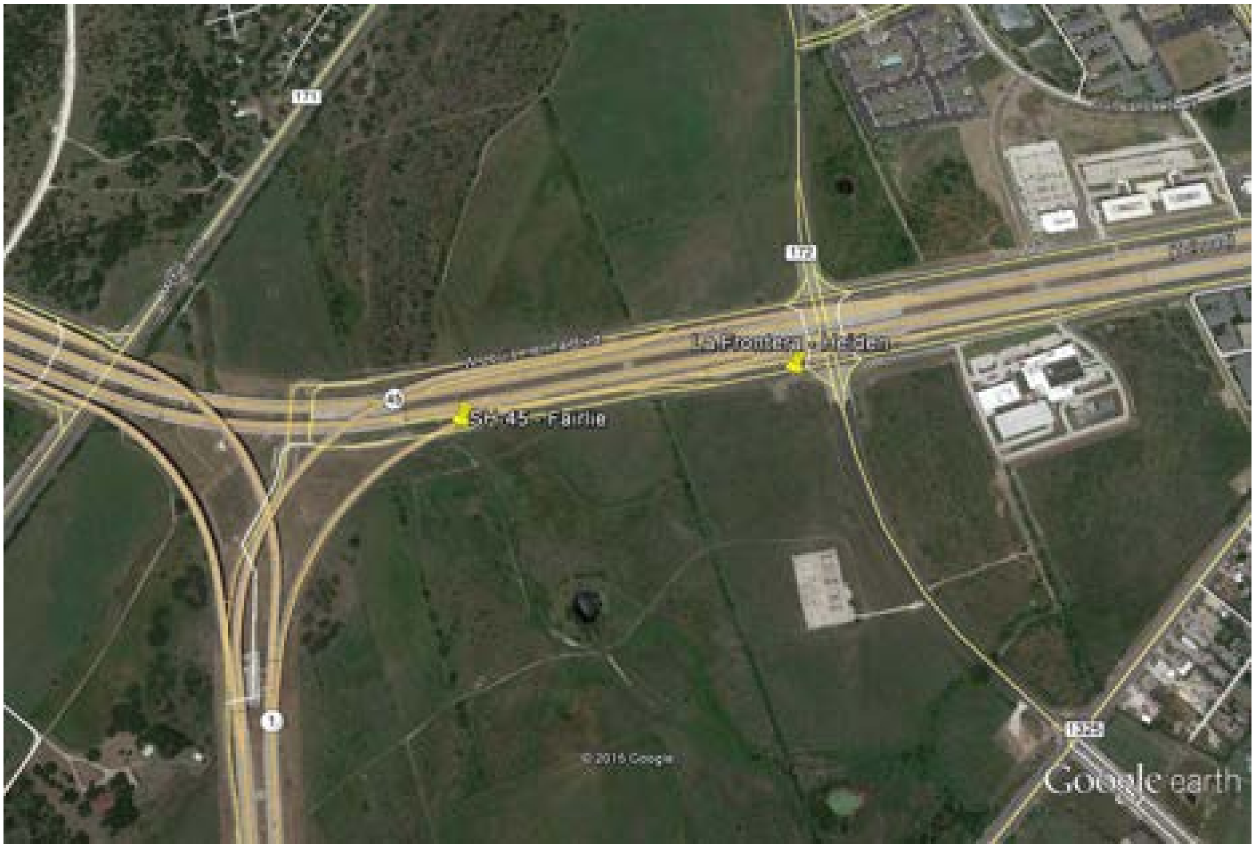


Figure 4.15: Map of Site 5 Location on SH-45 Frontage Road (Google 2014)

FaA	Fairlie clay, 0 to 1 percent slopes	48.4	22.9%
GsB	Georgetown stony clay loam, 1 to 3 percent slopes	1.2	0.6%
HeC2	Heiden clay, 3 to 5 percent slopes, eroded	11.6	5.5%
HeD2	Heiden clay, 5 to 8 percent slopes, eroded	12.9	6.1%

Figure 4.16: Soil Survey Map and Table at Site 5 (USDA 2013)

<u>Characterization of Fairlie Clay Samples [FR]</u>

The Fairlie Clay soil was air dried and processed for soil characterization and centrifuge tests. Atterberg Limits tests (Table 4.5) determined an average liquid limit of 59% and an average plastic limit of 25%. These results defined the plasticity index as 35%. The GSD curve produced from the Wet Sieve test is shown in Figure 4.17. The results of the wet sieve analysis showed that the soil was composed of about 40% sand-sized particles and 60% fine-sized particles. The optimum conditions given by the Standard Proctor compaction test were defined by the NAVDAC equations and gave an optimum moisture content of 25.0% and a maximum dry unit weight of 14.47 kN/m^3 (92 pcf).

Table 4.5: Results from Atterberg Limit Tests on Fairlie Clay Samples from Site 5

Test #	1	2	3
Predicted Liquid Limit, LL	63%	57%	58%
Selected Liquid Limit, LL	63%	57%	58%
Plastic Limit, PL	23%	26%	27%
Plasticity Index, PI	40%	31%	31%
Averaged Liquid Limit, LL_{avg}	**59%**		
Averaged Plastic Limit, PL_{avg}	**25%**		
Averaged Plasticity Index, PI_{avg}	**34%**		

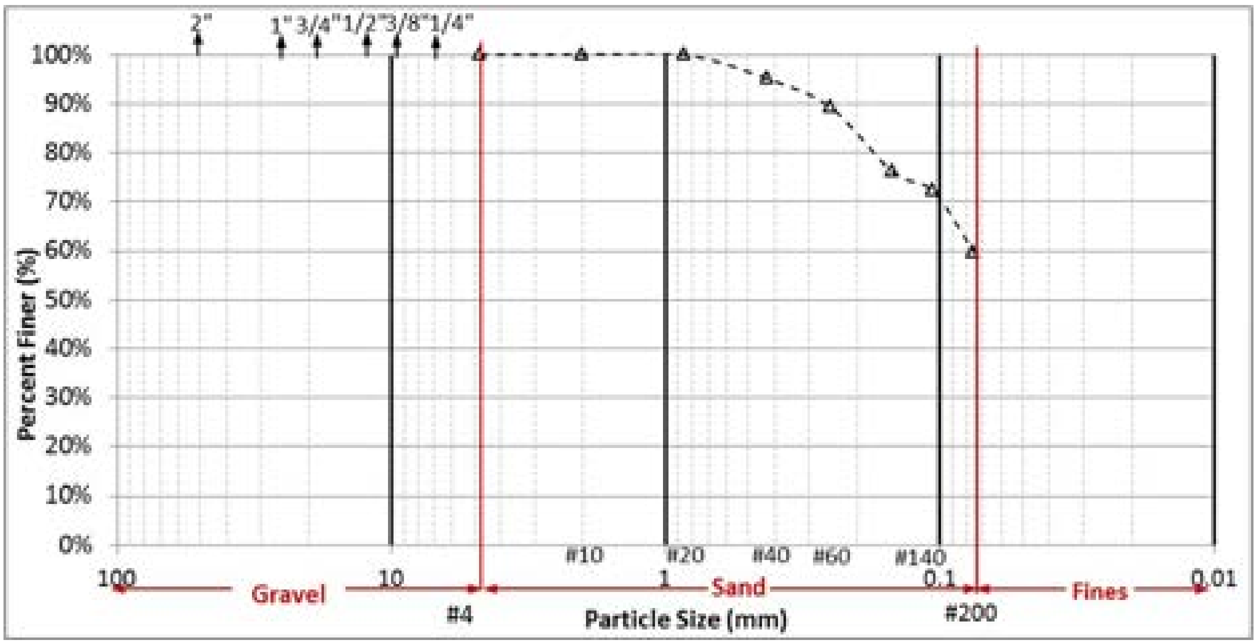


Figure 4.17: Grain Size Distribution Curve for Fairlie Clay at Site 5

4.1.5 Site 6: SH 45 and La Frontera Boulevard [Heiden Clay, HE-LF]

This section summarizes the findings at Site 6. On July 16, 2015, soil samples were collected from a strip of the frontage road south of SH 45 in Williamson County (Figure 4.18). The collected soil samples included a large amount of fines, and belong to the Del Rio Formation, according to geologic mapping of the area. The portion of frontage road near the site did not indicate significant distress, but the site was previously identified by geologic and agricultural maps, which indicated possible expansive subgrade deposits. The collected soil samples were extensively tested to identify soil characteristics and swelling properties.

Figure 4.18: Cracking of Heiden Clay near La Frontera Blvd

Location and Identification of Soil Samples

The location of Site 6 corresponds to a portion of the frontage road at the intersection of the frontage road of SH 45 and La Frontera Blvd near Round Rock, TX in southern Williamson County. The soil samples were collected using shovels to remove the topsoil and sample the soil at depth. A soil layer of tan soil was encountered and sampled to a depth of 3 inches below the ground surface; a nearby drainage ditch prevented deeper extraction. The GPS coordinates of the borehole locations at Site 6 were marked for soil identification purposes.

The GPS coordinates were input into Google Earth (the resulting image is shown in Figure 4.19), and the USGS geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Del Rio clay formation. To complement this information, an interactive map from the USDA was used to identify the soil found at the ground surface of Site 6. The information from the USDA soil survey indicates that the soil retrieved from the site is the Heiden Clay, as shown in Figure 4.20.

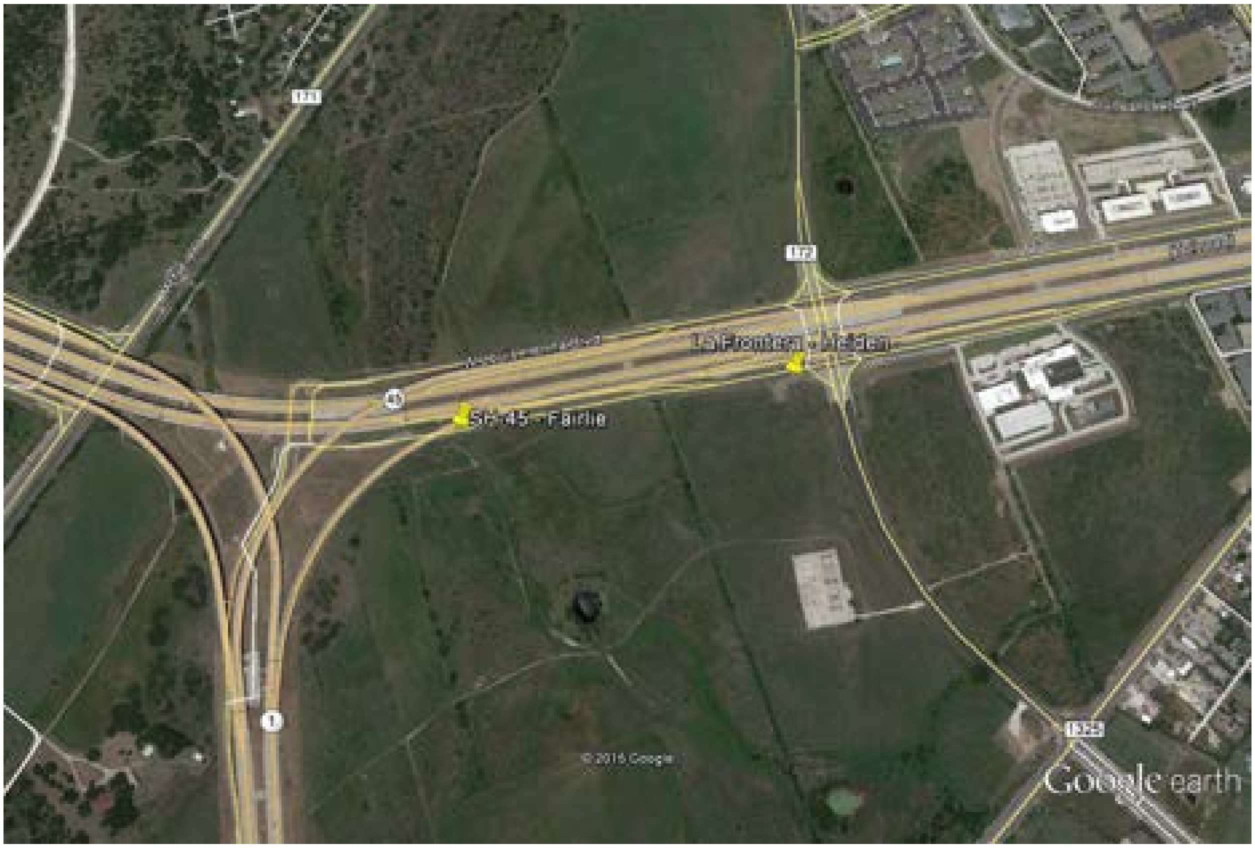


Figure 4.19: Map of Site 6 Location near La Frontera Blvd (Google 2014)

FaA	Fairlie clay, 0 to 1 percent slopes	48.4	22.9%
GsB	Georgetown stony clay loam, 1 to 3 percent slopes	1.2	0.6%
HeC2	Heiden clay, 3 to 5 percent slopes, eroded	11.6	5.5%
HeD2	Heiden clay, 5 to 8 percent slopes, eroded	12.9	6.1%

Figure 4.20: Soil Survey Map and Table at Site 6 (USDA 2013)

<u>Characterization of Heiden Clay Sample [HE-LF]</u>

The Heiden Clay soil was air dried and processed for the soil characterization and centrifuge tests. Atterberg Limits tests (Table 4.6) determined an average liquid limit of 55% and an average plastic limit of 21%. These results defined the plasticity index as 34%. The GSD curve produced from the Wet Sieve test is shown in Figure 4.21. The results of the wet sieve analysis showed that the soil was composed of about 15% sand-sized particles and 85% fine-sized particles. The optimum conditions given by the Standard Proctor compaction test were defined by the NAVDAC equations and gave an optimum moisture content of 25.4% and a maximum dry unit weight of 14.90 kN/m^3 (95 pcf).

Table 4.6: Results from Atterberg Limit Tests on Heiden Samples from Site 6

Test #	1	2	3
Predicted Liquid Limit, LL	63%	56%	56%
Selected Liquid Limit, LL	63%	55%	55%
Plastic Limit, PL	21%	21%	21%
Plasticity Index, PI	42%	34%	34%
Averaged Liquid Limit, LL_{avg}	**55%**		
Averaged Plastic Limit, PL_{avg}	**21%**		
Averaged Plasticity Index, PI_{avg}	**34%**		

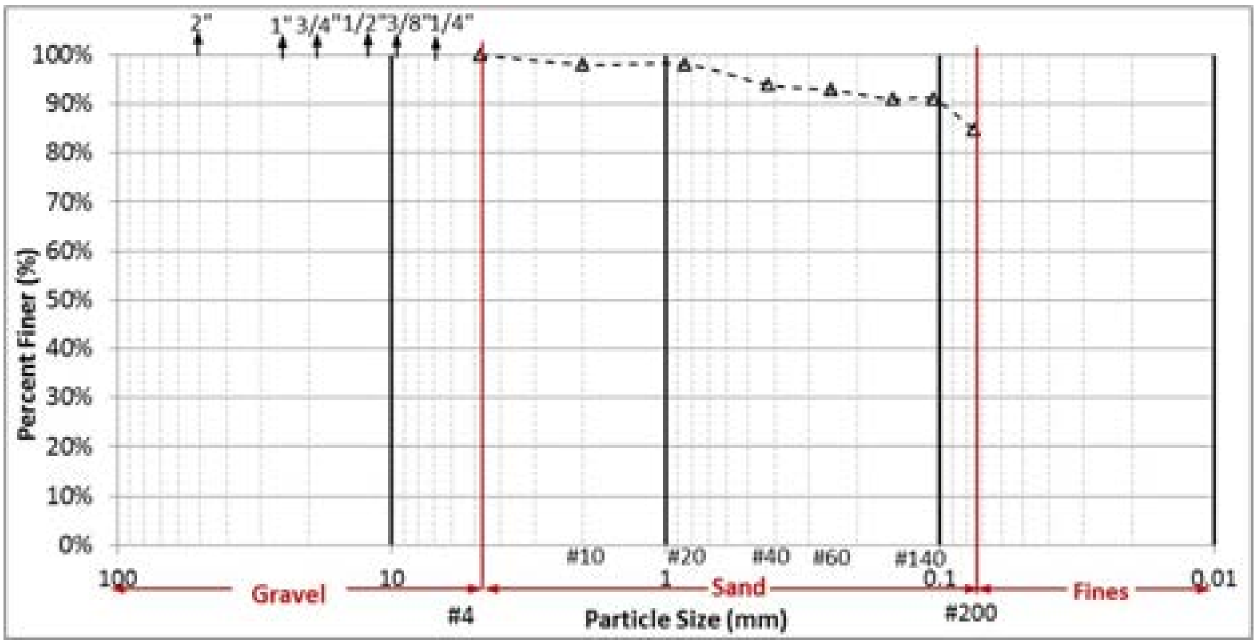


Figure 4.21: Grain Size Distribution Curve for Heiden Clay at Site 6

4.1.6 Site 7: FM 971 [Houston Black Clay, HB – 971]

This section summarizes the findings at Site 7. On August 12, 2015, soil samples were collected from the north side of FM 971 just east of Wier, TX in Williamson County (Figure 4.22). The collected soil samples included a large amount of fines, and belong to the Austin Chalk Formation, according to geologic mapping of the area. The side of the roadway showed extensive longitudinal cracking, indicative of an expansive subgrade. The collected soil samples were extensively tested to identify soil characteristics and swelling properties.

Figure 4.22: Augured Hole and Soil Samples from FM 971

<u>Location and Identification of Soil Samples</u>

The location of Site 7 corresponds to a site on the north side of FM 971 just west of the intersection of FM 154 and FM 971 near Wier, TX. The soil samples were collected using a trailer-mounted Simco 250 PTC auger to bore through the topsoil and reach the subgrade. A soil layer of grayish black, compacted, fat clay with a slight amount of gravel was encountered below the topsoil. This soil was identified as our target soil for Site 7. One borehole was drilled to a depth of 3 feet, and two 5-gallon buckets of soil samples were collected for further testing. The GPS coordinates of the borehole locations at Site 7 were marked for soil identification.

The GPS coordinates were input into Google Earth (the resulting image is shown in Figure 4.23), and the USGS geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Austin Chalk Formation. To complement this information, an interactive map from the USDA was used to identify the soil found at the ground surface of Site 7, as shown in Figure 4.24. The information from the USDA soil survey indicates that the soil retrieved from the site is the Houston Black Clay.

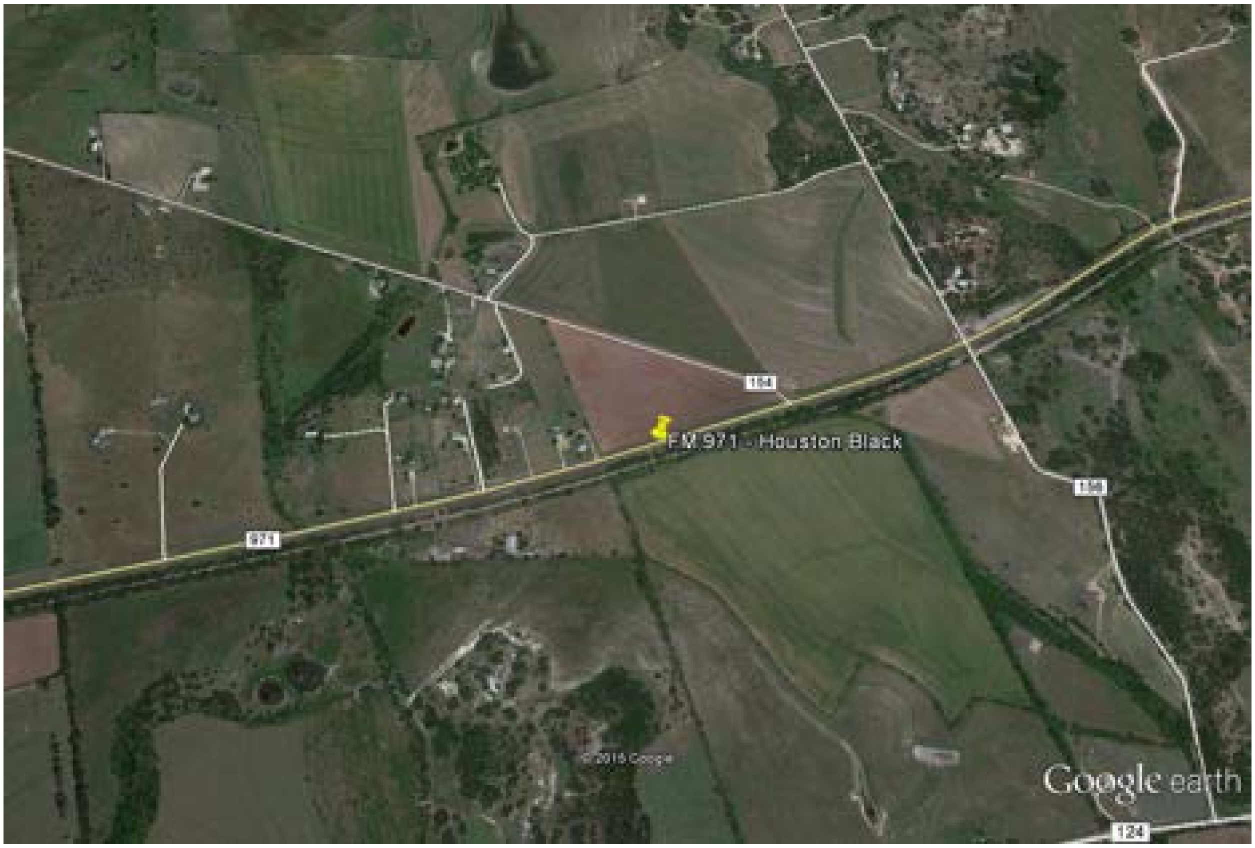


Figure 4.23: Map of Site 7 Location on FM 971 (Google 2014)

EyD	Eddy very gravelly clay loam, 3 to 8 percent slopes	30.6	8.1%
HuA	Houston Black clay, 0 to 1 percent slopes	93.1	24.6%
HuB	Houston Black clay, 1 to 3 percent slopes	131.0	34.6%

Figure 4.24: Soil Survey Map and Table at Site 7 (USDA 2013)

Characterization of Houston Black Samples [HB – 971]

The Houston Black Clay soil was air dried and processed for the soil characterization and centrifuge tests. Atterberg Limits tests (Table 4.7) determined an average liquid limit of 72%, and an average plastic limit of 25%. These results defined the plasticity index as 47%. The GSD curve produced from the Wet Sieve test is shown in Figure 4.25. The results of the wet sieve analysis showed that the soil was composed of about 14% sand-sized particles and 86% fine-sized particles. The optimum conditions given by the Standard Proctor compaction test were defined by the

NAVDAC equations and gave an optimum moisture content of 25.0% and a maximum dry unit weight of 13.33 kN/m^3 (85 pcf).

Table 4.7: Results from Atterberg Limit Tests on Houston Black Samples from Site 7

Test #	1	2	3
Predicted Liquid Limit, LL	70%	73%	72%
Selected Liquid Limit, LL	71%	73%	72%
Plastic Limit, PL	25%	26%	26%
Plasticity Index, PI	38%	37%	37%
Averaged Liquid Limit, LL_{avg}	**72%**		
Averaged Plastic Limit, PL_{avg}	**25%**		
Averaged Plasticity Index, PI_{avg}	**47%**		

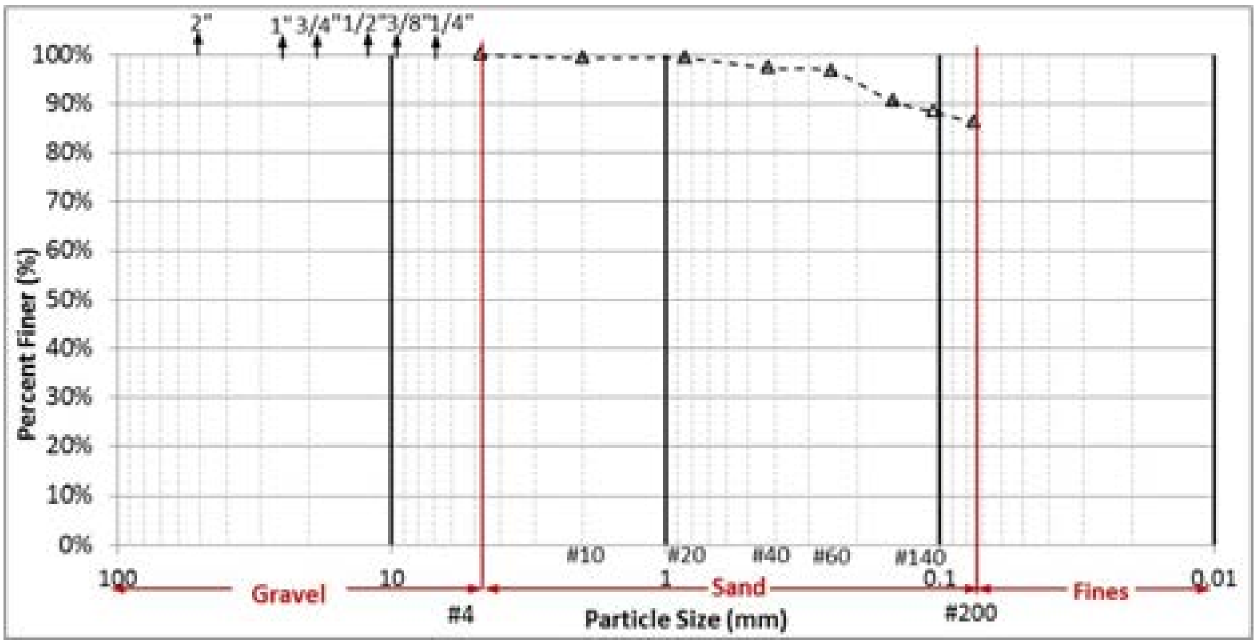


Figure 4.25: Grain Size Distribution Curve for Houston Black Clay at Site 7

4.1.7 Site 8: FM 972 [Branyon Clay, BR – 972]

This section summarizes the findings at Site 8. On August 12, 2015, soil samples were collected from the north side of FM 972 just north of Granger, TX in Williamson County (Figure 4.26). The collected soil samples included a large amount of fines, and belong to the Taylor and Navarro group, according to geologic mapping of the area. The side of the roadway showed extensive longitudinal cracking, indicative of an expansive subgrade. The collected soil samples were extensively tested to identify soil characteristics and swelling properties.

Figure 4.26: Augured Hole and Soil Samples from FM 972

Location and Identification of Soil Samples

The location of Site 8 corresponds to a site on the north side of FM 972 just west of the intersection of FM 972 and SH 45 near Granger, TX. The soil samples were collected using a trailer-mounted Simco 250 PTC auger to bore through the topsoil and reach the subgrade. A soil layer of grayish black, compacted, fat clay with a slight amount of gravel was encountered below the topsoil. This soil was identified as our target soil for Site 8. One borehole was drilled to a depth of 3 feet, and two 5-gallon buckets of soil samples were collected for further testing. The GPS coordinates of the borehole locations at Site 8 were marked for soil identification purposes.

The GPS coordinates were input into Google Earth (the resulting image is shown in Figure 4.27), and the USGS geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Navarro and Taylor group. To complement this information, an interactive map from the USDA was used to identify the soil found at the ground surface of Site 8. The information from the USDA soil survey indicates that the soil retrieved from the site is the Branyon Clay, as shown in Figure 4.28.

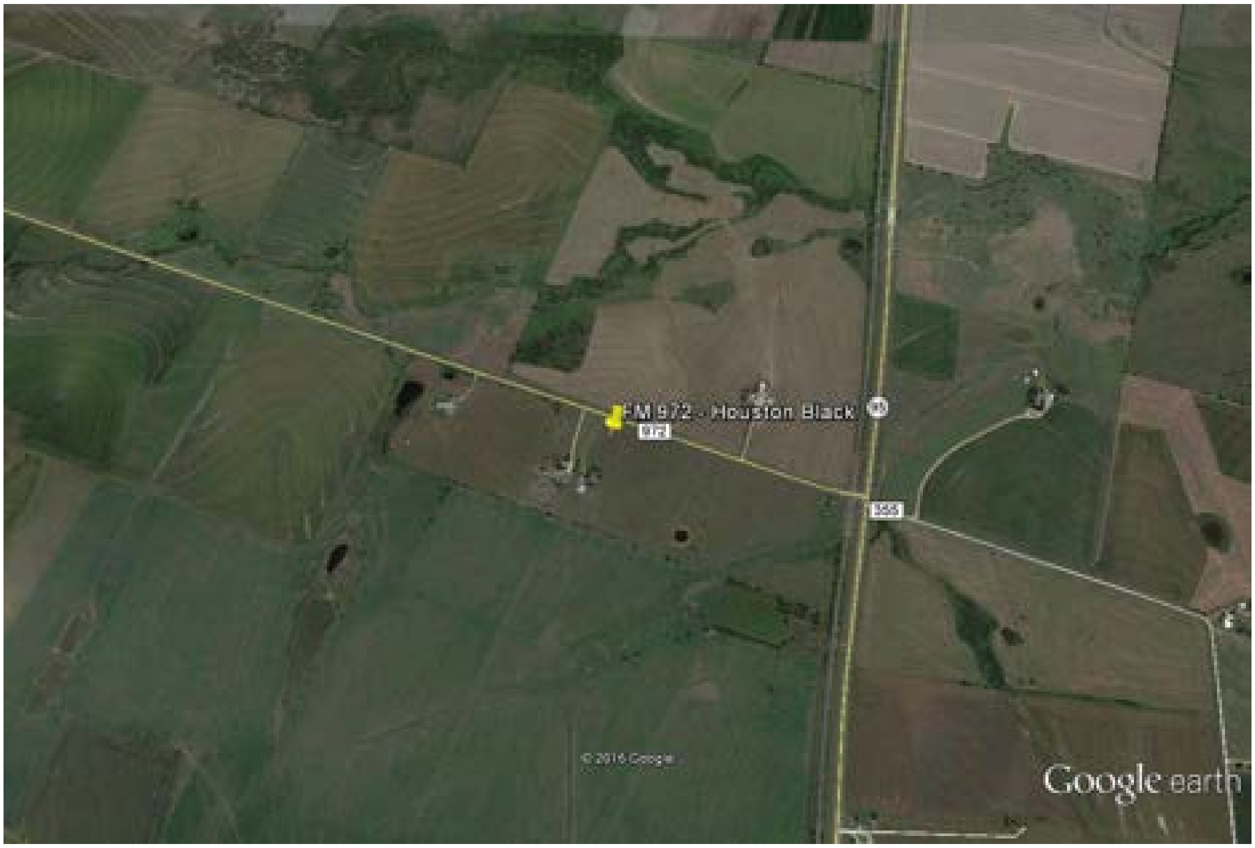
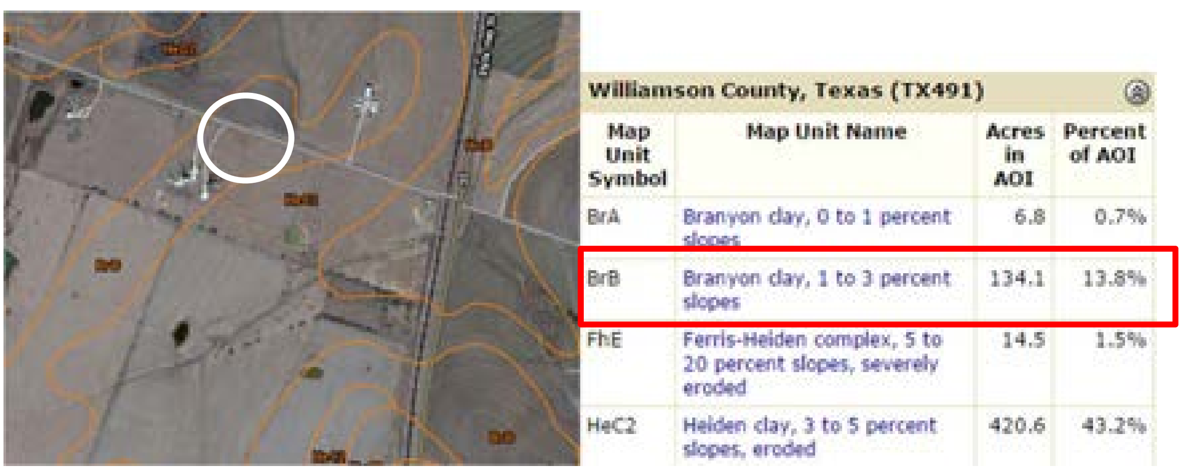


Figure 4.27: Map of Site 8 Location on FM 972 (Google 2014)



Williamson County, Texas (TX491)			
Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
BrA	Branyon clay, 0 to 1 percent slopes	6.8	0.7%
BrB	Branyon clay, 1 to 3 percent slopes	134.1	13.8%
FhE	Ferris-Heiden complex, 5 to 20 percent slopes, severely eroded	14.5	1.5%
HeC2	Heiden clay, 3 to 5 percent slopes, eroded	420.6	43.2%

Figure 4.28: Soil Survey Map and Table at Site 8 (USDA 2013)

<u>Characterization of Branyon Clay Samples [BR – 972]</u>

The Branyon Clay soil was air dried and processed for the soil characterization and centrifuge tests. Atterberg Limits tests (Table 4.8) determined an average liquid limit of 65% and an average plastic limit of 25%. These results defined the plasticity index as 40%. The GSD curve produced from the Wet Sieve test is shown in Figure 4.29. The results of the wet sieve analysis showed that the soil was composed of about 56% sand-sized particles and 44% fine-sized particles. The optimum conditions given by the Standard Proctor compaction test were defined by the

NAVDAC equations and gave an optimum moisture content of 23.7% and a maximum dry unit weight of 13.80 kN/m^3 (88 pcf).

Table 4.8: Results from Atterberg Limit Tests on Branyon Samples from Site 8

Test #	1	2	3
Predicted Liquid Limit, LL	65%	66%	67%
Selected Liquid Limit, LL	65%	65%	66%
Plastic Limit, PL	25%	25%	26%
Plasticity Index, PI	40%	40%	40%
Averaged Liquid Limit, LL_{avg}	**65%**		
Averaged Plastic Limit, PL_{avg}	**25%**		
Averaged Plasticity Index, PI_{avg}	**40%**		

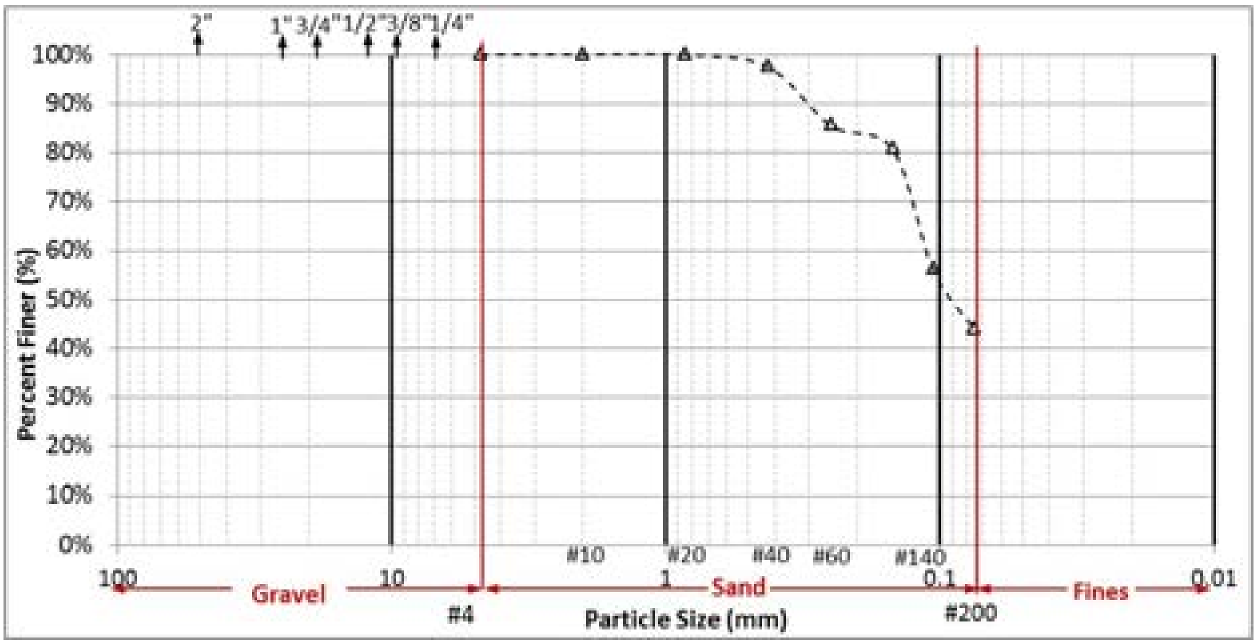


Figure 4.29: Grain Size Distribution Curve for Branyon Clay at Site 8

4.1.8 Site 9: SH 95 [Branyon Clay, BR – 95]

This section summarizes the findings at Site 9. On August 12, 2015, soil samples were collected from the west side of SH 95 just south of Granger, TX in Williamson County (Figure 4.30). The collected soil samples included a large amount of fines, and belong to the Taylor and Navarro group, according to geologic mapping of the area. The side of the roadway showed extensive longitudinal cracking, indicative of an expansive subgrade. The collected soil samples were extensively tested to identify soil characteristics and swelling properties.


Figure 4.30: Augured Hole and Soil Samples from SH 95

<u>Location and Identification of Soil Samples</u>

The location of Site 9 corresponds to a site on the west side of SH 95 approximately half a mile south of the intersection of FM 971 and SH 95 in Grange, TX. The soil samples were collected using a trailer-mounted Simco 250 PTC auger to bore through the topsoil and reach the subgrade. A soil layer of grayish black, compacted, fat clay with a slight amount of gravel was encountered below the topsoil. This soil was identified as our target soil for Site 9. One borehole was drilled to a depth of 3 feet, and two 5-gallon buckets of soil samples were collected for further testing. The GPS coordinates of the borehole locations at Site 9 were marked for soil identification purposes.

The GPS coordinates were input into Google Earth (the resulting image is shown in Figure 4.31), and the USGS geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Navarro and Taylor group. To complement this information, an interactive map from the USDA was used to identify the soil found at the ground surface of Site 9. The information from the USDA soil survey indicates that the soil retrieved from the site is the Branyon Clay as sown in Figure 4.32.

Figure 4.31: Map of Site 9 Location on SH 95 (Google 2014)

Williamson County, Texas (TX491)			
Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
BrA	Branyon clay, 0 to 1 percent slopes	238.8	70.7%
BrB	Branyon clay, 1 to 3 percent slopes	28.8	8.5%
KsA	Krum silty clay, 0 to 1 percent slopes	11.6	3.4%
KsB	Krum silty clay, 1 to 3 percent slopes	38.6	11.4%
Of	Oakalla silty clay loam, 0 to 2 percent slopes, frequently flooded	0.1	0.0%
Tn	Tinn clay, 0 to 1 percent slopes, frequently flooded	19.7	5.8%
Totals for Area of Interest		337.6	100.0%

Figure 4.32: Soil Survey Map and Table at Site 9 (USDA 2013)

Characterization of Branyon Clay Samples [BR – 95]

The Branyon Clay soil was air dried and processed for the soil characterization and centrifuge tests. Atterberg Limits tests (Table 4.9) determined an average liquid limit of 60%, and an average plastic limit of 34%. These results defined the plasticity index as 26%. The GSD curve produced from the Wet Sieve and Hydrometer tests is shown in Figure 4.33. The results of the wet sieve analysis showed that the soil was composed of about 8% sand-sized particles; the other 92% was fine-sized particles. The optimum conditions given by the Standard Proctor compaction test were defined by the NAVDAC equations and gave an optimum moisture content of 22.2% and a maximum dry unit weight of 14.24 kN/m^3 (91 pcf).

Table 4.9: Results from Atterberg Limit Tests on Branyon Samples from Site 9

Test #	1	2
Predicted Liquid Limit, LL	62%	57%
Selected Liquid Limit, LL	62%	57%
Plastic Limit, PL	34%	34%
Plasticity Index, PI	28%	23%
Averaged Liquid Limit, LL_{avg}	**60%**	
Averaged Plastic Limit, PL_{avg}	**34%**	
Averaged Plasticity Index, PI_{avg}	**26%**	

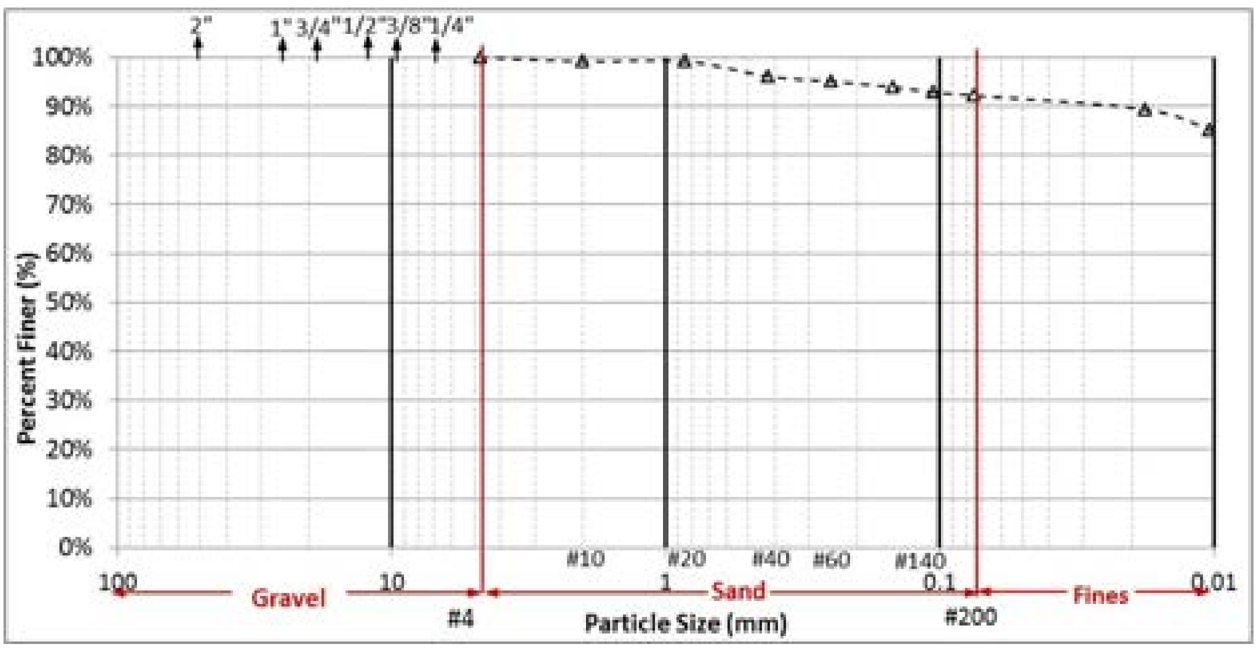


Figure 4.33: Grain Size Distribution Curve for Branyon Clay at Site 9

4.1.9 Site 10: TxDOT Maintenance Office in Taylor, TX [Houston Black Clay, HB – Taylor]

This section summarizes the findings at Site 10. On August 12, 2015, soil samples were collected from the southwest corner of the maintenance yard at the TxDOT office in Taylor, TX in Williamson County, as shown in Figure 4.34. The collected soil samples included a large amount of fines, and belong to the Pecan Gap Chalk, according to geologic mapping of the area. The office was known to sit on an expansive deposit due to cracking in the office floor as well as geologic and soil surveys of the area. The collected soil samples were extensively tested to identify soil characteristics and swelling properties.

Figure 4.34: Augured Hole and Soil Samples from Taylor Maintenance Office

<u>Location and Identification of Soil Samples</u>

The location of Site 10 corresponds to a site on the southern corner of the yard at the maintenance office in Taylor, TX. The soil samples were collected using a trailer-mounted Simco 250 PTC auger to bore through the topsoil and reach the subgrade. A soil layer of grayish black, compacted, fat clay with a slight amount of gravel was encountered below the topsoil. This soil was identified as our target soil for Site 10. One borehole was drilled to a depth of 3 feet, and two 5-gallon buckets of soil samples were collected for further testing. The GPS coordinates of the borehole locations at Site 10 were marked for soil identification purposes.

The GPS coordinates were input into Google Earth (the resulting image is shown in Figure 4.35), and the USGS geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Pecan Gap Chalk. To complement this information, an interactive map from the USDA was used to identify the soil found at the ground surface of Site 10. The information from the USDA soil survey indicates that the soil retrieved from the site is the Houston Black Clay, as shown in Figure 4.36.

Figure 4.35: Map of Site 10 Location at the TxDOT Maintenance Office in Taylor, TX (Google 2014)

BrA	Branyon clay, 0 to 1 percent slopes	2.0	0.7%
BrB	Branyon clay, 1 to 3 percent slopes	18.8	6.9%
FhE	Ferris-Heiden complex, 5 to 20 percent slopes, severely eroded	41.0	14.9%
HeD2	Heiden clay, 5 to 8 percent slopes, eroded	26.5	9.6%
HuB	Houston Black clay, 1 to 3 percent slopes	9.4	3.4%
HuC2	Houston Black clay, 3 to 5 percent slopes, moderately eroded	39.6	14.4%

Figure 4.36: Soil Survey Map and Table at Site 10 (USDA 2013)

<u>Characterization of Houston Black Clay Samples [HB - Taylor]</u>

The Houston Black clay was air dried and processed for the soil characterization and centrifuge tests. Atterberg Limits tests (Table 4.10) determined an average liquid limit of 55%, and an average plastic limit of 23%. These results defined the plasticity index as 32%. The GSD curve produced from the Wet Sieve and Hydrometer tests is shown in Figure 4.37. The results of the wet sieve analysis showed that the soil was composed of about 7% sand-sized particles; the other 93% was fine-sized particles. The optimum conditions given by the Standard Proctor compaction test were defined by the NAVDAC equations and gave an optimum moisture content of 23.7% and a maximum dry unit weight of 14.89 kN/m^3 (95 pcf).

Table 4.10: Results from Atterberg Limit Tests on Houston Black Samples from Site 10

Test #	1	2	3	4
Predicted Liquid Limit, LL	54%	54%	55%	59%
Selected Liquid Limit, LL	54%	54%	55%	57%
Plastic Limit, PL	23%	27%	26%	28%
Plasticity Index, PI	31%	28%	29%	29%
Averaged Liquid Limit, LL_{avg}	**55%**			
Averaged Plastic Limit, PL_{avg}	**23%**			
Averaged Plasticity Index, PI_{avg}	**32%**			

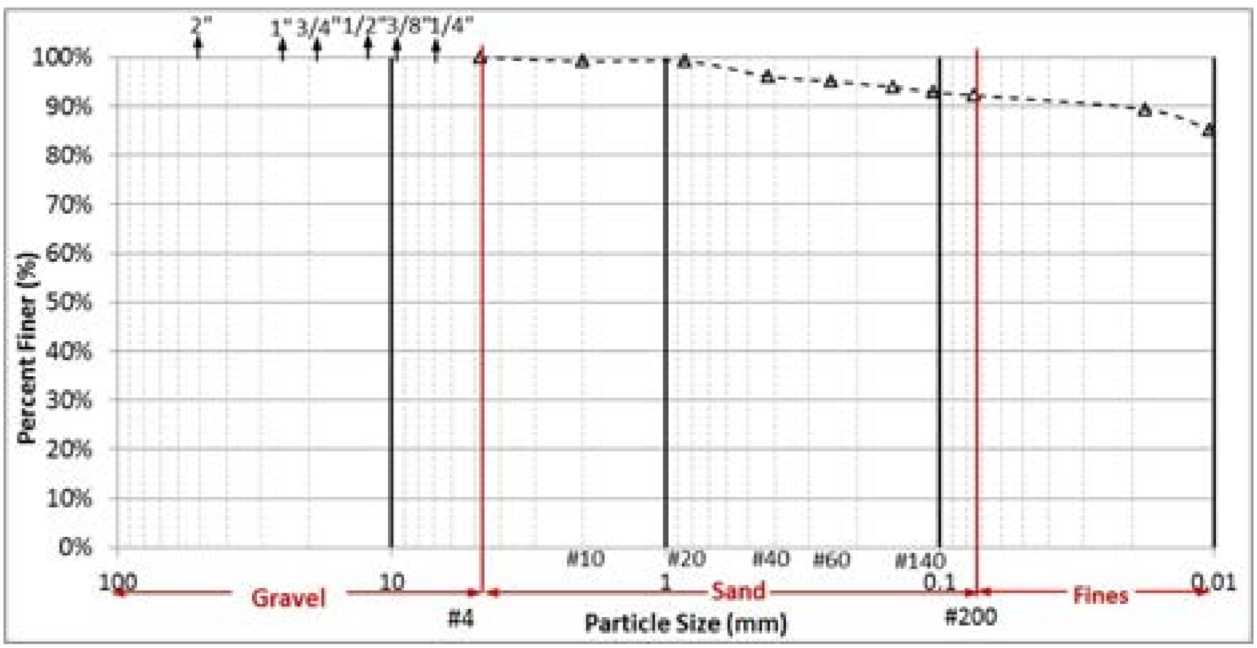


Figure 4.37: Grain Size Distribution Curve for Houston Black Clay at Site 10

4.1.10 Site 11: FM 535 [Behring Clay, BH – 535]

This section summarizes the findings at Site 11, shown in Figure 4.38. On August 13, 2015, soil samples were collected from the west side of FM 535 just northwest of Red Rock, TX in Bastrop County. The collected soil samples included a large amount of fines, and belong to the Wilcox group according to geologic mapping of the area. The side of the roadway showed extensive longitudinal cracking, indicative of an expansive subgrade. The collected soil samples were extensively tested to identify soil characteristics and swelling properties.

Figure 4.38: Augured Hole from FM 535

Location and Identification of Soil Samples

The location of Site 11 corresponds to a site on the west side FM 535 near Red Rock, TX just northwest of the intersection of FM 535 and FM 20. The soil samples were collected using a trailer-mounted Simco 250 PTC auger to bore through the topsoil and reach the subgrade. A soil layer of brown to tan, compacted, fat clay with a slight amount of gravel was encountered below the topsoil. This soil was identified as our target soil for Site 11. One borehole was drilled to a depth of 3 feet, and two 5-gallon buckets of soil samples were collected for further testing. The GPS coordinates of the borehole locations at Site 11 were marked for soil identification purposes.

The GPS coordinates were input into Google Earth (the resulting image is shown in Figure 4.39), and the USGS geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Wilcox group. To complement this information, an interactive map from the USDA was used to identify the soil found at the ground surface of Site 11. The information from the USDA soil survey indicates that the soil retrieved from the site is the Behring clay loam, as shown in Figure 4.40.

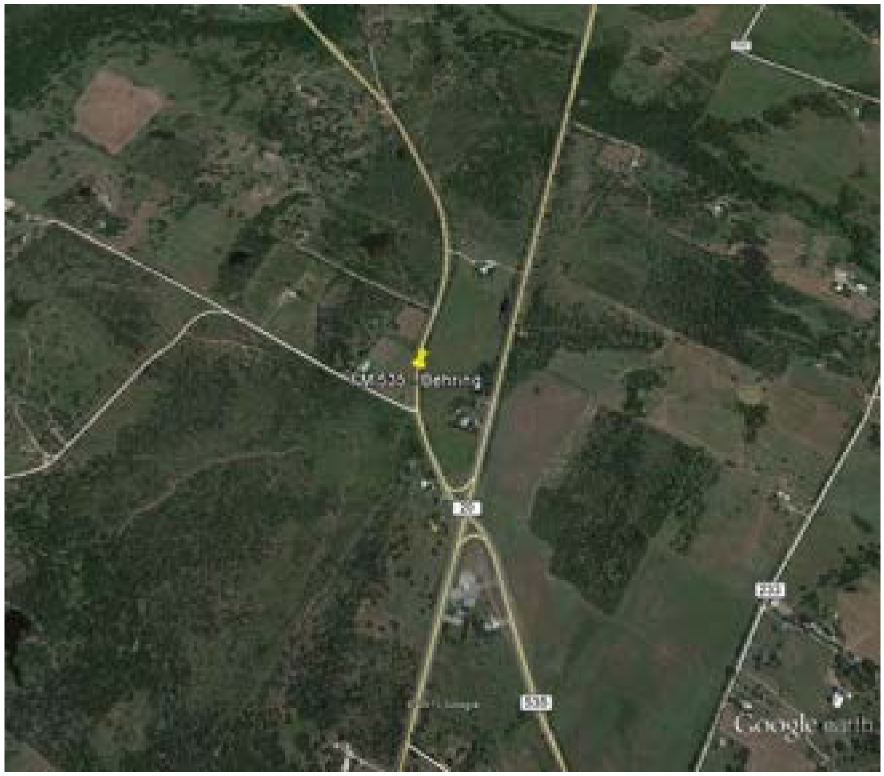


Figure 4.39: Map of Site 11 Location on FM 535 (Google 2014)

Bastrop County, Texas (TX021)			
Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
BeB	Behring clay loam, 1 to 3 percent slopes	22.2	20.7%
BeC2	Behring clay loam, 3 to 5 percent slopes, eroded	22.3	20.8%
BeD2	Behring clay loam, 5 to 8 percent slopes, eroded	5.9	5.5%

Figure 4.40: Soil Survey Map and Table at Site 11 (USDA 2013)

Characterization of Behring Clay Samples [BH - 535]

The Behring soil was air dried and processed for the soil characterization and centrifuge tests. Atterberg Limits tests (Table 4.11) determined an average liquid limit of 53%, and an average plastic limit of 21%. These results defined the plasticity index as 32%. The GSD curve produced from the Wet Sieve and Hydrometer tests is shown in Figure 4.41. The results of the wet sieve analysis showed that the soil was composed of about 22% sand-sized particles and 78% fine-sized particles. The optimum conditions given by the Standard Proctor compaction test were defined by

the NAVDAC equations and gave an optimum moisture content of 23.7% and a maximum dry unit weight of 14.89 kN/m^3 (95 pcf).

Table 4.11: Results from Atterberg Limit Tests on Behring Samples from Site 11

Test #	1	2	3
Predicted Liquid Limit, LL	53%	52%	53%
Selected Liquid Limit, LL	53%	52%	53%
Plastic Limit, PL	23%	19%	21%
Plasticity Index, PI	30%	34%	32%
Averaged Liquid Limit, LL_{avg}	**53%**		
Averaged Plastic Limit, PL_{avg}	**21%**		
Averaged Plasticity Index, PI_{avg}	**32%**		

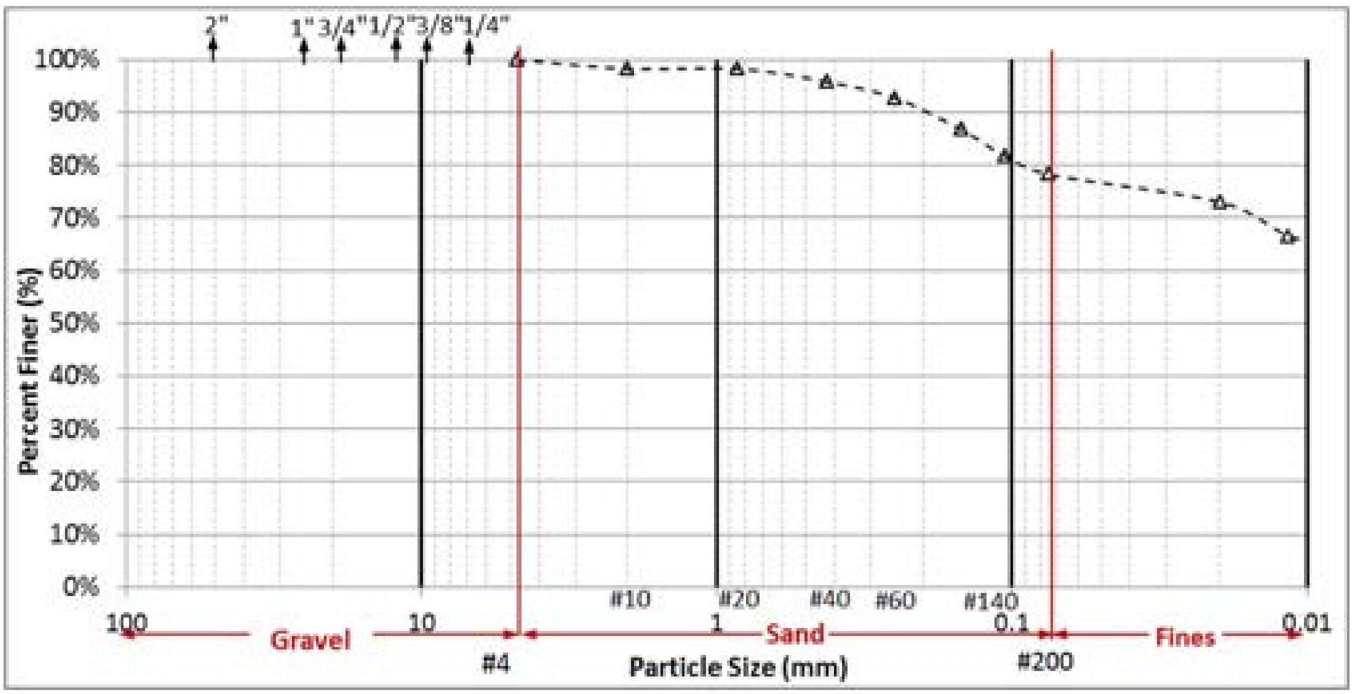


Figure 4.41: Grain Size Distribution Curve for Behring Clay Sample at Site 11

4.1.11 Site 12: FM 20 [Behring Clay, BH – 20]

This section summarizes the findings at Site 12. On August 13, 2015, soil samples were collected from the west side of FM 20 near Rockne, TX just south of the intersection of FM 20 and FM 235 in Bastrop County (Figure 4.42). The collected soil samples included a large amount of fines and belong to the Wilcox group, according to geologic mapping of the area. The side of the roadway showed extensive longitudinal cracking, indicative of an expansive subgrade. The collected soil samples were extensively tested to identify soil characteristics and swelling properties.

Figure 4.42: Augured Hole from FM 20

Location and Identification of Soil Samples

The location of Site 12 corresponds to a site on the west side of FM 20 near Rockne, TX, just south of the intersection of FM 20 and FM 235. The soil samples were collected using a trailer-mounted Simco 250 PTC auger to bore through the topsoil and reach the subgrade. A soil layer of brown-to-tan, compacted, fat clay with a slight amount of gravel was encountered below the topsoil. This soil was identified as our target soil for Site 11. One borehole was drilled to a depth of 3 feet, and two 5-gallon buckets of soil samples were collected for further testing. The GPS coordinates of the borehole locations at Site 12 were marked for soil identification purposes.

The GPS coordinates were input into Google Earth (the resulting image is shown in Figure 4.43), and the USGS geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Wilcox group. To complement this information, an interactive map from the USDA was used to identify the soil found at the ground surface of Site 12. The information from the USDA soil survey indicates that the soil retrieved from the site is the Behring clay loam, as shown in Figure 4.44.

Figure 4.43: Map of Site 12 Location on FM 20 (Google 2014)

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
Bastrop County, Texas (TX021)			
BeB	Behring clay loam, 1 to 3 percent slopes	17.5	9.2%
BeC2	Behring clay loam, 3 to 5 percent slopes, eroded	68.3	35.9%
CfB	Crockett fine sandy loam, 1 to 3 percent slopes	13.6	7.2%
CsC2	Crockett fine sandy loam, 2 to 5 percent slopes, eroded	73.8	38.8%
WsA	Wilson clay loam, 0 to 1 percent slopes	9.3	4.9%
WsB	Wilson clay loam, 1 to 3 percent slopes	7.6	4.0%
Totals for Area of Interest		190.1	100.0%

Figure 4.44: Soil Survey Map and Table at Site 12 (USDA 2013)

<u>Characterization of Behring Clay Samples [BH - 20]</u>

The Behring soil was air dried and processed for the soil characterization and centrifuge tests. As Table 4.12 indicates, Atterberg Limits tests determined an average liquid limit of 50%, and an average plastic limit of 24%. These results defined the plasticity index as 26%. The GSD curve produced from the Wet Sieve test is shown in Figure 4.45. The results of the wet sieve analysis showed that the soil was composed of about 30% sand-sized particles and 70% fine-sized particles. The optimum conditions given by the Standard Proctor compaction test were defined by

the NAVDAC equations and gave an optimum moisture content of 22.0% and a maximum dry unit weight of 15.40 kN/m^3 (98 pcf).

Table 4.12: Results from Atterberg Limit Tests on Behring Samples from Site 12

Test #	1	2
Predicted Liquid Limit, LL	49%	52%
Selected Liquid Limit, LL	49%	52%
Plastic Limit, PL	26%	21%
Plasticity Index, PI	22%	30%
Averaged Liquid Limit, LL_{avg}	**50%**	
Averaged Plastic Limit, PL_{avg}	**24%**	
Averaged Plasticity Index, PI_{avg}	**26%**	

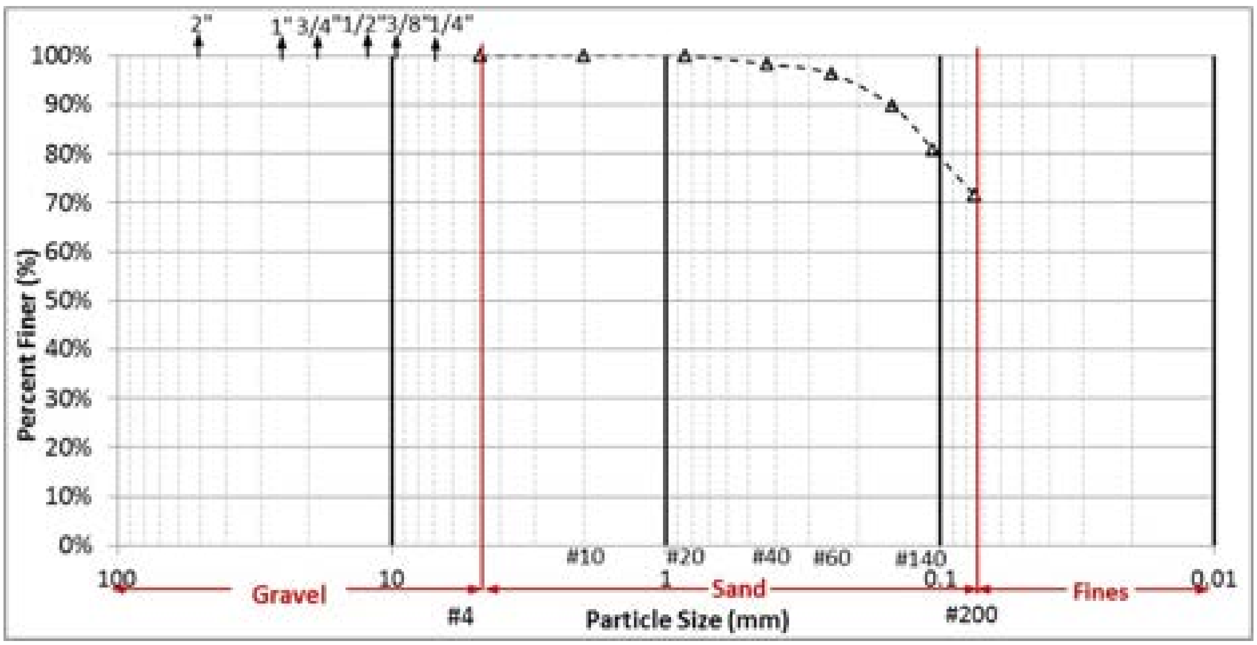


Figure 4.45: Grain Size Distribution Curve for Behring Clay Sample at Site 12

4.1.12 Site 13: FM 972 – North Site [Crockett Clay, CR – 672N]

This section summarizes the findings at Site 13. On August 13, 2015, soil samples were collected from the west side of FM 672 just southwest of the county line between Bastrop and Caldwell counties. The collected soil samples included a large amount of fines and belong to the Wilcox group, according to geologic mapping of the area. The side of the roadway showed extensive longitudinal cracking, indicative of an expansive subgrade. The collected soil samples were extensively tested to identify soil characteristics and swelling properties.

Location and Identification of Soil Samples

The location of Site 12 corresponds to a site on the west side FM 672 just south of the county line. The soil samples were collected using a trailer-mounted Simco 250 PTC auger to bore through the topsoil and reach the subgrade. A soil layer of dark-brown-to-brown, compacted, fat clay with a slight amount of gravel was encountered below the topsoil. This soil was identified as our target soil for Site 13. One borehole was drilled to a depth of 3 feet, and two 5-gallon buckets of soil samples were collected for further testing. The GPS coordinates of the borehole locations at Site 13 were marked for soil identification purposes.

The GPS coordinates were input into Google Earth (the resulting image is shown in Figure 4.46), and the USGS geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Wilcox group. To complement this information, an interactive map from the USDA was used to identify the soil found at the ground surface of Site 13. The information from the USDA soil survey indicates that the soil retrieved from the site is the Crockett fine sand loam as shown in Figure 4.47.

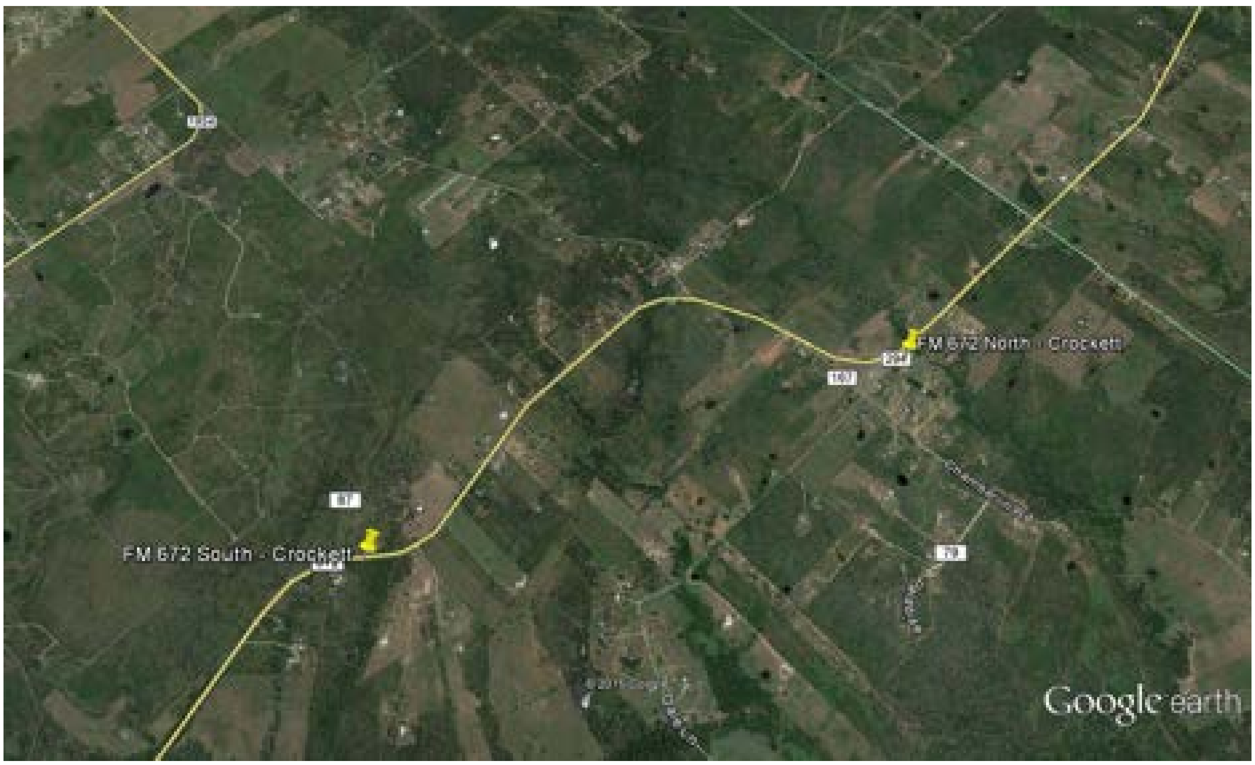
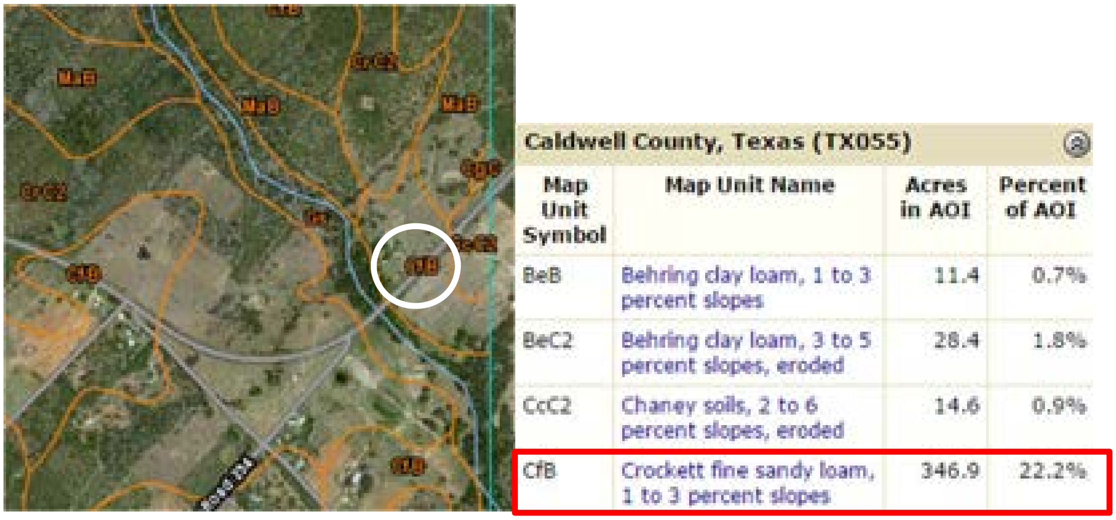


Figure 4.46: Map of Site 13 Location on FM 672 (Google 2014)



Caldwell County, Texas (TX055)			
Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
BeB	Behring clay loam, 1 to 3 percent slopes	11.4	0.7%
BeC2	Behring clay loam, 3 to 5 percent slopes, eroded	28.4	1.8%
CcC2	Chaney soils, 2 to 6 percent slopes, eroded	14.6	0.9%
CfB	Crockett fine sandy loam, 1 to 3 percent slopes	346.9	22.2%

Figure 4.47: Soil Survey Map and Table at Site 13 (USDA 2013)

<u>Characterization of Crockett Samples [CR – 672N]</u>

The Crockett soil was air dried and processed for the soil characterization and centrifuge tests. Atterberg Limits tests (Table 4.13) determined an average liquid limit of 40%, and an average plastic limit of 22%. These results defined the plasticity index as 18%. The GSD curve produced from the Wet Sieve test is shown in Figure 4.48. The results of the wet sieve analysis showed that the soil was composed of about 28% sand-sized particles; the other 72% was fine-sized particles. The optimum conditions given by the Standard Proctor compaction test were defined by the NAVDAC equations and gave an optimum moisture content of 20.2% and a maximum dry unit weight of 16.17 kN/m^3 (103 pcf).

Table 4.13: Results from Atterberg Limit Tests on Crockett Samples from Site 13

Test #	1	2	3
Predicted Liquid Limit, LL	41%	39%	39%
Selected Liquid Limit, LL	41%	39%	39%
Plastic Limit, PL	24%	23%	17%
Plasticity Index, PI	17%	16%	22%
Averaged Liquid Limit, LL_{avg}	**40%**		
Averaged Plastic Limit, PL_{avg}	**22%**		
Averaged Plasticity Index, PI_{avg}	**18%**		

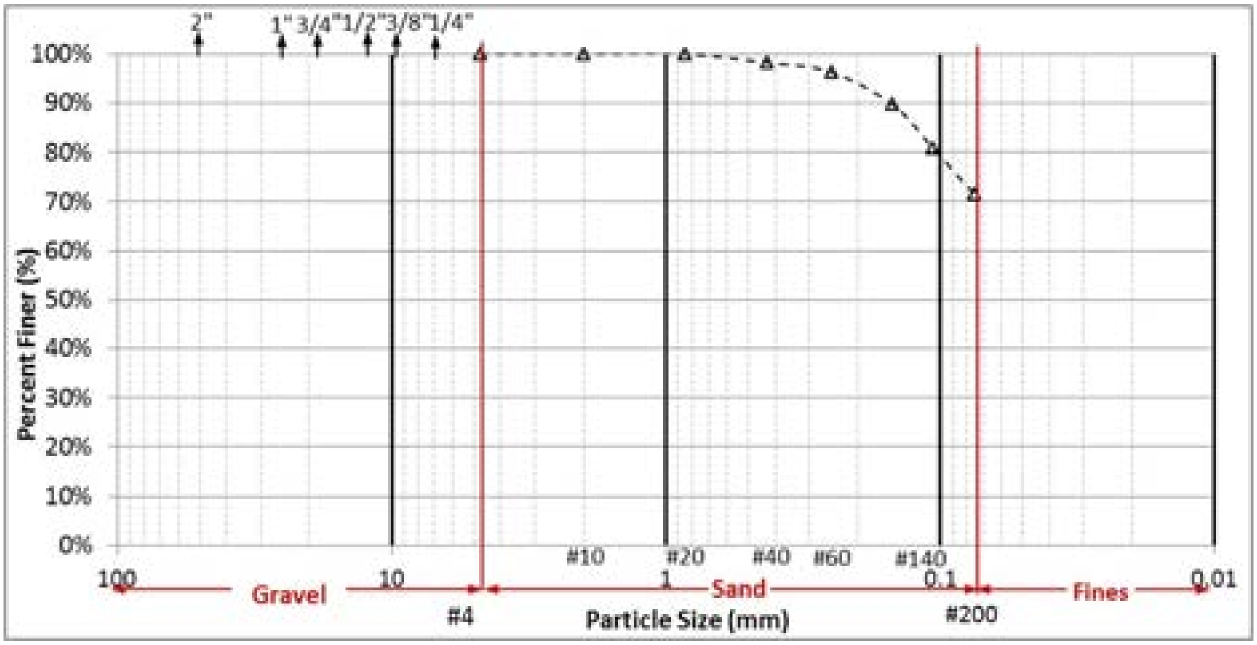


Figure 4.48: Grain Size Distribution Curve for Crockett Soil Sample at Site 13

4.1.13 Site 14: FM 672 – South Location [Crockett Soil, CR – 672S]

This section summarizes the findings at Site 14. On August 13, 2015, soil samples were collected from the west side of FM 672 just northeast of the intersection of FM 672 and FM 87 in Caldwell County (Figure 4.49). The collected soil samples included a large amount of fines and belong to the Wilcox group, according to geologic mapping of the area. The side of the roadway showed extensive longitudinal cracking, indicative of an expansive subgrade. The collected soil samples were extensively tested to identify soil characteristics and swelling properties.

Figure 4.49: Augured Hole from FM 672 South

<u>Location and Identification of Soil Samples</u>

The location of Site 14 corresponds to a site on the west side FM 672 just northeast of the intersection of FM 672 and FM 87. The soil samples were collected using a trailer-mounted Simco 250 PTC auger to bore through the topsoil and reach the subgrade. A soil layer of dark brown to brown, compacted, fat clay with a slight amount of gravel was encountered below the topsoil. This soil was identified as our target soil for Site 14. One borehole was drilled to a depth of 3 feet, and two 5-gallon buckets of soil samples were collected for further testing. The GPS coordinates of the borehole locations at Site 14 were marked for soil identification purposes.

The GPS coordinates were input into Google Earth (the resulting image is shown in Figure 4.50), and the USGS geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Wilcox group. To complement this information, an interactive map from the USDA was used to identify the soil found at the ground surface of Site 14. The information from the USDA soil survey indicates that the soil retrieved from the site is the Crockett fine sand loam as shown in Figure 4.51.

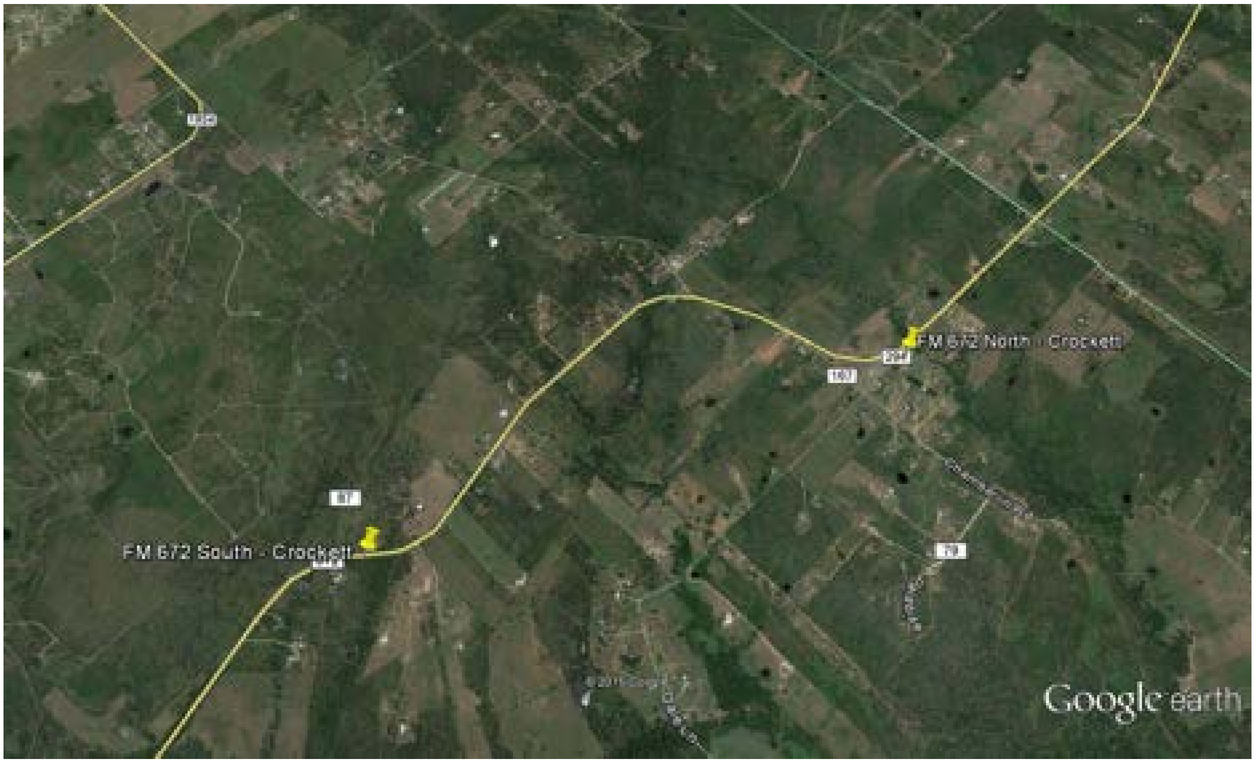
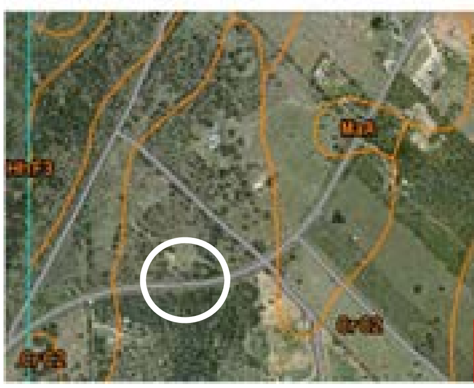


Figure 4.50: Map of Site 14 Location on FM 672 (Google 2014)



Caldwell County, Texas (TX055)

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
BeB	Behring clay loam, 1 to 3 percent slopes	11.4	0.7%
BeC2	Behring clay loam, 3 to 5 percent slopes, eroded	28.4	1.8%
CcC2	Chaney soils, 2 to 6 percent slopes, eroded	14.6	0.9%
CfB	Crockett fine sandy loam, 1 to 3 percent slopes	346.9	22.2%
CgC	Crockett gravelly fine sandy loam, 1 to 5 percent slopes	280.1	17.9%
CrC2	Crockett soils, 2 to 5 percent slopes, eroded	504.0	32.2%

Figure 4.51: Soil Survey Map and Table at Site 14 (USDA 2013)

<u>Characterization of Crockett Soil Samples [CR – 672S]</u>

The Crockett soil was air dried and processed for the soil characterization and centrifuge tests. Atterberg Limits tests (Table 4.14) determined an average liquid limit of 54%, and an average plastic limit of 23%. These results defined the plasticity index as 31%. The GSD curve produced from the Wet Sieve and Hydrometer tests is shown in Figure 4.52. The results of the wet sieve analysis showed that the soil was composed of about 15% sand-sized particles and 85% fine-sized particles. The optimum conditions given by the Standard Proctor compaction test were defined by

the NAVDAC equations and gave an optimum moisture content of 23.5% and a maximum dry unit weight of 14.97 kN/m^3 (95 pcf).

Table 4.14: Results from Atterberg Limit Tests on Crockett Samples from Site 14

Test #	1	2
Predicted Liquid Limit, LL	54%	54%
Selected Liquid Limit, LL	54%	54%
Plastic Limit, PL	25%	21%
Plasticity Index, PI	29%	33%
Averaged Liquid Limit, LL_{avg}	**54%**	
Averaged Plastic Limit, PL_{avg}	**23%**	
Averaged Plasticity Index, PI_{avg}	**31%**	

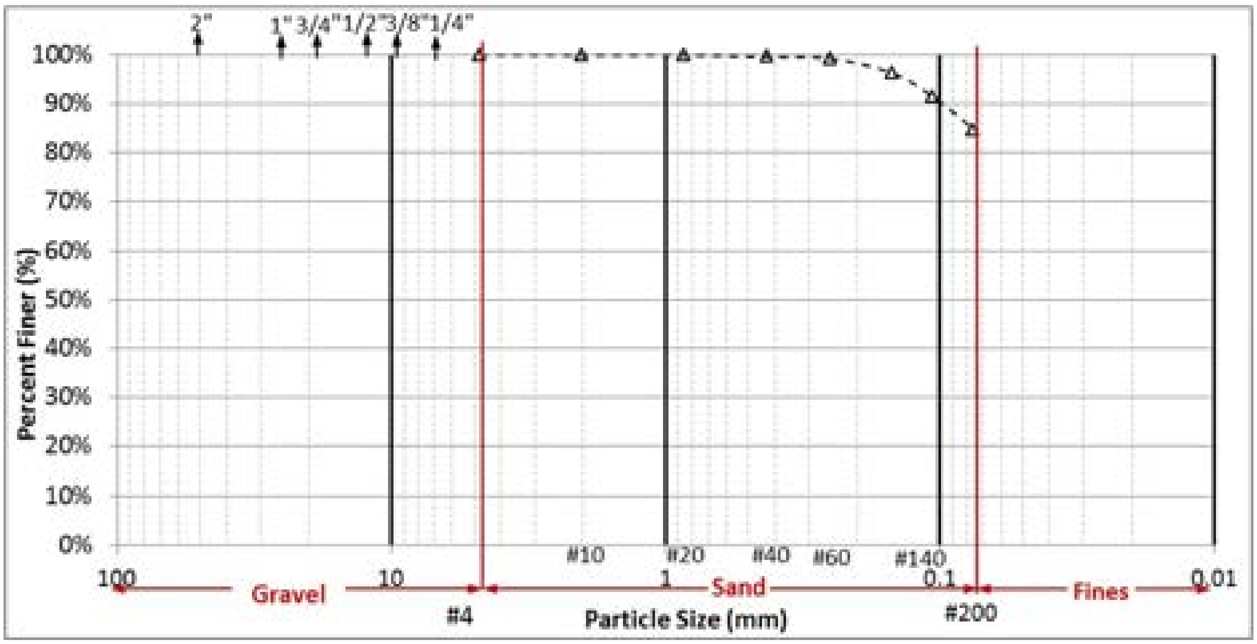


Figure 4.52: Grain Size Distribution Curve for Crockett Soil Samples at Site 14

4.1.14 Site 15: FM 1854 – East Location [Burleson Clay, BU]

This section summarizes the findings at Site 15. On August 13, 2015, soil samples were collected from the north side of FM 1854 just northeast of the intersection of FM 1854 and FM 672 in Caldwell County (Figure 4.53). The collected soil samples included a large amount of fines and belong to the Midway Group, according to geologic mapping of the area. The side of the roadway showed extensive amount of longitudinal cracking, indicative of an expansive subgrade. The collected soil samples were extensively tested to identify soil characteristics and swelling properties.

Figure 4.53: Augured Hole from FM 1854E

Location and Identification of Soil Samples

The location of Site 15 corresponds to a site on the north side of FM 1854 just northeast of the intersection of FM 1854 and FM 672. The soil samples were collected using a trailer-mounted Simco 250 PTC auger to bore through the topsoil and reach the subgrade. A soil layer of dark brown to brown, compacted, fat clay with a slight amount of gravel was encountered below the topsoil. This soil was identified as our target soil for Site 15. One borehole was drilled to a depth of 3 feet, and two 5-gallon buckets of soil samples were collected for further testing. The GPS coordinates of the borehole locations at Site 15 were marked for soil identification purposes.

The GPS coordinates were input into Google Earth (the resulting image is shown in Figure 4.54), and the USGS geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Midway Group. To complement this information, an interactive map from the USDA was used to identify the soil found at the ground surface of Site 15, indicating that the soil retrieved from the site is the Burleson clay as shown in Figure 4.55.

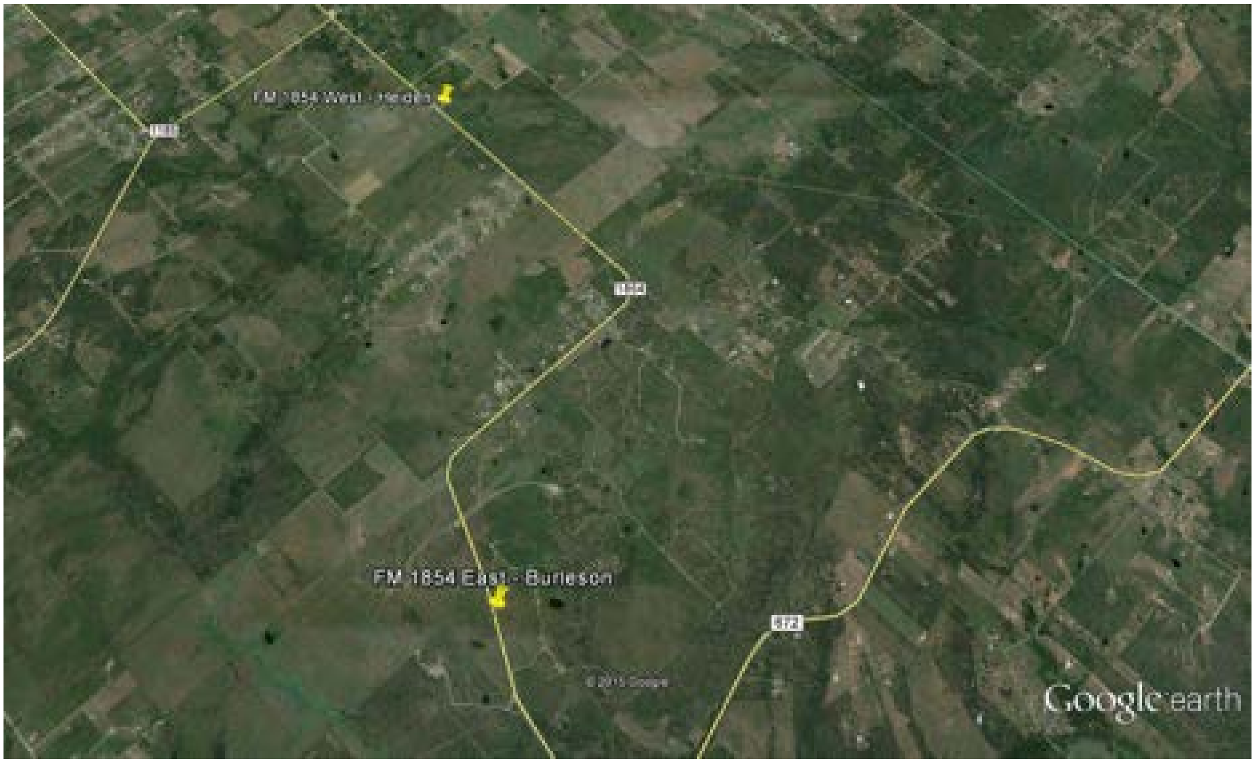
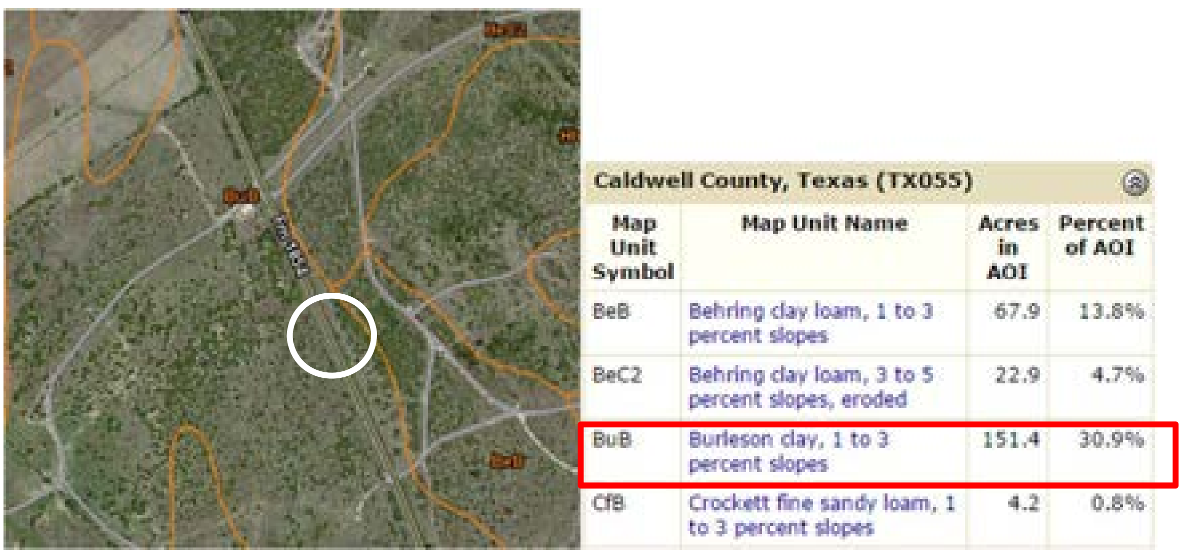


Figure 4.54: Map of Site 15 Location on FM 1854 (Google 2014)



Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
BeB	Behring clay loam, 1 to 3 percent slopes	67.9	13.8%
BeC2	Behring clay loam, 3 to 5 percent slopes, eroded	22.9	4.7%
BuB	Burleson clay, 1 to 3 percent slopes	151.4	30.9%
CfB	Crockett fine sandy loam, 1 to 3 percent slopes	4.2	0.8%

Figure 4.55: Soil Survey Map and Table at Site 15 (USDA 2013)

<u>Characterization of Burleson Soil Samples [BU]</u>

The Burleson soil was air dried and processed for the soil characterization and centrifuge tests. Atterberg Limits tests (Table 4.15) determined an average liquid limit of 62%, and an average plastic limit of 23%. These results defined the plasticity index as 39%. The GSD curve produced from the Wet Sieve and Hydrometer tests is shown in Figure 4.56. The results of the wet sieve analysis showed that the soil was composed of about 10% sand-sized particles and 90% fine-sized particles. The optimum conditions given by the Standard Proctor compaction test were defined by the NAVDAC equations and gave an optimum moisture content of 25.2% and a maximum dry unit weight of 14.32 kN/m^3 (91 pcf).

Table 4.15: Results from Atterberg Limit Tests on Burleson Samples from Site 15

Test #	1	2	3
Predicted Liquid Limit, LL	63%	61%	62%
Selected Liquid Limit, LL	63%	61%	62%
Plastic Limit, PL	23%	25%	21%
Plasticity Index, PI	40%	36%	41%
Averaged Liquid Limit, LL_{avg}	**62%**		
Averaged Plastic Limit, PL_{avg}	**23%**		
Averaged Plasticity Index, PI_{avg}	**39%**		

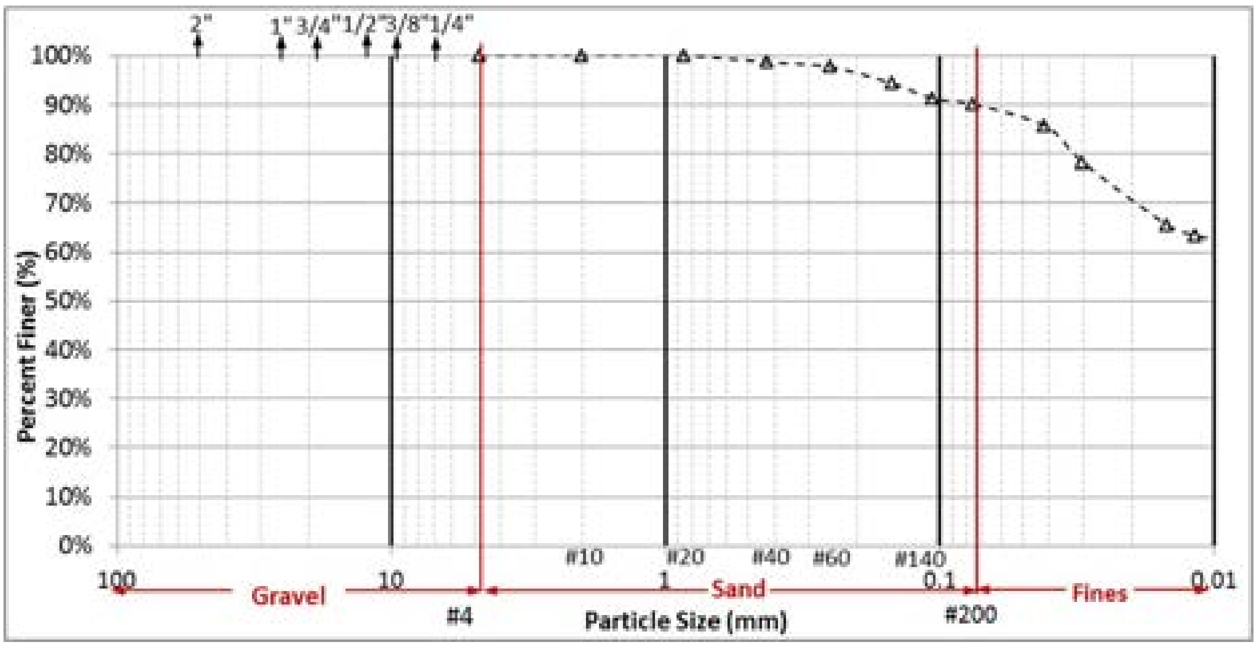


Figure 4.56: Grain Size Distribution Curve for Burleson Soil Samples at Site 15

4.1.15 Site 16: FM 1854 – West Location [Heiden Clay, HE – 1854W]

This section summarizes the findings at Site 16. On August 13, 2015, soil samples were collected from the north side of FM 1854 just east of the intersection of FM 1854 and FM 170 near Lytton Springs, TX in Caldwell County (Figure 4.57). The collected soil samples included a large amount of fines, and belong to the Wilcox group according to geologic mapping of the area. The side of the roadway showed extensive amount of longitudinal cracking, indicative of an expansive subgrade. The collected soil samples were extensively tested to identify soil characteristics and swelling properties.

Figure 4.57: Augured Hole from FM 1854W

<u>Location and Identification of Soil Samples</u>

The location of Site 16 corresponds to a site on the north side of FM 1854 just east of the intersection of FM 1854 and FM 170, or Crooked Rd. The soil samples were collected using a trailer-mounted Simco 250 PTC auger to bore through the topsoil and reach the subgrade. A soil layer of dark brown to brown, compacted, fat clay with a slight amount of gravel was encountered below the topsoil. This soil was identified as our target soil for Site 16. One borehole was drilled to a depth of 3 feet, and two 5-gallon buckets of soil samples were collected for further testing. The GPS coordinates of the borehole locations at Site 16 were marked for soil identification purposes.

The GPS coordinates were input into Google Earth (the resulting image is shown in Figure 4.58), and the USGS geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Wilcox group. To complement this information, an interactive map from the USDA was used to identify the soil found at the ground surface of Site 16, indicating that the soil retrieved from the site is the Heiden clay, as shown in Figure 4.59. Note that this soil description is different from that of the Heiden clay found at other locations, as it was dark brown as opposed to tan.

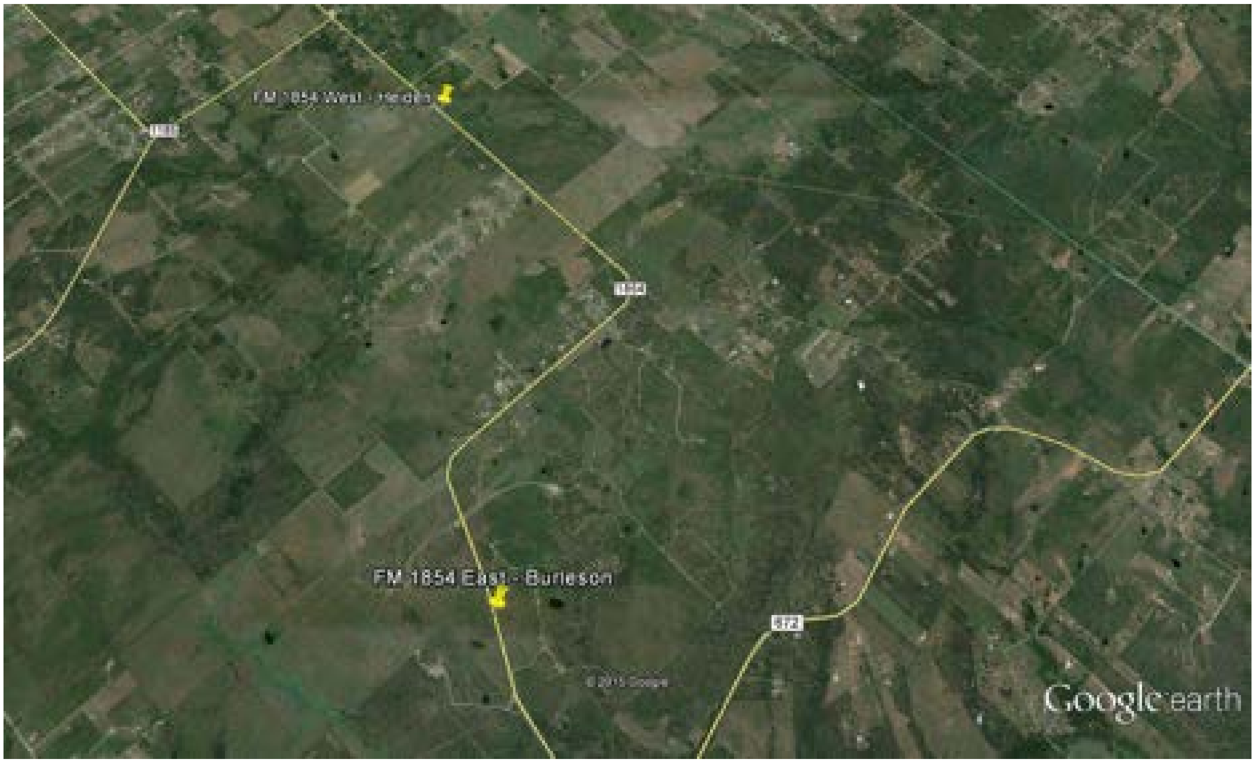


Figure 4.58: Map of Site 16 Location on FM 1854 (Google 2014)

Gs	Gowen soils, frequently flooded	22.0	18.7%
HeB	Heiden clay, 1 to 3 percent slopes	4.6	3.9%
HeC2	Heiden clay, 3 to 5 percent slopes, eroded	25.8	22.0%
HhF3	Heiden-Ferris complex, 5 to 20 percent slopes, severely eroded	15.5	13.2%

Figure 4.59: Soil Survey Map and Table at Site 16 (USDA 2013)

<u>Characterization of Heiden Soil Samples [HE – 1854W]</u>

The Heiden soil was air dried and processed for the soil characterization and centrifuge tests. Atterberg Limits tests (Table 4.16) determined an average liquid limit of 65%, and an average plastic limit of 27%. These results defined the plasticity index as 38%. The GSD curve produced from the Wet Sieve is shown in Figure 4.60. The results of the wet sieve analysis showed that the soil was composed of about 9% sand-sized particles and 91% fine-sized particles. The optimum conditions given by the Standard Proctor compaction test were defined by the NAVDAC equations and gave an optimum moisture content of 26.7% and a maximum dry unit weight of 13.89 kN/m^3 (88 pcf).

Table 4.16: Results from Atterberg Limit Tests on Heiden Clay Samples from Site 16

Test #	1	2	3
Predicted Liquid Limit, LL	62%	65%	65%
Selected Liquid Limit, LL	65%	65%	65%
Plastic Limit, PL	28%	27%	27%
Plasticity Index, PI	37%	38%	38%
Averaged Liquid Limit, LL_{avg}	**65%**		
Averaged Plastic Limit, PL_{avg}	**27%**		
Averaged Plasticity Index, PI_{avg}	**38%**		

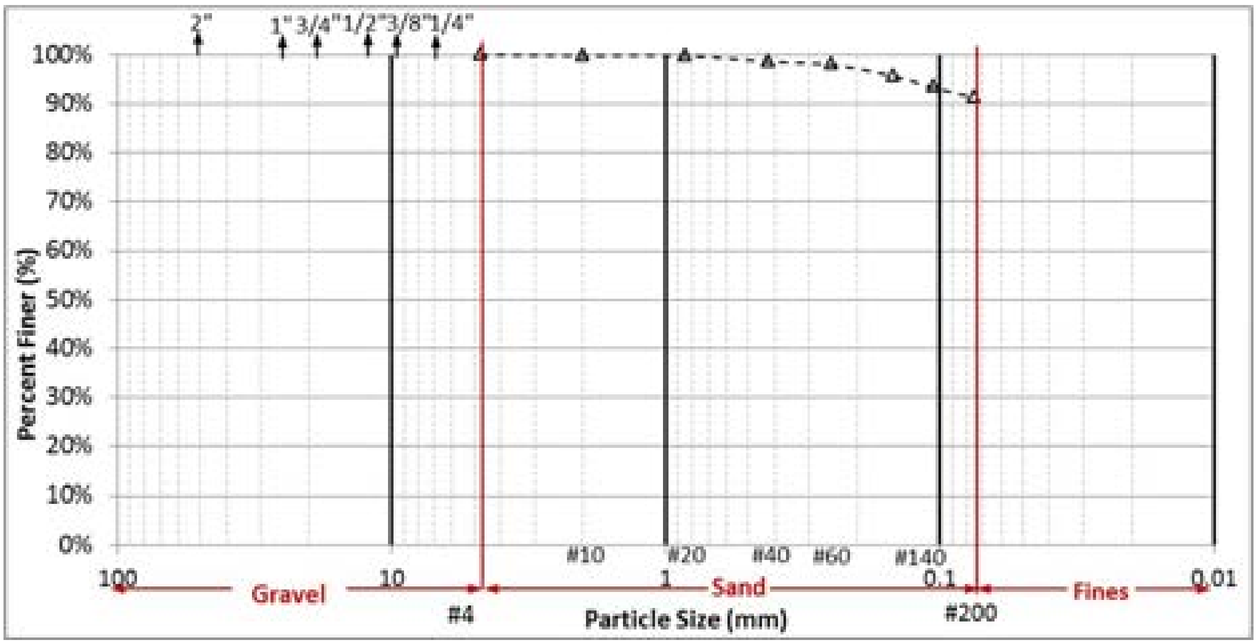


Figure 4.60: Grain Size Distribution Curve for Heiden Clay Samples at Site 16

4.1.16 Site 17: FM 1854 and SH 21[Heiden Clay, HE – 1854 & SH 21]

This section summarizes the findings at Site 17. On August 13, 2015, soil samples were collected from the north side of FM 1854 just east of the intersection of FM 1854 and SH 21 near Lytton Springs, TX in Caldwell County (Figure 4.61). The collected soil samples included a large amount of fines, and belong to the Navarro group according to geologic mapping of the area. The side of the roadway showed extensive longitudinal cracking, indicative of an expansive subgrade. The collected soil samples were extensively tested to identify soil characteristics and swelling properties.

Figure 4.61: Augured Hole from FM 1854 and SH 21

Location and Identification of Soil Samples

The location of Site 17 corresponds to a site on the north side of FM 1854 just east of the intersection of FM 1854 and SH 21, between SH 21 and Tomahawk Trail. The soil samples were collected using a trailer-mounted Simco 250 PTC auger to bore through the topsoil and reach the subgrade. A soil layer of light brown to tan, compacted, fat clay with a slight amount of gravel was encountered below the topsoil. This soil was identified as our target soil for Site 17. One borehole was drilled to a depth of 3 feet, and two 5-gallon buckets of soil samples were collected for further testing. The GPS coordinates of the borehole locations at Site 17 were marked for soil identification purposes.

The GPS coordinates were input into Google Earth (the resulting image is shown in Figure 4.62), and the USGS geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Navarro group. To complement this information, an interactive map from the USDA was used to identify the soil found at the ground surface of Site 17, indicating that the soil retrieved from the site is the Heiden clay, as shown in Figure 4.63.

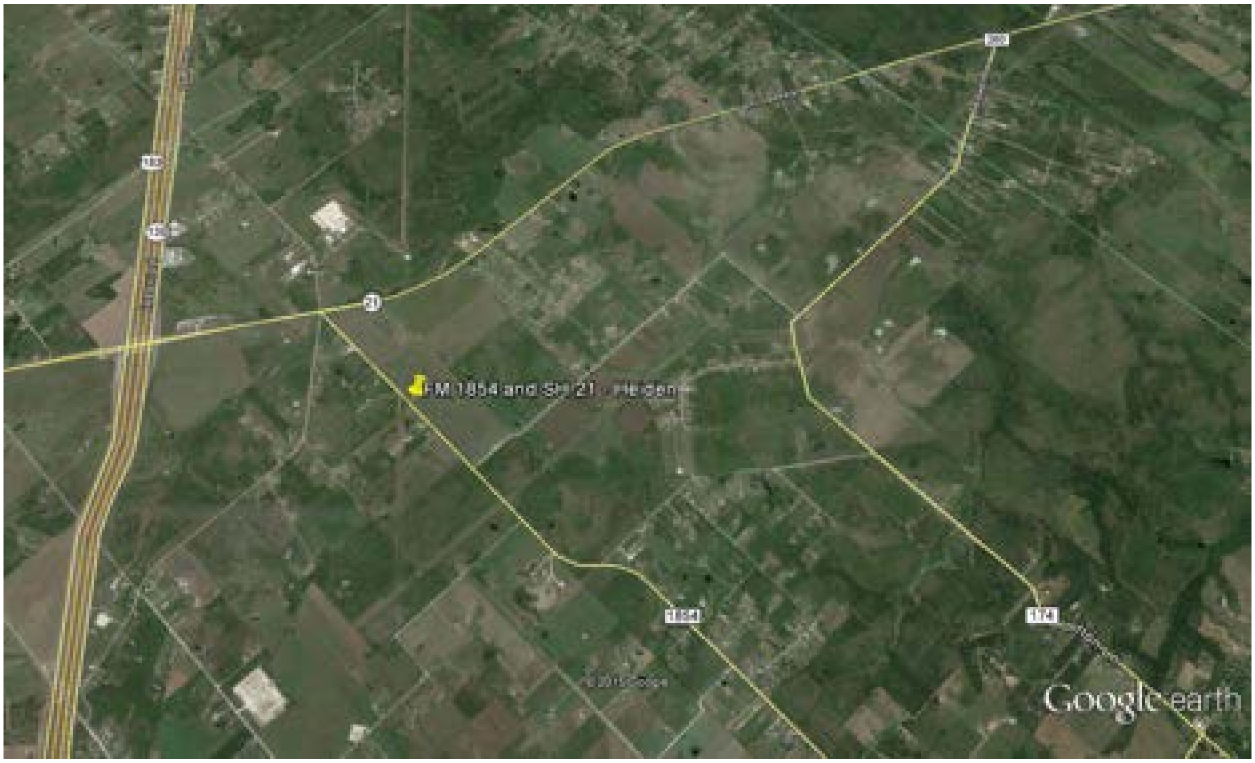
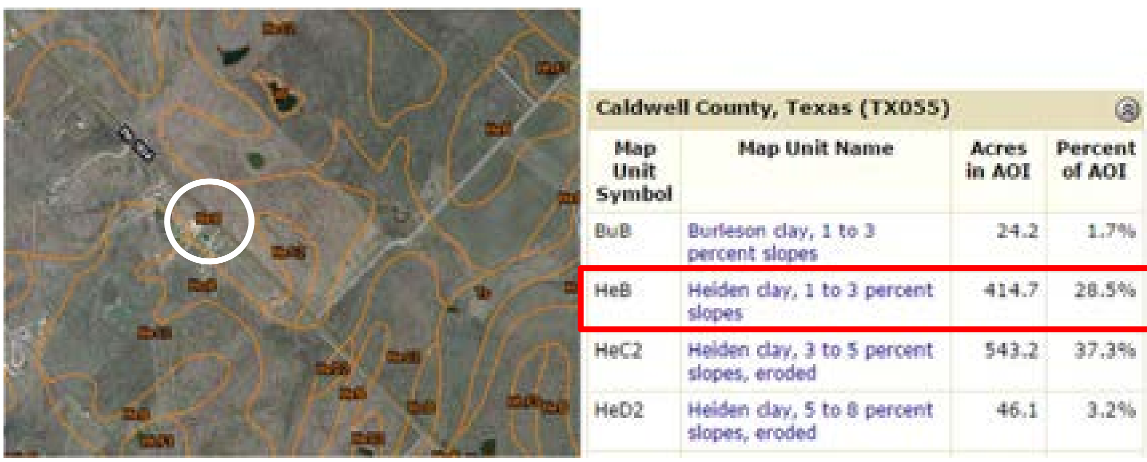


Figure 4.62: Map of Site 17 Location on FM 1854 (Google 2014)



Caldwell County, Texas (TX055)			
Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
BuB	Burleson clay, 1 to 3 percent slopes	24.2	1.7%
HeB	Heiden clay, 1 to 3 percent slopes	414.7	28.5%
HeC2	Heiden clay, 3 to 5 percent slopes, eroded	543.2	37.3%
HeD2	Heiden clay, 5 to 8 percent slopes, eroded	46.1	3.2%

Figure 4.63: Soil Survey Map and Table at Site 17 (USDA 2013)

<u>Characterization of Heiden Soil Samples [HE – 1854 & SH 21]</u>

The Heiden soil was air dried and processed for the soil characterization and centrifuge tests. Atterberg Limits tests (Table 4.17) determined an average liquid limit of 63%, and an average plastic limit of 22%. These results defined the plasticity index as 41%. The GSD curve produced from the Wet Sieve is shown in Figure 4.64. The results of the wet sieve analysis showed that the soil was composed of about 9% sand-sized particles and 91% fine-sized particles. The optimum conditions given by the Standard Proctor compaction test were defined by the NAVDAC equations and gave an optimum moisture content of 25.3% and a maximum dry unit weight of 14.2 kN/m^3 (91 pcf).

Table 4.17: Results from Atterberg Limit Tests on Heiden Clay Samples from Site 17

Test #	1	2	3
Predicted Liquid Limit, LL	69%	69%	69%
Selected Liquid Limit, LL	63%	63%	63%
Plastic Limit, PL	22%	22%	23%
Plasticity Index, PI	41%	41%	40%
Averaged Liquid Limit, LL_{avg}	**63%**		
Averaged Plastic Limit, PL_{avg}	**22%**		
Averaged Plasticity Index, PI_{avg}	**41%**		

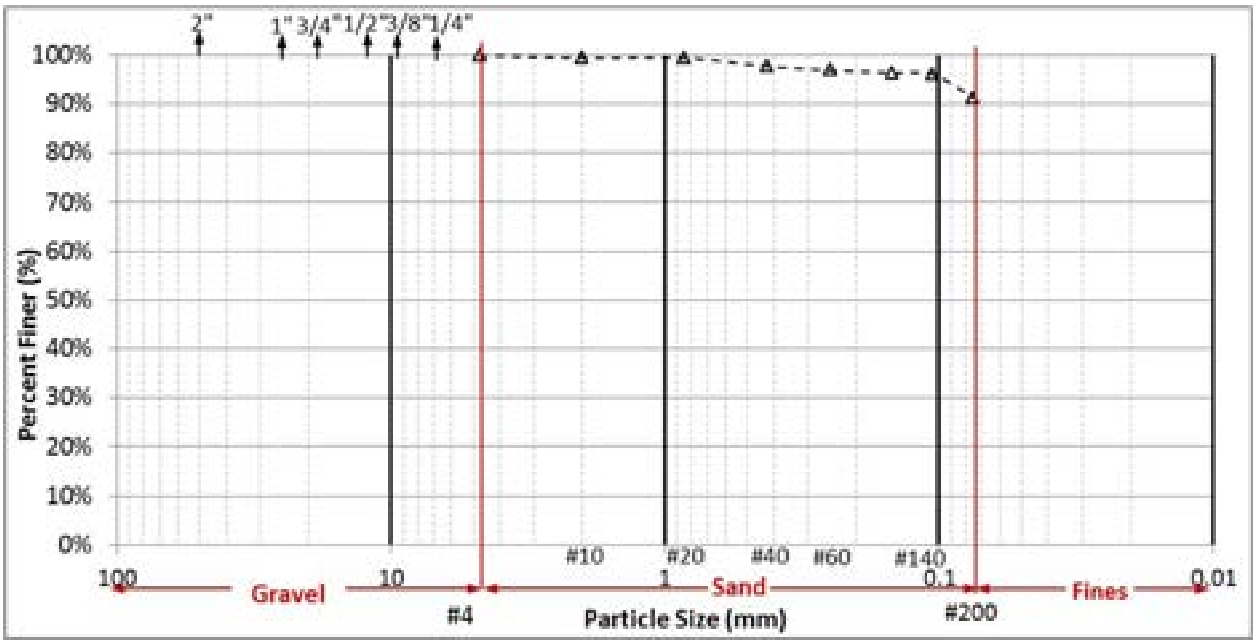


Figure 4.64: Grain Size Distribution Curve for Heiden Clay Samples at Site 17

4.1.17 Site 18: FM 685 [Branyon Clay, BR – 685]

This section summarizes the findings at Site 18, shown in Figure 4.65. In December 2015, soil samples were collected during the installation of moisture and suction sensors used later in Chapter 5. The collected soil samples included a large amount of fines, and belong to the Austin Chalk, according to geologic mapping of the area. The side of the roadway showed extensive longitudinal cracking, indicative of an expansive subgrade. The collected soil samples were extensively tested to identify soil characteristics and swelling properties.

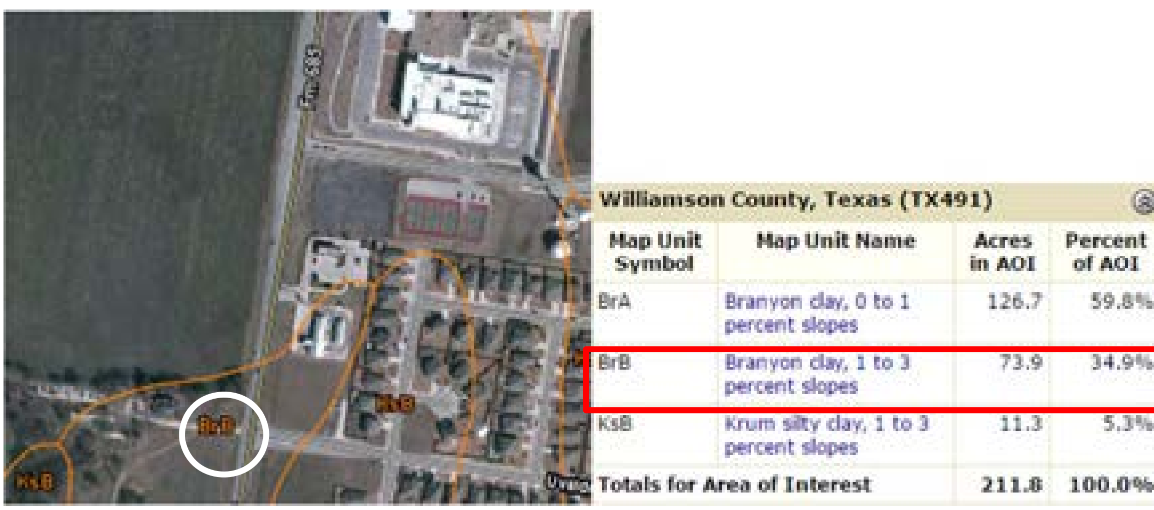
Figure 4.65: Sampled Slope near FM 685

Location and Identification of Soil Samples

The location of Site 18 corresponds to a site across from the intersection of Uvalde Drive and FM 685 in Hutto, TX. The site is south of Hutto High School and the intersection of FM 685 and US 79. The collection of samples consisted of sampling soil using shovel from a site to the left of the drainage box on the shoulder of the reconstructed road. Note that the soil below the sampled soil, the Krum series, was sampled as well and found to be non-expansive. Results from this soil sampling will be detailed further in Chapter 4 that covers the sensors at the given location. The GPS coordinates of the borehole locations at Site 18 were marked for soil identification purposes.

The GPS coordinates were input into Google Earth (the resulting image is shown in Figure 4.66), and the USGS geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Austin Chalk. To complement this information, an interactive map from the USDA was used to identify the soil found at the ground surface of Site 18, indicating that the soil retrieved is the Branyon clay, as shown in Figure 4.67.

Figure 4.66: Map of Site 18 Location on FM 685(Google 2014)



Williamson County, Texas (TX491)

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
BrA	Branyon clay, 0 to 1 percent slopes	126.7	59.8%
BrB	Branyon clay, 1 to 3 percent slopes	73.9	34.9%
KsB	Krum silty clay, 1 to 3 percent slopes	11.3	5.3%
Totals for Area of Interest		211.8	100.0%

Figure 4.67: Soil Survey Map and Table at Site 18 (USDA 2013)

<u>Characterization of Branyon Clay Sample [BR – 685]</u>

The Branyon clay was air dried and processed for the soil characterization and centrifuge tests. Atterberg Limits tests (Table 4.18) determined an average liquid limit of 63%, and an average plastic limit of 22%. These results defined the plasticity index as 41%. The GSD curve produced from the Wet Sieve is shown in Figure 4.68. The results of the wet sieve analysis showed that the soil was composed of about 7% sand-sized particles and 93% fine-sized particles. The optimum conditions given by the Standard Proctor compaction test were defined by the NAVDAC equations and gave an optimum moisture content of 27.6% and a maximum dry unit weight of 13.70 kN/m^3 (86 pcf).

Table 4.18: Results from Atterberg Limit Tests on Branyon Clay Samples from Site 18

Test #	1	2
Predicted Liquid Limit, LL	64%	66%
Selected Liquid Limit, LL	64%	66%
Plastic Limit, PL	31%	-
Plasticity Index, PI	34%	-
Averaged Liquid Limit, LL_{avg}	**65%**	
Averaged Plastic Limit, PL_{avg}	**31%**	
Averaged Plasticity Index, PI_{avg}	**34%**	

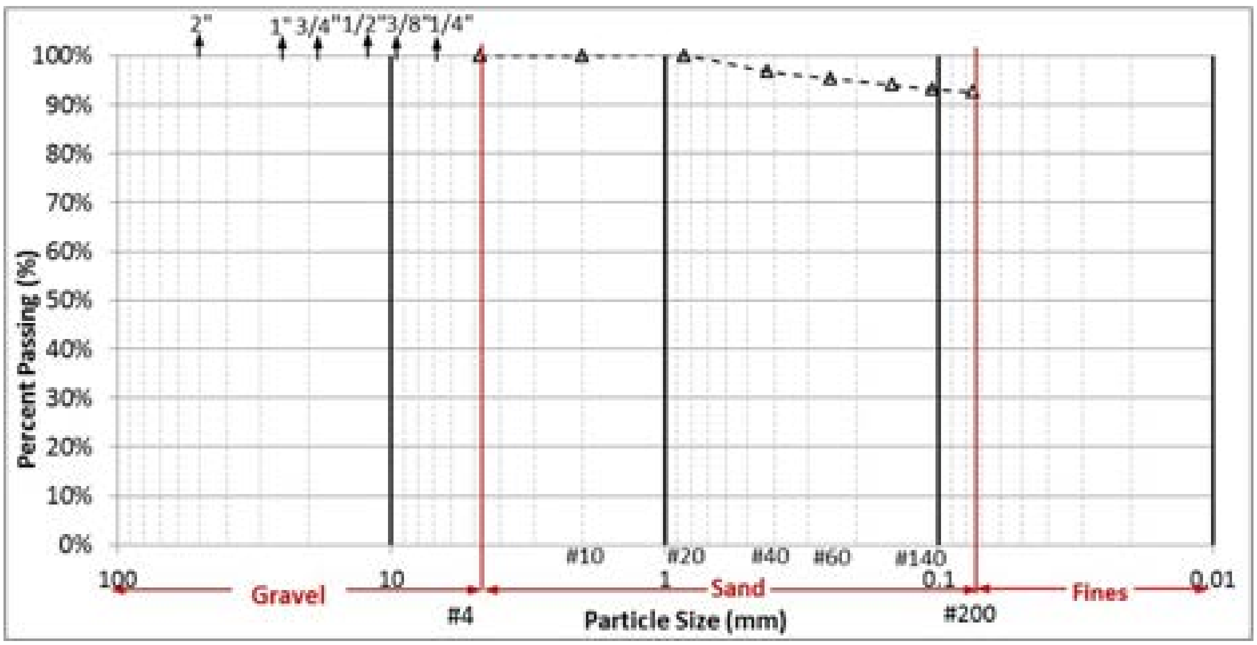


Figure 4.68: Grain Size Distribution Curve for Branyon Clay Samples at Site 18

4.1.18 Site 19: SH-21 [Behring Clay – Cook Mountain Clay – CM]

This section summarizes the findings at Site 18. The soil was sampled during the course of an additional project that involved the placement of various geomembranes to examine the separate between an expansive subgrade and the base for the expansion of the roadway. The location is south of the intersection of SH-21 and US-290 as shown in (Figure 4.69). The collected soil samples included a large amount of fines, and belong to the Navarro group according to geologic mapping of the area. The side of the roadway showed extensive longitudinal cracking, indicative of an expansive subgrade. The collected soil samples were extensively tested to identify soil characteristics and swelling properties.

Figure 4.69: Cut in Cook Mountain Clay during Sensor Installation

Location and Identification of Soil Samples

The location of Site 18 corresponds to a site that was located on the east side of SH-21 over the course of 4000 ft. A soil layer of light brown to brown clay with mottled yellow and red stained hues throughout the soil was located on the excavation for the expansion of SH-21. The soil was identified later as the Cook Mountain clay due to the geologic location of the site and was determined to be the subgrade soil beneath the shoulder expansion. Samples were collected at 7 locations over 3,500 ft in order to test the specimens for their liquid limits.

The location was inputted into Google Earth (the resulting image is shown in Figure 4.70), and the USGS geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Cook Mountain clay, as shown in Figure 4.71. To complement this information, an interactive map from the USDA was used to identify the soil found at the ground surface of Site 17, indicating that the soil retrieved from the site is the Behring clay loam, as shown in Figure 4.72 along the course of the project, albeit at different horizons.

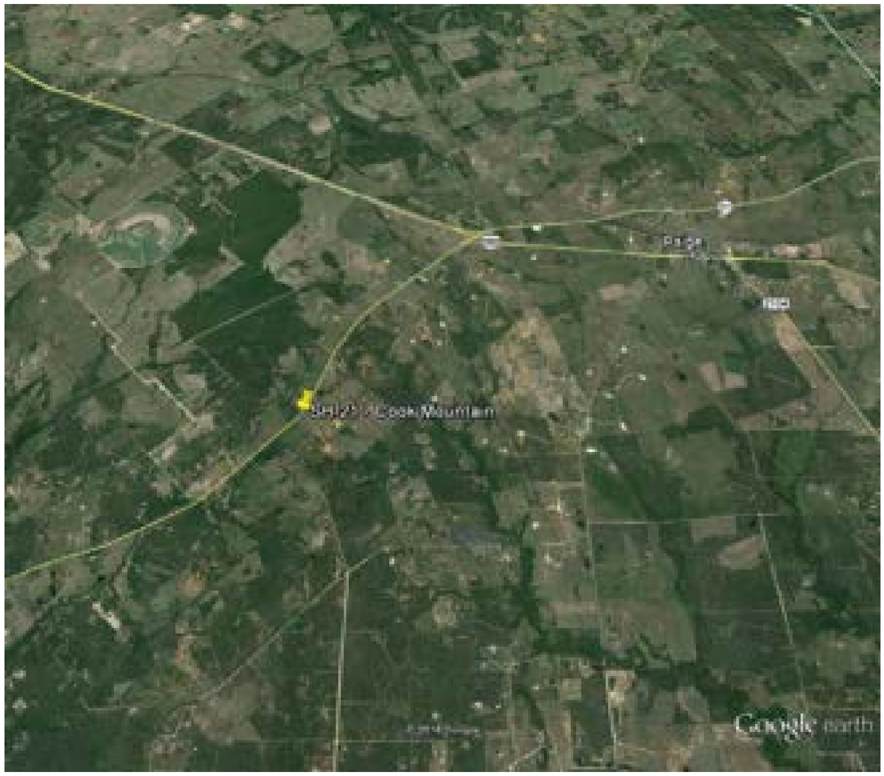


Figure 4.70: Map of Site 18 Location on SH-21 (Google 2014)

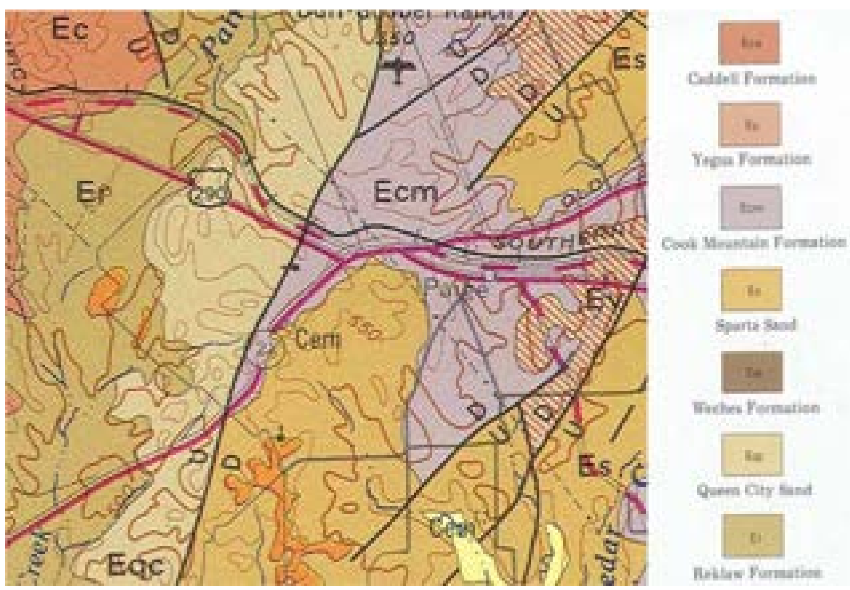


Figure 4.71: Geologic Map of Location (Barnes 1981)

Bastrop County, Texas (TX021)			
Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
AfC	Edge fine sandy loam, 1 to 5 percent slopes	1.8	1.2%
AfC2	Edge fine sandy loam, 2 to 5 percent slopes, moderately eroded	8.0	5.4%
BeB	Behring clay loam, 1 to 3 percent slopes	42.6	29.0%
BeC2	Behring clay loam, 3 to 5 percent slopes, eroded	25.3	17.2%
CsC2	Crockett fine sandy loam, 2 to 5 percent slopes, eroded	39.4	26.8%
CsD3	Crockett fine sandy loam, 3 to 8 percent slopes, severely eroded	4.9	3.3%
HeC2	Heiden clay, 3 to 5 percent slopes, eroded	22.1	15.0%
TfB	Tabor fine sandy loam, 1 to 3 percent slopes	2.9	2.0%
Totals for Area of Interest		**147.0**	**100.0%**

Figure 4.72: Soil Survey Map and Table at Site 18 (USDA 2013)

<u>Characterization of Cook Mountain Soil Samples [CM]</u>

The Behring soil sampled at the given locations were air dried and processed separately for Atterberg Limit testing as shown in Figure 4.73. In order to determine the liquid limit of the combined mass, the soils were randomly combined into three separate "combined" soils to test the liquid limit versus the results from three separate operates. The results are shown in Figure 4.74 and are reasonably consistent, indicating that the soils are similar along the section of the road. The results from the Atterberg Limits tests (Table 4.19) determined an average liquid limit of 55%, and an average plastic limit of 17%. These results defined the plasticity index as 41%. The GSD curve produced from the Wet Sieve is shown in Figure 4.75. The results of the wet sieve analysis showed that the soil was composed of about 18% sand-sized particles and 82% fine-sized particles. The optimum conditions given by the Standard Proctor compaction test were measured via compaction tests following ASTM D698, as shown in Figure 4.76, and the soil was shown to have an optimum moisture content of 20.0% with a maximum dry unit weight of 15.42 kN/m^3.

Figure 4.73: Drying of Specimens taken from SH-21

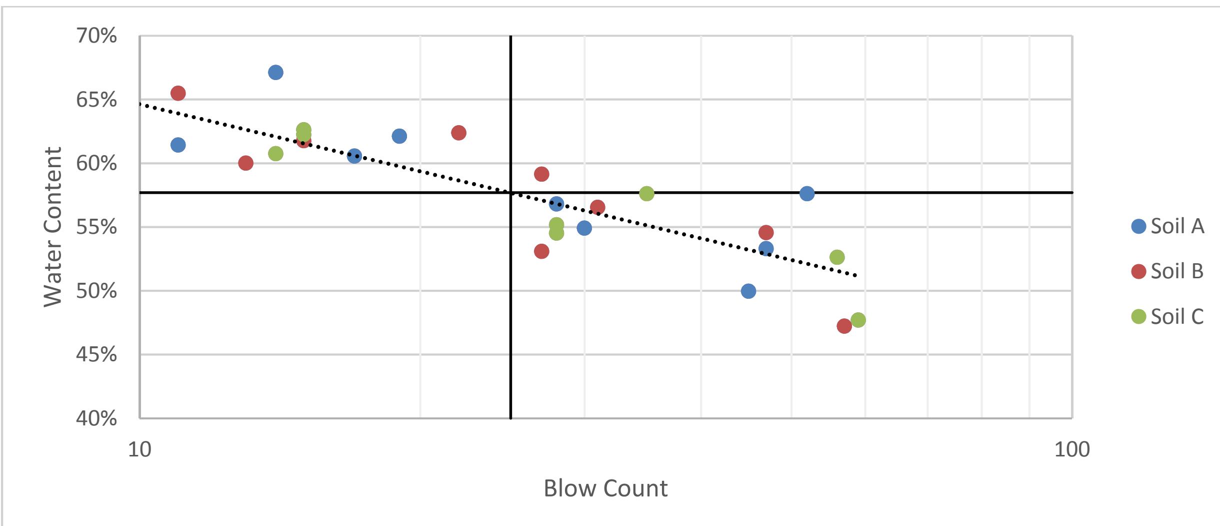


Figure 4.74: Liquid Limit Testing for combined SH-21 Soils

Table 4.19: Results from Atterberg Limit Tests on Behring Clay Samples from Site 18

Liquid Limit (LL)	58
Plastic Limit (PL)	17
Plasticity Index (PI)	41
USCS Classification	CH

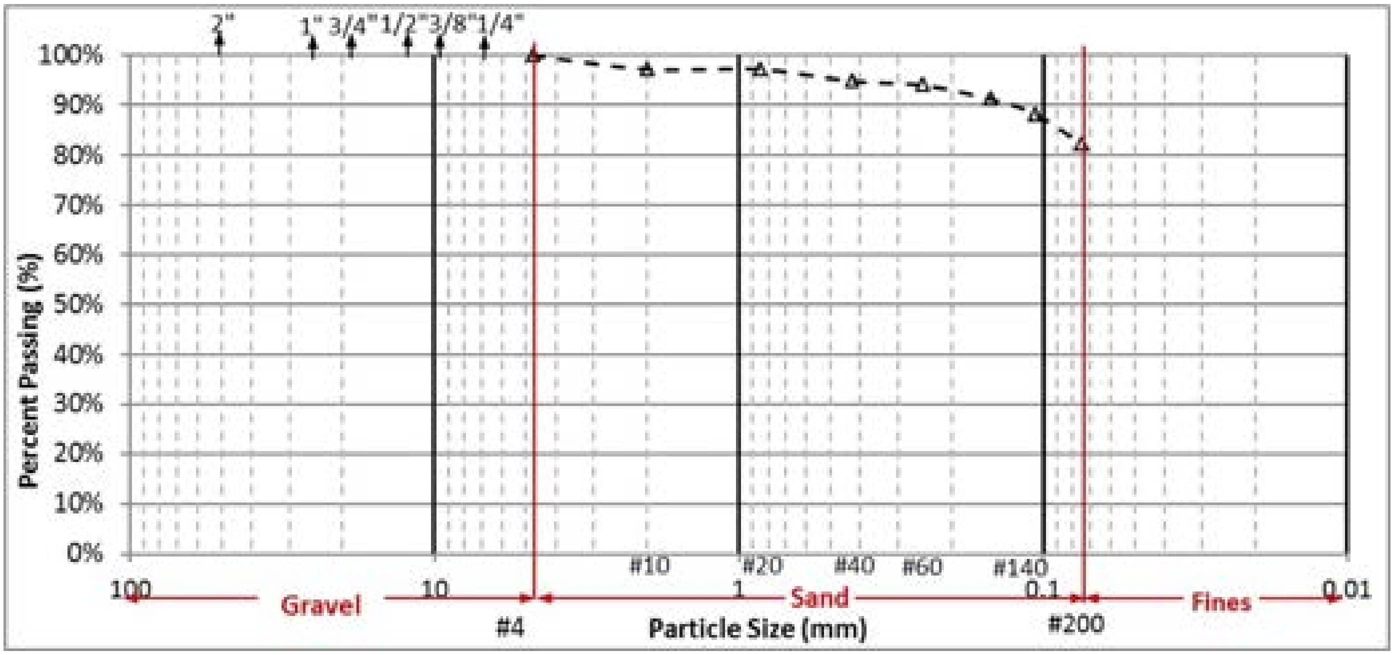


Figure 4.75: Grain Size Distribution Curve for Cook Mountain Clay Samples at Site 18

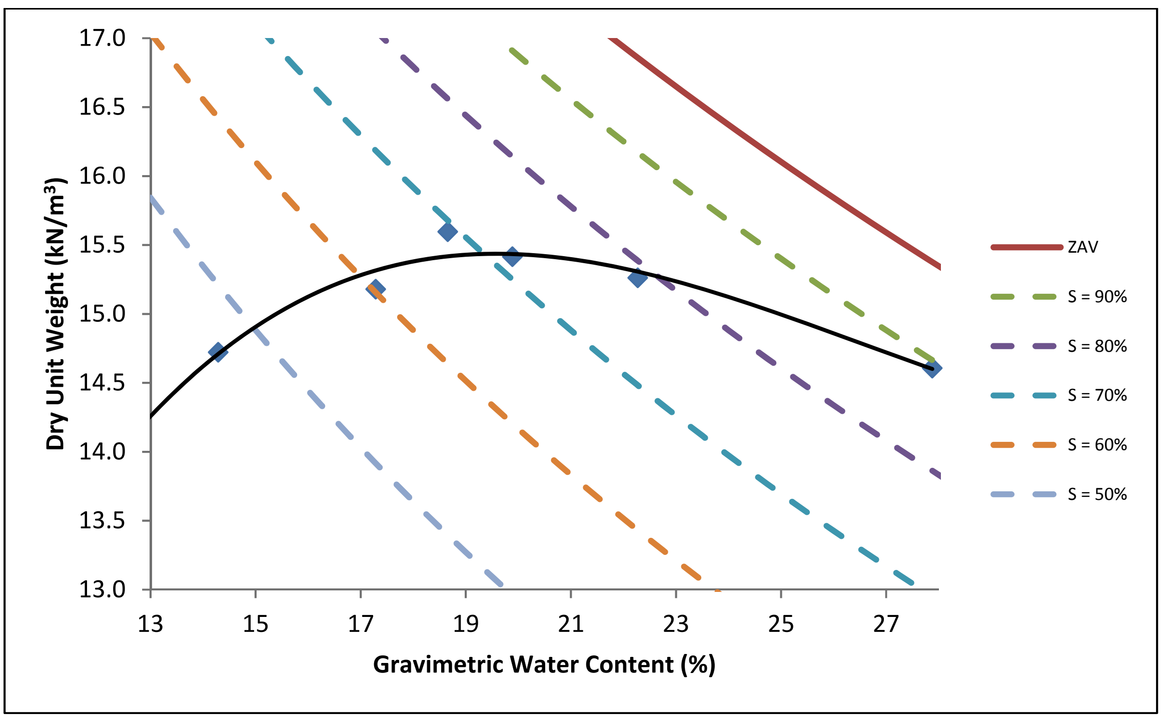


Figure 4.76: Compaction Curve using Standard Effort for the Cook Mountain Clay

4.1.19 Site 20: FM 487 [Branyon Clay, BR – 487]

This section summarizes the findings at Site 19. In February 2014, soil samples were collected from the borrow pit for the roadway expansion of FM487, west of Bartlett, TX in Williamson County as shown in Figure 2.4a. The collected soil samples included a large amount of fines, and belong to the Taylor and Navarro group, according to geologic mapping of the area.

The location of the sampled soils were known to be from an expansive deposit from previous TxDOT experience. The collected soil samples were extensively tested to identify soil characteristics and swelling properties.

<u>Location and Identification of Soil Samples</u>

The location of Site 19 corresponds to a site in the borrow pit for the expansion of FM487, just southwest of Bartlett, TX. The soil samples were collected using shovels and buckets to collect samples from the side of the borrow pit to avoid contamination of the specimens. A soil layer of grayish black, compacted, fat clay with a slight amount of gravel was encountered below the topsoil. This soil was identified as our target soil for Site 19.

The location was input into Google Earth (the resulting image is shown in Figure 4.77), and the USGS geologic overlay was used to identify the lithology of the soil. The overlay indicated that the collected soil samples belong to the Navarro and Taylor group. To complement this information, an interactive map from the USDA was used to identify the soil found at the ground surface of Site 9. The information from the USDA soil survey indicates that the soil retrieved from the site is the Branyon Clay as sown in Figure 4.78.

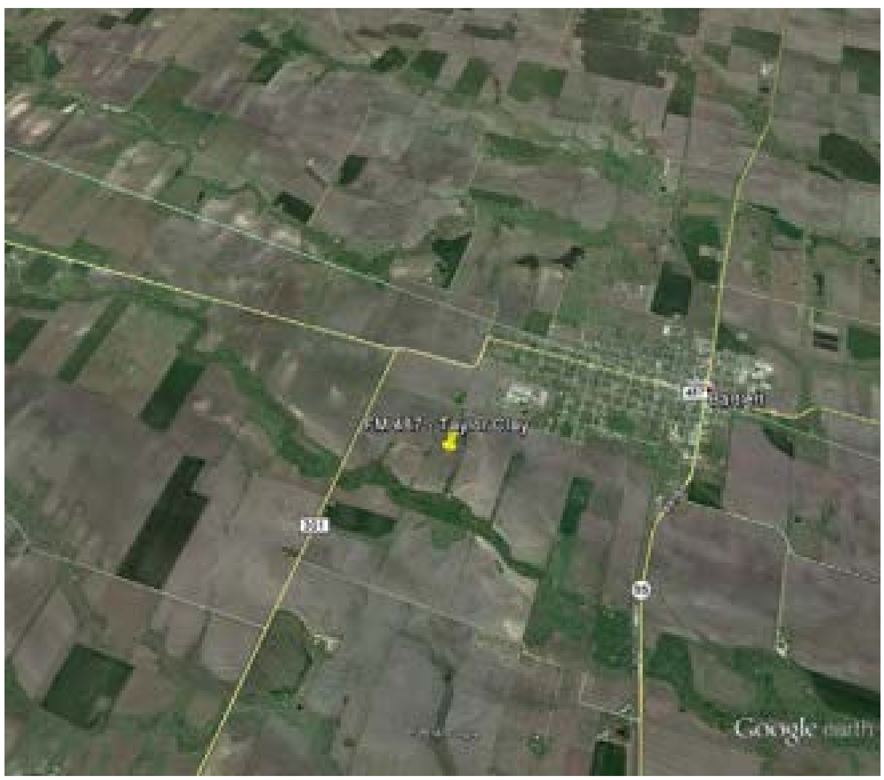


Figure 4.77: Map of Site 9 Location on SH 95 (Google 2014)

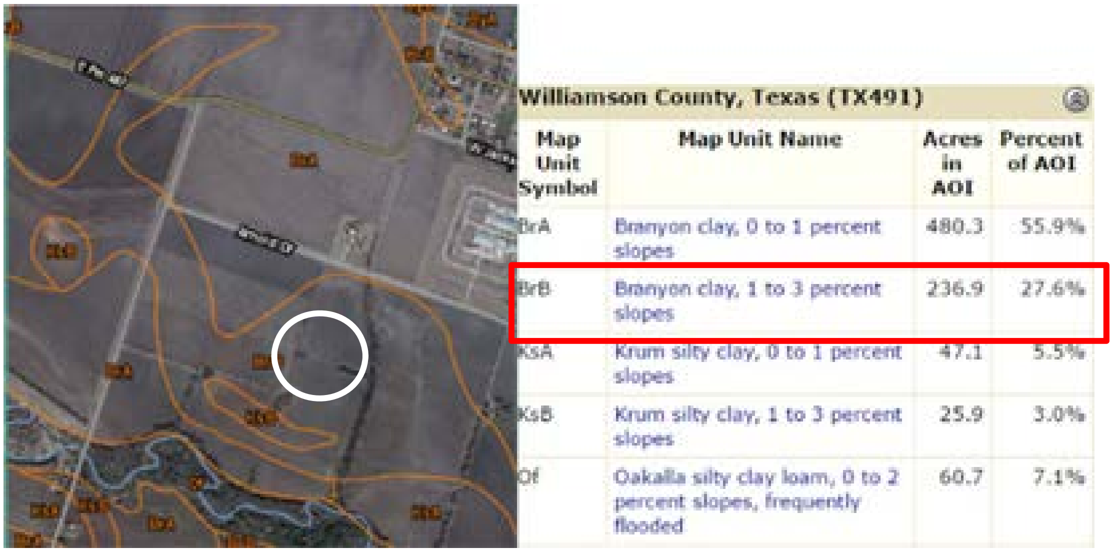


Figure 4.78: Soil Survey Map and Table at Site 19 (USDA 2013)

Characterization of Branyon Clay Samples [BR – 95]

The Branyon Clay soil was air dried and processed for the soil characterization and centrifuge tests. Atterberg Limits tests (Table 4.20) determined an average liquid limit of 60%, and an average plastic limit of 34%. These results defined the plasticity index as 2. The optimum conditions given by the Standard Proctor compaction test were measured via compaction tests following ASTM D698, as shown in Figure 4.79, and the soil was shown to have an optimum moisture content of 26.0% with a maximum dry unit weight of 14.30 kN/m^3.

Table 4.20: Results from Atterberg Limit Tests on Branyon Samples from Site 19

Averaged Liquid Limit, LL_{avg}	**52%**
Averaged Plastic Limit, PL_{avg}	**28%**
Averaged Plasticity Index, PI_{avg}	**24%**

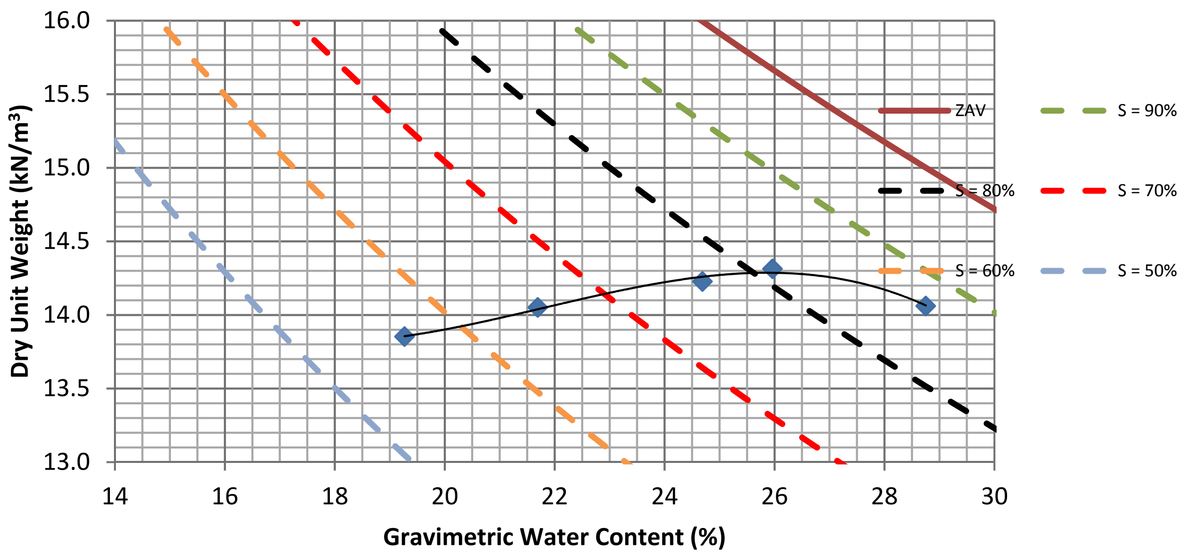


Figure 4.79: Compaction Curve using Standard Effort for the Cook Mountain Clay for the Branyon Clay at Site 19

4.1.20 Summary of Geotechnical Characterization of Sites

A summary of the general geotechnical characterization is shown in Table 4.21. A general trend is evident: the soils typically encountered in Williamson and Travis Counties have a higher plasticity index, and thus a greater range of plasticity over which the soil can be expansive, than those soils found in Bastrop and Caldwell Counties, especially for the Behring and Crockett soils.

Table 4.21: General Geotechnical Characteristics for the 18 Sites

Site #	Soil Name	County	Liquid Limit	Plasticity Index	Fines Content
1	**CR**	Travis	71	43	50%
2	**TR**	Travis	69	45	96%
3	**HB – M36**	Travis	52	28	87%
4	**HB – M127**	Travis	55	32	83%
5	**FR**	Williamson	59	34	60%
6	**HE - LF**	Williamson	63	42	85%
7	**HB - 971**	Williamson	72	47	86%
8	**BR - 972**	Williamson	66	41	44%
9	**BR - 95**	Williamson	60	26	92%
10	**HB - Taylor**	Williamson	55	32	93%
11	**BH - 535**	Bastrop	53	32	78%
12	**BH - 20**	Bastrop	50	29	82%
13	**CR – 672N**	Caldwell	40	18	72%
14	**CR – 672S**	Caldwell	54	31	85%
15	**BU**	Caldwell	62	39	90%
16	**HE – 1854W**	Caldwell	65	38	91%
17	**HE – 1854 and SH 21**	Caldwell	63	41	91%
18	**BR - 685**	Williamson	65	34	93%
19	**CM**	Bastrop	58	17	82%
20	**BR - 487**	Williamson	52	24	-

4.2 PVR Calculations and Field Performance

4.2.1 Site 1: Yett Creek Neighborhood Park [Crawford Clay, CR]

After the soil characterization and centrifuge testing program was completed on the Crawford Clay from Site 1, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. A field visit in March 2016 was also performed to determine the longitudinal cracking severity outside the outer wheel path to determine the field behavior of each subgrade.

Assumed Soil Profile

The soil samples for this site were taken within the top 1 foot; therefore, the profile had to be fully assumed. The depth of the asphalt was taken to be 4 inches with a base layer depth of 6 inches. This assumption is consistent among the sites in order to provide similar comparisons in terms of the range of stresses. The Crawford Clay was assumed to be at a dry of optimum moisture content of 24% and a relative compaction of 100%, which resulted in a dry unit weight of 92 pcf and a total unit weight of 114 pcf. The soil profile used for both methods is shown in Table 4.22.

Table 4.22: Assumed Soil Profile for Crawford Clay at Site 1

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	123	0.9
1	0	1	Crawford Clay	71	28	43	24	114	180	1.3
2	1	2							294	2.0
3	2	3							409	2.8
4	3	4							523	3.6
5	4	5							637	4.4
6	5	6							751	5.2
7	6	7							865	6.0
8	7	8							979	6.8
9	8	9							1093	7.6
10	9	10							1207	8.4
*Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer										

PVR Calculations

The soil conditions for the centrifuge testing program on the Crawford Clay from Site 1 included an initial moisture content of 24% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from three centrifuge samples and three free swell samples were input into the DMS-C spreadsheet, yielding the results shown in Figure 4.80.

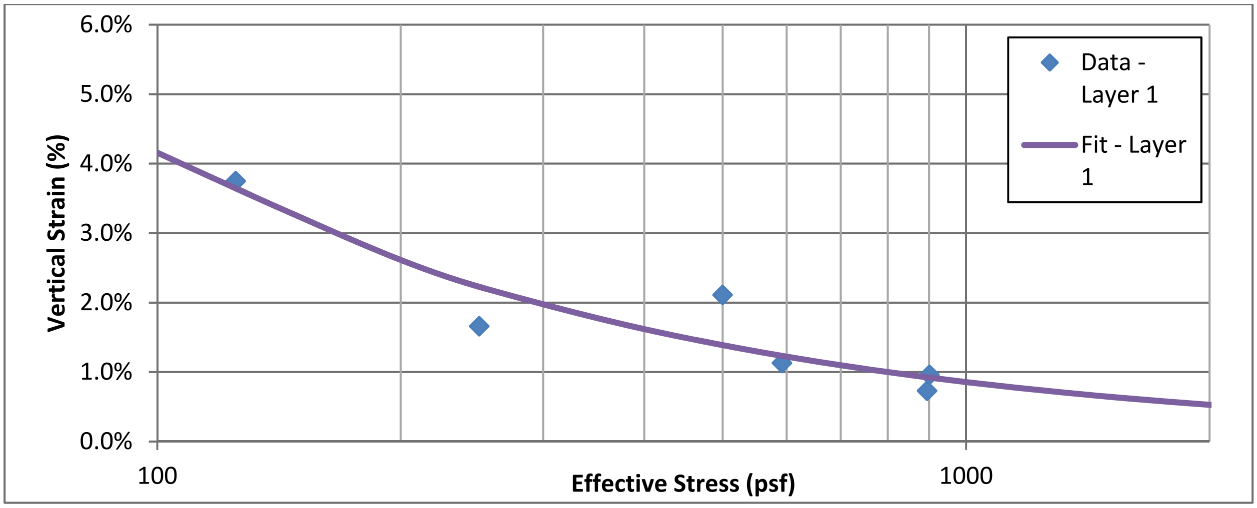


Figure 4.80: Swelling Results and Curve Fitting for Site 1

From the figure, it becomes clear that the soil tested in the centrifuge was moderately expansive with the potential for swelling occurring even at very high stresses.

For the Tex-124-E method, the soil profile from Table 3.1 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 96% of the soil passing the No. 40 sieve, as determined from the Wet Sieve tests, was assumed. Also assumed were the dry conditions for the tests, which corresponded to a moisture content of 24.6% from the correlations in Tex-124-E. The sample unit weights as determined from Equations 2.1 and 2.2 were used, giving a density correction of 0.85 and a modified No. 40 factor of 0.96 for the sample. The inputs used for the PVR calculations are shown in Table 4.23.

Table 4.23: PVR Input Parameters for Tex-124-E for Site 1

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		173	1.2	-	-		-	-
2	1	CR	385.5	2.7	71	25.3	106	96.0	43
4	1	CR	597.6	4.2	71	25.3	106	96.0	43
6	1	CR	809.8	5.6	71	25.3	106	96.0	43
8	1	CR	1022.0	7.1	71	25.3	106	96.0	43
10	1	CR	1234.1	8.6	71	25.3	106	96.0	43

By integrating the curve fitted function from Figure 3.1 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1207 psf, the PVR of the subgrade was determined to be 1.53 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 1.49 in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.81, and the comparison between the cumulative PVR versus depth is shown in Table 4.24.

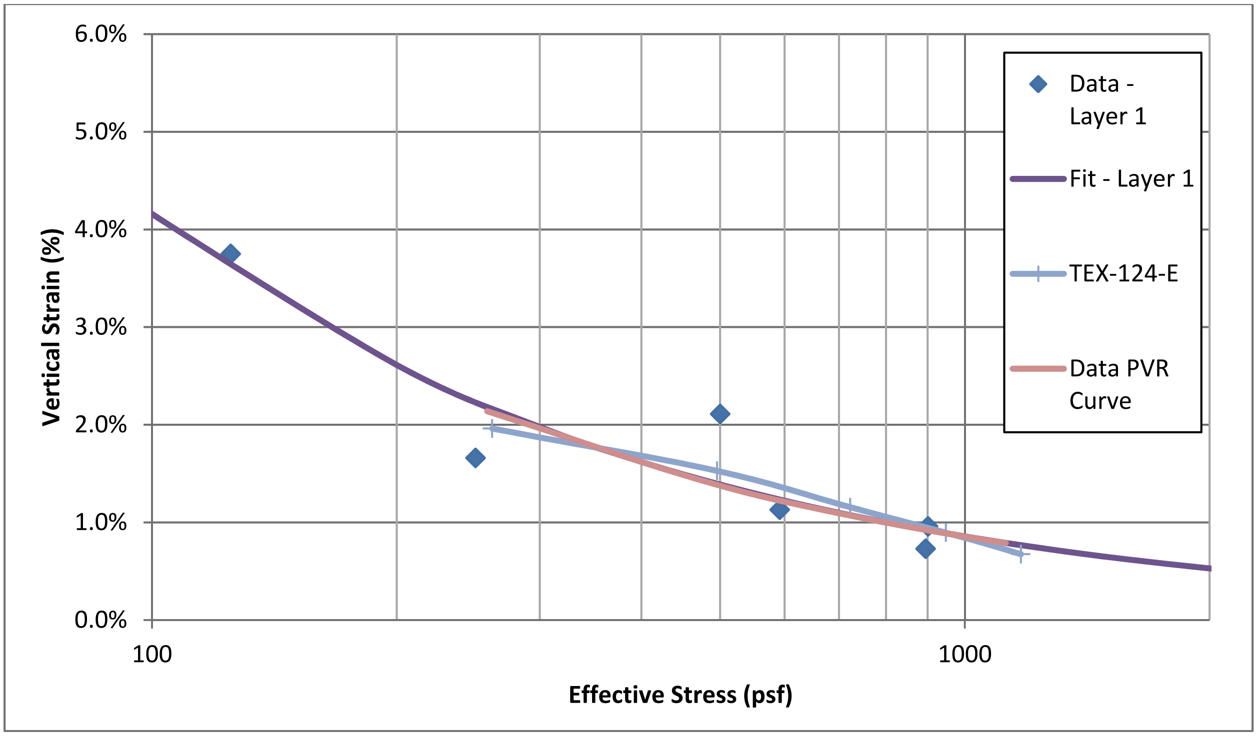


Figure 4.81: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Site 1

Table 4.24: Comparison of PVR Results for Site 1

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**1.49**	**1.53**
2.0	1	CR	262	1.02	1.01
4.0	1	CR	496	0.65	0.67
6.0	1	CR	723	0.38	0.41
8.0	1	CR	948	0.16	0.19
10.0	1	CR	1172	0.00	0.00

Based upon both the centrifuge testing of the Crawford Clay specimens and the Tex-124-E results, the site is considered to rest on an expansive subgrade with remediation techniques necessary. A site visit was performed in March 2016 near the location sampled in order to determine the amount of longitudinal cracking outside of the outer wheel path. The location that was surveyed for this particular condition is shown in Figure 4.82.

Figure 4.82: Condition Survey Location for Site 1

The road is within a neighborhood, and extensive cracking was found along the outside edge of the pavement. A length of 112 ft of roadway was surveyed, and the amount of longitudinal cracking was determined to be approximately 76 ft per 100 ft of roadway with the cracks ranging from 3 mm to 15 mm in width. This amount of cracking is very extensive and with the amount of sealing and cracking, the roadway would be considered to have environmental cracking from an expansive subgrade.

4.2.2 Site 2: Greenlawn Boulevard and IH 35 [Tinn Clay, TN]

After the soil characterization and centrifuge testing program was completed on the Tinn Clay from Site 2, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. A field visit in March 2016 was also performed to determine the longitudinal cracking severity outside of the outer wheel path to determine the field behavior of each subgrade.

Assumed Soil Profile

The soil samples for this site were taken within the top 1 foot; therefore, the profile had to be fully assumed. The depth of the asphalt was taken to be 4 inches with a base layer depth of 6 inches. This assumption is consistent among the sites in order to provide similar comparisons in terms of the range of stresses. The Tinn Clay was assumed to be at a dry of optimum moisture content of 25% and a relative compaction of 100%, which resulted in a dry unit weight of 86 pcf and a total unit weight of 108 pcf. The soil profile used for both methods is shown in Table 4.25.

Table 4.25: Assumed Soil Profile for Tinn Clay at Site 2

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	123	0.9
1	0	1	Tinn Clay	67	30	37	25	108	177	1.2
2	1	2							285	2.0
3	2	3							392	2.7
4	3	4							500	3.5
5	4	5							607	4.2
6	5	6							715	5.0
7	6	7							822	5.7
8	7	8							930	6.5
9	8	9							1037	7.2
10	9	10							1145	8.0
***Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer**										

PVR Calculations

The soil conditions for centrifuge testing program on the Tinn Clay from Site 2 included an initial moisture content of 25% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from three centrifuge samples and one free swell sample were input into the DMS-C spreadsheet, yielding the results shown in Figure 4.83.

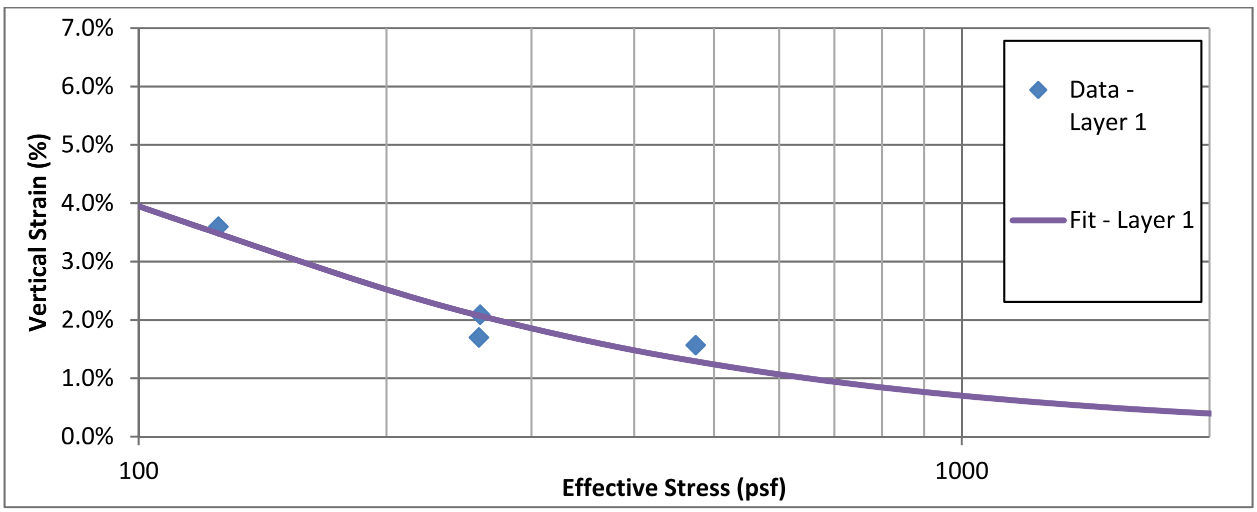


Figure 4.83: Swelling Results and Curve Fitting for Site 2

From the figure, it becomes clear that the soil tested in the centrifuge was moderately expansive with the potential for swelling occurring even at very high stresses.

For the Tex-124-E method, the soil profile from Table 4.25 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 98% of the soil passing the No. 40 sieve, as determined from the Wet Sieve tests, was assumed as well as dry conditions for the tests, which corresponded to a moisture content of 23.7% from the correlations in Tex-124-E. The sample unit weights as determined from equations 1 and 2 were

used, giving a density correction of 0.86 and a modified No. 40 factor of 0.98 for the sample. The inputs used for the PVR calculations are shown in Table 4.26.

Table 4.26: PVR Input Parameters for Tex-124-E for Site 2

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		173	1.2	-	-		-	-
2	1	TN	262	1.8	67	24.8	108	100.0	37
4	1	TN	496	3.4	67	24.8	108	100.0	37
6	1	TN	723	5.0	67	24.8	108	100.0	37
8	1	TN	948	6.6	67	24.8	108	100.0	37
10	1	TN	1172	8.1	67	24.8	108	100.0	37

By integrating the curve fitted function from Figure 3.4 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1145 psf, the PVR of the subgrade was determined to be 1.34 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 1.17 in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.84, and the comparison between the cumulative PVR versus depth is shown in Table 4.27.

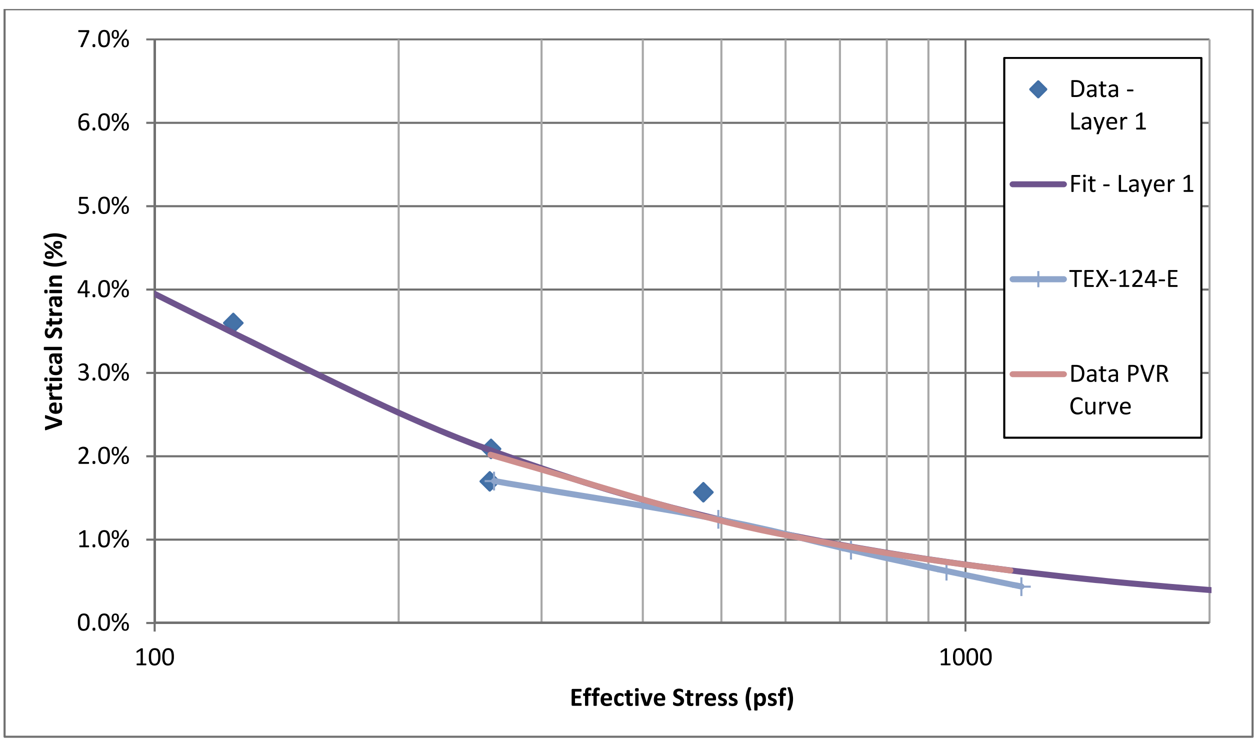


Figure 4.84: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Site 2

Table 4.27: Comparison of PVR Results for Site 2

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**1.17**	**1.34**
2.0	1	TN	262	0.76	0.86
4.0	1	TN	496	0.46	0.56
6.0	1	TN	723	0.25	0.33
8.0	1	TN	948	0.10	0.15
10.0	1	TN	1172	0.00	0.00

Based upon both the centrifuge testing of the Tinn Clay specimens and the Tex-124-E results, the site is considered to rest on an expansive subgrade with remediation techniques necessary. A site visit was performed in March 2016 near the location sampled in order to determine the amount of longitudinal cracking outside of the outer wheel path. The location that was surveyed for this particular condition is shown in Figure 4.85.

Figure 4.85: Condition Survey Location for Site 2

The road is within a neighborhood, and extensive cracking was found along the outside edge of the pavement. A length of 100 ft of roadway was surveyed, and the amount of longitudinal cracking was determined to be approximately 31 ft per 100 ft of roadway with the cracks ranging from 6 to 7 mm in width. This amount of cracking in this short period of road is fairly extensive, and additional remediation techniques at this site are recommended.

4.2.3 Sites 3 and 4: Manor Retaining Wall Site [Houston Black, HB – M36 and HB – M127]

After the soil characterization and centrifuge testing program was completed on the Houston Black clay specimens taken from the Manor Retaining Wall at two separate depths, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. No field visit was performed to determine the longitudinal cracking at the site as there is no pavement structure near the sampled location. This site's PVR was completed to add to the database as well as examine how the PVR of a site varies with further reconstituted specimens at depth.

Assumed Soil Profile

The soil samples from this site were taken at two separate depths of approximately 3 ft and 11 ft. The assumed pavement structure used for PVR calculations had an asphalt depth of 4 inches and a base layer 6 inches. This assumption is consistent among the sites in order to provide a similar comparison between sites in terms of the range of stresses. The Houston Black Clay was assumed to be at a dry of optimum moisture contents of 20% for M36 and 21% for M127 and a relative compaction of 100%, which resulted in a dry unit weight of 96 pcf and a total unit weight of 115 pcf for M36 and a dry unit weight of 95 pcf and a total unit weight of 114 pcf for M127. The soil profile used for both methods is shown in Table 4.28.

Table 4.28: Assumed Soil Profile for Houston Black Clay at Sites 3 and 4

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	123	0.9
1	0	1	Houston Black Clay	52	24	28	20	115	181	1.3
2	1	2							296	2.1
3	2	3							411	2.9
4	3	4	Houston Black Clay	55	23	32	21	114	526	3.7
5	4	5							640	4.4
6	5	6							754	5.2
7	6	7							868	6.0
8	7	8							982	6.8
9	8	9							1096	7.6
10	9	10							1210	8.4
*Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer										

PVR Calculations

The soil conditions for centrifuge testing program on the Houston Black Clay from Site 3 included an initial moisture content of 20% and a relative compaction of 100% and Houston Black Clay from Site 4 included an initial moisture content of 21% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from three centrifuge samples from Site 3 and four centrifuge specimens from Site 4 were input into the DMS-C spreadsheet, yielding the results shown in Figure 4.86.

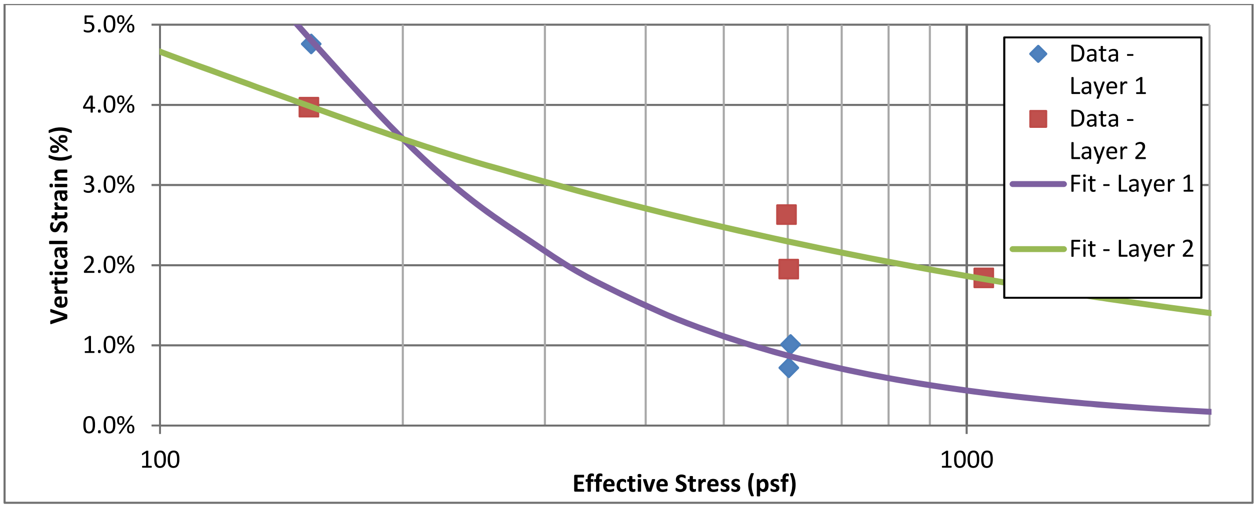


Figure 4.86: Swelling Results and Curve Fitting for Sites 3 and 4

From the figure, it becomes clear that the soil tested in the centrifuge was moderately expansive with the potential for swelling occurring even at very high stresses.

For the Tex-124-E method, the soil profile from Table 3.7 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 93% of the soil passing the No. 40 sieve, as determined from the Wet Sieve tests, was assumed as well as dry conditions for the tests, which corresponded to a moisture content of 20.4% from the correlations in Tex-124-E. The sample unit weights as determined from equations 1 and 2 were used, giving a binder correction of 0.92and a modified density factor of 0.98 for the sample from Site 3. For Site 4, 95% of the soil passed through the No. 40 Sieve with the dry condition being 20.7% from the correlations given in Tex-124-E. The soil binder correction was 0.95 and modified density factor was 0.91 for Site 4. Table 4.29 provides the inputs used for the PVR calculations.

Table 4.29: PVR Input Parameters for Tex-124-E for Sites 3 and 4

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		173	1.2	-	-		-	-
2	1	M36	262	1.8	52	20.3	115	100.0	28
4	1	M36	496	3.4	52	20.3	115	100.0	28
6	1	M36	723	5.0	52	20.3	115	100.0	28
8	2	M127	948	6.6	55	20.7	114	70.0	32
10	2	M127	1172	8.1	55	20.7	114	70.0	32

By integrating the curve fitted function from Figure 4.86 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1210 psf, the PVR of the subgrade was determined to be 1.87 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 0.70 in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.87, and the comparison between the cumulative PVR versus depth is shown in Table 4.30.

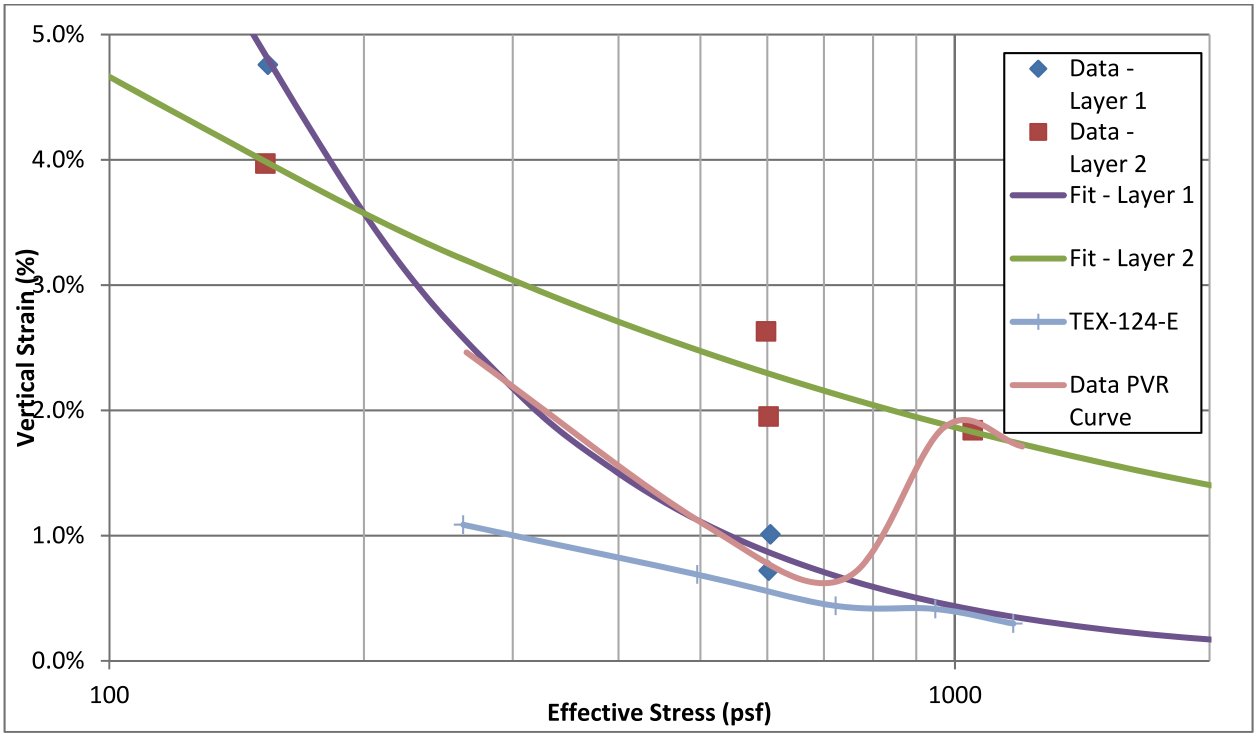


Figure 4.87: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Sites 3 and 4

Table 4.30: Comparison of PVR Results for Sites 3 and 4

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**0.70**	**1.87**
2.0	1	M36	262	0.44	1.28
4.0	1	M36	496	0.28	1.02
6.0	1	M36	723	0.17	0.86
8.0	2	M127	948	0.07	0.41
10.0	2	M127	1172	0.00	0.00

Based upon both the centrifuge testing of the Houston Black Clay specimens and the Tex-124-E results, the site is considered to rest on an expansive subgrade with remediation techniques necessary. The results from these sites indicate that the sampling of reconstituted specimens at depth of the same soil deposit may not be necessary. Also, the site indicated that the original Houston Black Clay specimen from previous TxDOT projects was not contaminated when sampled.

4.2.4 Site 5: SH 45 and MoPac Interchange [Fairlie Clay, FR]

After the soil characterization and centrifuge testing program was completed on the Fairlie Clay from Site 5, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. A field visit in March 2016 was also performed to determine the longitudinal cracking severity outside of the outer wheel path to determine the field behavior of each subgrade.

<u>Assumed Soil Profile</u>

The soil samples that were taken for this site did not extend beyond a depth of a foot, therefore the profile had to be fully assumed. The depth of the asphalt was taken to be 4 inches with the depth of the base layer being 6 inches. This result is consistent among the sites in order to provide a similar comparison between sites in terms of the range of stresses. The Fairlie Clay was assumed to be at a dry of optimum moisture content of 22% and a relative compaction of 100%, which resulted in a dry unit weight of 92 pcf and a total unit weight of 112 pcf. The soil profile used for both methods is shown in Table 4.31.

Table 4.31: Assumed Soil Profile for Fairlie Clay at Site 5

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	173	1.2
1	0	1	Fairlie Clay	59	25	34	22	112	229	1.6
2	1	2							341	2.4
3	2	3							453	3.1
4	3	4							565	3.9
5	4	5							677	4.7
6	5	6							789	5.5
7	6	7							901	6.3
8	7	8							1013	7.0
9	8	9							1126	7.8
10	9	10							1238	8.6
*Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer										

<u>PVR Calculations</u>

The soil conditions for centrifuge testing program on the Fairlie Clay from Site 5 included an initial moisture content of 22% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from four centrifuge samples and three free swell samples were input into the DMS-C spreadsheet and shown in Figure 4.88.

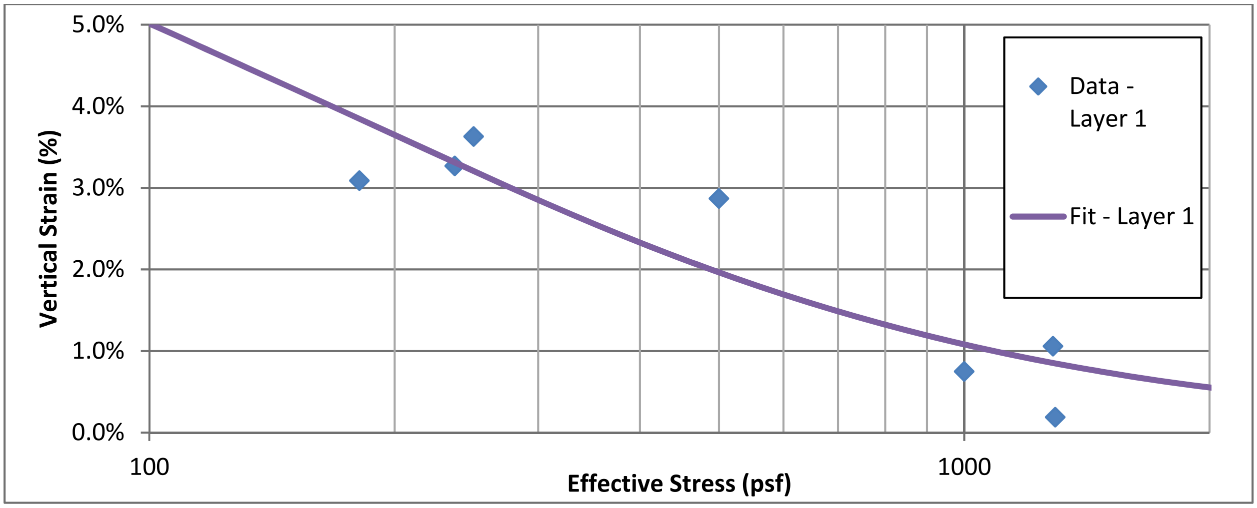


Figure 4.88: Swelling Results and Curve Fitting for Site 5

From the figure, it becomes clear that the soil tested in the centrifuge was moderately expansive with the potential for swelling occurring even at very high stresses.

For the Tex-124-E method, the soil profile from Table 3.4 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 95% of the soil passing the No. 40 sieve, as determined from the Wet Sieve tests, was assumed as well as dry conditions for the tests, which corresponded to a moisture content of 22.0% from the correlations in Tex-124-E. The sample unit weights as determined from equations 1 and 2 were used, giving a density correction of 0.90 and a modified No. 40 factor of 0.95 for the sample. The inputs used for the PVR calculations are shown in Table 4.32.

Table 4.32: PVR Input Parameters for Tex-124-E for Site 5

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		173	1.2	-	-		-	-
2	1	FR	262	1.8	59	22.0	112	100.0	34
4	1	FR	496	3.4	59	22.0	112	100.0	34
6	1	FR	723	5.0	59	22.0	112	100.0	34
8	1	FR	948	6.6	59	22.0	112	100.0	34
10	1	FR	1172	8.1	59	22.0	112	100.0	34

By integrating the curve fitted function from Figure 4.88 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1188 psf, the PVR of the subgrade was determined to be 2.03 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 1.01 in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.89, and the comparison between the cumulative PVR versus depth is shown in Table 4.33.

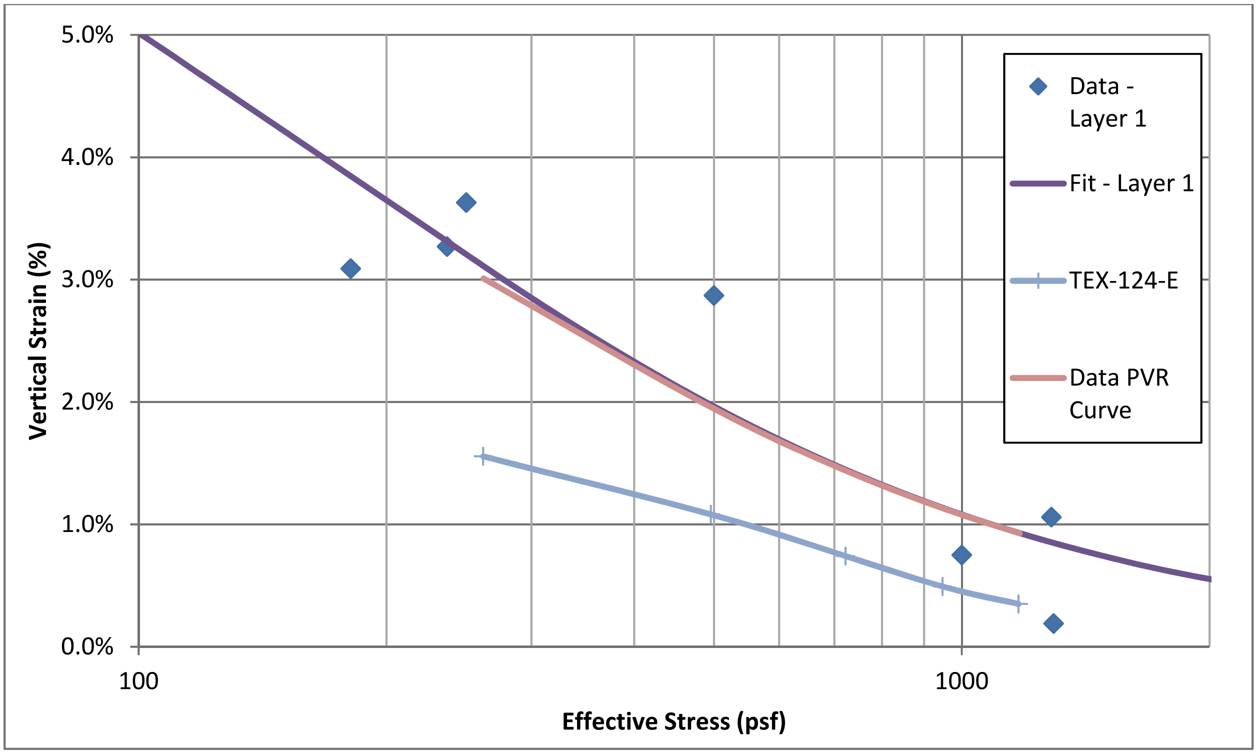


Figure 4.89: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Site 5

Table 4.33: Comparison of PVR Results for Site 5

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**1.01**	**2.03**
2.0	1	FR	262	0.64	1.31
4.0	1	FR	496	0.38	0.84
6.0	1	FR	723	0.20	0.49
8.0	1	FR	948	0.08	0.22
10.0	1	FR	1172	0.00	0.00

Based upon both the centrifuge testing of the Fairlie Clay specimens and the Tex-124-E results, the site is considered to rest on an expansive subgrade with remediation techniques necessary. A site visit was performed in March 2016 near the location sampled in order to determine the amount of longitudinal cracking outside of the outer wheel path. The location that was surveyed for this particular condition is shown in Figure 4.90.

Figure 4.90: Condition Survey Location for Site 5

The road is next to a major frontage road from the toll road SH 45. A length of 150 ft of roadway was surveyed, and the amount of longitudinal cracking was determined to be approximately 0 ft per 100 ft of roadway. This lack of cracking is likely due to remedial techniques used during construction at this location, and the relatively new placement of the pavement structure.

4.2.5 Site 6: SH 45 and La Frontera Boulevard [Heiden Clay, HE - LF]

After the soil characterization and centrifuge testing program was completed on the Heiden Clay from Site 6, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. A field visit in March 2016 was also performed to determine the longitudinal cracking severity outside of the outer wheel path to determine the field behavior of each subgrade.

Assumed Soil Profile

The soil samples that were taken for this site did not extend beyond a depth of a foot, therefore the profile had to be fully assumed. The depth of the asphalt was taken to be 4 inches with the depth of the base layer being 6 inches. This result is consistent among the sites in order to provide a similar comparison between sites in terms of the range of stresses. The Heiden Clay was assumed to be at a dry of optimum moisture content of 20% and a relative compaction of 100%, which resulted in a dry unit weight of 95 pcf and a total unit weight of 114 pcf. The soil profile used for both methods is shown in Table 4.34.

Table 4.34: Assumed Soil Profile for Heiden Clay at Site 6

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	123	0.9
1	0	1	Heiden Clay	55	21	34	20	114	181	1.3
2	1	2							295	2.0
3	2	3							409	2.8
4	3	4							524	3.6
5	4	5							638	4.4
6	5	6							752	5.2
7	6	7							867	6.0
8	7	8							981	6.8
9	8	9							1096	7.6
10	9	10							1210	8.4
***Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer**										

PVR Calculations

The soil conditions for centrifuge testing program on the Heiden Clay from Site 5 included an initial moisture content of 20% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from seven centrifuge samples were input into the DMS-C spreadsheet, yielding the results shown in Figure 4.91.

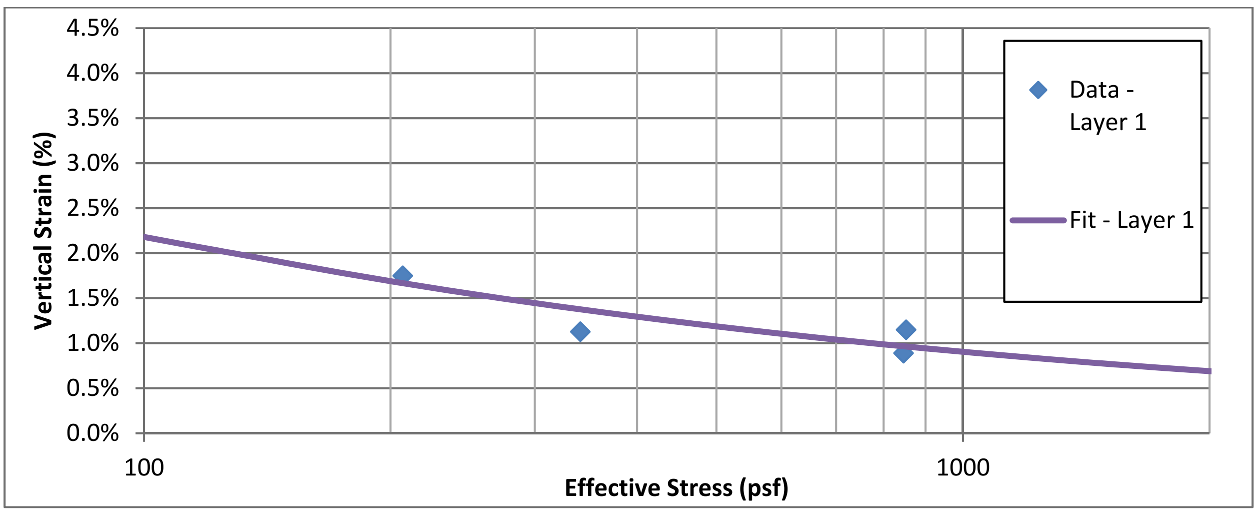


Figure 4.91: Swelling Results and Curve Fitting for Site 6

From the figure, it becomes clear that the soil tested in the centrifuge was moderately expansive with the potential for swelling occurring even at very high stresses.

For the Tex-124-E method, the soil profile from Table 4.34 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 94% of the soil passing the No. 40 sieve, as determined from the Wet Sieve tests, was assumed as well as dry conditions for the tests, which corresponded to a moisture content of 20.0% from the correlations in Tex-124-E. The sample unit weights as determined from equations 1 and 2 were

used, giving a density correction of 0.92 and a modified No. 40 factor of 0.94 for the sample. The inputs used for the PVR calculations are shown in Table 4.35.

Table 4.35: PVR Input Parameters for Tex-124-E for Site 6

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		173	1.2	-	-		-	-
2	1	HE	262	1.8	55	20.3	114	100.0	34
4	1	HE	496	3.4	55	20.3	114	100.0	34
6	1	HE	723	5.0	55	20.3	114	100.0	34
8	1	HE	948	6.6	55	20.3	114	100.0	34
10	1	HE	1172	8.1	55	20.3	114	100.0	34

By integrating the curve fitted function from Figure 3.12 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1210 psf, the PVR of the subgrade was determined to be 1.31 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 1.03 in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.92, and the comparison between the cumulative PVR versus depth is shown in Table 4.36.

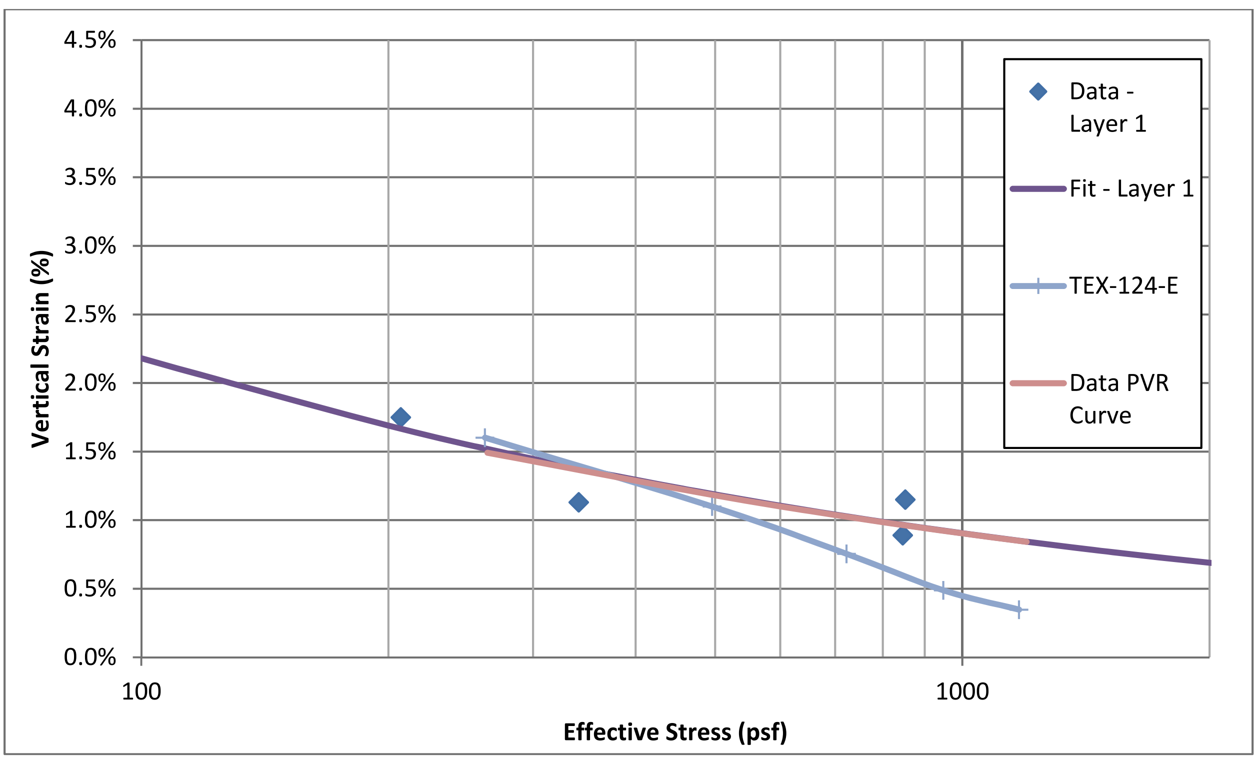


Figure 4.92: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Site 6

Table 4.36: Comparison of PVR Results for Site 6

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**1.03**	**1.31**
2.0	1	HE	262	0.65	0.95
4.0	1	HE	496	0.38	0.67
6.0	1	HE	723	0.20	0.42
8.0	1	HE	948	0.08	0.20
10.0	1	HE	1172	0.00	0.00

Based upon both the centrifuge testing of the Heiden Clay specimens and the Tex-124-E results, the site is considered to rest on an expansive subgrade with remediation techniques necessary. A site visit was performed in March 2016 near the location sampled in order to determine the amount of longitudinal cracking outside of the outer wheel path. The location that was surveyed for this particular condition is shown in Figure 4.93.

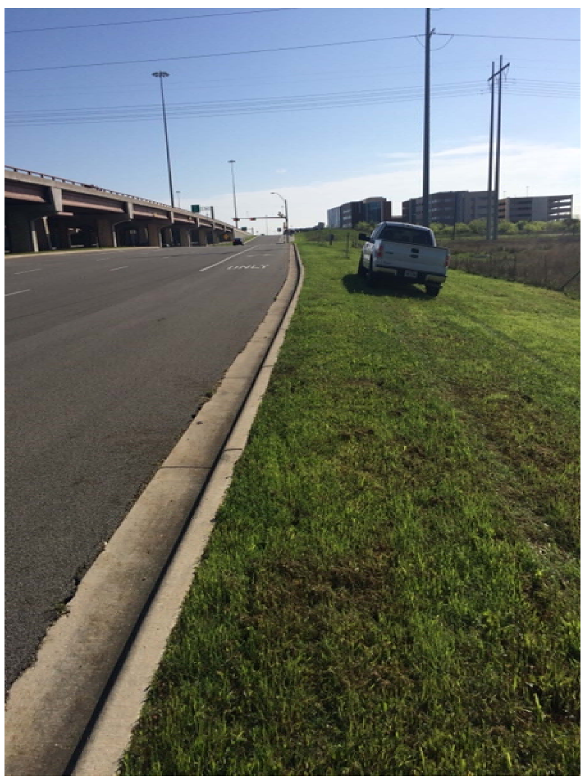


Figure 4.93: Condition Survey Location for Site 6

The road is next to a major frontage road from the toll road, SH 45. A length of 150 ft of roadway was surveyed, and the amount of longitudinal cracking was determined to be approximately 0 ft per 100 ft of roadway. This lack of cracking is likely due to remedial techniques used during construction at this location, and the relatively new placement of the pavement structure. However, one thing to note is that there is a 5-in. deep crack, shown in Figure 4.94, between the concrete curb and the pavement structure itself. However, the crack's cause—whether from construction of the roadway itself or the expansive subgrade beneath the pavement structure—is undetermined.

Figure 4.94: Gap between Curb and Asphalt at Site 6

4.2.6 Site 7: FM 971 [Houston Black Clay, HB - 971]

After the soil characterization and centrifuge testing program was completed on the Houston Black Clay from Site 7, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. A field visit in March 2016 was also performed to determine the longitudinal cracking severity outside of the outer wheel path to determine the field behavior of each subgrade.

<u>Assumed Soil Profile</u>

The soil samples from this site were taken were taken within the top 3 feet. The assumed pavement structure used for PVR calculations had an asphalt depth of 4 inches and a base layer of 6 inches. This assumption is consistent among the sites in order to provide a similar comparison between sites in terms of the range of stresses. The Houston Black Clay was assumed to be at a dry of optimum moisture content of 22% and a relative compaction of 100%, which resulted in a dry unit weight of 85 pcf and a total unit weight of 106 pcf. The soil profile used for both methods is shown in Table 4.37.

Table 4.37: Assumed Soil Profile for Houston Black Clay at Site 7

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	123	0.9
1	0	1	Houston Black Clay	72	25	47	25	106	176	1.2
2	1	2							283	2.0
3	2	3							389	2.7
4	3	4							495	3.4
5	4	5							601	4.2
6	5	6							707	4.9
7	6	7							814	5.7
8	7	8							920	6.4
9	8	9							1026	7.1
10	9	10							1132	7.9
***Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer**										

PVR Calculations

The soil conditions for centrifuge testing program on the Houston Black Clay from Site 7 included an initial moisture content of 25% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from five centrifuge samples and one free swell sample were input into the DMS-C spreadsheet, yielding the results shown in Figure 4.95.

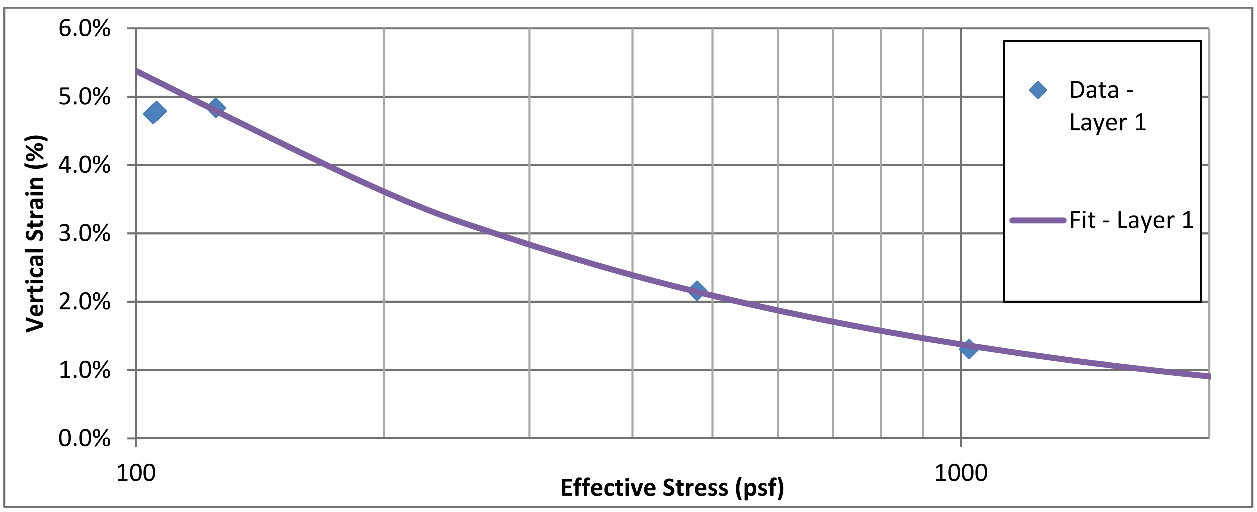


Figure 4.95: Swelling Results and Curve Fitting for Site 7

From the figure, it becomes clear that the soil tested in the centrifuge was moderately expansive with the potential for swelling occurring even at very high stresses.

For the Tex-124-E method, the soil profile from Table 3.16 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 97% of the soil passing the No. 40 sieve, as determined from the Wet Sieve tests, was assumed as well as dry conditions for the tests, which corresponded to a moisture content of 24.8% from the correlations in Tex-124-E. The sample unit weights as determined from equations 1 and 2 were

used, giving a density correction of 0.85 and a modified No. 40 factor of 0.97 for the sample. The inputs used for the PVR calculations are shown in Table 4.38.

Table 4.38: PVR Input Parameters for Tex-124-E for Site 7

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		173	1.2	-	-		-	-
2	1	HB	259	1.8	72	24.9	106	100.0	47
4	1	HB	480	3.3	72	24.9	106	100.0	47
6	1	HB	696	4.8	72	24.9	106	100.0	47
8	1	HB	911	6.3	72	24.9	106	100.0	47
10	1	HB	1124	7.8	72	24.9	106	100.0	47

By integrating the curve fitted function from Figure 4.95 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1132 psf, the PVR of the subgrade was determined to be 2.30 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 1.76 in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.96, and the comparison between the cumulative PVR versus depth is shown in Table 4.39.

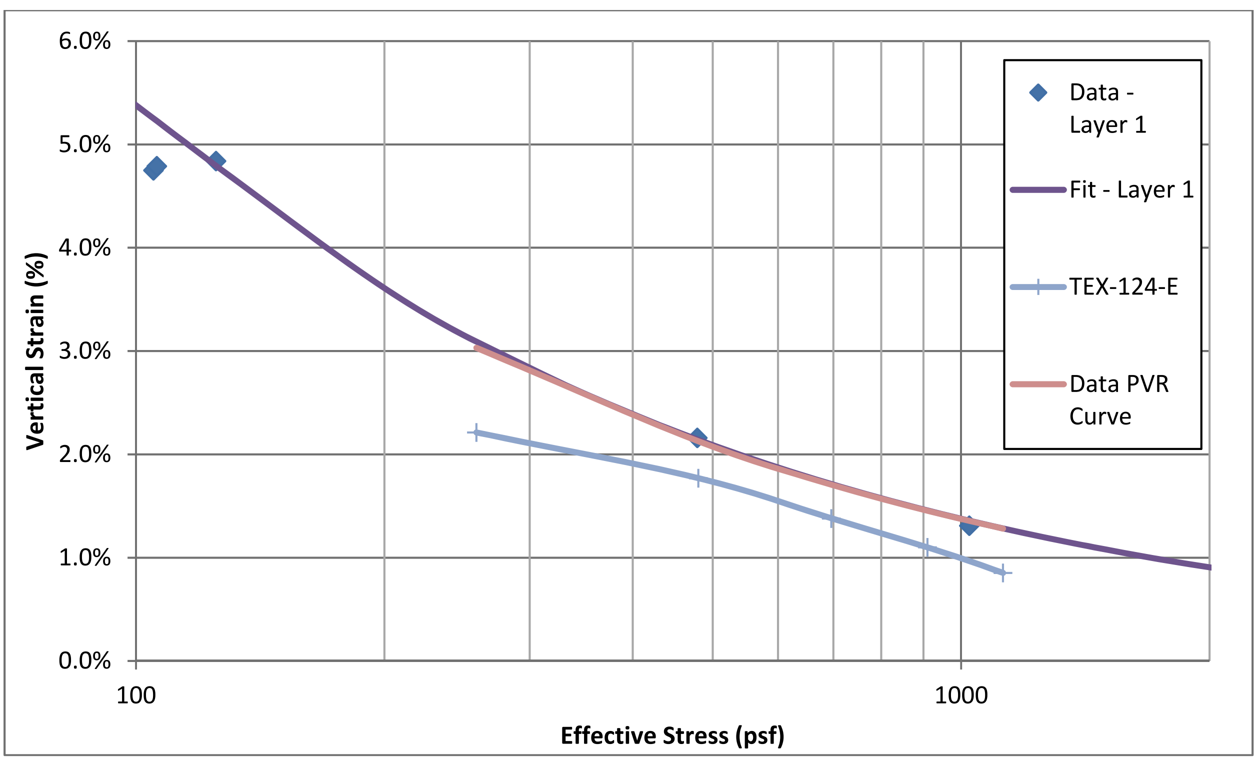


Figure 4.96: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Site 7

Table 4.39: Comparison of PVR Results for Site 7

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**1.76**	**2.30**
2.0	1	HB	259	1.22	1.58
4.0	1	HB	480	0.80	1.07
6.0	1	HB	696	0.47	0.66
8.0	1	HB	911	0.20	0.31
10.0	1	HB	1124	0.00	0.00

Based upon both the centrifuge testing of the Houston Black Clay specimens and the Tex-124-E results, the site is considered to rest on an expansive subgrade with remediation techniques necessary. A site visit was performed in March 2016 near the location sampled in order to determine the amount of longitudinal cracking outside of the outer wheel path. The location that was surveyed for this particular condition is shown in Figure 4.97.

Figure 4.97: Condition Survey Location for Site 7

The road is located just east of Weir, TX, and extensive cracking was found along the outside edge of the pavement. A length of 115 ft of roadway was surveyed, and the amount of longitudinal cracking was determined to be approximately 43 ft per 100 ft of roadway with the cracks ranging from 3 mm to 5 mm in width. This amount of cracking is extensive and based upon testing and field performance, the roadway is determined to have issues with environmental cracking from the expansive subgrade.

4.2.7 Site 8: FM 972 [Branyon Clay, BR - 972]

After the soil characterization and centrifuge testing program was completed on the Branyon Clay from Site 8, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. A field visit in March 2016 was also performed to determine the longitudinal cracking severity outside of the outer wheel path to determine the field behavior of each subgrade.

Assumed Soil Profile

The soil samples from this site were taken were taken within the top 3 feet. The assumed pavement structure used for PVR calculations had an asphalt depth of 4 inches and a base layer of 6 inches. This assumption is consistent among the sites in order to provide a similar comparison between sites in terms of the range of stresses. The Branyon Clay was assumed to be at a dry of optimum moisture content of 23% and a relative compaction of 100%, which resulted in a dry unit weight of 89 pcf and a total unit weight of 109 pcf. The soil profile used for both methods is shown in Table 4.40.

Table 4.40: Assumed Soil Profile for Branyon Clay at Site 8

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	173	1.2
1	0	1	Branyon Clay	65	25	40	23	109	228	1.6
2	1	2							337	2.3
3	2	3							447	3.1
4	3	4							556	3.9
5	4	5							666	4.6
6	5	6							775	5.4
7	6	7							884	6.1
8	7	8							994	6.9
9	8	9							1103	7.7
10	9	10							1212	8.4
***Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer**										

PVR Calculations

The soil conditions for centrifuge testing program on the Branyon Clay from Site 8 included an initial moisture content of 25% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from eleven centrifuge samples were input into the DMS-C spreadsheet, yielding the results shown in Figure 4.98.

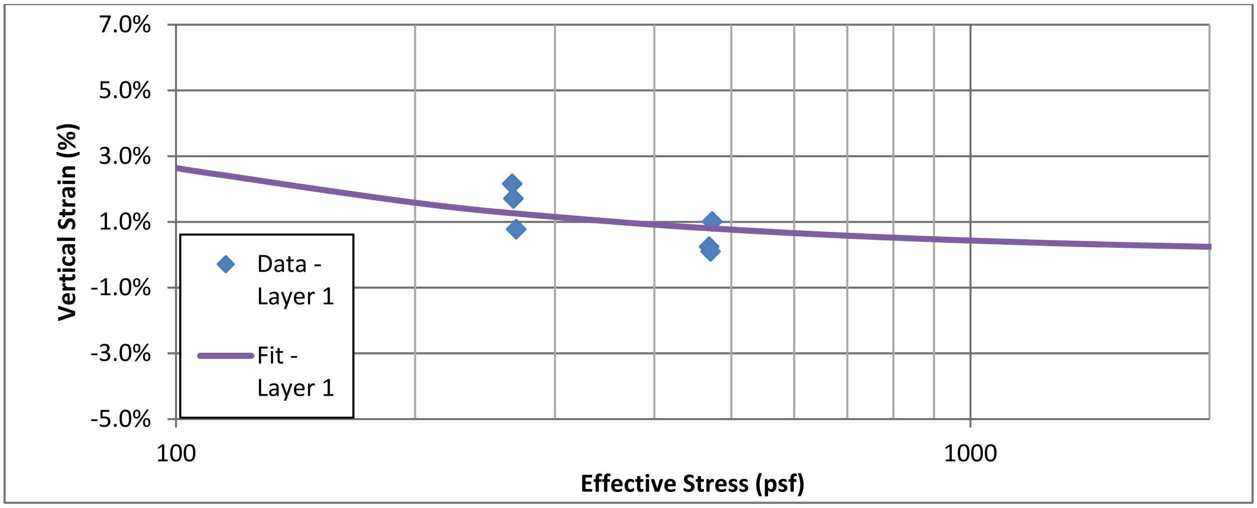


Figure 4.98: Swelling Results and Curve Fitting for Site 8

From the figure, it becomes clear that the soil tested in the centrifuge was not considered to be expansive and actually saw collapse occurring after approximately 500 psf.

For the Tex-124-E method, the soil profile from Table 4.40 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 98% of the soil passing the No. 40 sieve, as determined from the Wet Sieve tests, was assumed as well as dry conditions for the tests, which corresponded to a moisture content of 23.3% from the correlations in Tex-124-E. The sample unit weights as determined from equations 1 and 2 were used, giving a density correction of 0.88 and a modified No. 40 factor of 0.98 for the sample. The inputs used for the PVR calculations are shown in Table 4.41.

Table 4.41: PVR Input Parameters for Tex-124-E for Site 8

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		173	1.2	-	-		-	-
2	1	BR	262	1.8	65	23.3	109	100.0	40
4	1	BR	496	3.4	65	23.3	109	100.0	40
6	1	BR	723	5.0	65	23.3	109	100.0	40
8	1	BR	948	6.6	65	23.3	109	100.0	40
10	1	BR	1172	8.1	65	23.3	109	100.0	40

By integrating the curve fitted function from Figure 3.19 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1212 psf, the PVR of the subgrade was determined to be 0.82 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 1.40 in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.99, and the comparison between the cumulative PVR versus depth is shown in Table 4.42.

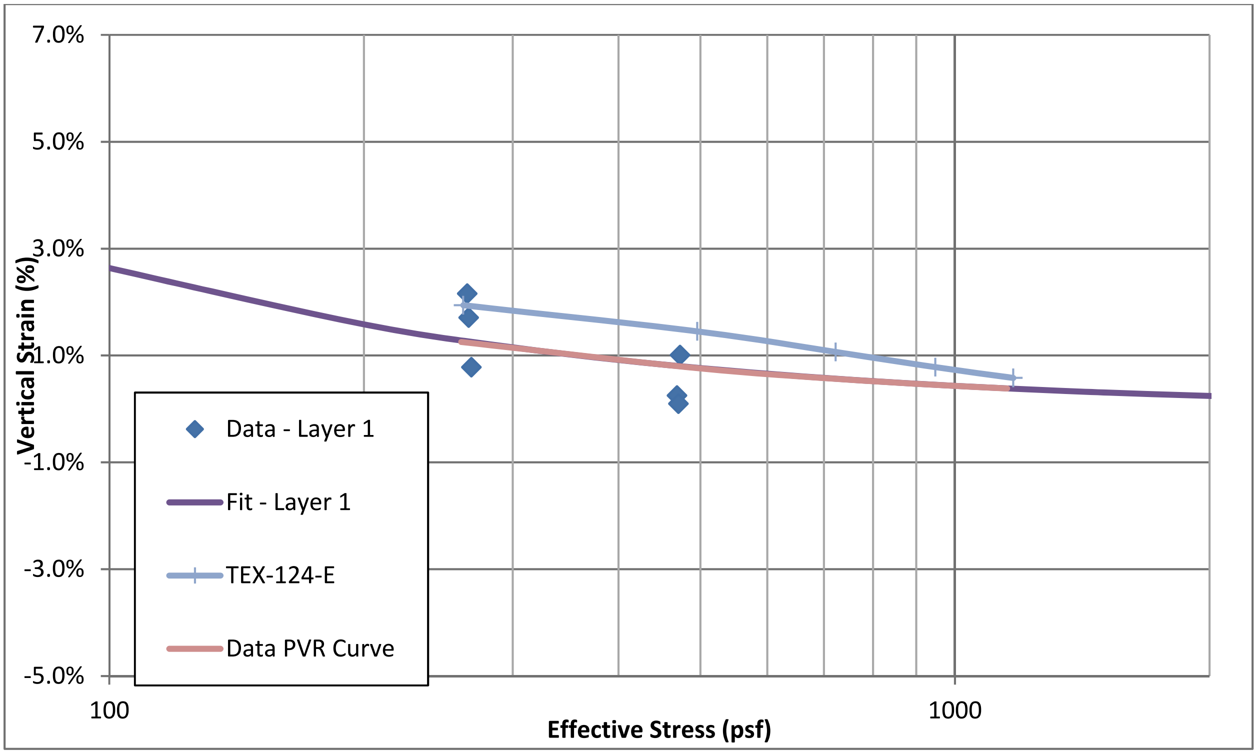


Figure 4.99: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Site 8

Table 4.42: Comparison of PVR Results for Site 8

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**1.40**	**0.82**
2.0	1	BR	262	0.93	0.52
4.0	1	BR	496	0.58	0.34
6.0	1	BR	723	0.33	0.20
8.0	1	BR	948	0.14	0.09
10.0	1	BR	1172	0.00	0.00

The results from the PVRs disagree, with Tex-124-E indicating that this site will sit on an expansive subgrade whereas the results from testing of the soils indicating that this is not necessarily the case. A site visit was performed in March 2016 near the location sampled in order to determine the amount of longitudinal cracking outside of the outer wheel path. The location that was surveyed for this particular condition is shown in Figure 4.100.

Figure 4.100: Condition Survey Location for Site 8

The road is located just west of the intersection of SH 95 and FM 972, and extensive cracking was found along the outside edge of the pavement. A length of 100 ft of roadway was surveyed, and the amount of longitudinal cracking was determined to be approximately 42 ft per 100 ft of roadway with the cracks ranging from 3 mm to 5 mm in width. This amount of cracking is extensive, but further explanations were needed as the site was not performing as expected. Examination of the change in profile of the road over time was performed using a total station surveying set-up and is further detailed in Chapter 5.

4.2.8 Site 9: SH 95 [Branyon Clay, BR - 95]

After the soil characterization and centrifuge testing program was completed on the Branyon Clay from Site 9, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. A field visit in March 2016 was also performed to determine the longitudinal cracking severity outside of the outer wheel path to determine the field behavior of each subgrade.

<u>Assumed Soil Profile</u>

The soil samples from this site were taken were taken within the top 3 feet. The assumed pavement structure used for PVR calculations had an asphalt depth of 4 inches and a base layer of 6 inches. This assumption is consistent among the sites in order to provide a similar comparison between sites in terms of the range of stresses. The Branyon Clay was assumed to be at a dry of optimum moisture content of 24% and a relative compaction of 100%, which resulted in a dry unit weight of 89 pcf and a total unit weight of 110 pcf. The soil profile used for both methods is shown in Table 4.43.

Table 4.43: Assumed Soil Profile for Branyon Clay at Site 9

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	123	0.9
1	0	1	Branyon Clay	60	34	26	24	110	178	1.2
2	1	2							288	2.0
3	2	3							398	2.8
4	3	4							508	3.5
5	4	5							618	4.3
6	5	6							728	5.1
7	6	7							838	5.8
8	7	8							948	6.6
9	8	9							1058	7.3
10	9	10							1168	8.1
***Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer**										

PVR Calculations

The soil conditions for centrifuge testing program on the Branyon Clay from Site 9 included an initial moisture content of 24% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from seven centrifuge samples and two free swell samples were input into the DMS-C spreadsheet, yielding the results shown in Figure 4.101.

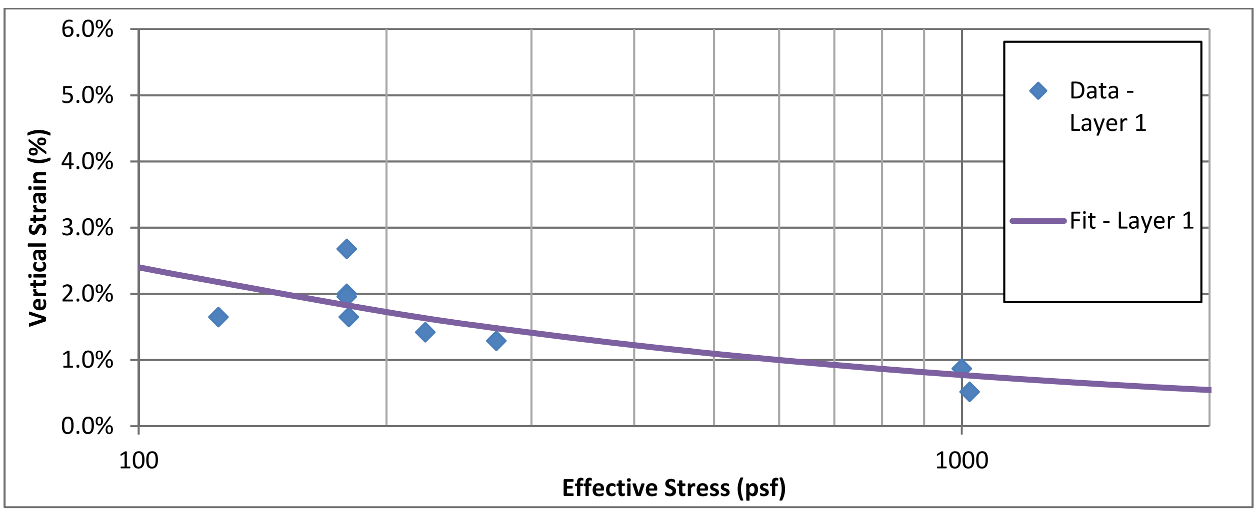


Figure 4.101: Swelling Results and Curve Fitting for Site 9

From the figure, it becomes clear that the soil tested in the centrifuge was moderately expansive with the potential for swelling occurring even at very high stresses.

For the Tex-124-E method, the soil profile from Table 4.43 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 96% of the soil passing the No. 40 sieve, as determined from the Wet Sieve tests, was assumed as well as dry conditions for the tests, which corresponded to a moisture content of 22.2% from the correlations in Tex-124-E. The sample unit weights as determined from equations 1 and 2 were

used, giving a density correction of 0.88 and a modified No. 40 factor of 0.96 for the sample. The inputs used for the PVR calculations are shown in Table 4.44.

Table 4.44: PVR Input Parameters for Tex-124-E for Site 9

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		173	1.2	-	-		-	-
2	1	BR	262	1.8	60	24.1	110	100.0	26
4	1	BR	496	3.4	60	24.1	110	100.0	26
6	1	BR	723	5.0	60	24.1	110	100.0	26
8	1	BR	948	6.6	60	24.1	110	100.0	26
10	1	BR	1172	8.1	60	24.1	110	100.0	26

By integrating the curve fitted function from Figure 3.22 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1168 psf, the PVR of the subgrade was determined to be 1.20 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 0.18 in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.102, and the comparison between the cumulative PVR versus depth is shown in Table 4.45.

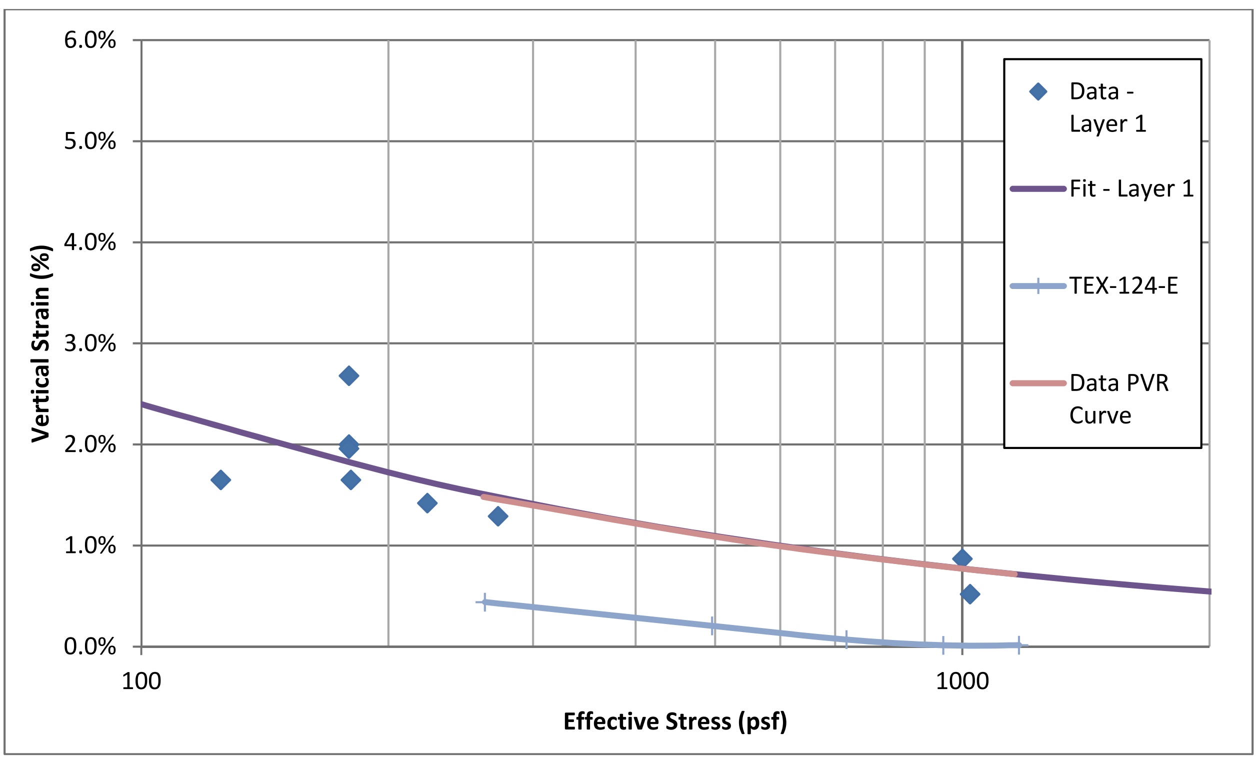


Figure 4.102: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Site 9

Table 4.45: Comparison of PVR Results for Site 9

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**0.18**	**1.20**
2.0	1	BR	262	0.07	0.85
4.0	1	BR	496	0.02	0.58
6.0	1	BR	723	0.01	0.36
8.0	1	BR	948	0.00	0.17
10.0	1	BR	1172	0.00	0.00

Based upon both the centrifuge testing of the Branyon Clay specimens and the Tex-124-E results, the site is considered to rest on an expansive subgrade with remediation techniques necessary. A site visit was performed in March 2016 near the location sampled in order to determine the amount of longitudinal cracking outside of the outer wheel path. The location that was surveyed for this particular condition is shown in Figure 4.103.

Figure 4.103: Condition Survey Location for Site 9

The road is located just south of Granger, TX, and extensive cracking was found along the outside edge of the pavement. A length of 130 ft of roadway was surveyed, and the amount of longitudinal cracking was determined to be approximately 218 ft per 100 ft of roadway with the cracks ranging from 3 to 5 mm in width. This amount of cracking is extensive and based upon testing and field performance, the roadway is determined to have issues with environmental cracking from the expansive subgrade. However, the extent to which the cracking is so pervasive at this area may come from the repairs that have taken place. Since the site is in a high volume traffic area, the asphalt has not been replaced, and sealing the cracks seems to be the most common maintenance operation.

4.2.9 Site 10: TxDOT Maintenance Office in Taylor, TX [Houston Black Clay, HB - Taylor]

After the soil characterization and centrifuge testing program was completed on the Houston Black Clay from Site 10, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. A field visit in March 2016 was also performed to determine the longitudinal cracking severity outside of the outer wheel path to determine the field behavior of each subgrade.

Assumed Soil Profile

The soil samples from this site were taken were taken within the top 3 feet. The assumed pavement structure used for PVR calculations had an asphalt depth of 4 inches and a base layer of 6 inches. This assumption is consistent among the sites in order to provide a similar comparison between sites in terms of the range of stresses. The Houston Black Clay was assumed to be at a dry of optimum moisture content of 21% and a relative compaction of 100%, which resulted in a dry unit weight of 95 pcf and a total unit weight of 114 pcf. The soil profile used for both methods is shown in Table 4.46.

Table 4.46: Assumed Soil Profile for Houston Black Clay at Site 10

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	123	0.9
1	0	1	Houston Black Clay	55	23	32	21	114	180	1.3
2	1	2							294	2.0
3	2	3							408	2.8
4	3	4							523	3.6
5	4	5							637	4.4
6	5	6							751	5.2
7	6	7							865	6.0
8	7	8							979	6.8
9	8	9							1093	7.6
10	9	10							1207	8.4
***Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer**										

PVR Calculations

The soil conditions for centrifuge testing program on the Houston Black Clay from Site 10 included an initial moisture content of 21% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from eight centrifuge specimens were input into the DMS-C spreadsheet, yielding the results shown in Figure 4.104.

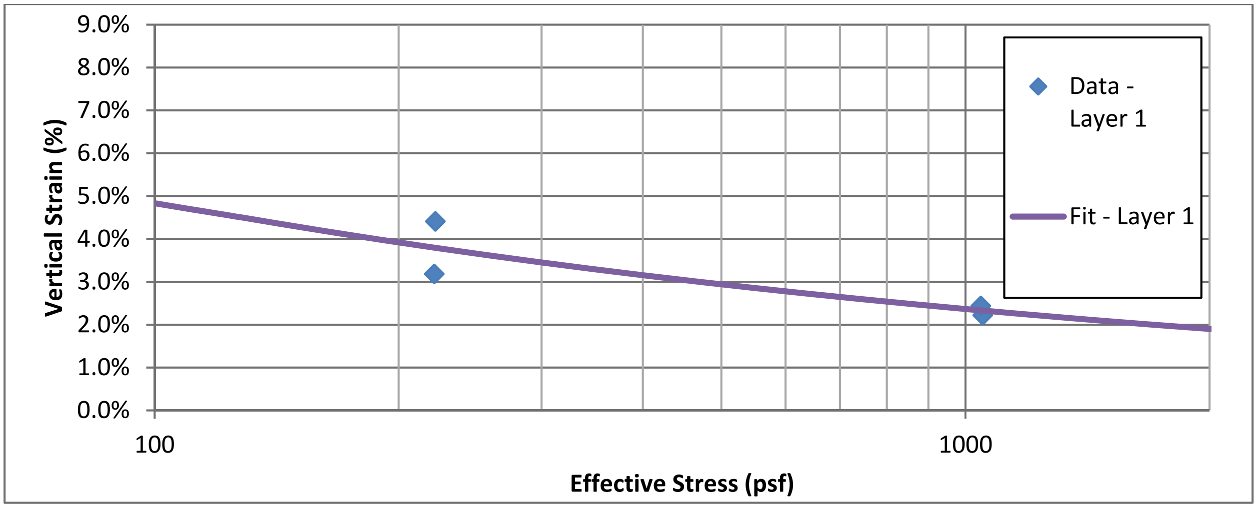


Figure 4.104: Swelling Results and Curve Fitting for Site 10

From the figure, it becomes clear that the soil tested in the centrifuge was moderately expansive with the potential for swelling occurring even at very high stresses.

For the Tex-124-E method, the soil profile from Table 4.46 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 97% of the soil passing the No. 40 sieve, as determined from the Wet Sieve tests, was assumed as well as dry conditions for the tests, which corresponded to a moisture content of 21.1% from the correlations in Tex-124-E. The sample unit weights as determined from equations 1 and 2 were used, giving a density correction of 0.91 and a modified No. 40 factor of 0.97 for the sample. The inputs used for the PVR calculations are shown in Table 4.47.

Table 4.47: PVR Input Parameters for Tex-124-E for Site 10

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		173	1.2	-	-		-	-
2	1	HB	264	1.8	55	20.7	114	95.0	32
4	1	HB	503	3.5	55	20.7	114	95.0	32
6	1	HB	735	5.1	55	20.7	114	95.0	32
8	1	HB	965	6.7	55	20.7	114	95.0	32
10	1	HB	1194	8.3	55	20.7	114	95.0	32

By integrating the curve fitted function from Figure 4.104 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1207 psf, the PVR of the subgrade was determined to be 3.28 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 0.93in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.105, and the comparison between the cumulative PVR versus depth is shown in Table 4.48.

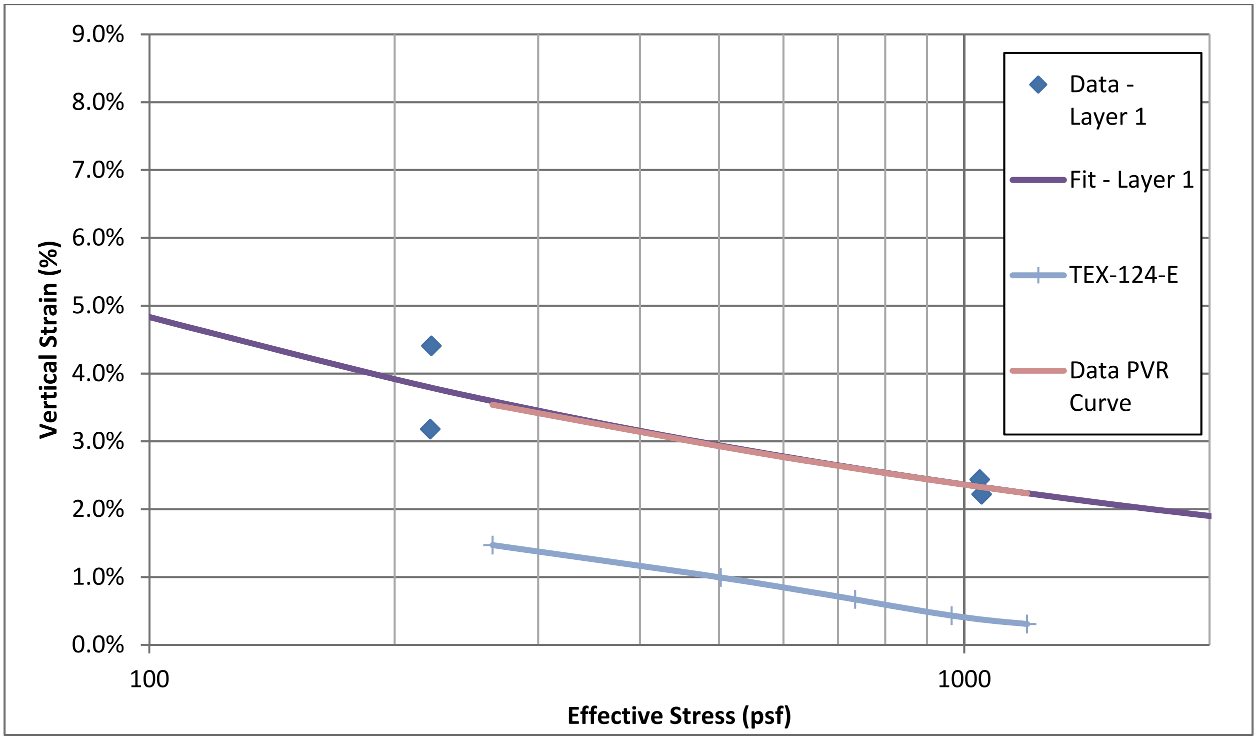


Figure 4.105: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Site 10

Table 4.48: Comparison of PVR Results for Site 10

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**0.93**	**3.28**
2.0	1	HB	264	0.58	2.44
4.0	1	HB	503	0.34	1.73
6.0	1	HB	735	0.18	1.11
8.0	1	HB	965	0.07	0.54
10.0	1	HB	1194	0.00	0.00

Based upon both the centrifuge testing of the Houston Black Clay specimens and the Tex-124-E results, the site is considered to rest on an expansive subgrade with remediation techniques necessary. A site visit was performed in March 2016 near the location sampled in order to determine the amount of longitudinal cracking outside of the outer wheel path. The location that was surveyed for this particular condition is shown in Figure 4.106.

Figure 4.106: Condition Survey Location for Site 10

The road is located just south of the maintenance yard, and extensive cracking was found along the outside edge of the pavement. A length of 100 ft of roadway was surveyed, and the amount of longitudinal cracking was determined to be approximately 97 ft per 100 ft of roadway with the cracks ranging from 3 mm to 5 mm in width. This amount of cracking is extensive and based upon testing and field performance, the roadway is determined to be lying on an expansive subgrade.

4.2.10 Site 11: FM 535 [Behring Clay, BH - 535]

After the soil characterization and centrifuge testing program was completed on the Behring Clay from Site 11, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. A field visit in March 2016 was also performed to determine the longitudinal cracking severity outside of the outer wheel path to determine the field behavior of each subgrade.

Assumed Soil Profile

The soil samples from this site were taken were taken within the top 3 feet. The assumed pavement structure used for PVR calculations had an asphalt depth of 4 inches and a base layer of 6 inches. This assumption is consistent among the sites in order to provide a similar comparison between sites in terms of the range of stresses. The Behring Clay was assumed to be at a dry of optimum moisture content of 20% and a relative compaction of 100%, which resulted in a dry unit weight of 96 pcf and a total unit weight of 115 pcf. The soil profile used for both methods is shown in Table 4.49.

Table 4.49: Assumed Soil Profile for Behring Clay at Site 11

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	123	0.9
1	0	1	Behring Clay	53	21	32	20	115	181	1.3
2	1	2							296	2.1
3	2	3							411	2.9
4	3	4							527	3.7
5	4	5							642	4.5
6	5	6							757	5.3
7	6	7							872	6.1
8	7	8							987	6.9
9	8	9							1103	7.7
10	9	10							1218	8.5
***Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer**										

PVR Calculations

The soil conditions for centrifuge testing program on the Behring Clay from Site 11 included an initial moisture content of 20% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from five centrifuge samples and two free swell specimens were input into the DMS-C spreadsheet, yielding the results shown in Figure 4.107.

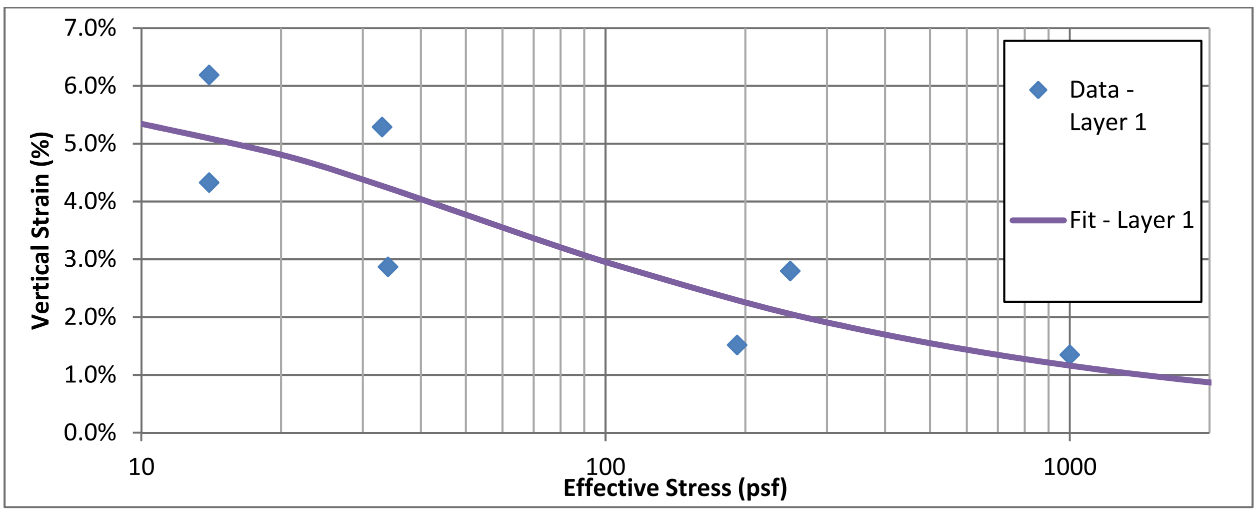


Figure 4.107: Swelling Results and Curve Fitting for Site 11

From the figure, it becomes clear that the soil tested in the centrifuge was moderately expansive with the potential for swelling occurring even at very high stresses.

For the Tex-124-E method, the soil profile from Table 3.28 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 96% of the soil passing the No. 40 sieve, as determined from the Wet Sieve tests, was assumed as well as dry conditions for the tests, which corresponded to a moisture content of 20.7% from the correlations in Tex-124-E. The sample unit weights as determined from equations 1 and 2 were

used, giving a density correction of 0.92 and a modified No. 40 factor of 0.96 for the sample. The inputs used for the PVR calculations are shown in Table 4.50.

Table 4.50: PVR Input Parameters for Tex-124-E for Site 11

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		173	1.2	-	-		-	-
2	1	HB	262	1.8	53	19.8	115	96.0	32
4	1	HB	496	3.4	53	19.8	115	96.0	32
6	1	HB	723	5.0	53	19.8	115	96.0	32
8	1	HB	948	6.6	53	19.8	115	96.0	32
10	1	HB	1172	8.1	53	19.8	115	96.0	32

By integrating the curve fitted function from Figure 4.107 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1218 psf, the PVR of the subgrade was determined to be 1.69 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 0.93 in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.108, and the comparison between the cumulative PVR versus depth is shown in Table 4.51.

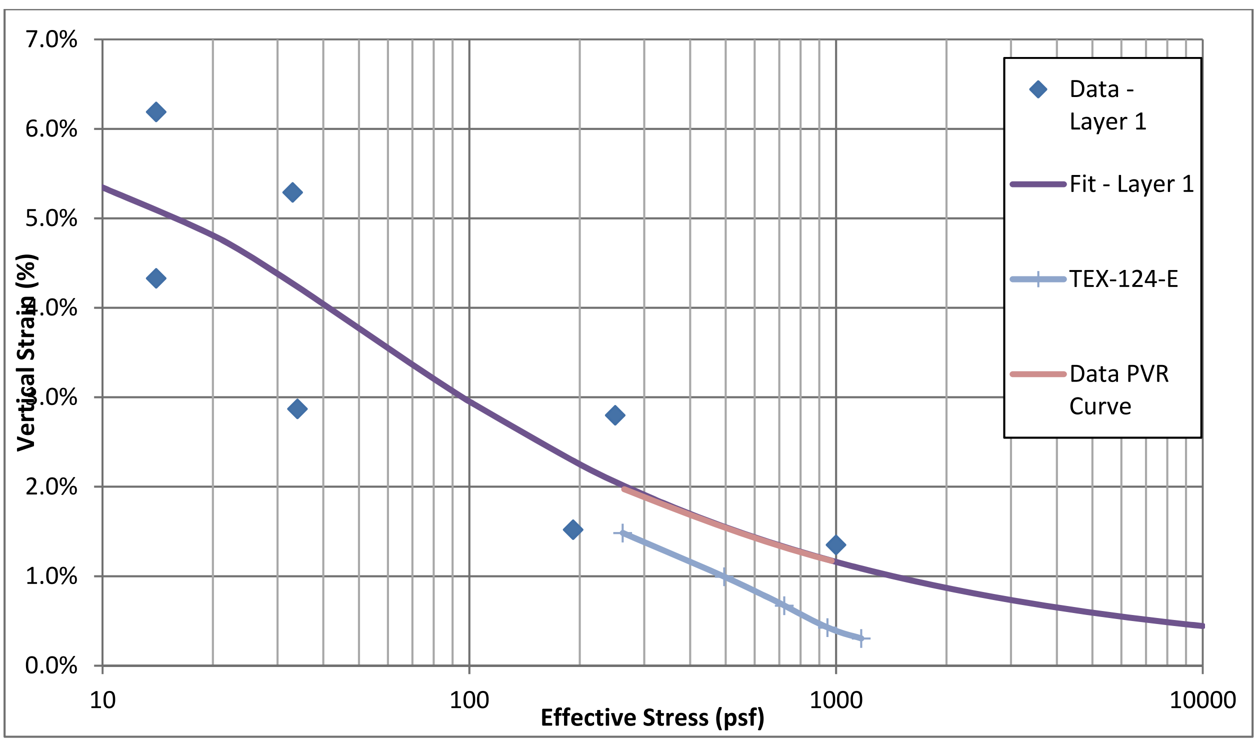


Figure 4.108: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Site 11

Table 4.51: Comparison of PVR Results for Site 11

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**0.93**	**1.69**
2.0	1	HB	262	0.58	1.22
4.0	1	HB	496	0.34	0.85
6.0	1	HB	723	0.18	0.54
8.0	1	HB	948	0.07	0.26
10.0	1	HB	1172	0.00	0.00

Based upon both the centrifuge testing of the Behring Clay specimens and the Tex-124-E results, the site is considered to rest on an expansive subgrade with remediation techniques necessary. A site visit was performed in March 2016 near the location sampled in order to determine the amount of longitudinal cracking outside of the outer wheel path. The location that was surveyed for this particular condition is shown in Figure 4.109.

Figure 4.109: Condition Survey Location for Site 11

The road is located just northwest of the intersection of FM 535 and FM 20, and cracking was not found along the edge of the roadway. There was a crack that was approximately 6 inches within the shoulder line, but the crack appears to be from a maintenance operation as opposed to any environmental conditions. More document research into the maintenance undertaken at this location, but the roadway does not show signs of environmental cracking.

4.2.11 Site 12: FM 20 [Behring Clay, BH - 20]

After the soil characterization and centrifuge testing program was completed on the Behring Clay from Site 12, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. A field visit in March 2016 was also performed to determine the longitudinal cracking severity outside of the outer wheel path to determine the field behavior of each subgrade.

Assumed Soil Profile

The soil samples from this site were taken were taken within the top 3 feet. The assumed pavement structure used for PVR calculations had an asphalt depth of 4 inches and a base layer of 6 inches. This assumption is consistent among the sites in order to provide a similar comparison between sites in terms of the range of stresses. The Behring Clay was assumed to be at a dry of optimum moisture content of 20% and a relative compaction of 100%, which resulted in a dry unit weight of 97 pcf and a total unit weight of 116 pcf. The soil profile used for both methods is shown in Table 4.52.

Table 4.52: Assumed Soil Profile for Behring Clay at Site 12

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	123	0.9
1	0	1	Behring Clay	50	24	26	20	116	181	1.3
2	1	2							297	2.1
3	2	3							413	2.9
4	3	4							529	3.7
5	4	5							645	4.5
6	5	6							761	5.3
7	6	7							877	6.1
8	7	8							993	6.9
9	8	9							1109	7.7
10	9	10							1225	8.5
***Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer**										

PVR Calculations

The soil conditions for centrifuge testing program on the Behring Clay from Site 12 included an initial moisture content of 20% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from five centrifuge specimens and one free swell specimen were input into the DMS-C spreadsheet, yielding the results shown in Figure 4.110.

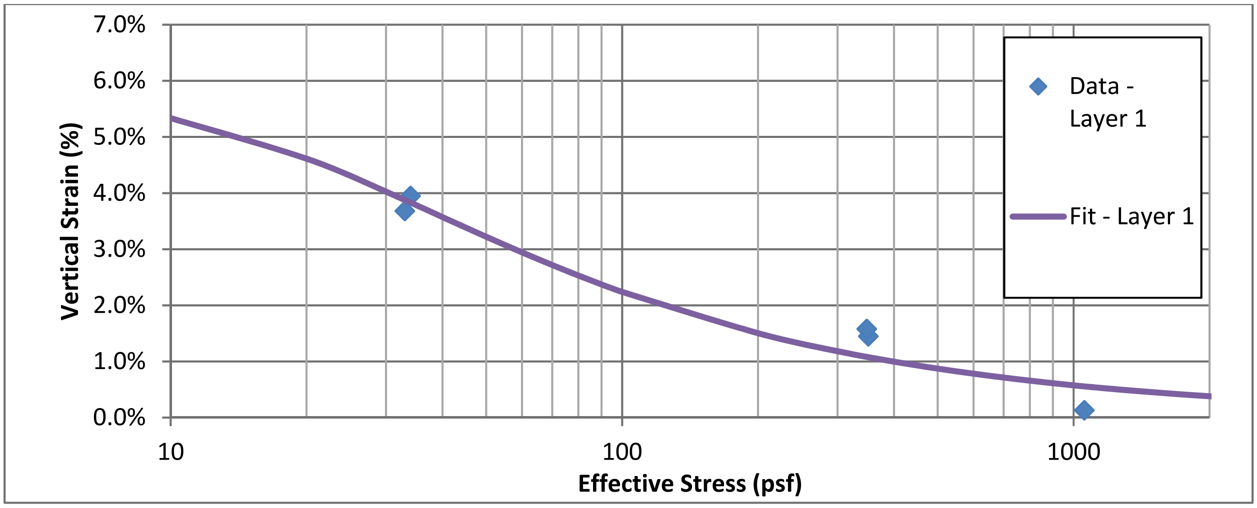


Figure 4.110: Swelling Results and Curve Fitting for Site 12

From the figure, it becomes clear that the soil tested in the centrifuge was moderately expansive with a small potential for swelling occurring even at very high stresses.

For the Tex-124-E method, the soil profile from Table 3.31 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 95% of the soil passing the No. 40 sieve, as determined from the Wet Sieve tests, was assumed as well as dry conditions for the tests, which corresponded to a moisture content of 20.0% from the correlations in Tex-124-E. The sample unit weights as determined from equations 1 and 2 were used, giving a density correction of 0.93 and a modified No. 40 factor of 0.95 for the sample. The inputs used for the PVR calculations are shown in Table 4.53.

Table 4.53: PVR Input Parameters for Tex-124-E for Site 12

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		173	1.2	-	-		-	-
2	1	BH	265	1.8	50	19.8	116	100.0	26
4	1	BH	508	3.5	50	19.8	116	100.0	26
6	1	BH	744	5.2	50	19.8	116	100.0	26
8	1	BH	978	6.8	50	19.8	116	100.0	26
10	1	BH	1212	8.4	50	19.8	116	100.0	26

By integrating the curve fitted function from Figure 4.110 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1225 psf, the PVR of the subgrade was determined to be 0.93 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 0.54 in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.111, and the comparison between the cumulative PVR versus depth is shown in Table 4.54.

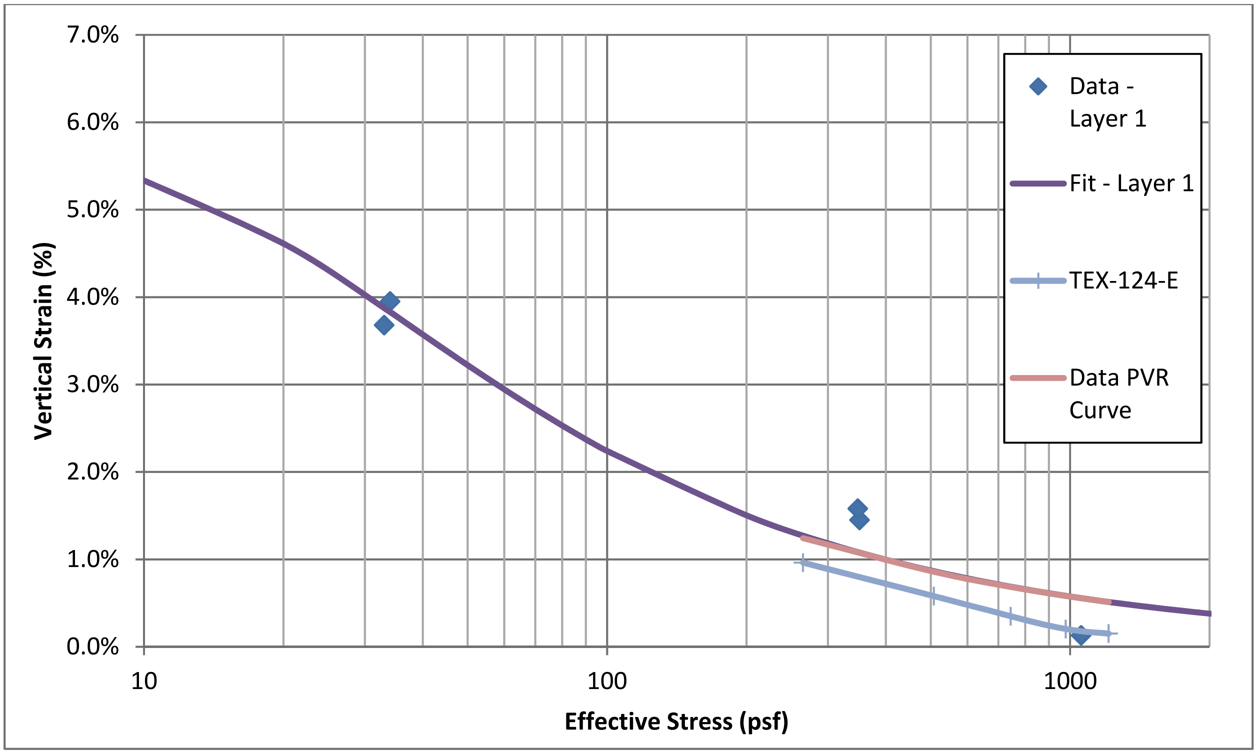


Figure 4.111: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Site 12

Table 4.54: Comparison of PVR Results for Site 12

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**0.54**	**0.93**
2.0	1	BH	265	0.31	0.63
4.0	1	BH	508	0.17	0.43
6.0	1	BH	744	0.08	0.26
8.0	1	BH	978	0.04	0.12
10.0	1	BH	1212	0.00	0.00

Based upon both the centrifuge testing of the Behring Clay specimens and the Tex-124-E results, the site is considered to rest on an expansive subgrade with remediation techniques necessary. A site visit was performed in March 2016 near the location sampled in order to determine the amount of longitudinal cracking outside of the outer wheel path. The location that was surveyed for this particular condition is shown in Figure 4.112.

Figure 4.112: Condition Survey Location for Site 12

The road is located along FM 20 north of Rockne, TX, and extensive cracking was found along the outside edge of the pavement. A length of 150 ft of roadway was surveyed, and the amount of longitudinal cracking was determined to be approximately 82 ft per 100 ft of roadway with the cracks ranging from 3 to 7 mm in width. This amount of cracking is extensive and based upon testing and field performance, the roadway is determined to be lying on an expansive subgrade.

4.2.12 Site 13: FM 672 North Site [Crockett Soil, CR – 672N]

After the soil characterization and centrifuge testing program was completed on the Crockett Soil from Site 13, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. A field visit in March 2016 was also performed to determine the longitudinal cracking severity outside of the outer wheel path to determine the field behavior of each subgrade.

Assumed Soil Profile

The soil samples from this site were taken were taken within the top 3 feet. The assumed pavement structure used for PVR calculations had an asphalt depth of 4 inches and a base layer of 6 inches. This assumption is consistent among the sites in order to provide a similar comparison between sites in terms of the range of stresses. The Crockett Soil was assumed to be at a dry of optimum moisture content of 17% and a relative compaction of 100%, which resulted in a dry unit weight of 103 pcf and a total unit weight of 120 pcf. The soil profile used for both methods is shown in Table 4.55.

Table 4.55: Assumed Soil Profile for Crockett Soil at Site 13

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	123	0.9
1	0	1	Crockett Soil	40	22	18	17	120	183	1.3
2	1	2							304	2.1
3	2	3							424	2.9
4	3	4							544	3.8
5	4	5							664	4.6
6	5	6							785	5.4
7	6	7							905	6.3
8	7	8							1025	7.1
9	8	9							1145	8.0
10	9	10							1266	8.8
***Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer**										

PVR Calculations

The soil conditions for centrifuge testing program on the Crockett Soil from Site 13 included an initial moisture content of 17% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from seven centrifuge specimens were input into the DMS-C spreadsheet, yielding the results shown in Figure 4.113.

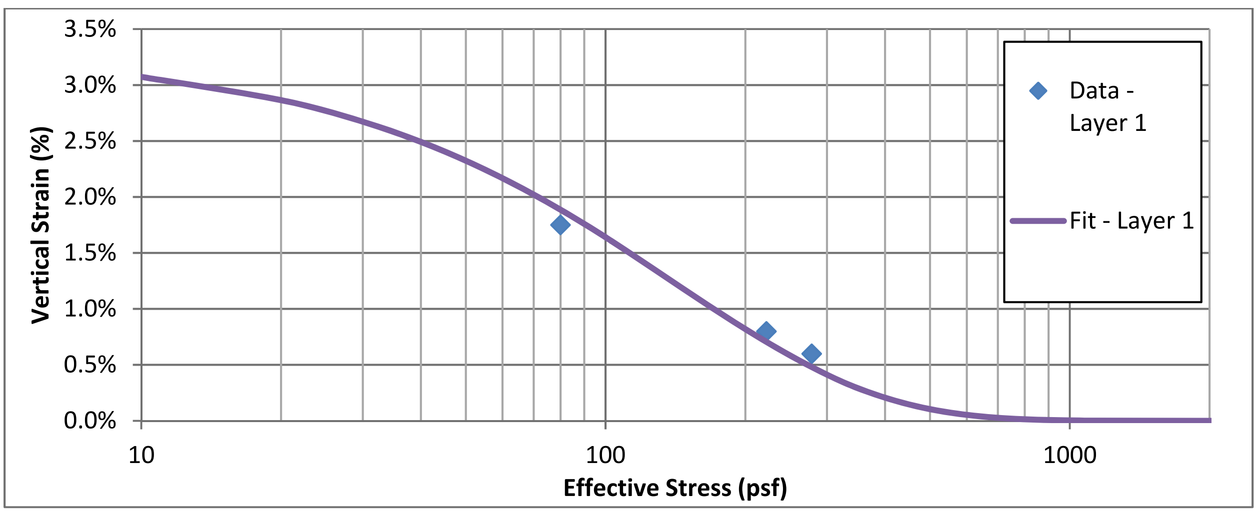


Figure 4.113: Swelling Results and Curve Fitting for Site 13

From the figure, it becomes clear that the soil tested in the centrifuge was not very expansive and began to see collapse at higher stresses.

For the Tex-124-E method, the soil profile from Table 4.55 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 98% of the soil passing the No. 40 sieve, as determined from the Wet Sieve tests, was assumed as well as dry conditions for the tests, which corresponded to a moisture content of 17.8% from the correlations in Tex-124-E. The sample unit weights as determined from equations 1 and 2 were

used, giving a binder correction of 0.96 and a modified No. 40 factor of 0.98 for the sample. The inputs used for the PVR calculations are shown below in Table 4.56.

Table 4.56: PVR Input Parameters for Tex-124-E for Site 13

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		173	1.2	-	-		-	-
2	1	CR	268	1.9	40	17.2	120	100.0	18
4	1	CR	520	3.6	40	17.2	120	100.0	18
6	1	CR	765	5.3	40	17.2	120	100.0	18
8	1	CR	1008	7.0	40	17.2	120	100.0	18
10	1	CR	1250	8.7	40	17.2	120	100.0	18

By integrating the curve fitted function from Figure 4.113 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1266 psf, the PVR of the subgrade was determined to be 0.14 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 0.17 in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.114, and the comparison between the cumulative PVR versus depth is shown in Table 4.57.

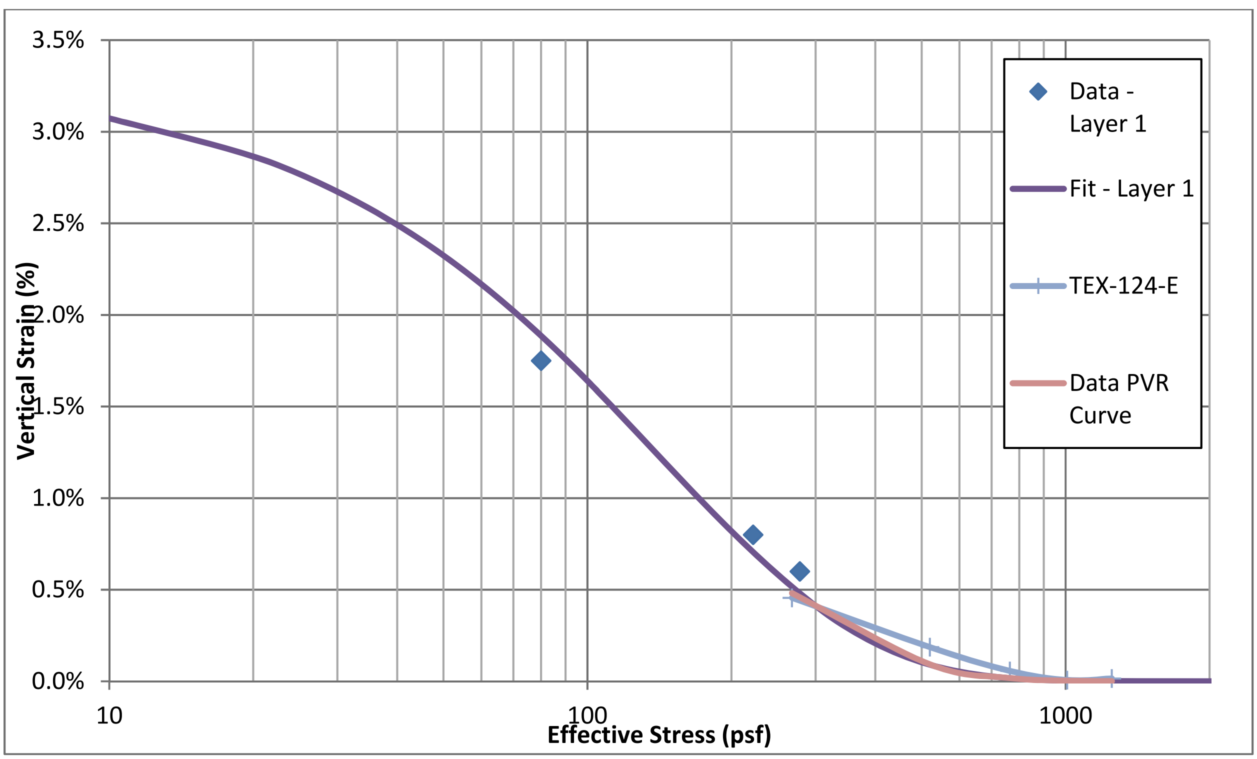


Figure 4.114: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Site 13

Table 4.57: Comparison of PVR Results for Site 13

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**0.17**	**0.14**
2.0	1	CR	268	0.06	0.03
4.0	1	CR	520	0.02	0.01
6.0	1	CR	765	0.01	0.00
8.0	1	CR	1008	0.00	0.00
10.0	1	CR	1250	0.00	0.00

Based upon both the centrifuge testing of the Crockett Soil specimens and the Tex-124-E results, the site is not considered to rest on an expansive subgrade. A site visit was performed in March 2016 near the location sampled in order to determine the amount of longitudinal cracking outside of the outer wheel path. The location that was surveyed for this particular condition is shown in Figure 4.115.

Figure 4.115: Condition Survey Location for Site 13

The road is located just south of St. John's Colony, and extensive cracking was found along the outside edge of the pavement. A length of 150 ft of roadway was surveyed, and the amount of longitudinal cracking was determined to be approximately 75 ft per 100 ft of roadway with the cracks ranging from 3 to 5 mm in width. This amount of cracking is extensive, but the cracking may have come from settlement of the roadway. There appears to not have been maintenance operations relaying the asphalt; only sealing the cracks has been done. Thus, the site's maintenance history must be examined, but the testing results indicate that the underlying subgrade is non-expansive.

4.2.13 Site 14: FM 672 South Site [Crockett Soil, CR – 672S]

After the soil characterization and centrifuge testing program was completed on the Crockett Soil from Site 14, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. A field visit in March 2016 was also performed to determine the longitudinal cracking severity outside of the outer wheel path to determine the field behavior of each subgrade.

<u>Assumed Soil Profile</u>

The soil samples from this site were taken were taken within the top 3 feet. The assumed pavement structure used for PVR calculations had an asphalt depth of 4 inches and a base layer of 6 inches. This assumption is consistent among the sites in order to provide a similar comparison between sites in terms of the range of stresses. The Crockett Soil was assumed to be at a dry of optimum moisture content of 20% and a relative compaction of 100%, which resulted in a dry unit weight of 95 pcf and a total unit weight of 114 pcf. The soil profile used for both methods is shown in Table 4.58.

Table 4.58: Assumed Soil Profile for Crockett Soil at Site 14

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	123	0.9
1	0	1	Crockett Soil	54	21	34	20	114	181	1.3
2	1	2							295	2.0
3	2	3							410	2.8
4	3	4							524	3.6
5	4	5							638	4.4
6	5	6							753	5.2
7	6	7							867	6.0
8	7	8							982	6.8
9	8	9							1096	7.6
10	9	10							1211	8.4
***Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer**										

<u>PVR Calculations</u>

The soil conditions for centrifuge testing program on the Crockett Soil from Site 14 included an initial moisture content of 20% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from one centrifuge specimen and three free swell specimens were input into the DMS-C spreadsheet, yielding the results shown in Figure 4.116.

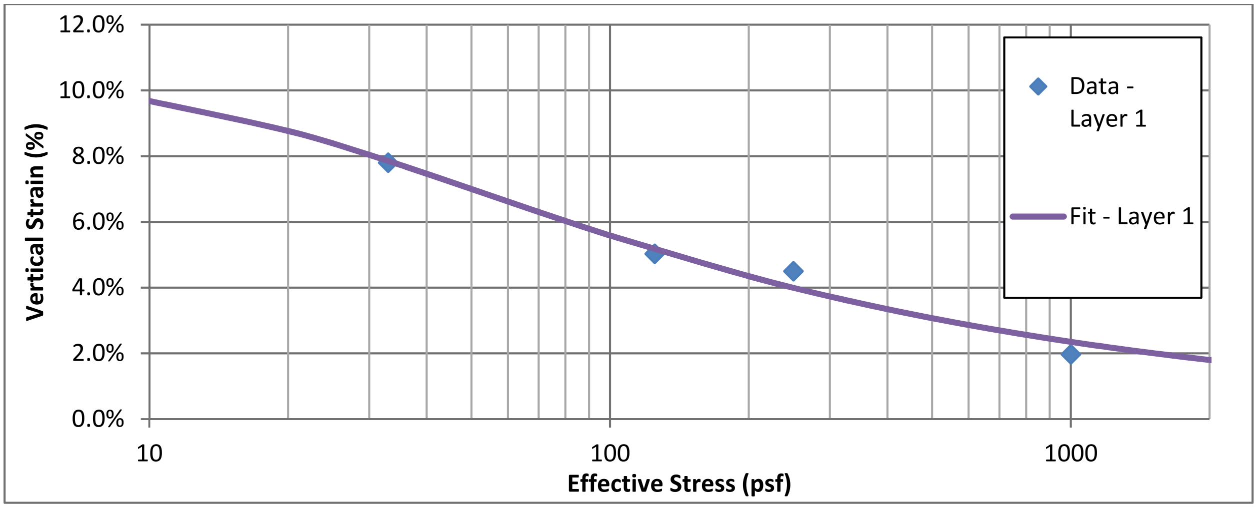


Figure 4.116: Swelling Results and Curve Fitting for Site 14

From the figure, it becomes clear that the soil tested in the centrifuge was moderately expansive with a small potential for swelling occurring even at very high stresses.

For the Tex-124-E method, the soil profile from Table 4.58 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 100% of the soil passing the No. 40 sieve, as determined from the Wet Sieve tests, was assumed as well as dry conditions for the tests, which corresponded to a moisture content of 17.8% from the correlations in Tex-124-E. The sample unit weights as determined from equations 1 and 2 were used, giving a density correction of 0.96 and a modified No. 40 factor of 1.00 for the sample. The inputs used for the PVR calculations are shown in Table 4.59.

Table 4.59: PVR Input Parameters for Tex-124-E for Site 14

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		173	1.2	-	-		-	-
2	1	CR	264	1.8	54	20.5	114	100.0	31
4	1	CR	504	3.5	54	20.5	114	100.0	31
6	1	CR	737	5.1	54	20.5	114	100.0	31
8	1	CR	968	6.7	54	20.5	114	100.0	31
10	1	CR	1198	8.3	54	20.5	114	100.0	31

By integrating the curve fitted function from Figure 4.116 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1211 psf, the PVR of the subgrade was determined to be 3.38 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 0.89 in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.117, and the comparison between the cumulative PVR versus depth is shown in Table 4.60.

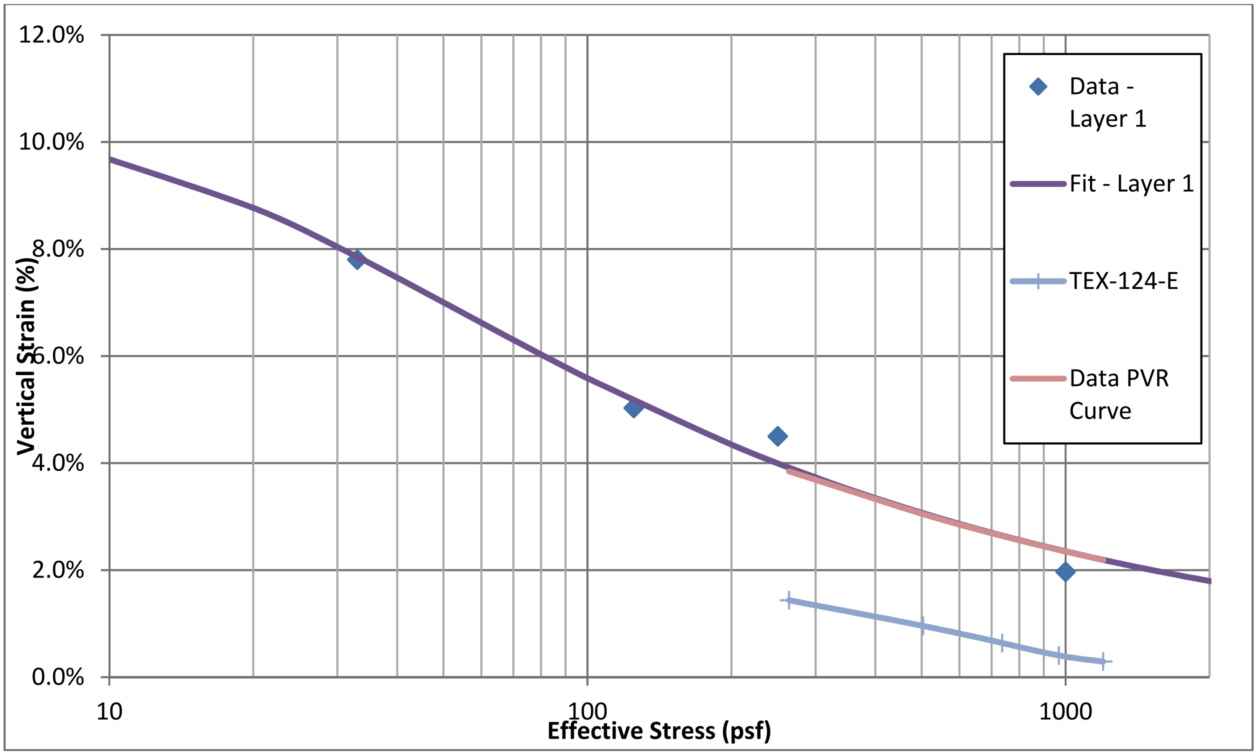


Figure 4.117: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Site 14

Table 4.60: Comparison of PVR Results for Site 14

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**0.89**	**3.38**
2.0	1	CR	264	0.55	2.46
4.0	1	CR	504	0.32	1.73
6.0	1	CR	737	0.17	1.10
8.0	1	CR	968	0.07	0.53
10.0	1	CR	1198	0.00	0.00

Based on both the centrifuge testing of the Crockett Soil specimens and the Tex-124-E results, the site is considered to rest on an expansive subgrade and will need techniques to prevent environmental cracking. A site visit was performed in March 2016 near the location sampled to determine the amount of longitudinal cracking outside the outer wheel path. The location that was surveyed for this particular condition is shown in Figure 4.118.

Figure 4.118: Condition Survey Location for Site 14

The road is located just north of Dale, and cracking was found along the outside edge of the pavement. A length of 100 ft of roadway was surveyed, and the amount of longitudinal cracking was determined to be approximately 29 ft per 100 ft of roadway, with the cracks ranging from 3 to 5 mm in width. This amount of cracking is not very extensive, but the results from testing indicate that the soil here will be expansive. Combined with the results from Site 13, these results are indicative that the Crockett Soil may see a high amount of heterogeneity, and further studies into this soil series should be conducted.

4.2.14 Site 15: FM 1854 – East Site [Burleson Clay, BU]

After the soil characterization and centrifuge testing program was completed on the Burleson Clay from Site 15, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. A field visit in March 2016 was also performed to determine the longitudinal cracking severity outside of the outer wheel path to determine the field behavior of each subgrade.

<u>Assumed Soil Profile</u>

The soil samples from this site were taken were taken within the top 3 feet. The assumed pavement structure used for PVR calculations had an asphalt depth of 4 inches and a base layer of 6 inches. This assumption is consistent among the sites in order to provide a similar comparison between sites in terms of the range of stresses. The Burleson Clay was assumed to be at a dry of optimum moisture content of 23% and a relative compaction of 100%, which resulted in a dry unit weight of 90 pcf and a total unit weight of 110 pcf. The soil profile used for both methods is shown in Table 4.61.

Table 4.61: Assumed Soil Profile for Burleson Clay at Site 15

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	123	0.9
1	0	1	Burleson Clay	63	24	39	23	110	179	1.2
2	1	2							289	2.0
3	2	3							399	2.8
4	3	4							510	3.5
5	4	5							620	4.3
6	5	6							731	5.1
7	6	7							841	5.8
8	7	8							952	6.6
9	8	9							1062	7.4
10	9	10							1173	8.1
***Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer**										

PVR Calculations

The soil conditions for centrifuge testing program on the Burleson Clay from Site 15 included an initial moisture content of 23% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from seven centrifuge specimens and one free swell specimen were input into the DMS-C spreadsheet, yielding the results shown in Figure 4.119.

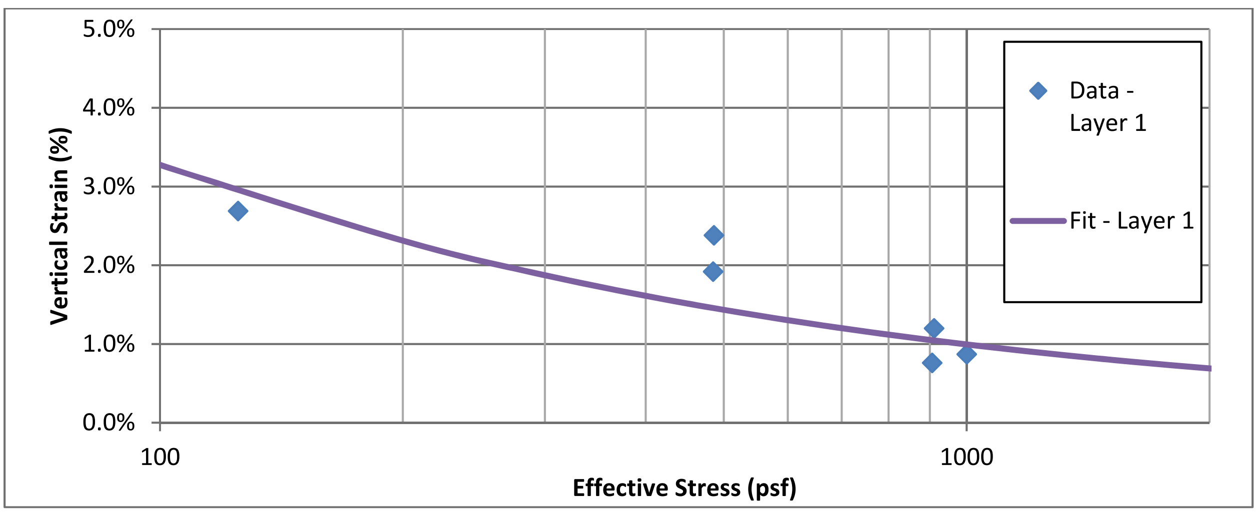


Figure 4.119: Swelling Results and Curve Fitting for Site 15

From the figure, it becomes clear that the soil tested in the centrifuge was only slightly expansive with a small potential for swelling occurring even at very high stresses.

For the Tex-124-E method, the soil profile from Table 4.61 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 99% of the soil passing the No. 40 sieve, as determined from the Wet Sieve tests, was assumed as well as dry conditions for the tests, which corresponded to a moisture content of 22.9% from the correlations in Tex-124-E. The sample unit weights as determined from equations 1 and 2 were

used, giving a binder correction of 0.88 and a modified No. 40 factor of 0.99 for the sample. The inputs used for the PVR calculations are shown in Table 4.62.

Table 4.62: PVR Input Parameters for Tex-124-E for Site 15

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		173	1.2	-	-		-	-
2	1	BU	261	1.8	63	22.7	110	99.0	39
4	1	BU	492	3.4	63	22.7	110	99.0	39
6	1	BU	717	5.0	63	22.7	110	99.0	39
8	1	BU	940	6.5	63	22.7	110	99.0	39
10	1	BU	1162	8.1	63	22.7	110	99.0	39

By integrating the curve fitted function from Figure 4.119 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1173 psf, the PVR of the subgrade was determined to be 1.57 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 1.35 in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.120, and the comparison between the cumulative PVR versus depth is shown in Table 4.63.

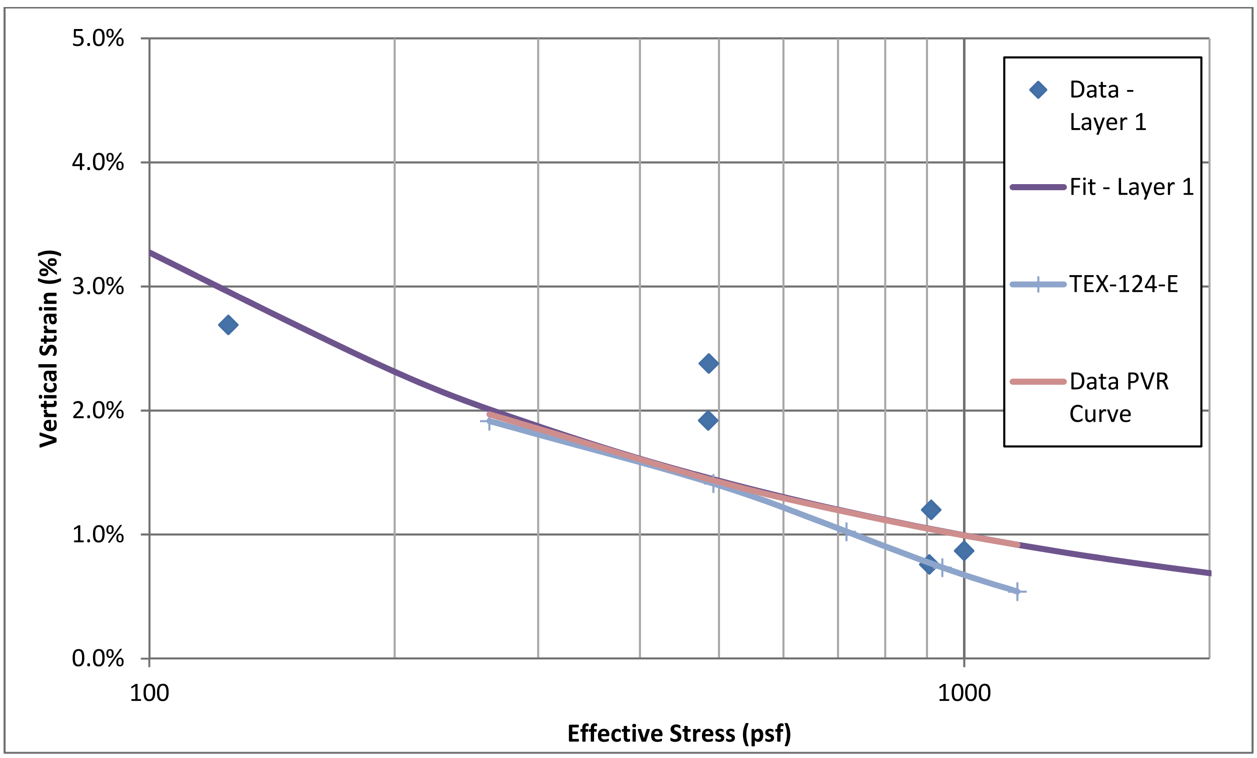


Figure 4.120: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Site 15

Table 4.63: Comparison of PVR Results for Site 15

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**1.35**	**1.57**
2.0	1	BU	261	0.89	1.09
4.0	1	BU	492	0.55	0.75
6.0	1	BU	717	0.31	0.47
8.0	1	BU	940	0.13	0.22
10.0	1	BU	1162	0.00	0.00

Based upon both the centrifuge testing of the Behring Clay specimens and the Tex-124-E results, the site is considered to rest on an expansive subgrade with remediation techniques necessary. A site visit was performed in March 2016 near the location sampled in order to determine the amount of longitudinal cracking outside of the outer wheel path. The location that was surveyed for this particular condition is shown in Figure 4.121.

Figure 4.121: Condition Survey Location for Site 15

The road is located along FM 1854 just west of Dale, and extensive cracking was found along the outside edge of the pavement. A length of 150 ft of roadway was surveyed, and the amount of longitudinal cracking was determined to be approximately 93 ft per 100 ft of roadway with the cracks ranging from 3 to 7 mm in width. This amount of cracking is extensive and based upon testing and field performance, the roadway is determined to be lying on an expansive subgrade.

4.2.15 Site 16: FM 1854 – West Site [Heiden Clay, HE – 1854W]

After the soil characterization and centrifuge testing program was completed on the Heiden Clay from Site 16, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. A field visit in March 2016 was also performed to determine the longitudinal cracking severity outside of the outer wheel path to determine the field behavior of each subgrade.

Assumed Soil Profile

The soil samples from this site were taken were taken within the top 3 feet. The assumed pavement structure used for PVR calculations had an asphalt depth of 4 inches and a base layer of 6 inches. This assumption is consistent among the sites in order to provide a similar comparison between sites in terms of the range of stresses. The Heiden Clay was assumed to be at a dry of optimum moisture content of 24% and a relative compaction of 100%, which resulted in a dry unit weight of 88 pcf and a total unit weight of 109 pcf. The soil profile used for both methods is shown in Table 4.64.

Table 4.64: Assumed Soil Profile for Heiden Clay at Site 16

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	123	0.9
1	0	1	Heiden Clay	65	27	38	24	109	178	1.2
2	1	2							287	2.0
3	2	3							396	2.7
4	3	4							505	3.5
5	4	5							614	4.3
6	5	6							723	5.0
7	6	7							832	5.8
8	7	8							941	6.5
9	8	9							1050	7.3
10	9	10							1159	8.0
***Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer**										

PVR Calculations

The soil conditions for centrifuge testing program on the Heiden Clay from Site 16 included an initial moisture content of 24% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from seven centrifuge specimens and one free swell specimen were input into the DMS-C spreadsheet, yielding the results shown in Figure 4.122.

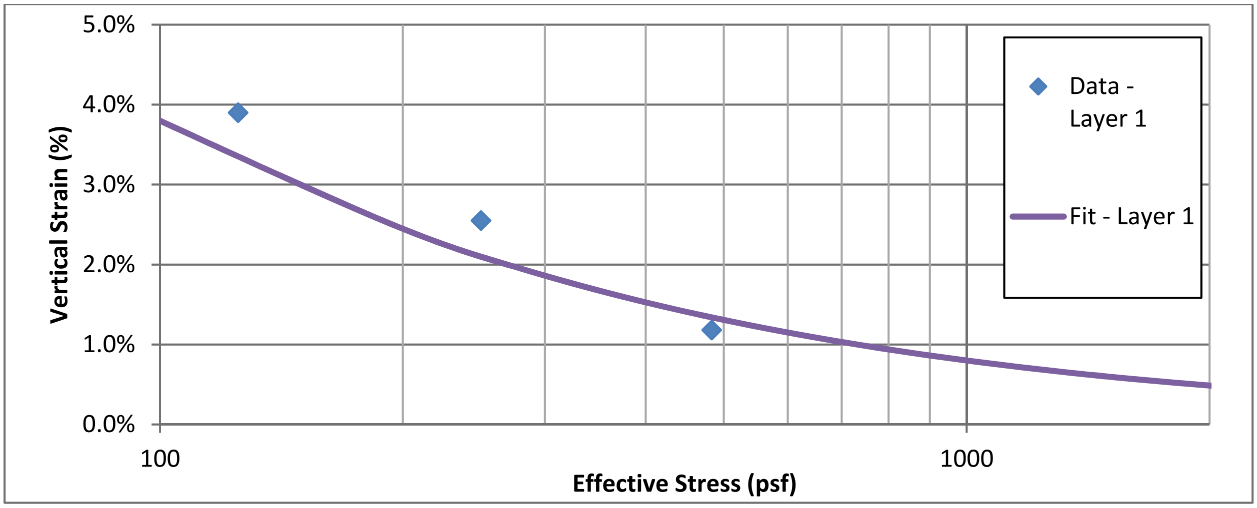


Figure 4.122: Swelling Results and Curve Fitting for Site 16

From the figure, it becomes clear that the soil tested in the centrifuge was slightly expansive with a small potential for swelling occurring even at very high stresses.

For the Tex-124-E method, the soil profile from Table 4.64 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 99% of the soil passing the No. 40 sieve, as determined from the Wet Sieve tests, was assumed as well as dry conditions for the tests, which corresponded to a moisture content of 23.3% from the correlations in Tex-124-E. The sample unit weights as determined from equations 1 and 2 were used, giving a density correction of 0.87 and a modified No. 40 factor of 0.99 for the sample. The inputs used for the PVR calculations are shown in Table 4.65.

Table 4.65: PVR Input Parameters for Tex-124-E for Site 16

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		173	1.2	-	-		-	-
2	1	HE	262	1.8	65	23.7	109	99.0	38
4	1	HE	496	3.4	65	23.7	109	99.0	38
6	1	HE	723	5.0	65	23.7	109	99.0	38
8	1	HE	948	6.6	65	23.7	109	99.0	38
10	1	HE	1172	8.1	65	23.7	109	99.0	38

By integrating the curve fitted function from Figure 4.122 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1159 psf, the PVR of the subgrade was determined to be 1.22 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 1.42 in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.123, and the comparison between the cumulative PVR versus depth is shown in Table 4.66.

Figure 4.123: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Site 16

Table 4.66: Comparison of PVR Results for Site 16

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**1.27**	**1.42**
2.0	1	HE	260	0.83	0.94
4.0	1	HE	488	0.51	0.62
6.0	1	HE	710	0.28	0.38
8.0	1	HE	930	0.12	0.17
10.0	1	HE	1149	0.00	0.00

Based upon both the centrifuge testing of the Heiden Clay specimens and the Tex-124-E results, the site is considered to rest on an expansive subgrade with remediation techniques necessary. A site visit was performed in March 2016 near the location sampled in order to determine the amount of longitudinal cracking outside of the outer wheel path. The location that was surveyed for this particular condition is shown in Figure 4.124.

Figure 4.124: Condition Survey Location for Site 16

The road is located along FM 1854 just east of Lytton Springs, and extensive cracking was found along the outside edge of the pavement. A length of 100 ft of roadway was surveyed, and the amount of longitudinal cracking was determined to be approximately 136 ft per 100 ft of roadway with the cracks ranging from 3 to 5 mm in width. This amount of cracking is extensive; based upon testing and field performance, the roadway is determined to be lying on an expansive subgrade.

4.2.16 Site 17: FM 1854 & SH 21 [Heiden Clay, HE – 1854 & SH21]

After the soil characterization and centrifuge testing program was completed on the Heiden Clay from Site 17, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. A field visit in March 2016 was also performed to determine the longitudinal cracking severity outside of the outer wheel path to determine the field behavior of each subgrade.

Assumed Soil Profile

The soil samples from this site were taken were taken within the top 3 feet. The assumed pavement structure used for PVR calculations had an asphalt depth of 4 inches and a base layer of 6 inches. This assumption is consistent among the sites in order to provide a similar comparison between sites in terms of the range of stresses. The Heiden Clay was assumed to be at a dry of optimum moisture content of 23% and a relative compaction of 100%, which resulted in a dry unit weight of 90 pcf and a total unit weight of 110 pcf. The soil profile used for both methods is shown in Table 4.67.

Table 4.67: Assumed Soil Profile for Heiden Clay at Site 17

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	123	0.9
1	0	1	Heiden Clay	63	25	38	24	109	178	1.2
2	1	2							289	2.0
3	2	3							399	2.8
4	3	4							509	3.5
5	4	5							620	4.3
6	5	6							730	5.1
7	6	7							840	5.8
8	7	8							950	6.6
9	8	9							1061	7.4
10	9	10							1171	8.1
***Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer**										

PVR Calculations

The soil conditions for centrifuge testing program on the Heiden Clay from Site 17 included an initial moisture content of 24% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from seven centrifuge specimens and one free swell specimen were input into the DMS-C spreadsheet, yielding the results shown in Figure 4.125.

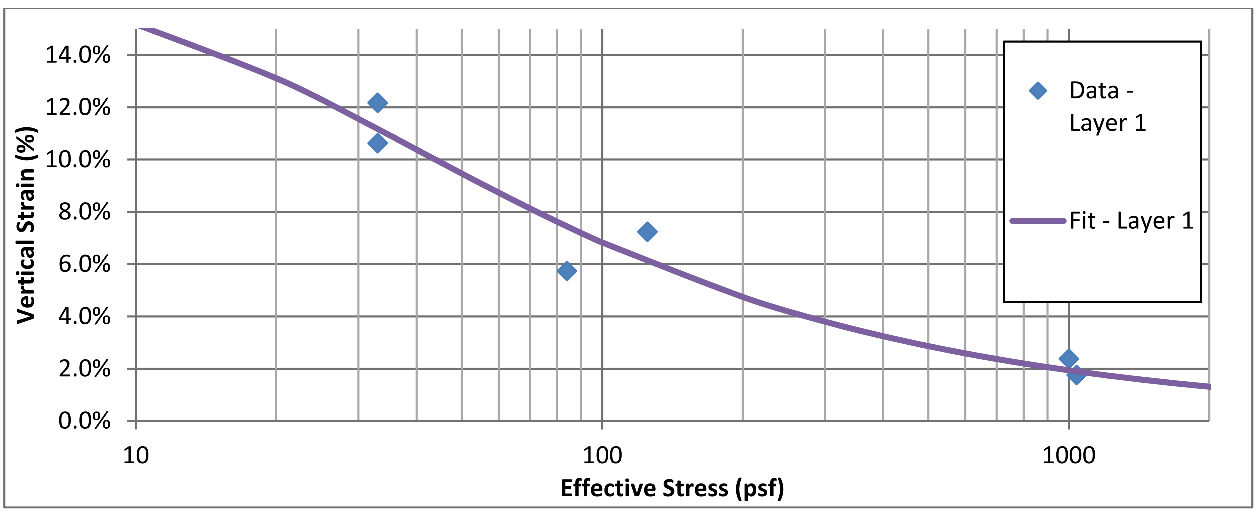


Figure 4.125: Swelling Results and Curve Fitting for Site 17

From the figure, it becomes clear that the soil tested in the centrifuge was very expansive with a small potential for swelling occurring even at very high stresses.

For the Tex-124-E method, the soil profile from Table 4.67 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 98% of the soil passing the No. 40 sieve, as determined from the Wet Sieve tests, was assumed as well as dry conditions for the tests, which corresponded to a moisture content of 22.9 % from the correlations in Tex-124-E. The sample unit weights as determined from equations 1 and 2 were

used, giving a density correction of 0.87 and a modified No. 40 factor of 0.98 for the sample. The inputs used for the PVR calculations are shown in Table 4.68.

Table 4.68: PVR Input Parameters for Tex-124-E for Site 17

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		173	1.2	-	-		-	-
2	1	HE	261	1.8	63	22.9	110	98.0	38
4	1	HE	492	3.4	63	22.9	110	98.0	38
6	1	HE	716	5.0	63	22.9	110	98.0	38
8	1	HE	939	6.5	63	22.9	110	98.0	38
10	1	HE	1161	8.1	63	22.9	110	98.0	38

By integrating the curve fitted function from Figure 4.125 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1171 psf, the PVR of the subgrade was determined to be 3.12 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 1.28 in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.126, and the comparison between the cumulative PVR versus depth is shown in Table 4.69.

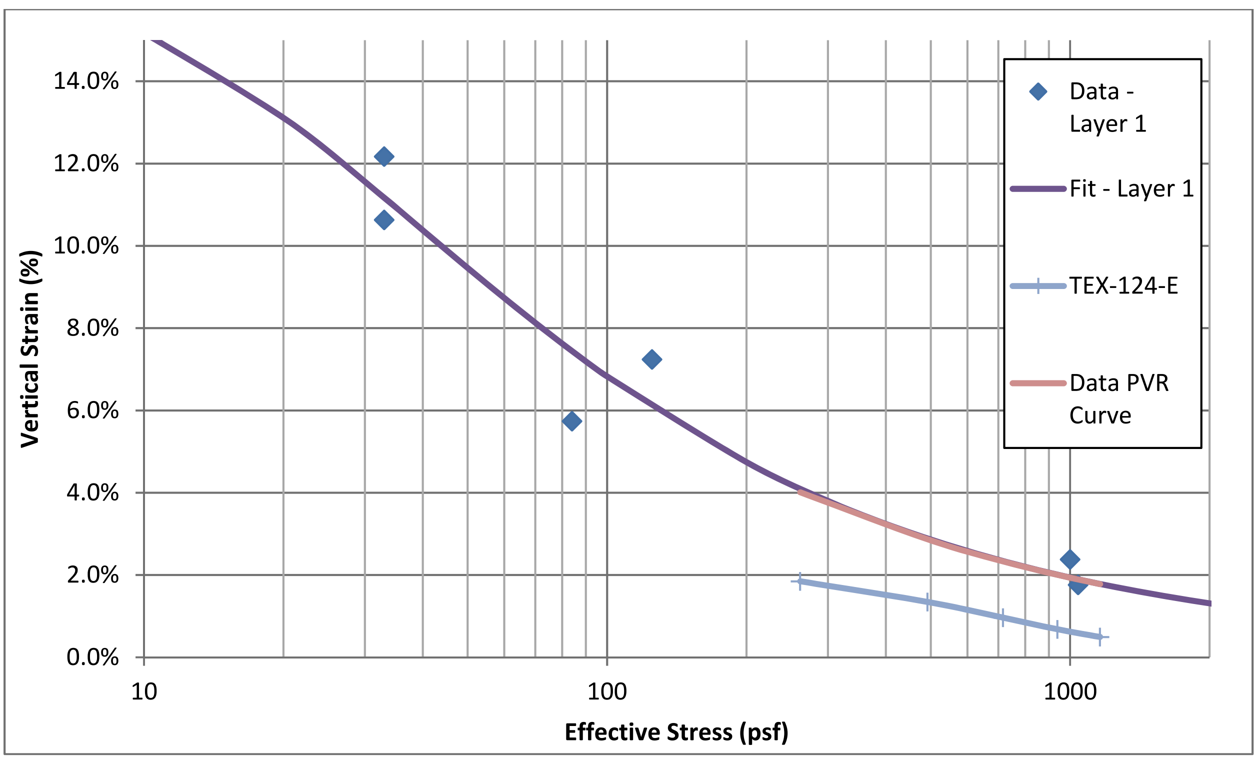


Figure 4.126: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Site 17

Table 4.69: Comparison of PVR Results for Site 17

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**1.28**	**3.12**
2.0	1	HE	261	0.84	2.16
4.0	1	HE	492	0.51	1.47
6.0	1	HE	716	0.28	0.91
8.0	1	HE	939	0.12	0.43
10.0	1	HE	1161	0.00	0.00

Based upon both the centrifuge testing of the Heiden Clay specimens and the Tex-124-E results, the site is considered to rest on an expansive subgrade with remediation techniques necessary. A site visit was performed in March 2016 near the location sampled in order to determine the amount of longitudinal cracking outside of the outer wheel path. The location that was surveyed for this particular condition is shown in Figure 4.127.

Figure 4.127: Condition Survey Location for Site 17

The road is located along FM 1854 just east of Lytton Springs, and extensive cracking was found along the outside edge of the pavement. A length of 100 ft of roadway was surveyed, and the amount of longitudinal cracking was determined to be approximately 45 ft per 100 ft of roadway with the cracks ranging from 3 to 5 mm in width. This amount of cracking is extensive;

based upon testing and field performance, the roadway is determined to be lying on an expansive subgrade.

4.2.17 Site 18: FM 685 [Branyon Clay, BR - 685]

After the soil characterization and centrifuge testing program was completed on the Branyon Clay from Site 18, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. A field visit was not performed as the roadway had just been placed in Winter 2015. Moisture and suction sensors were placed and are further explored in Chapter 5.

Assumed Soil Profile

The soil samples that were taken for this site allowed for the determination of the subgrade bedding with depth and indicated that the top five feet of soil is Branyon Clay underlain by five feet of Krum soil. The Krum soil is not considered to be expansive due to testing that took place in the lab. For the overburden, the depth of the asphalt was taken to be 4 inches with the depth of the base layer being 6 inches. This assumption is consistent among the sites in order to provide a similar comparison between sites in terms of the range of stresses. The Branyon Clay was assumed to be at a dry of optimum moisture content of 25% and a relative compaction of 100%, which resulted in a dry unit weight of 87 pcf and a total unit weight of 108 pcf. The Krum Soil was assumed to be at a dry of optimum moisture content of 16% with a total unit weight of 122 pcf. The soil profile used for both methods is shown in Table 4.70.

Table 4.70: Assumed Soil Profile for Branyon Clay at Site 18

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	123	0.9
1	0	1	Branyon Clay	65	31	34	25	108	177	1.2
2	1	2							286	2.0
3	2	3							394	2.7
4	3	4							502	3.5
5	4	5							611	4.2
6	5	6	Krum Soil	36	19	17	16	122	726	5.0
7	6	7							848	5.9
8	7	8							970	6.7
9	8	9							1092	7.6
10	9	10							1214	8.4
***Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer**										

PVR Calculations

The soil conditions for centrifuge testing program on the Branyon Clay from Site 18 included an initial moisture content of 24% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from seven centrifuge specimens and one free swell specimen were input into the DMS-C spreadsheet, yielding the results shown in Figure 4.128.

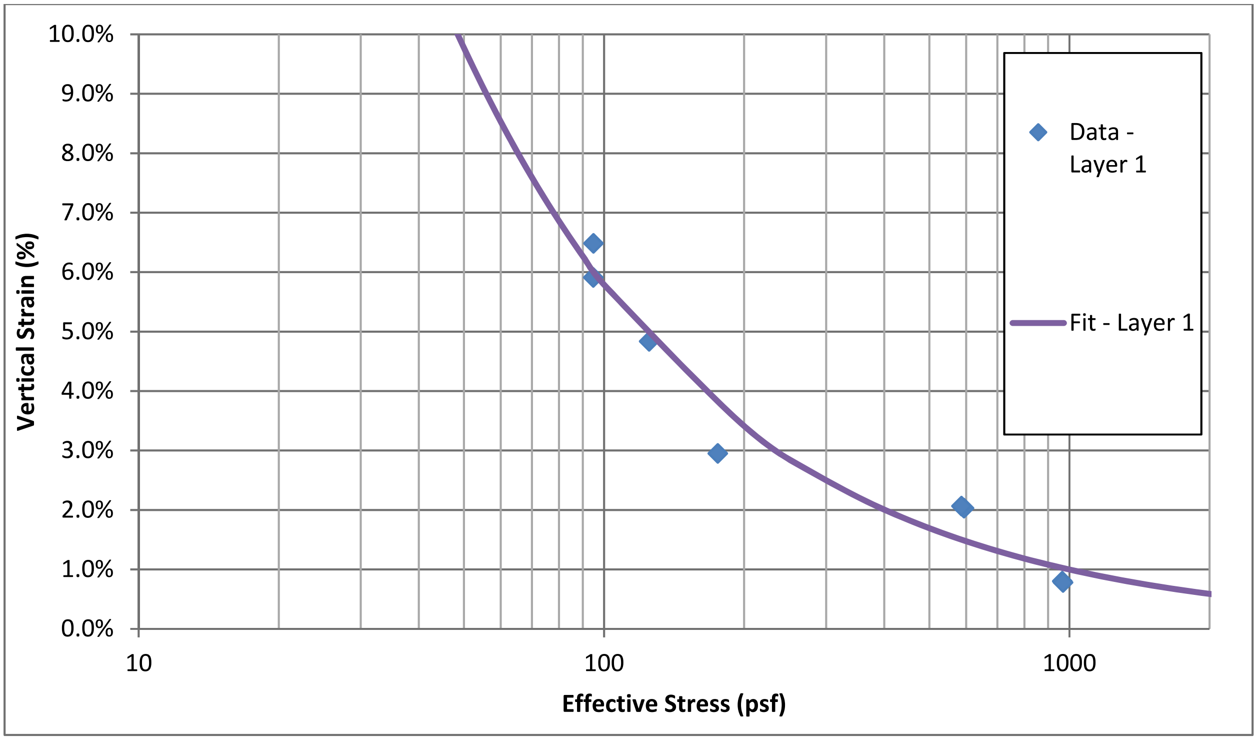


Figure 4.128: Swelling Results and Curve Fitting for Site 18

From the figure, it becomes clear that the soil tested in the centrifuge was only slightly expansive with a small potential for swelling occurring even at very high stresses.

For the Tex-124-E method, the soil profile from Table 4.70 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 97% of the soil passing the No. 40 sieve, as determined from the Wet Sieve tests, was assumed as well as dry conditions for the tests, which corresponded to a moisture content of 23.3% from the correlations in Tex-124-E. The sample unit weights as determined from equations 1 and 2 were used, giving a density correction of 0.87 and a modified No. 40 factor of 0.97 for the sample. The inputs used for the PVR calculations are shown in Table 4.71.

Table 4.71: PVR Input Parameters for Tex-124-E for Site 18

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		173	1.2	-	-		-	-
2	1	HB	260	1.8	65	24.6	108	97.0	34
4	1	HB	486	3.4	65	24.6	108	97.0	34
6	1	HB	706	4.9	65	24.6	108	97.0	34
8	1	HB	925	6.4	65	24.6	108	97.0	34
10	1	HB	1143	7.9	65	24.6	108	97.0	34

By integrating the curve fitted function from Figure 4.128 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1214 psf, the PVR of the subgrade was determined to be 1.85 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 0.95 in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.129, and the comparison between the cumulative PVR versus depth is shown in Table 4.72.

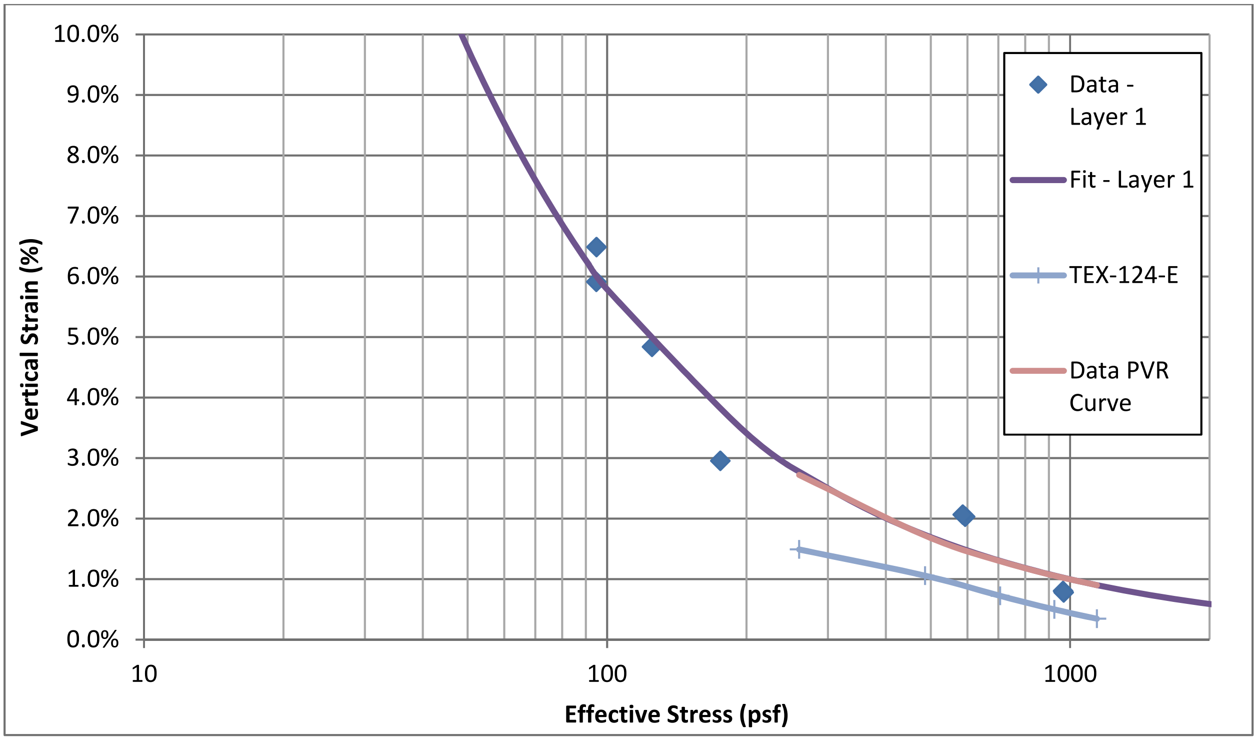


Figure 4.129: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Site 18

Table 4.72: Comparison of PVR Results for Site 18

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**0.99**	**1.85**
2.0	1	HB	260	0.63	1.19
4.0	1	HB	486	0.38	0.78
6.0	1	HB	706	0.20	0.47
8.0	1	HB	925	0.08	0.22
10.0	1	HB	1143	0.00	0.00

Based upon both the centrifuge testing of the Branyon Clay specimens and the Tex-124-E results, the site is considered to rest on an expansive subgrade with remediation techniques necessary. No condition survey was performed because construction of the roadway was scheduled for completed in winter 2015.

4.2.18 Site 19: SH-21 [Cook Mountain Clay – CM]

After the soil characterization and centrifuge testing program was completed on the Cook Mountain Clay from Site 19, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. Regular field visits have taken place to perform condition surveys at the site since the installation in 2013.

Assumed Soil Profile

The soil samples from this site were taken were taken directly at the base-subgrade interface. The assumed pavement structure used for PVR calculations had an asphalt depth of 4 inches and a base layer of 6 inches. This assumption is consistent among the sites in order to provide a similar comparison between sites in terms of the range of stresses. The Cook Mountain Clay was assumed to be at a dry of optimum moisture content of 17% and a relative compaction of 100%, which resulted in a dry unit weight of 95 pcf and a total unit weight of 114 pcf. The soil profile used for both methods is shown in Table 4.73.

Table 4.73: Assumed Soil Profile for Heiden Clay at Site 16

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	178	1.2
1	0	1	Cook Mountain Clay	58	17	41	17	114	287	2.0
2	1	2							401	2.8
3	2	3							515	3.6
4	3	4							629	4.4
5	4	5							743	5.2
6	5	6							857	6.0
7	6	7							971	6.7
8	7	8							1085	7.5
9	8	9							1199	8.3
10	9	10							1313	9.1
***Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer**										

PVR Calculations

The soil conditions for centrifuge testing program on the Cook Mountain Clay from Site 19 included an initial moisture content of 17% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from four centrifuge specimens were input into the DMS-C spreadsheet, yielding the results shown in Figure 4.130.

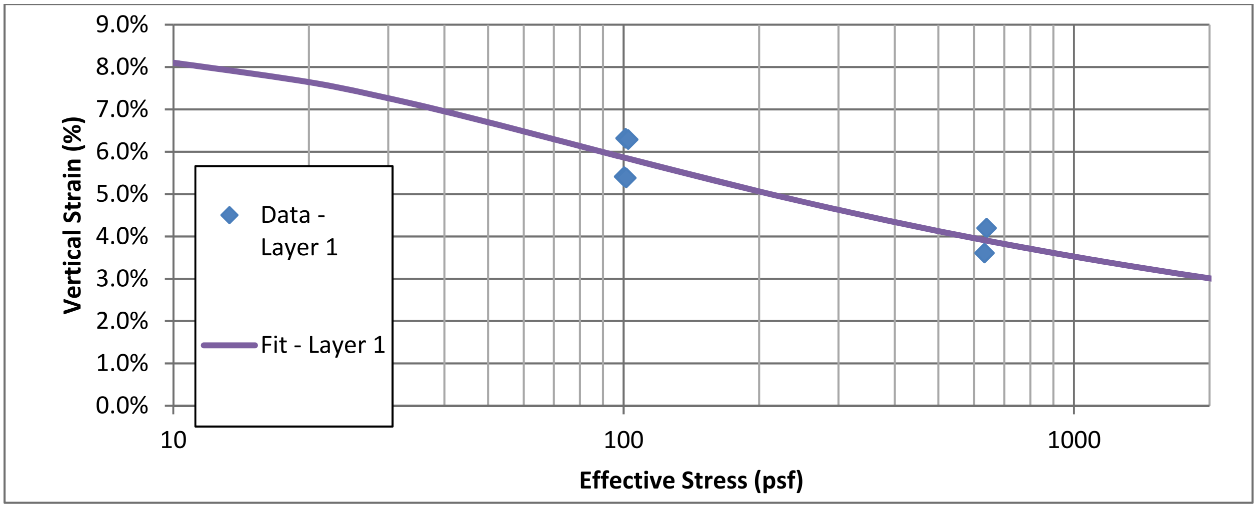


Figure 4.130: Swelling Results and Curve Fitting for Site 19

From the figure, it becomes clear that the soil tested in the centrifuge was slightly expansive with a small potential for swelling occurring even at very high stresses.

For the Tex-124-E method, the soil profile from Table 4.73 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 90% of the soil passing the No. 40 sieve, as determined from the Wet Sieve tests, was assumed as well as dry conditions for the tests, which corresponded to a moisture content of 23.3% from the correlations in Tex-124-E. The sample unit weights as determined from equations 1 and 2 were used, giving a density correction of 0.87 and a modified No. 40 factor of 0.90 for the sample. The inputs used for the PVR calculations are shown in Table 4.74.

Table 4.74: PVR Input Parameters for Tex-124-E for Site 19

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		173	1.2	-	-		-	-
2	1	CM	264	1.8	58	20.1	114	90.0	41
4	1	CM	502	3.5	58	20.1	114	90.0	41
6	1	CM	733	5.1	58	20.1	114	90.0	41
8	1	CM	963	6.7	58	20.1	114	90.0	41
10	1	CM	1192	8.3	58	20.1	114	90.0	41

By integrating the curve fitted function from Figure 4.130 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1159 psf, the PVR of the subgrade was determined to be 4.69 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 1.35 in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.131, and the comparison between the cumulative PVR versus depth is shown in Table 4.75.

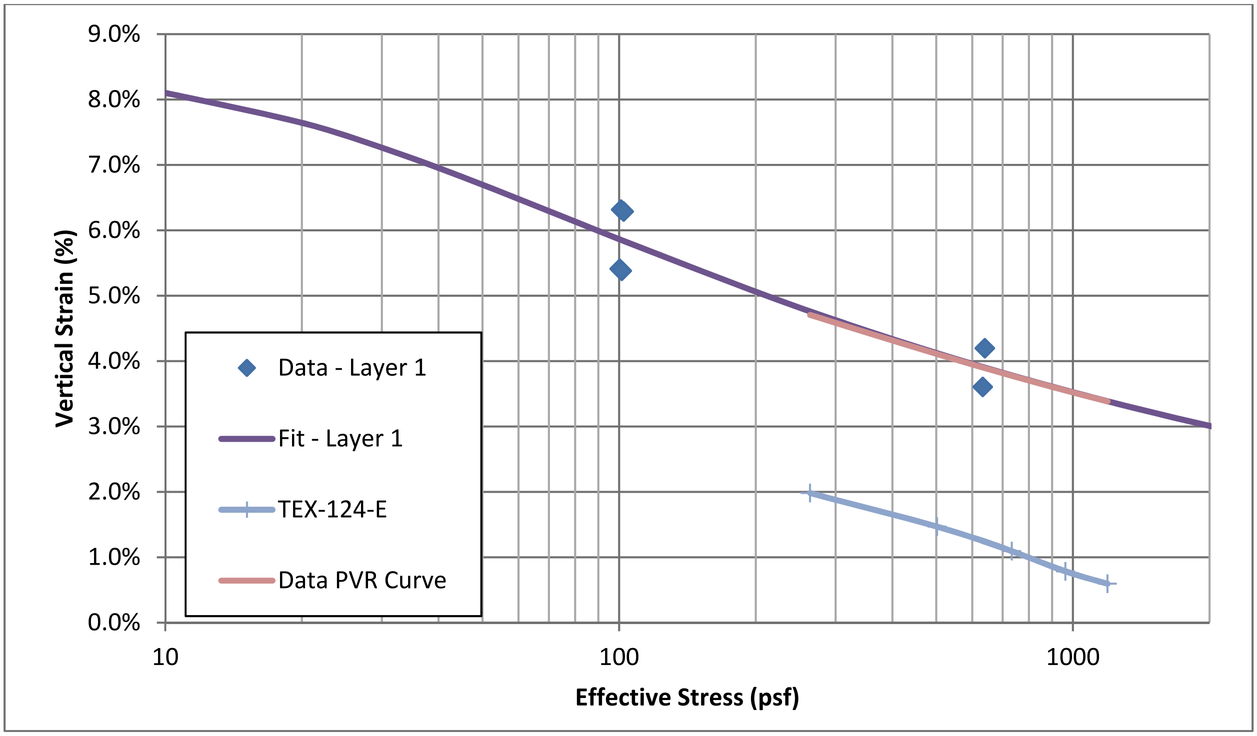


Figure 4.131: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Site 19

Table 4.75: Comparison of PVR Results for Site 19

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**1.42**	**4.69**
2.0	1	CM	264	0.95	3.56
4.0	1	CM	502	0.59	2.57
6.0	1	CM	733	0.33	1.66
8.0	1	CM	963	0.14	0.81
10.0	1	CM	1192	0.00	0.00

Based on both the centrifuge testing of the Cook Mountain Clay specimens and the Tex-124-E results, the site is considered to rest on an expansive subgrade with remediation techniques necessary. Multiple site visits have been performed, and longitudinal edge cracking has occurred as shown in Figure 4.132.

Figure 4.132: Edge Cracking at SH-21 (Garcia 2015)

Distresses in the road range from approximately 7 to 25 percent length of the road over the course of the seven sections in which the Behring occurs. This failure rate is very highly, especially considering the relatively new expansion of the roadway.

4.2.19 Site 20: FM 487 [Branyon Clay, BR - 487]

After the soil characterization and centrifuge testing program was completed on the Branyon Clay from Site 20, the PVR calculations for the DMS-C and Tex-124-E approaches were determined. A field visit was not performed as the roadway had just been placed in Winter 2015. Moisture and suction sensors were placed and are further explored in Chapter 4.

<u>Assumed Soil Profile</u>

The soil samples that were taken for this site allowed for the determination of the subgrade bedding with depth and indicated that the top five feet of soil is Branyon Clay underlain a non-expansive soil. For the overburden, the depth of the asphalt was taken to be 4 inches with the depth of the base layer being 6 inches. This assumption is consistent among the sites in order to provide a similar comparison between sites in terms of the range of stresses. The Branyon Clay was assumed to be at a dry of optimum moisture content of 23% and a relative compaction of 100%, which resulted in a dry unit weight of 91 pcf and a total unit weight of 112 pcf. The soil profile used for both methods is shown in Table 4.76.

Table 4.76: Assumed Soil Profile for Branyon Clay at Site 20

Layer	Depths [ft]		Soil	Liquid Limit	Plastic Limit	Plasticity Index	Water Content [%]	Unit Weight [pcf]	Average Pressure	
	From	To							[psf]	[psi]
-	+0.8	0	*Asphalt + Base Material	0	0	0	-	Varies	123	0.9
1	0	1	Branyon Clay	52	28	24	23	112	177	1.2
2	1	2							286	2.0
3	2	3							394	2.7
4	3	4							502	3.5
5	4	5							611	4.2
***Asphalt + Base Material Pressure is Assumed as a Total Applied Surcharge Load on Top of Soil Layer**										

PVR Calculations

The soil conditions for centrifuge testing program on the Branyon Clay from Site 18 included an initial moisture content of 23% and a relative compaction of 100%. Tests were completed at the prescribed g-levels in the centrifuge to determine the swelling properties for the sample at different stress conditions. In total, data from four centrifuge specimens were input into the DMS-C spreadsheet, yielding the results shown in Figure 4.133.

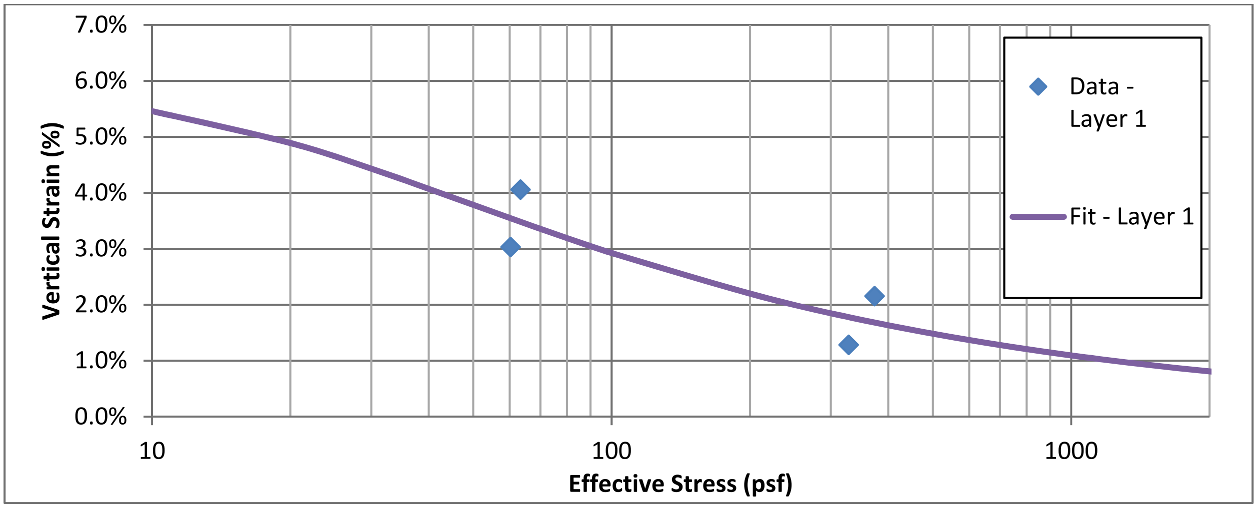


Figure 4.133: Swelling Results and Curve Fitting for Site 20

From the figure, it becomes clear that the soil tested in the centrifuge was only slightly expansive with a small potential for swelling occurring even at very high stresses.

For the Tex-124-E method, the soil profile from Table 4.76 was used with the sample moisture content and unit weight. In order to give the worst case scenario, a fine soil that saw 95% of the soil passing the No. 40 sieve was assumed as well as dry conditions for the tests, which corresponded to a moisture content of 19.4% from the correlations in Tex-124-E. The sample unit weights as determined from equations 1 and 2 were used, giving a binder correction of 0.90 and a modified No. 40 factor of 0.95 for the sample. The inputs used for the PVR calculations are shown in Table 4.77.

Table 4.77: PVR Input Parameters for Tex-124-E for Site 20

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Average Load [psi]	Liquid Limit (LL)	Percent Moisture	Unit Weight [pcf]	Percent - No.40	Plasticity Index (PI)
0	-		123	0.9	-	-		-	-
2	1	BR	262	1.8	52	19.4	112	95.0	28
4	1	BR	496	3.4	52	19.4	112	95.0	28
6	1	BR	723	5.0	52	19.4	112	95.0	28

By integrating the curve fitted function from Figure 4.133 numerically using the trapezoidal rule with 1,000 divisions between the top and bottom stresses of 123 and 1214 psf, the PVR of the subgrade was determined to be 1.71 in. For the Tex-124-E method, an Excel workbook calculated the PVR based upon the input parameters from above and produced a PVR of 0.70 in. The results for both methods, including the PVR curves—i.e., the swelling of each subgrade layer versus the original height of the subgrade layer—are shown in Figure 4.134, and the comparison between the cumulative PVR versus depth is shown in Table 4.78.

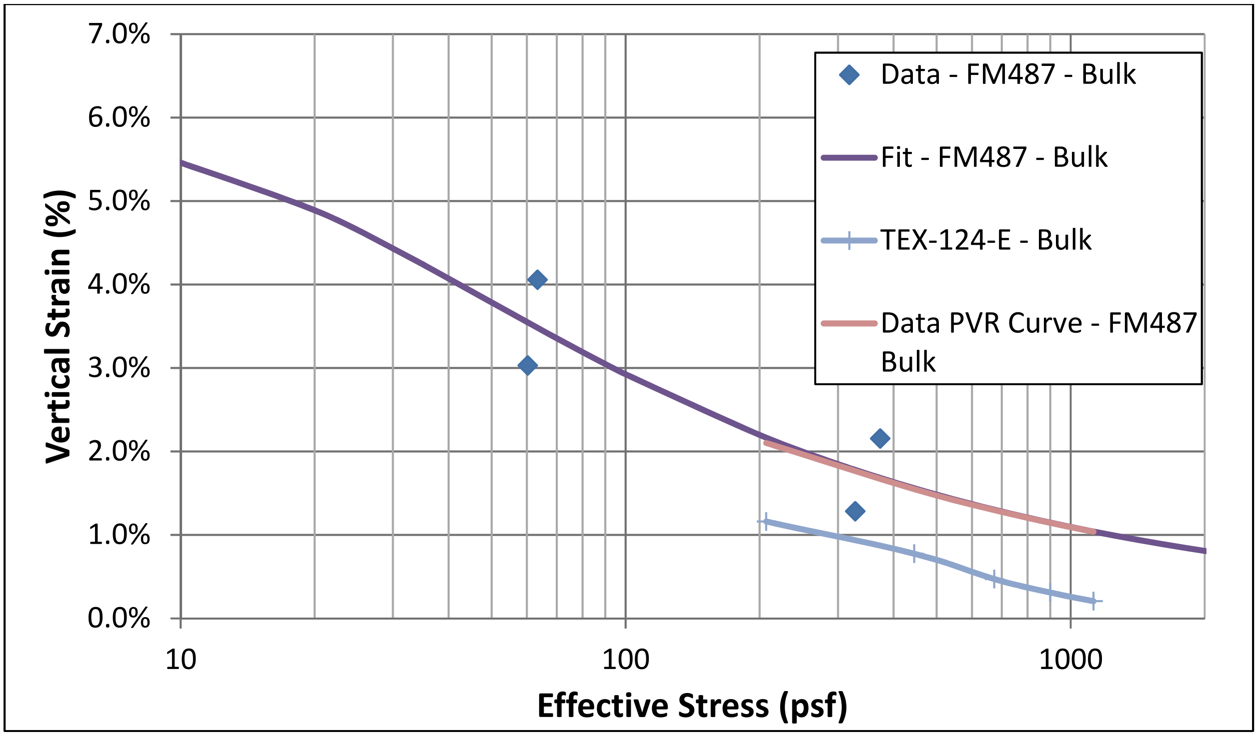


Figure 4.134: Comparison of Swelling Curves from Centrifuge Data and Tex-124-E for Site 20

Table 4.78: Comparison of PVR Results for Site 20

Depth to Bottom of Layer [ft]	Layer	Soil	Average Load [psf]	Tex-124-E PVR (in)	Data PVR (in)
0	-	-	-	**0.70**	**1.71**
2.0	1	BR	207	0.42	1.21
4.0	1	BR	445	0.24	0.84
6.0	1	BR	674	0.12	0.52
8.0	1	BR	900	0.05	0.25
10.0	1	BR	1125	0.00	0.00

4.3 Conclusions from Experimental Testing of Field Sites

In this chapter, the testing of 20 field sites using the newly developed methods was performed. The results show the importance of testing field sites, as the results are not consistent in their prediction of the PVR at each site. These results were used in conjunction with previous results to further expand the database shown in Appendix A.

Chapter 5. Field Validation of Results

This chapter examines PVR versus performance in the field (condition surveys for the summer 2015 sites), as well as the results from FM487 and FM685. Additionally, the early stages of the inundation project are detailed.

5.1 CAPEC Site

During the course of the project, soil samples were provided to the University of Texas at Austin in conjunction with an expanded research project from the Capital Area Pavement Engineer's Council, CAPEC. These soils were taken from formations that are prevalent east of the Balcones fault zone in Central Texas, near various roadways that are currently experiencing pavement distresses. The locations tested were not a part of TxDOT roadways, but the locations had subgrades typical of problematic soil deposits in the TxDOT Austin district. This section covers the laboratory testing for the centrifuge method of expansive soil characterization of undisturbed specimens, as well as other general geotechnical tests and the single and double infiltration approaches for centrifuge characterization of expansive soils for the bulk samples provided at a condition prescribed by Tex-124-E. Overall, four sites provided ten borings, each containing a few inches from a single strata in the subgrade, along with bulk samples for three of the four generalized locations.

5.1.1 Geologic Characterization of CAPEC Soils

Boring data was provided by Rodriguez Engineering Laboratories, LLC (boring B-8) and PaveTex (borings B-11, B-12, B-13, B-15, B-16, B-17, B-18, B-19, and B-20). The general soil characterization testing program was split into two sections; one using samples taken from each of the borings and the other using bulk samples from three of the four general locations. For the bulk samples, Table 3.1 provides results from testing of the bulk samples that were performed at the University of Texas at Austin (UT) which included the determination of the Atterberg Limits of the samples using ASTM D4318 Method A and the determination of the soluble sulfate content using OkDOT OHD L-49 and a colorimeter. For the samples taken from the borings, Table 3.2 provides the results from both the commercial laboratories as well as those performed at the UT. For the laboratory test performed at the university, a single point Liquid Limit test, as prescribed by ASTM D4318 Method B, was performed on each boring in order to the determine the moisture content targeted for moisture conditioning based on the "Dry" condition in Tex-124-E. Also note that for boring B-12, the sample provided to UT was in a different geologic stratum than that tested by the commercial laboratory which explains the difference in the result from Liquid Limit testing. Issues during the general characterization arose during the targeting of specific moisture contents for the moisture conditioning to the "dry" moisture content of specimens prior to centrifuge testing as there were localized points in the provided boring samples that differed from the measured moisture content prior to moisture conditioning. Thus, the moisture contents for the moisture adjusted specimens were not as accurate as desired, but all samples still provided a result from a moisture content that was less than the in-situ moisture content which would give an indication of how the soil would react at a moisture drying than those encountered in the field.

The geologic maps include three main formations on which the borings are located; the Navarro group, the Taylor group and the Pecan Gap chalk. For some scales, however, the Taylor and Navarro groups are combined. The Navarro group is typically described as a clay that is generally dark gray to brown, silty, and contains sandy and calcareous concentrations interbedded with the clay (Dillon et al. 1992). The Taylor group is described as a clay, dark gray to green-gray, that generally has more calcareous cementations in the middle portion of the geologic formation. Finally, the Pecan Gap chalk is described as a limestone unit that is chalk to chalky marl. The geologic maps were obtained from the Geologic Map of the Austin Area, Texas (Dillon et al. 1992) for the Kelly Lane, Forest Bluff Subdivision, and Turnersville Road location and from Geologic Atlas of Texas, Austin Sheet (Barnes 1981) for the Limmer Loop section.

In order to give a comparison between sites, a Single Point PVR Method, based off of Tex-124-E and Zornberg et al. (2013), was developed to determine the PVR for each boring location. This method is outlined in Section 3.1.3. While the Single Point PVR Method only predicts a PVR for the given geologic strata the soil was sampled out, comparisons with the PVR of nearby borings at a different stratum that is present in the boring location will give an idea of whether the boring will be problematic.

Table 5.1: Characterization of Bulk Samples

Location	Liquid Limit	Plastic Limit	Plasticity Index	Soluble Sulfate (PPM)
Turnersville Road	56	19	37	198
Limmer Loop	55	19	36	210
Kelly Lane	70	27	43	116

Table 5.2: Commercial Laboratory Geotechnical Characterization of CAPEC Boring Samples

Boring	Soil Series	Commercial Lab Elev (ft)	LL	PL	PI	ω (%)	γ_d (pcf)	Clay Fraction (%)	Bulk SG
B-8	Houston Black	4 to 6	68	26	42	16.3	101.7	19.4	-
B-11	Ferris Heiden	6 to 8	75	21	54	19.4	100.6	6.3	2.43
B-12	Ferris Heiden	4 to 6	57	18	39	15.2	108.9	7.7	2.513
B-13	Ferris Heiden	3.5 to 5	33	23	10	16.5	-	5.5	2.604
B-15	Houston Black	4 to 6	70	21	49	18	103.6	7.2	2.451
B-16	Houston Black	4 to 6	71	26	45	20	96.6	6.4	2.358
B-17	Branyon	1.5 to 2.5	64	20	44	18	100.9	6.8	2.404
B-18	Branyon	4 to 5	75	25	50	10.1	-	5	2.688
B-19	Branyon	5 to 7	68	22	46	13.5	106.4	7.4	2.53
B-20	Branyon	8.5 to 10	40	14	26	10.5	116	12.5	2.554

Table 5.3: University of Texas Geotechnical Characterization of CAPEC Boring Samples

Boring	Soil Series	UT Elev (ft)	LL (1 Pt)	ω (%)	$\omega_{dry\ target}$ (%)	$\omega_{dry\ actual}$ (%)
B-8	Houston Black	2 to 4	71.36	34.35	23.3	27.7
B-11	Ferris Heiden	8 to 10	76.8	28.13	24.4	23.2
B-12	Ferris Heiden	2 to 4	58.59	24.91	20.7	18.5
B-13	Ferris Heiden	6 to 7	68.79	25.23	22.8	21.7
B-15	Houston Black	2 to 4	64.87	30.63	22	20.9
B-16	Houston Black	2 to 4	74.13	34.64	23.8	22.7
B-17	Branyon	2.5 to 4	74.06	26.78	23.8	23.7
B-18	Branyon	2 to 4	62.13	25.58	21.4	20.1
B-19	Branyon	1.5 to 3.5	60.19	25.89	21	21.4
B-20	Branyon	6 to 7.5	72.52	26.36	23.5	23.1

5.1.2 Single Point PVR Methodology for CAPEC Soils

Since only a few inches of each boring was provided to the University of Texas at Austin with the need for a minimum of four tests performed per boring, the typical centrifuge swelling test regime for each boring consisted of three tests; two tests on specimens at their in-situ moisture condition and one test at a moisture adjusted condition, all at the same stress level that correlated to an expected vertical stress felt in the field. In order to provide a comparison between individual sites by giving a value for the Potential Vertical Rise (PVR), a stress-swell curve was generated for each boring and condition utilizing a single point PVR method. This single point method involves utilizing data from either the Central Texas soils database compiled for TxDOT Austin in a concurrent research project or by using the results from bulk samples' swelling characterization. This database was examined to find a similar soil to those encountered in the field as determined by the USDA NRCS soil surveys and thereby use the previous curve fitting of a similar soil as a tool to generate a stress-swell curve for each of the CAPEC borings and moisture condition. In order to generate the stress-swell curve, a log-linear fit of the vertical strain and vertical effective stress was assumed based on Equation 5.1.

$$Swell\ (\%) = A * \ln(\ \sigma\ [psf]\ ) + B \tag{5.1}$$

For this assumed curve fitting, the "A" coefficient is the slope of the swell-stress curve in semi-log linear space and indicates how rapidly swelling decreases with an increase in vertical effective stress. The "B" coefficient correlates to the amount of swelling a soil would experience if the stress was only 1 psf (i.e. a natural log value of 0), essentially giving a fitting parameter for the "free swell" condition. Since the results from CAPEC only generate the vertical swelling at a single stress, one of these parameters needs to be determined or assumed in order to estimate the stress-swell curve. Assuming that soils of the same geologic and agricultural classification behave similarly, the soils were compared to their best geologic matches in the database for a given moisture condition (i.e. a stress-swell curve generated at a similar moisture content). For example, the samples taken from location B-11 that lies on the Ferris-Heiden soil, according to NRCS soil surveys, would behave similarly to the Tan Taylor/Ferris-Heiden soils previously collected and tested from US-183 and Riverside Dr. Thus, the slope, or the "A" coefficient, for the moisture adjusted condition is assumed to have the same slope as the dry condition tested, a moisture content of optimum minus 3%, on the soils from the Tan Taylor soil. Note that for the three locations with bulk specimens, the most geologically appropriate soil was taken to be the bulk samples from the sites. With this slope, the "B" parameter can be determined using the data solver function in Excel, thereby giving an estimated stress-swell curve for the location. By integrating this stress-swell curve for a given range of stresses that accounts for the conditions in the field (i.e. taking into account the pavement structure above the subgrade and any overlaying geologic stratum), the PVR for each location can be determined for both the in-situ and moisture adjusted condition which gives a relative indication of the level of distress the pavement would experience for a given subgrade. Note that this estimation assumes that if multiple stratums are located in a given boring, any other stratum present would be non-expansive which may be overly conservative. Note that the locations that sit on the Branyon or Houston Black clays typically have an unweather portion of the Taylor Formation that lies beneath the initial deposit which may be expansive.

In order to examine the accuracy of this single-point method, a location which did not see variation in its soil stratum between borings would need to be examined at two separate stress levels for both the in-situ and moisture adjusted condition. The borings from Limmer Loop were selected for this test due to the close proximity between borings and a uniform soil deposit of the Branyon Clay as according to the USDA NRCS soil survey maps. Limmer Loop had three specimens (B-17, B-18, and B-19) that were given at an upper portion of the soil deposit, between 1.5 ft and 4 ft, with another specimen (B-20) that was at a deeper portion of the soil strata, between 6 and 7.5 ft. While the boring logs given by the drillers indicated that the soil stratums would vary between the B-17, B-18, and B-19 borings and the B-20 boring, the B-20 sample had a similar visual complexion and liquid limit from the single point method to the B-17 sample, thus indicating that these samples were from a similar geologic deposit. This assumption allows for a direct comparison between the single point PVR stress-swell curve and a theoretically "correct" curve for the soil stratum. Two curves were generated for both the moisture adjusted and in-situ condition; the first of these curves was using the "A" coefficient from the Central Texas database, and the second was using the results from the test to generate a swell-stress curve. These two curves are shown below in Figure 5.1 using the dry condition and in Figure 5.2for the in-situ condition. In order to understand the sensitivity of the value for "A," the database was gave a value of –1.13%/log cycle from the Limmer Loop bulk samples whereas the fit using B-20 gave an "A" value of -1.52%/log cycle. In order to verify whether there is a significant impact between in the PVR, a PVR was calculated for both the single point, curved fitted results for B-17 and B-20 separately and for the two data points used in conjunction to generate a log-linear stress-swell curve. A cross section of 4 in of asphalt underlain by 10 in of base on top of 8 ft of subgrade was assumed, giving a stress range of 173 to 923 psf. The PVR values assuming these conditions were 2.18 in for B-17 alone, 1.82 in for B-20 alone, and 1.92 in for the log-linear fit between both borings. Thus, these changes are relatively small, and using the geologically similar soil as an estimate for the "A" coefficient will be suitable for this project.

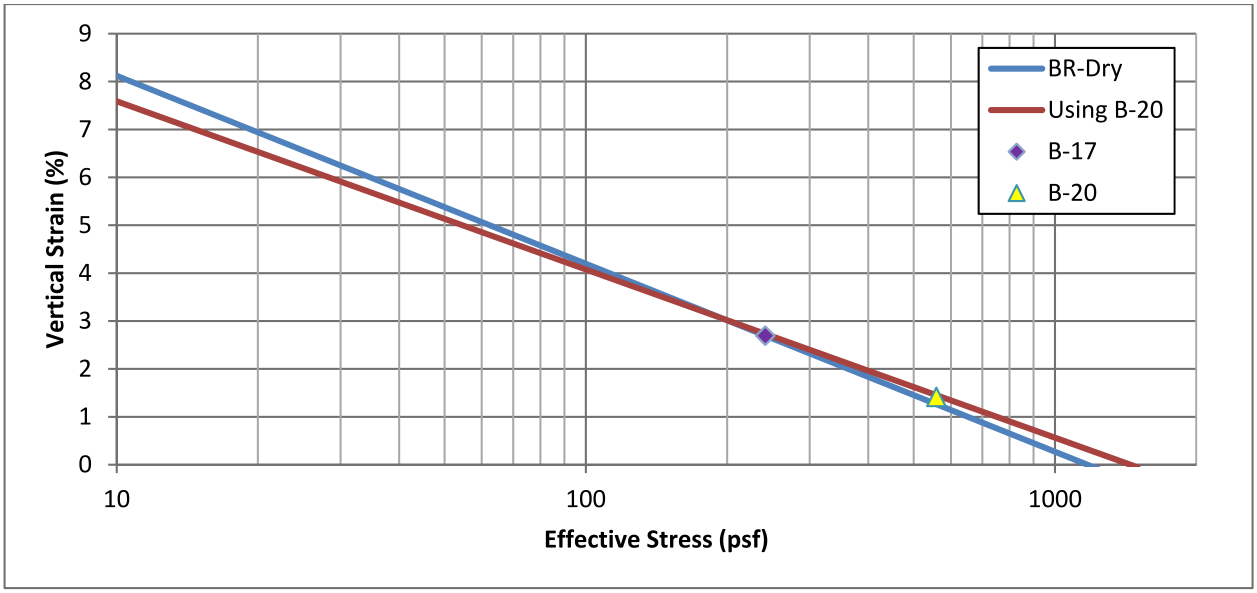


Figure 5.1: Stress-Swell Curves for Moisture Adjusted Conditions

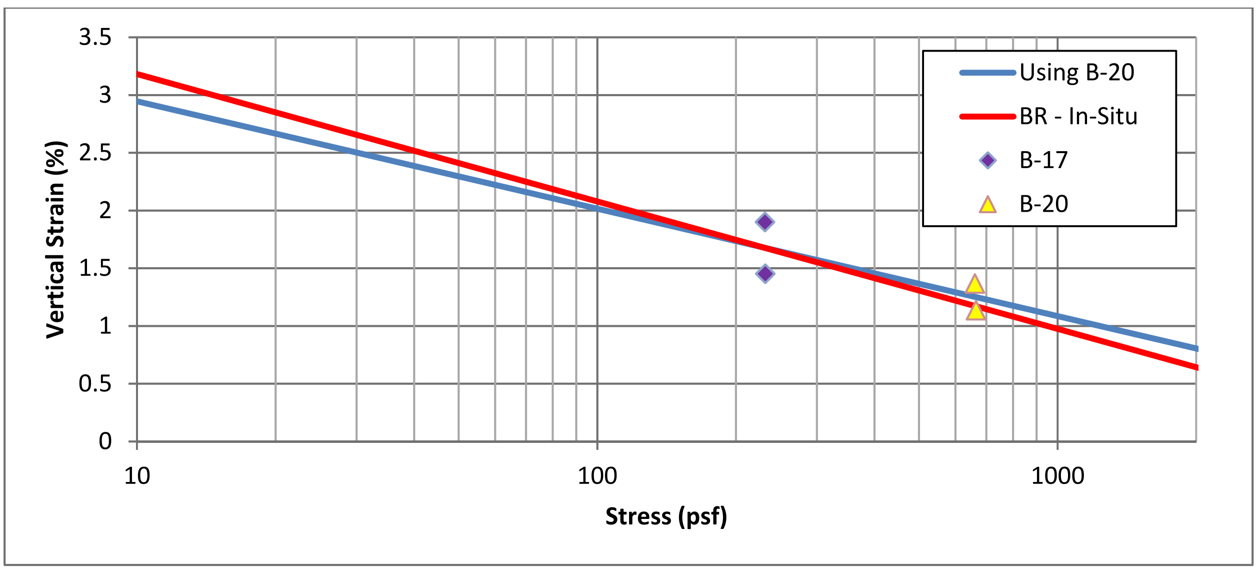


Figure 5.2: Stress-Swell Curves for In-Situ Conditions

The list of soils from the database which were used as well for which borings they were used for are shown in Table 5.1.

Table 5.4: Properties of Database Soils used for Single Point PVR Method

Soil	Liquid Limit	Moisture Condition	ω (%)	"A" Coefficient	Borings Used
Houston Black Clay (HB -Kelly Ln)	70	Dry	22.1	-1.39	B-8
Tan Taylor Clay (TT)	69	DOPT	19.5	-6.369	B-11; B-13
Houston Black Clay (HB - Manor)	62	DOPT	22.5	-1.412	B-12
Branyon Clay (BR - Limmer Loop)	55	Dry	19.3	-1.129	B-17; B-18; B-19; B-20

Note that for some of the location and moisture conditions, the stress-swell curve became negative, i.e. compression instead of expansion, in the range of stresses in the stratum. In order to give a more representative answer, the bottom stress was then assumed to be the stress at which zero swelling occurred as opposed to the calculated bottom stress. The top and bottom stresses were calculated by assuming a unit weight of 150 pcf for the pavement and base material as well as assuming a unit weight of the stratum as determined by the testing program.

5.1.3 Swelling Characterization of CAPEC Soils

Kelly Lane

The location and geologic map for the Kelly Lane is shown in Figure 5.3. The site is located 3.5 miles northeast of Pflugerville and lies on the Taylor formation. A sample soil collected from a depth of 2 to 4 ft below the surface is shown in Figure 5.4. Table 5.2 shows stratigraphy from the driller's boring log that indicates a stratum below the base of the Taylor formation, a dark brown clay that has intermixed sand, gravel and calcareous cementations. The Atterberg Limits and clay fractions suggest that the soil may have a significant potential to swell based on empirical correlations from Tex-124-E. The liquid limit from the boring was 71% based on the single point test from the boring trimmings with a higher than expected moisture content of 34.4% in the boring sample provided to UT. The liquid limit from this location is similar to those from the commercial laboratory indicating that the dark brown clay stratum is fairly homogenous.

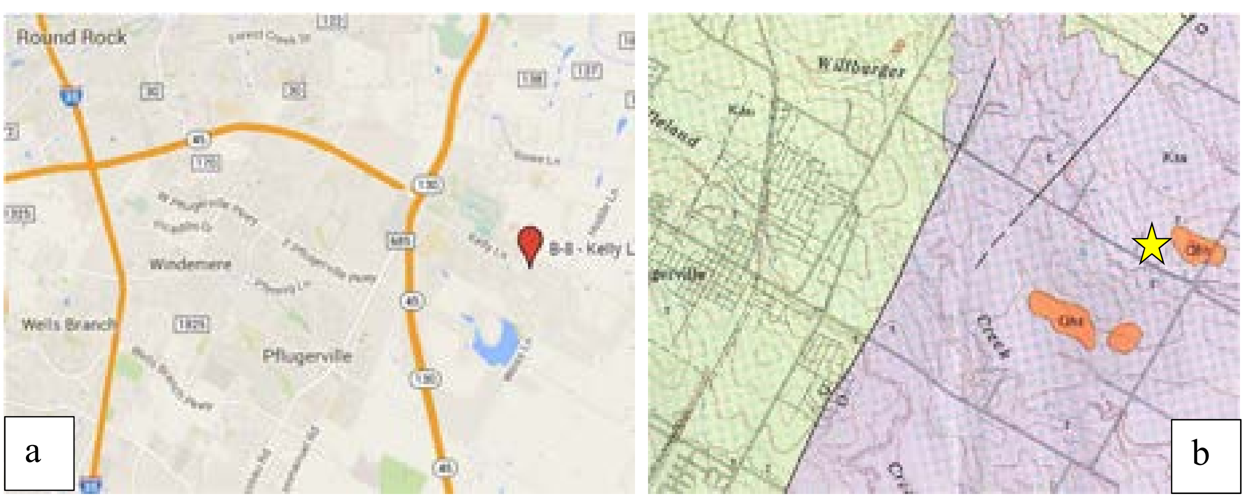


Figure 5.3: Location of Kelly Lane (a) and Geologic Map (b)

Figure 5.4: Sample Provided for B-8

Table 5.5: Boring Log for B-8

Depth	Layer Description	ω (%)	Liquid Limit (%)	Plasticity Index (%)	γ_d (pcf)	Clay Fraction (%)
	Pavement/Base					
5	Dark Brown Clay (SC) with fine sand, gravel, and calcium carbonate (Fill)	16.3	68	42	101.7	19.4
	Light Brown Clay (CH) with fine rounded gravel (LCD)					
10	Tan Clay (CL) with calcium carbonate seams	19.2	34	18	104.5	14.7

The samples used for centrifuge testing are shown in Figure 5.5, which includes a view of the outer portion of the slices shown for those tested at the in-situ conditions. The initial conditions of the soil along with the results of testing are shown in Table 5.3 along with the vertical strain versus time of inundation for the tests in Figure 5.6. A free swell test was run in order to verify the results taken from the in-situ specimens and that, due to the limited amount of soil provided to UT, a reconstituted specimen from the trimmings of the soil, compacted at a similar dry density and moisture content of the in-situ soil specimens, was moisture conditioned for the moisture adjusted specimen.

Figure 5.5: B-8 Samples tested at In-Situ (a) and Moisture Adjusted (b) Conditions

Table 5.6: Summary of Results for B-8

Date of Testing	ω_i (%)	γ_d (pcf)	Primary Swelling (%)	End of Primary Swelling (hr)	Vertical Stress (psf)
5/28/14	34.8	80	0.4	17.7	252
5/28/14	33.9	84	1.4	3.9	255
11/9/14	27.7	90	3.0	9.8	263
5/31/14 – Free Swell	32.7	86	1.0	3.5	250

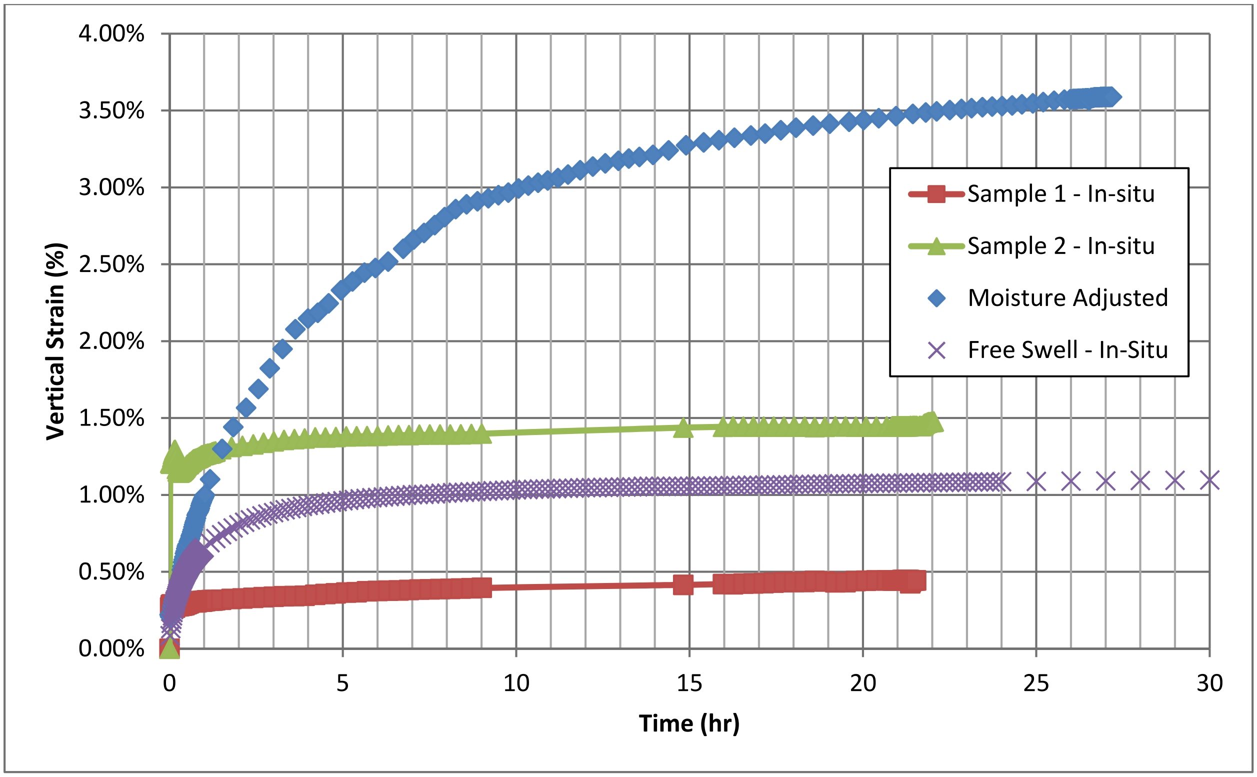


Figure 5.6: Swell-Time Curves for B-8 Samples

The results indicate that, for a vertical stress of approximately 250 psf which corresponds to the vertical stress at the top of the boring specimen in the field, the stratum does not swell a significant amount. An interesting observation, however, is that the moisture adjusted specimen saw primary swelling occur over a longer timeframe than those samples that were tested at their in-situ conditions. Overall, using the single point PVR method with the soil being correlated to existing data for the Branyon clay from the reconstituted specimens taken from the site, this sublayer from a depth of 2 to 5 ft has a PVR of 0.62 in for the moisture adjusted condition. Both of these results are well below the TxDOT tolerances and indicate that the soil layer should not be problematic for low-rise infrastructure.

Additionally, bulk samples were taken at this location and provided to UT in order to test the specimens compacted at the TxDOT "dry" condition to generate swelling results over a wide range of effective stress to compare to results from the boring samples. The bulk sample provided was a dark brown fat clay with a liquid limit of 70% as shown in Table 3.1. A stress-swell curve was generated for the bulk sample provided to UT and is shown in Figure 5.7.

Figure 5.7: Stress-Swell Curves for Bulk Samples from Kelly Lane

While the bulk soil sample provided visually appeared similar to the soil taken from the borings and gave a similar liquid limit, the bulk soil samples experienced a significantly higher amount of swelling than those taken from the boring specimen. However, based on the PVR approach and integrating the stress-swell curve over a similar stress range as the boring specimen, the PVR for this soil is similar with a value of 0.73 in. However, the soil will be very problematic if the confining pressure is very low. Note that the Atterberg Limits between the bulk sample and boring are similar, indicating that there may be an issue or possible contamination with either specimen and illustrates the issue with using only the Atterberg Limits as Tex-124-E does.

Forest Bluff Subdivision (Borings B-11, B-12, and B-13)

The location for the next three borings is in a residential area, the Forest Bluff subdivision, which is located approximately 10 miles east of Austin and is shown in Figure 5.8 along with a geologic map of the surrounding area. The subdivision is geologically close to older Taylor group deposits surrounded by much more recent Quaternary deposits. These Quaternary deposits consist of more granular soils from the weathering of previous rocks and alluvial deposition from the Colorado River and are typically not considered an area where expansive soils would be an issue. Three borings were taken in the subdivision and indicate that the general subgrade consists of a dark brown fat clay underlain by a tan fat clay, a typical stratigraphy encountered in the Taylor group. This soil strata indicates that the depth to the unweather Taylor Shale is much lower, which would be indicative of the alluvial deposits near the Taylor Group. As such, the lower clay is a "Tan Taylor" clay, which is also known as the Heiden Clay.

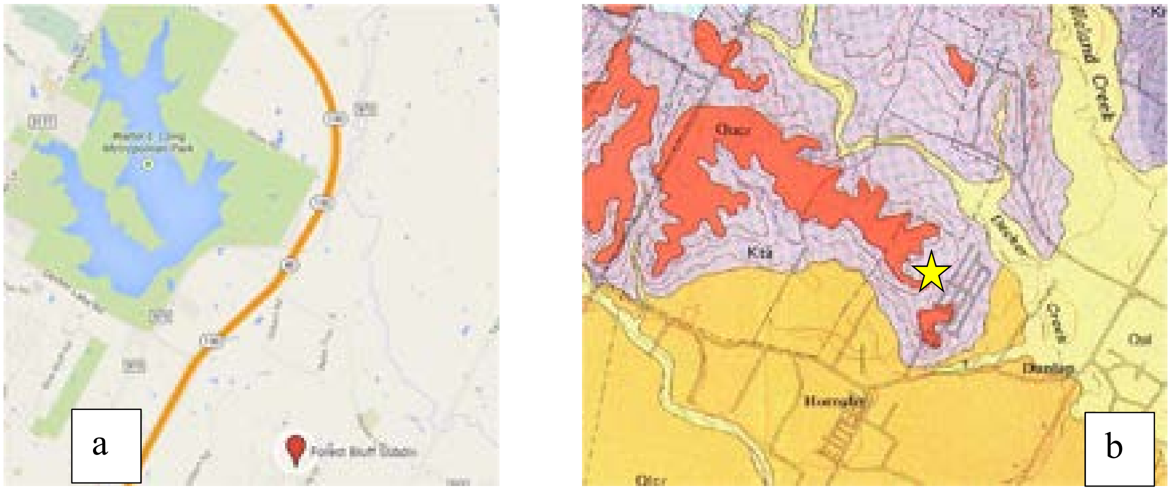


Figure 5.8: Location of Forrest Bluff Subdivision (a) and Geologic Map (b)

<u>*B-11*</u>

The first boring from this location is taken from a depth of 8-10 feet below the pavement surface and is shown in Figure 5.9 with the boring log and results from the commercial laboratory testing shown in Table 5.4. The liquid limit from the boring was 75% based on the single point test from the boring trimmings, similar to the liquid limit from the commercial laboratory, with a higher than expected moisture content of 28 % in the boring sample provided to UT. The Atterberg Limits suggest that the soil may have a significant potential to swell based on empirical correlations from Tex-124-E.

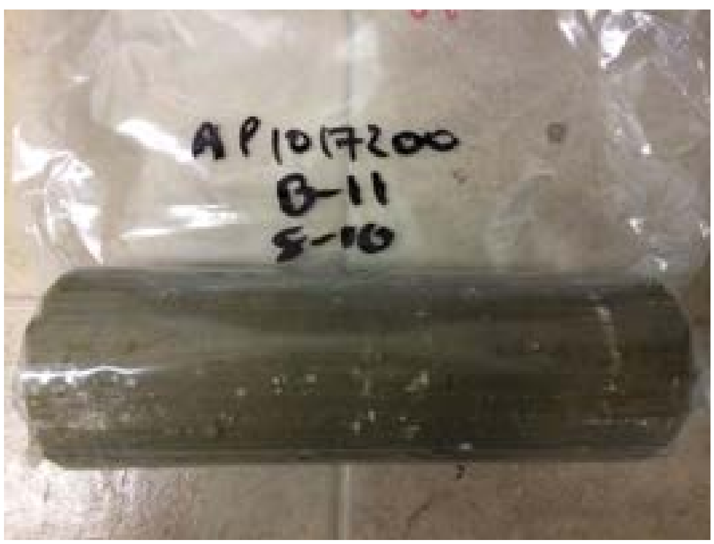


Figure 5.9: Sample Provided for B-11

Table 5.7: Boring Log for B-11

Depth	Layer Description	ω (%)	Liquid Limit (%)	Plasticity Index (%)	γ_d (pcf)	Clay Fraction (%)
	Pavement/Base (Note: Depth ends at 1.5 ft)					
5	Tan Fat Clay (CH) with calcite deposits, moist -light gray	16.7	68	45	101.5	7.7
		19.4	75	54	100.6	6.3
10						

While the boring appears to be a dark brown clay from the outer portion of the boring, the soil is a tan clay with some calcite deposits within the soil matrix, matching the boring log stratum and the soil series, which classifies the surficial soils as the Ferris-Heiden complex. The samples used for centrifuge testing are shown below in Figure 5.10 for the three specimens. The initial conditions of the soil along with the results of testing are shown in Table 5.5 along with the vertical strain versus time of inundation for the tests in Figure 5.11.

Figure 5.10: B-11 Samples at In-situ (a,b) and Moisture Adjusted (c) Conditions

Table 5.8: Summary of Results for B-11

Date of Testing	ω_i (%)	γ_d (pcf)	Primary Swelling (%)	End of Primary Swelling (hr)	Vertical Stress (psf)
6/21/2014	27.5	93	2.4	4.0	1004
6/21/2014	28.8	94	2.7	4.0	1007
10/28/2014	23.2	101	5.5	14.6	954

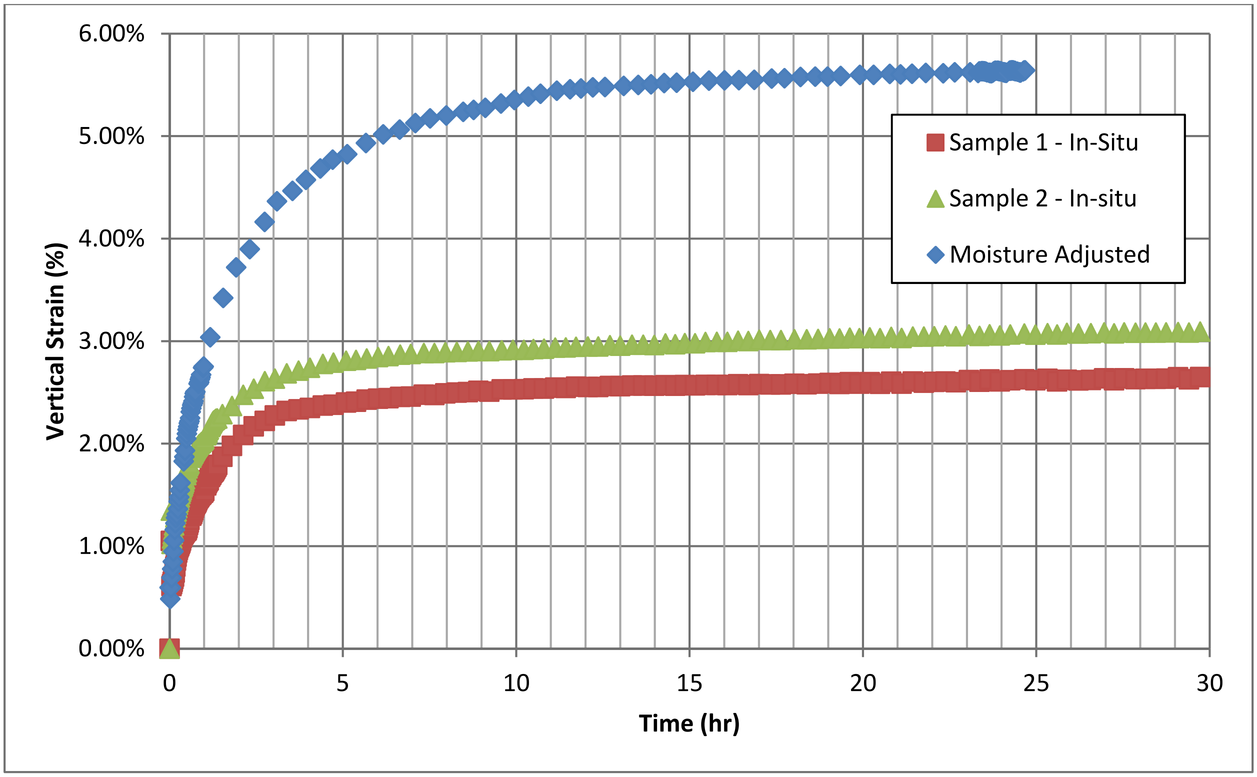


Figure 5.11: Swell-Time Curves for B-11 Samples

The results indicate that, for a vertical stress of approximately 900 psf which corresponds to the vertical stress felt at the middle of the depths of the sample provided, the soil does have a high capacity to swell, even under a significant amount of stress as shown by a single-point PVR of 7.24 in for the moisture adjusted specimens. Thus, this tan fat clay appears to be a stratum that will be extremely problematic for the Forest Bluff subdivision.

<u>*B-12*</u>

The second boring from the Forest Bluff subdivision is shown in Figure 5.12 and was collected from a depth of 2 to 4 feet below the pavement surface with the boring log and summary of laboratory testing shown in Table 5.6. The liquid limit from the boring was 58.6% based on the single point test from the boring trimmings, similar to the liquid limit from the commercial lab, with a higher than expected moisture content of 24.91% in the boring sample provided to UT. The Atterberg Limits suggest that the soil may have a significant potential to swell based on empirical correlations from Tex-124-E.

Figure 5.12: Sample Provided for B-12

Table 5.9: Boring Log for B-12

Depth	Layer Description	ω (%)	Liquid Limit (%)	Plasticity Index (%)	γ_d (pcf)	Clay Fraction (%)
	Pavement/Base					
	Brown to black Fat Clay (CH) with sand and gravel, moist					
5	-light gray	15.2	57	39	108.9	7.7
		19.4	48	30	109.3	9.2
10						

The boring is visually a dark brown clay with a significant amount of gravel and possible calcite deposits in the soil matrix. Several portions of the boring also indicate the possibility of vugs, voids, and local discontinuities within the soil. Note that the boring specimen was divided and trimmed prior to the moisture conditioning stage of testing, thus a reconstituted specimen from the trimmings of the in-situ specimen was compacted to a similar dry density and moisture content as the original boring specimen provided to UT and then moisture conditioned for this moisture adjusted specimen. The samples used for centrifuge testing are shown below in Figure 5.13 for the three specimens. The initial conditions of the soil along with the results of testing are shown in Table 5.7 along with the vertical strain versus time of inundation for the tests in Figure 5.14.

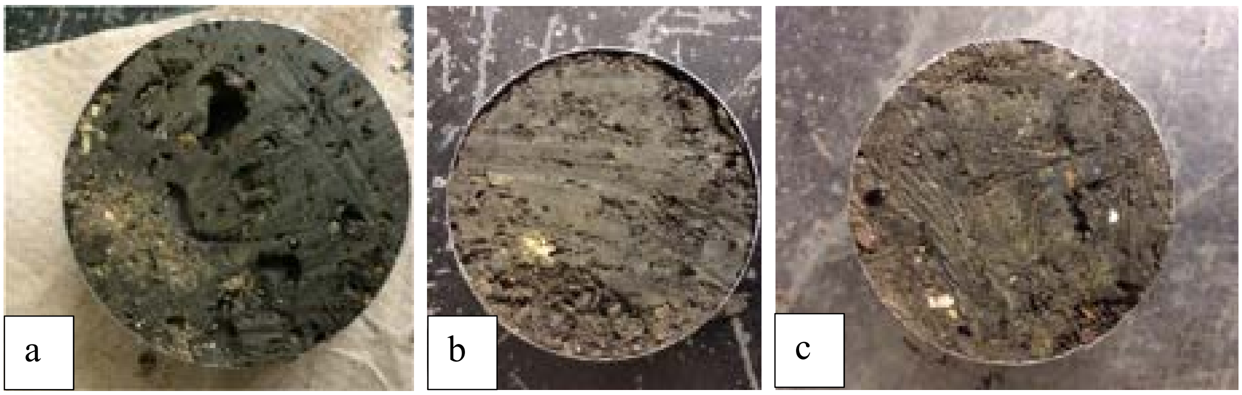


Figure 5.13: B-12 Samples at In-situ (a,b) and Moisture Adjusted (c) Conditions

Table 5.10: Summary of Results for B-12

Date of Testing	ω_i (%)	γ_d (pcf)	Primary Swelling (%)	End of Primary Swelling (hr)	Vertical Stress (psf)
6/10/2014	24.4	79	0.33	7.2	170
6/10/2014	25.5	83	0.07	1.0	167
11/7/2014	18.5	106	4.5	7.6	203

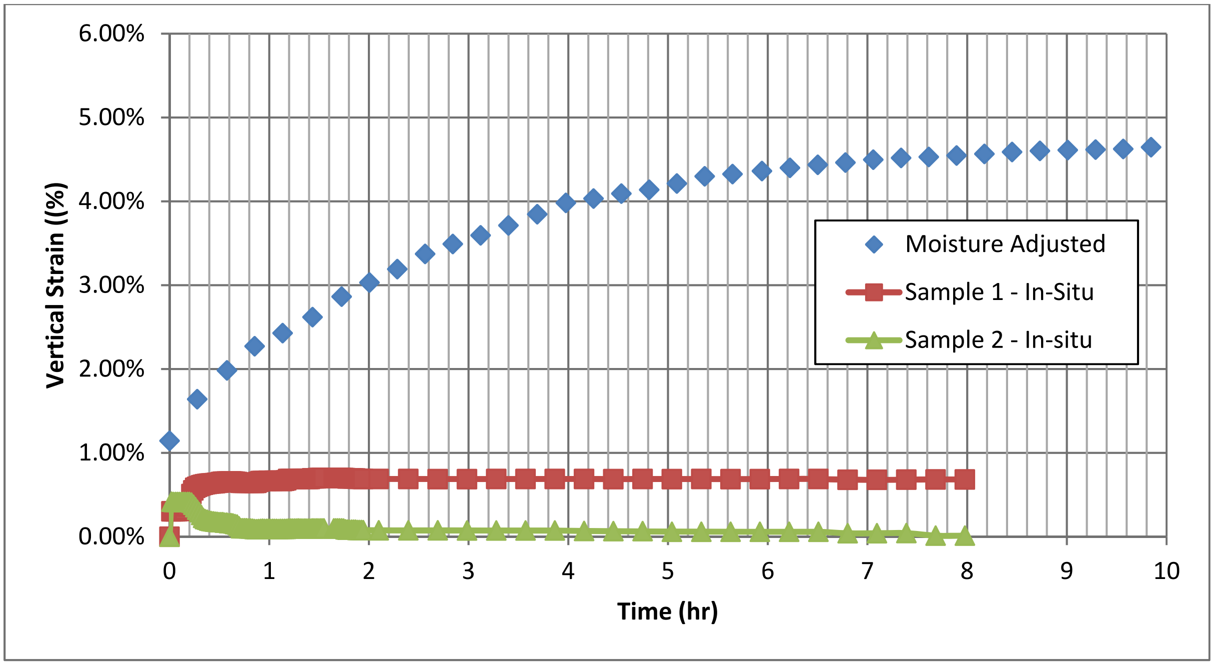


Figure 5.14: Swell-Time Curves for B-12 Samples

The results indicate that, for a vertical stress of approximately 150 psf which corresponds to the vertical stress at the top of the boring specimen in the field, the soil will see only a limited amount of swelling for the in-situ condition which is greatly increased when the specimen becomes dry. The single point PVR indicates that there would a PVR of 3.12 in for the moisture adjusted scenario. Therefore, preventing loss of moisture into this location will be key as a drier specimen will experience a significantly higher amount of swelling that pushes the stratum into an unacceptable level of PVR than those at the in-situ conditions at the time of sampling.

B-13

The last boring from the Forest Bluff subdivision is shown in Figure 5.15 and was collected from a depth of 6 to 7 feet below the pavement surface with a summary of the boring log and laboratory testing in Table 5.8. The liquid limit from the boring was 68.8% based on the single point test from the boring trimmings, similar to that taken from the commercial laboratory, with a higher than expected moisture content of 25.23% for the boring sample provided to UT. The Atterberg Limits suggest that the soil may have a significant potential to swell based on empirical correlations from Tex-124-E.

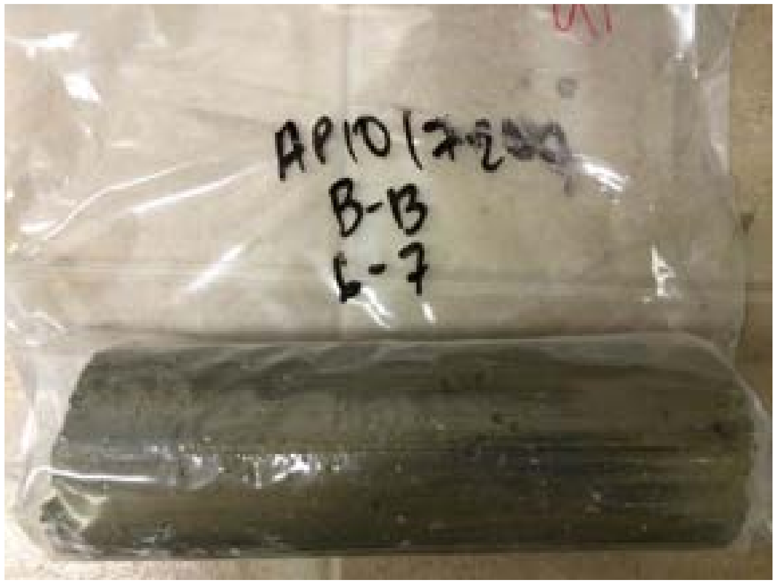


Figure 5.15: Sample Provided for B-13

Table 5.11: Boring Log for B-13

Depth	Layer Description	ω (%)	Liquid Limit (%)	Plasticity Index (%)	γ_d (pcf)	Clay Fraction (%)
	Pavement/Base (Note: Depth ends at 1.5 ft)					
	Brown to black Fat Clay (CH) with sand, moist (Fill)	16.5	33	10		5.5
5	-sands and gravel seams intermixed to depth of 6.5'					
	Tan and light gray Fat Clay (CH) with calcite deposits, moist					
10		15.3	71	46	108.6	8.2

The soil is a tan and light gray clay that does have a significant amount of calcite deposits and vugs, similar to the sample from boring B-11. The samples used for centrifuge testing are shown below in Figure 5.16for the three specimens. The initial conditions of the soil along with the results of testing are shown in Table 5.9 along with the vertical strain versus time of inundation for the tests in Figure 5.17.

Figure 5.16: B-13 Samples at In-situ (a, b) and Moisture Adjusted (c) Conditions

Table 5.12: Summary of Results for B-13

Date of Testing	ω_i (%)	γ_d (pcf)	Primary Swelling (%)	End of Primary Swelling (hr)	Vertical Stress (psf)
6/20/2014	25.2	199	2.2	27.5	702
6/20/2014	25.3	109	1.7	14.9	703
10/26/2014	21.7	103	4.2	13.7	658

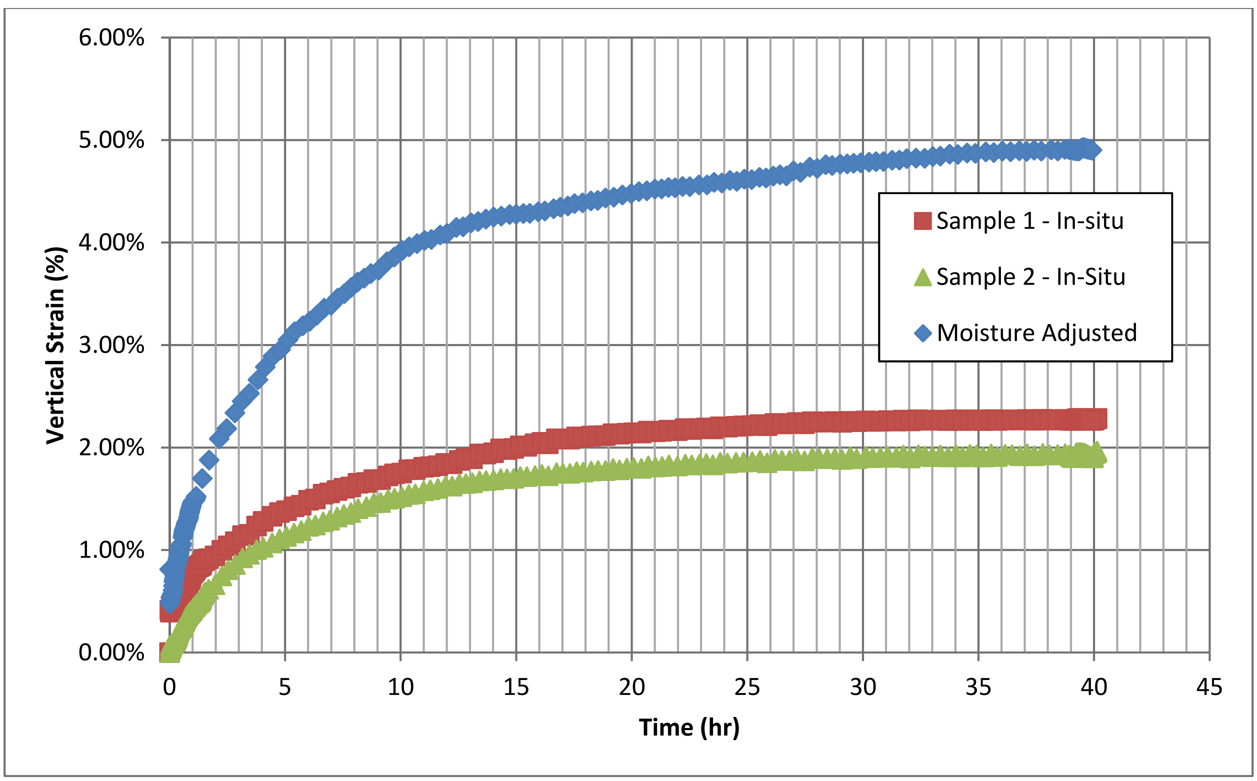


Figure 5.17: Swell-Time Curves for B-13

The results indicate that, for a vertical stress of approximately 650 psf which corresponds to the vertical stress at the top of the boring specimen in the field, the soil will swell significantly under moisture fluctuations at a high vertical effective stress. Based on the single point PVR method, the moisture adjusted PVR is 0.38 in for the stratum which is significant as the single point PVR method does not account for any potential expansive nature of the overlaying stratum and only for the bottom 3.5 ft stratum. Based on the results of the dark brown fat clay from the nearby boring B-12, the dark brown stratum may experience a significant amount of swelling as well if the specimen is dried out, thus a drier subgrade condition will significantly affect the performance of the pavement in this subdivision.

Bulk samples were not provided to UT for testing. Based on the testing done at this location, the dark brown clay will only become problematic as the soil begins to dry, and the tan clay will be expansive at both the in-situ and moisture adjusted conditions. Therefore, prevention of moisture fluctuations in the dark brown clay will be important in the subdivision as a drier stratum there will increase the issue with expansive soils. Further subsurface investigations may be required to determine the stratigraphy in this location in order to see the prevalence of the tan fat clay and whether it is an issue throughout the subdivision.

<u>Turnersville Road (Borings B-15 and B-16)</u>

The site is located approximately 12 miles south of Austin off of Turnersville Road as shown in Figure 5.18 along with a geologic map of the area. The geologic map of the area indicates that the location lies entirely on the Taylor group. Both boring were collected from a depth of 2 to 4 feet below the top surface of the pavement and contain a dark brown fat clay with calcites and sands that is typical of the Taylor group.

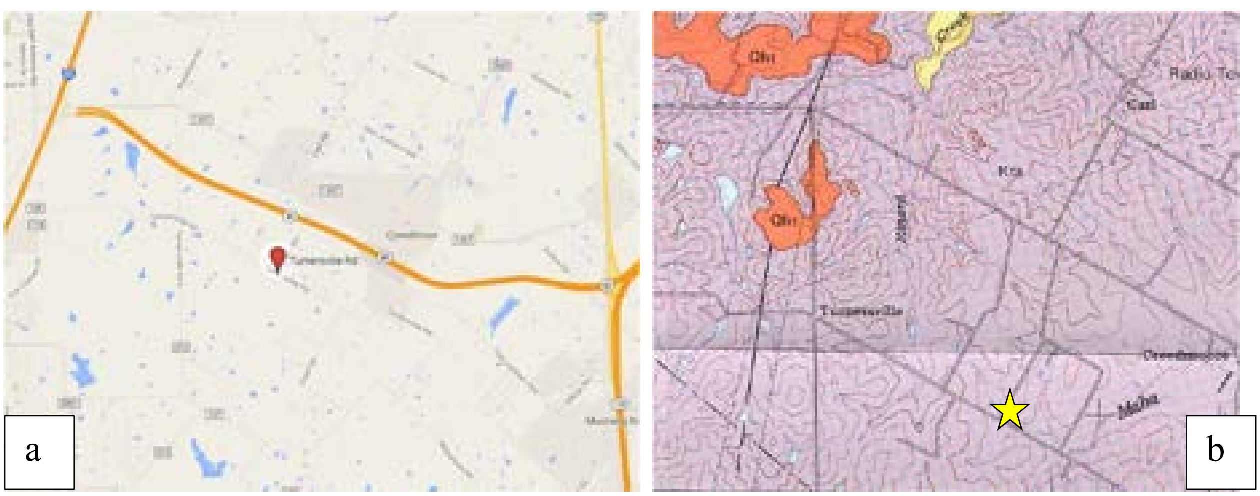


Figure 5.18: Location of Turnersville Road (a) and Geologic Map (b)

<u>B-15</u>

The first boring from Turnersville Road is shown in Figure 5.19 and was collected from a depth of 2 to 4 feet below the pavement surface with the boring log and summary of laboratory testing shown in Table 5.10. The liquid limit from the boring was 64.9% based on the single point test from the boring trimmings, approximately 5% less than the liquid limit from the commercial laboratory results indicating some heterogeneity within the stratum, with a higher than expected moisture content of 30.63% for the boring sample provided to UT. The Atterberg Limits suggest that the soil may have a significant potential to swell based on empirical correlations from Tex-124-E.

Figure 5.19: Sample Provided for B-15

Table 5.13: Boring Log for B-15

Depth	Layer Description	ω (%)	Liquid Limit (%)	Plasticity Index (%)	γ_d (pcf)	Clay Fraction (%)
	Pavement/Base					
5	Dark gray Fat Clay (CH), with calcite and sand	18	70	49	103.6	7.2
10	Tan Fat Clay (CH), with calcite seams	14.6	71	49	112.2	8.3

The soil lies within the dark gray and brown fat clay with some noticeable portions of calcite and sand. A significant amount of vugs and local discontinuities were located in the sample, therefore leading to a lower than expected dry density for one of the in-situ samples tested. The samples used for centrifuge testing are shown below in Figure 5.20 with a view of the outer portion of the slices shown for those tested at the in-situ conditions. The initial conditions of the soil along with the results of testing are shown in Table 5.11 along with the vertical strain versus time of inundation for the tests in Figure 5.21.

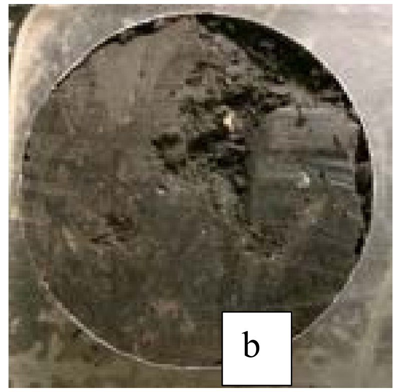


Figure 5.20: B-15 Samples at In-situ (a) and Moisture Adjusted (b) Conditions

Table 5.14: Summary of Results for B-15

Date of Testing	ω_i (%)	γ_d (pcf)	Primary Swelling (%)	End of Primary Swelling (hr)	Vertical Stress (psf)
5/28/2014	16.7	97	-1.2	0.4	244
5/28/2014	30.6	78	1.9	3.9	268
10/24/2014	20.9	100	3.7	9.72	282

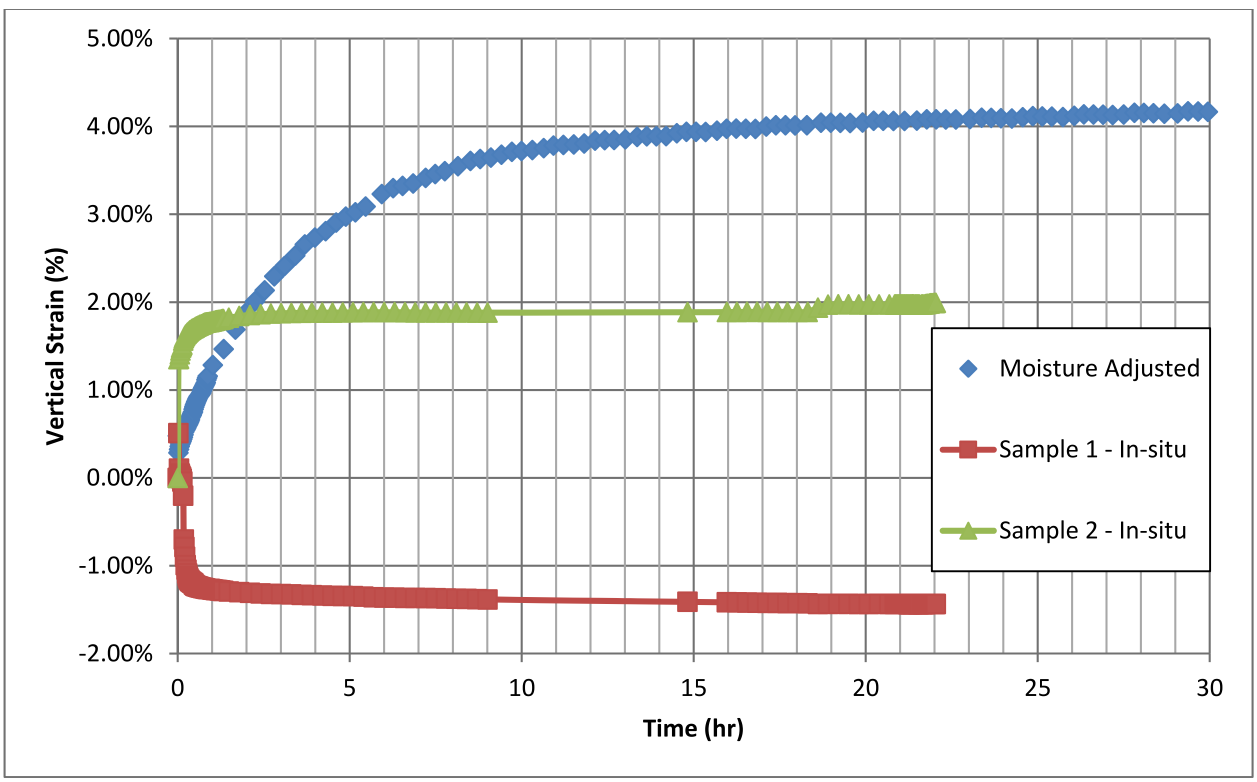


Figure 5.21: Swell-Time Curves for B-15 Samples

Examining Figure 5.21, an issue to note is that one of the in-situ specimens collapsed during wetting as a portion of the specimen included the base layer that contaminated the top portion of the boring. This base layer consisted of much more granular material which skewed the initial conditions by lowering the moisture content and increasing the dry density. The additional base thus consolidated under the addition of water, causing the sample to show a reduction in volumetric strain which is a flawed result. This result is discarded from the subsequent PVR analysis as it is not representative of data from the field. The single point PVR is 1.80 in for the moisture adjusted condition.

<u>*B-16*</u>

The second boring from Turnersville Road is shown in Figure 5.22 and was collected from a depth of 2 to 4 feet below the pavement surface with the boring log and summary of laboratory testing shown in Table 5.12. The liquid limit from the boring was 74.1% based on the single point test from the boring trimmings, slightly higher than the liquid limit from the commercial lab, with a higher than expected moisture content of 34.64%. The Atterberg Limits suggest that the soil may have a significant potential to swell based on empirical correlations from Tex-124-E.

Figure 5.22: Sample Provided for B-16

Table 5.15: Boring Log for B-16

Depth	Layer Description	ω (%)	Liquid Limit (%)	Plasticity Index (%)	γ_d (pcf)	Clay Fraction (%)
	Pavement/Base					
5	Dark gray Fat Clay (CH), with calcite and sand - with sand	20	71	45	96.6	6.4
		18.3	80	58	96	7.6
10						

The soil lies within the same dark gray and brown fat clay stratum. The samples used for testing showed a significant amount of vugs and fissures, indicating that this portion may have natural voids that allow for a relatively quicker flow of water which is evident in the vertical strain vs. time curves for the in-situ specimens. The samples used for centrifuge testing are shown below in Figure 5.23. The initial conditions of the soil along with the results of testing are shown in Table 5.13 along with the vertical strain versus time of inundation for the tests in Figure 5.24.

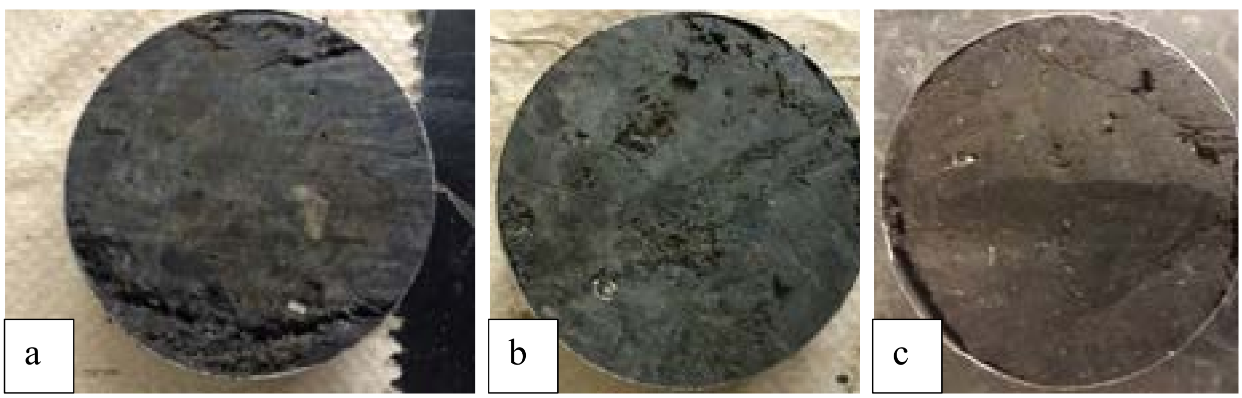


Figure 5.23: B-16 Samples at In-situ (a,b) and Moisture Adjusted (c) Conditions

Table 5.16: Summary of Results for B-16

Date of Testing	ω_i (%)	γ_d (pcf)	Primary Swelling (%)	End of Primary Swelling (hr)	Vertical Stress (psf)
6/13/2014	34.9	81	0.53	1.2	231
6/13/2014	34.4	83	0.51	0.9	233
10/31/2014	22.7	92	7.3	16.6	267

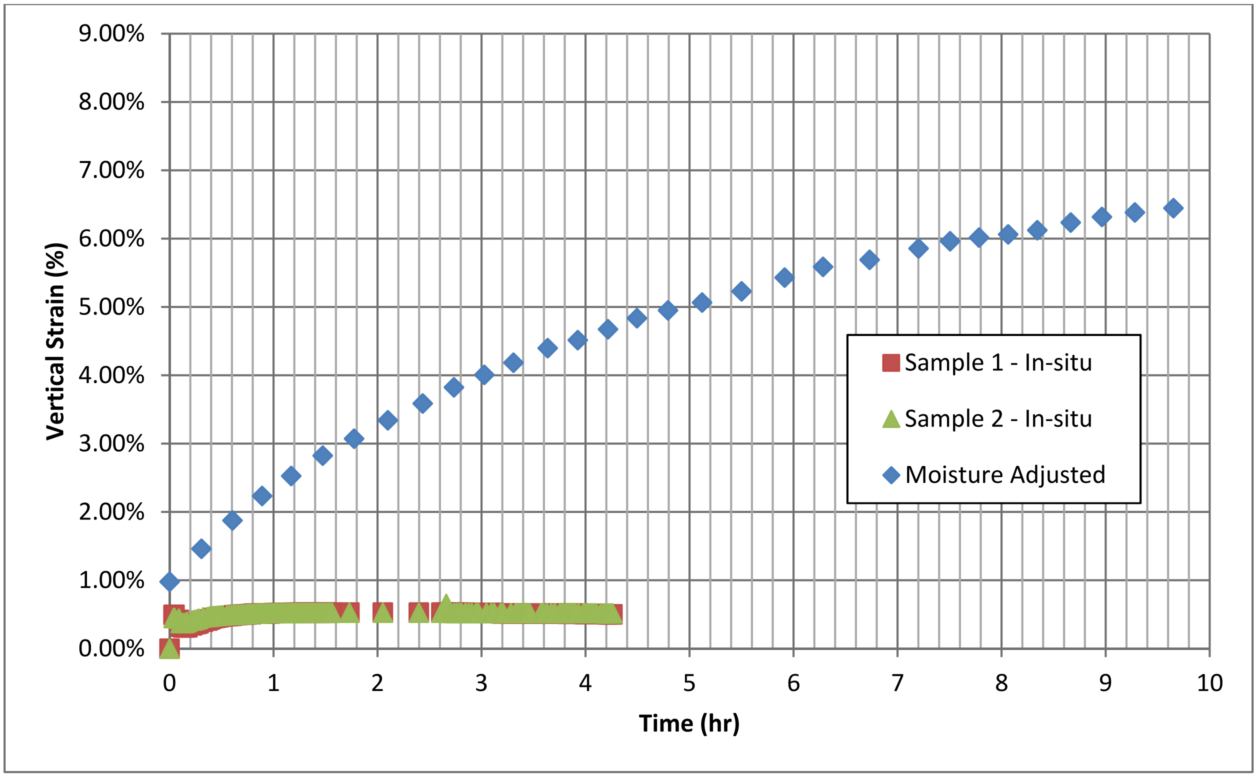


Figure 5.24: Swell-Time Curves for B-16

The results indicate that the soil isn't as expansive as the soil from B-15 in its in-situ condition due to a higher initial moisture content. However, the amount of swelling from a moisture adjusted specimen is very significant and indicates that a drier stratum at this location will show a very significant potential to expand under wetting as opposed to the current in-situ conditions. The significant difference between the in-situ and moisture adjusted specimen is illustrated in the single point PVR results, which gives a single point PVR of 5.53 in for the moisture adjusted condition. Therefore, for Turnersville Road, borings B-15 and B-16 indicate that the stratum will be problematic if the soil experience significant amount of drying during seasonal fluctuations.

Bulk samples were provided to UT to test as well and the stress-swell curve for the given soil is shown below in Figure 5.25. The bulk samples were similar to the soils in terms of complexion, but the liquid limit was significantly less than those taken from the borings. Based on the results, the reconstituted bulk samples would significantly under predict the amount of swelling based on a PVR of 2.47 assuming the same conditions as B-15. Sulfates do not seem to be an issue at the site as shown in Table 5.15, indicating that lime treatment may be a suitable method for treating these soils.

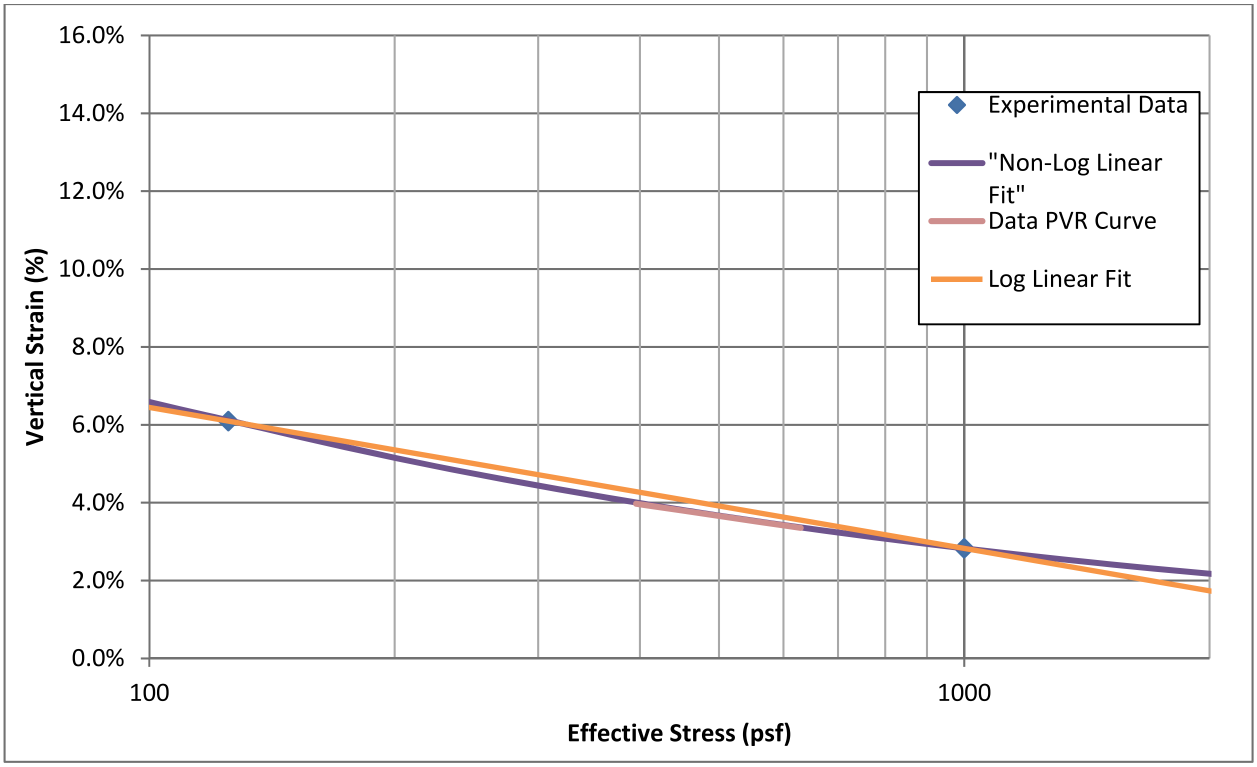


Figure 5.25: Swell-Stress Curves for Turnersville Road Bulk Samples

Limmer Loop (Borings B-17, B-18, B-19, and B-20)

The final location of borings was at a TxDOT test road section, Limmer Loop, which lies between Hutto and Taylor as shown in Figure 5.26, along with the geologic map of the area. The geologic map is from a different geologic map than the other locations, thus the Taylor and Navarro groups are not divided but grouped together. The location lies on the Pecan Gap Chalk according to the geologic map as shown in Figure 5.26, but the areas of interest in the boring indicate that this formation may only be surficial with the Taylor/Navarro formation being the geologic formation at which the borings were taken from. The boring logs indicate a typical cross section of a dark brown fat clay underlain by a tan fat clay, both with a significant portion of gravels and sand. Borings B-17, B-18, and B-19 were taken from the upper portion of the sublayer in the dark brown fat clay strata whereas boring B-20 was taken from the lighter brown strata of the soil.

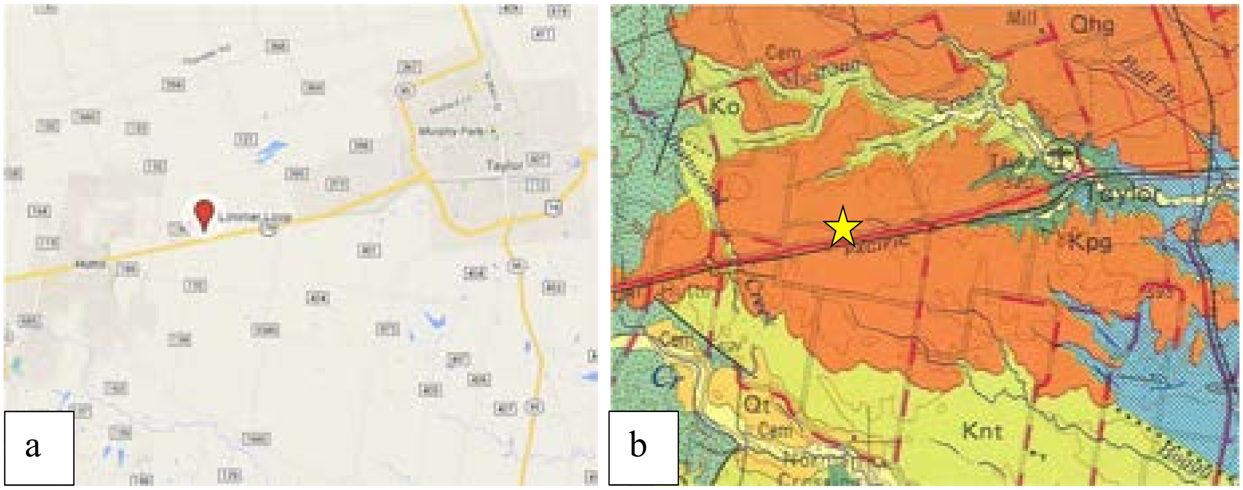


Figure 5.26: Location of Limmer Loop (a) and Geologic Map (b)

<u>*B-17*</u>

The first boring from Limmer Loop was collected from a depth of 2.5-4 feet beneath the pavement surface and is shown in Figure 5.27with the boring log and results from the commercial laboratory testing shown in Table 5.14. The Atterberg Limits suggest that the soil may have a significant potential to swell based on empirical correlations from Tex-124-E. An interesting thing to note is the decrease in the plasticity as the depth increase that is accompanied by an increase in the clay fraction which may indicate that the dark brown stratum can be divided into a stratum that is a fat clay and one which is a lean clay at a lower depth.

Figure 5.27: Sample Provided for B-17

Table 5.17: Boring Log for B-17

Depth	Layer Description	ω (%)	Liquid Limit (%)	Plasticity Index (%)	γ_d (pcf)	Clay Fraction (%)
	Pavement/Base					
	Dark Brown Clay with Gravel - no other discernable information	18	64	44	100.9	6.8
5						
		11.9	38	22		13
10						

The boring sample provided is of a brown clay with some noticeable traces of granular particles. Unfortunately, the boring log was smeared and thus geologic information from the driller's log was not able to be incorporated for this boring. The liquid limit from the boring was 74% based on the single point test from the boring trimmings with a higher than expected moisture content of 26.8% for the sample provided to UT. The samples used for centrifuge testing are shown below in Figure 5.28 with a view of the outer portion of the slices shown for those tested at the in-situ conditions. The initial conditions of the soil along with the results of testing are shown in Table 5.15 along with the vertical strain versus time of inundation for the tests in Figure 5.29.

Figure 5.28: B-17 Samples at In-situ (a,) and Moisture Adjusted (b) Conditions

Table 5.18: Summary of Results for B-17

Date of Testing	ω_i (%)	γ_d (pcf)	Primary Swelling (%)	End of Primary Swelling (hr)	Vertical Stress (psf)
6/6/2014	26.8	95	1.8	22.0	242
6/6/2014	26.8	95	1.4	22.0	242
10/24/2014	23.69	85	2.7	14.2	274

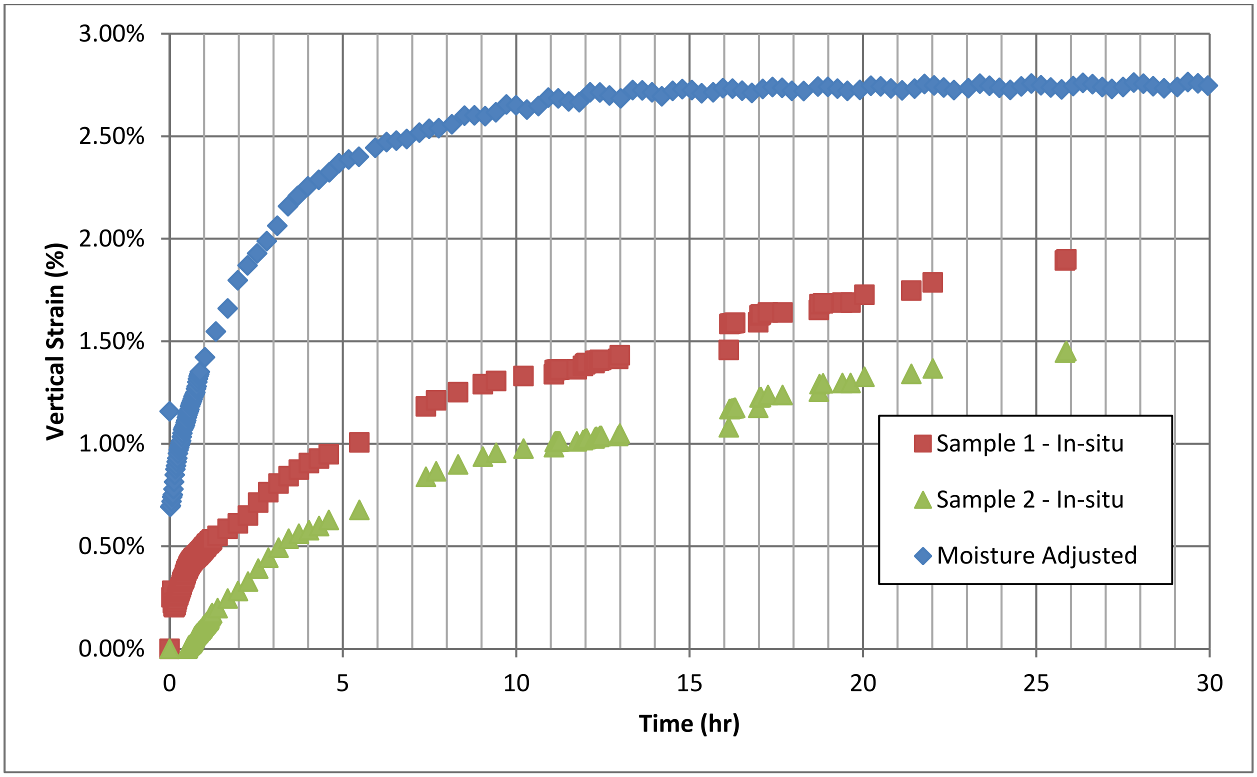


Figure 5.29: Swell-Time Curves for B-17

The results indicate that, for a vertical stress of approximately 250 psf which corresponds to the vertical stress at the top of the boring specimen in the field, the soil will not swell significantly under moisture fluctuations. This indication is further shown in the single point PVR method which gives a value of 1.91 in for the moisture adjusted condition.

B-18

The next boring from this Limmer Loop was collected from a depth of 2-4 feet beneath the pavement surface and is shown in Figure 5.30 with the boring log and results from the commercial laboratory testing shown in Table 5.16. The Atterberg Limits suggest that the soil may have a significant potential to swell based on empirical correlations from Tex-124-E. The liquid limit from the boring was 62% based on the single point test from the boring trimmings with a higher than expected moisture content of 25.6% for the boring sample provided to UT. The liquid limit between the lab and UT tested samples is significantly different, indicating that the soil strata may show some heterogeneity between locations.

Figure 5.30: Sample Provided for B-18

Table 5.19: Boring Log for B-18

Depth	Layer Description	ω (%)	Liquid Limit (%)	Plasticity Index (%)	γ_d (pcf)	Clay Fraction (%)
	Pavement/Base					
	Dark brown and black Fat Clay (CH) - with sand seams	10.1	75	50		5
5						
	Tan to light brown Fat Clay (CH), with sand and chalk/calcite deposits, moist	12	76	34		8.8
10						

The soil lies within the dark brown to black fat clay that was previously seen in B-17 and shows a significant amount of granular material on the external portion of the sample. The samples used for centrifuge testing are shown below in Figure 5.31 with a view of the outer portion of the slices shown for those tested at the in-situ conditions. The initial conditions of the soil along with the results of testing are shown in Table 5.17 along with the vertical strain versus time of inundation for the tests in Figure 5.32. Note that a free swell test was run on trimmings from the in-situ specimens compacted at a similar dry density and moisture as the in-situ conditions that swells over a much longer timeframe.

Figure 5.31: B-18 Samples at In-situ (a) and Moisture Adjusted (b) Conditions

Table 5.20: Summary of Results for B-18

Date of Testing	ω_i (%)	γ_d (pcf)	Primary Swelling (%)	End of Primary Swelling (hr)	Vertical Stress (psf)
5/30/2014	25.7	96	1.2	19.9	292
5/30/2014	25.5	98	1.2	9.6	295
11/9/2014	20.1	101	2.5	8.3	268
6/6/14 – Free Swell of Trimmings	24.5	96	2.8	41	250

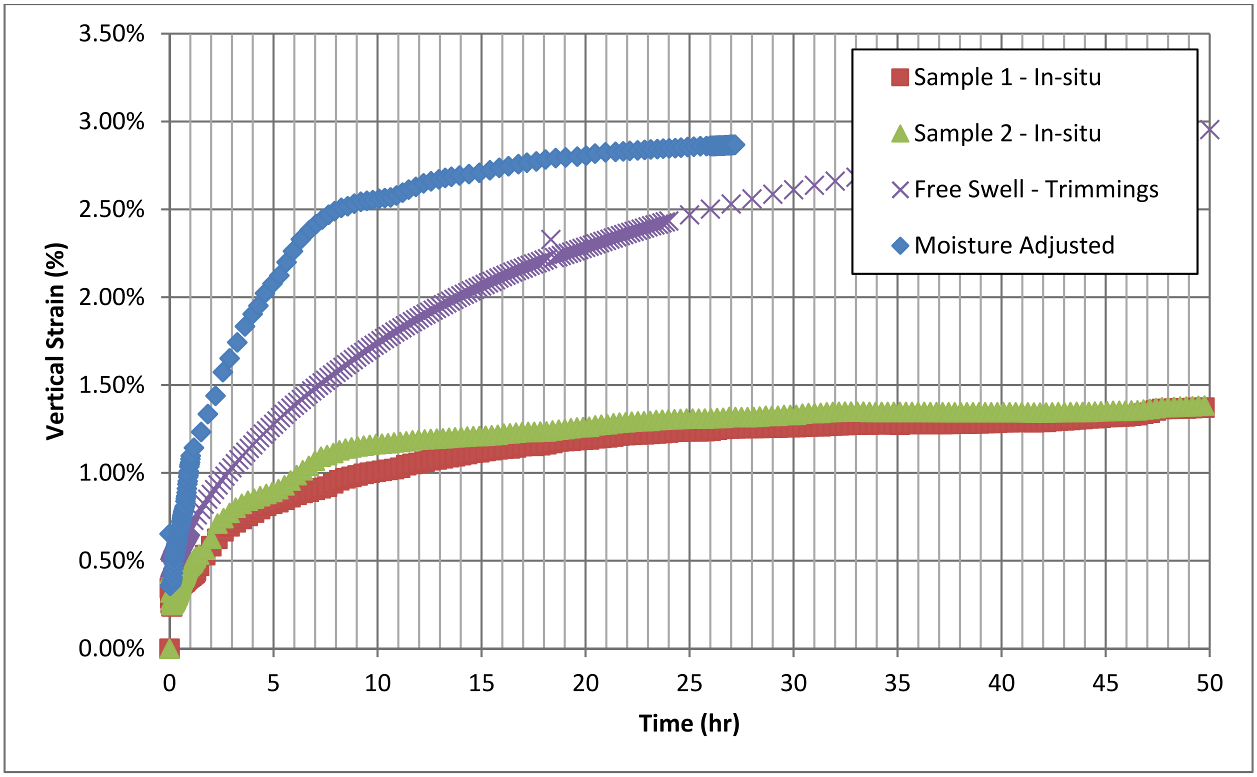


Figure 5.32: Swell-Time Curves for B-18

The results indicate that, for a vertical stress of approximately 250 psf which corresponds to the vertical stress at the top of the boring specimen in the field, the soil will not swell significantly under moisture fluctuations. This conclusion is drawn out in the single point PVR results of 1.13 in for the moisture adjusted condition. The result from the free swell test indicates some of the issues of solely relying on reconstituted samples as they may over predict the amount of swelling seen based on reconstitution of naturally formed due to natural voids and bias from the selection of non-granular material for reconstitution. Overall, the results at this location are consistent with the dark brown fat clay results from B-17.

B-19

The next boring from Limmer Loop was collected from a depth of 2-4 feet beneath the pavement surface and is shown in Figure 5.33 with the boring log and results from the commercial laboratory testing shown in Table 5.18. The Atterberg Limits suggest that the soil may have a significant potential to swell based on empirical correlations from Tex-124-E and that the tan fat clay at the bottom may not have a significant potential to swell. The liquid limit from the boring was 60.2% based on the single point test from the boring trimmings with a higher than expected moisture content of 25.9% which the liquid limit once again showing heterogeneity in the results between laboratories as also illustrated in boring B-18.

Figure 5.33: Sample Provided for B-19

Table 5.21: Boring Log for B-19

Depth	Layer Description	ω (%)	Liquid Limit (%)	Plasticity Index (%)	γ_d (pcf)	Clay Fraction (%)
	Pavement/Base					
	Dark brown and black Fat Clay (CH), with gravel seams, moist					
5	- with sand seams					
		13.5	68	46	106.4	7.4
	Tan Fat Clay (CH), with sand and chalk/calcite deposits, moist					
10		14.1	38	24	97.3	11.5

The soil lies within the dark brown to black fat clay from the previous borings and shows a significant amount of granular material on the external portion of the sample. The samples used for centrifuge testing are shown below in Figure 5.34 with two samples being at the in-situ conditions and two samples being at the moisture adjusted condition. The initial conditions of the soil along with the results of testing are shown in Table 5.19 along with the vertical strain versus time of inundation for the tests in Figure 5.35.

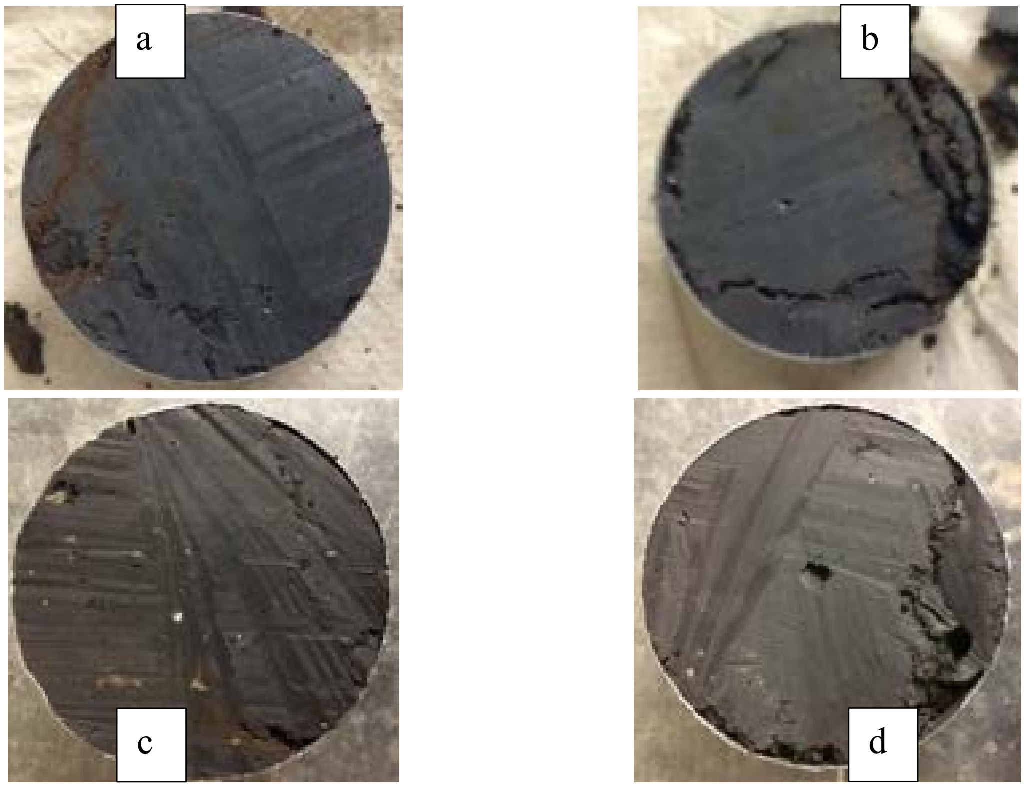


Figure 5.34: B-19 Samples at In-situ (a,b) and Moisture Adjusted (c,d) Conditions

Table 5.22: Summary of Results for B-19

Date of Testing	ω_i (%)	γ_d (pcf)	Primary Swelling (%)	End of Primary Swelling (hr)	Vertical Stress (psf)
6/3/2014	26.4	92	1.1	26.0	253
6/3/2014	25.4	97	1.0	4.8	257
10/24/2014	21.7	104	2.7	4.9	282
10/24/2014	21.1	98	3.1	15.7	283

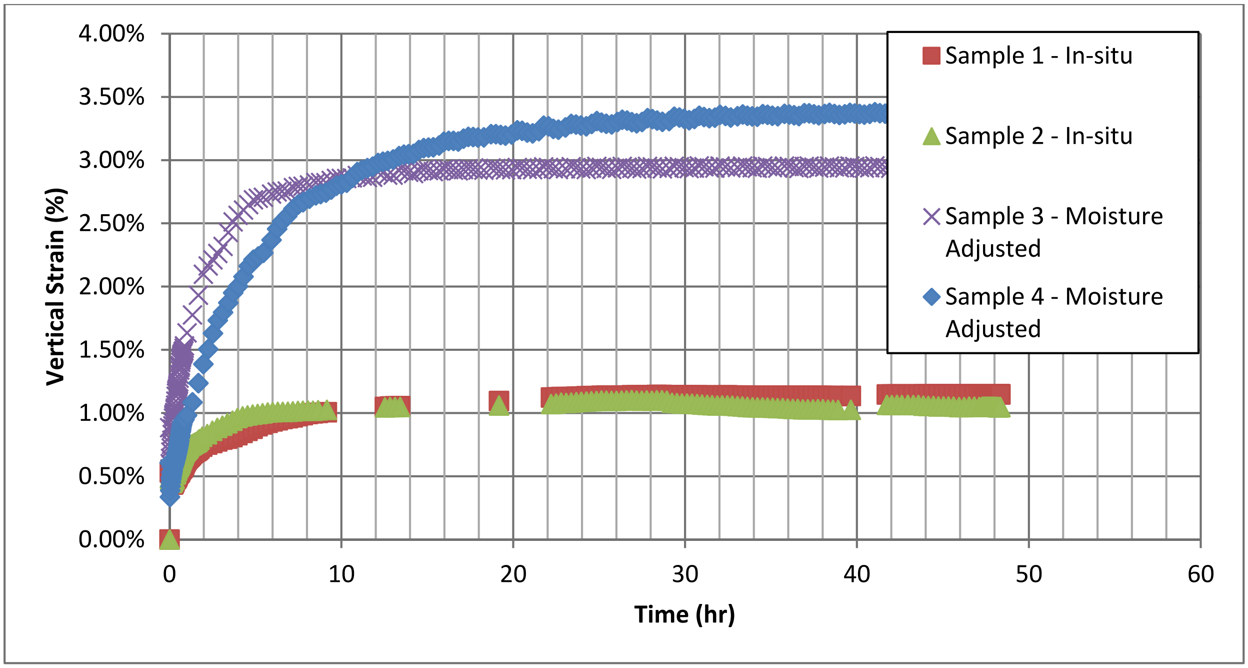


Figure 5.35: Swell-Time Curves for B-19

The results indicate that, for a vertical stress of approximately 250 psf which corresponds to the vertical stress at the top of the boring specimen in the field, the soil will not swell significantly under moisture fluctuations. The first thing to note is that the in-situ conditions are showing a similar amount of swelling previously encountered in borings B-17 and B-18 whereas the moisture adjusted specimens are at a slightly higher amount of primary swelling. The results indicate that the soil is not problematic at low stresses with a single point PVR of 2.37 in for the moisture adjusted conditions.

B-20

The next boring from Limmer Loop was collected from a depth of 6-7.5 feet beneath the pavement surface and is shown in Figure 5.36with the boring log and results from the commercial laboratory testing shown in Table 5.20. The Atterberg Limits suggest that the soil may have a significant potential to swell based on empirical correlations from Tex-124-E and that the tan fat clay at the bottom may not have a significant potential to swell. The liquid limit from the boring was 72.5% based on the single point test from the boring trimmings, which is significantly higher than the amount seen further in the boring of the same layer indicating possible heterogeneity, with a higher than expected moisture content of 26.4% for the sample provided to UT.

Figure 5.36: Sample Provided for B-20

Table 5.23: Boring Log for B-20

Depth	Layer Description	ω (%)	Liquid Limit (%)	Plasticity Index (%)	γ_d (pcf)	Clay Fraction (%)
	Pavement/Base					
5	Dark brown and black Fat Clay (CH), moist - with sand seams	16	73	51	104.6	4.1
10	Tan to light brown Fat Clay (CH), with sand and chalk/calcite deposits	10.5	40	26	116	12.5

The soil stratum lies with the tan to light brown section of the boring with the predominant feature being a light brown clay that shows significant amount of vugs and calcite deposits, possibly transitioning to a limestone layer of the Pecan Gap Chalk at the bottom section of the sample. The results of liquid limit tests obtained by the commercial laboratory and UT tested samples are significantly different. This indicates that the light brown soil strata may be heterogeneous based on the differences in the Atterberg Limits as the previous dark brown strata had different liquid limit between the testing laboratories that were significant. The samples used for centrifuge testing are shown below in Figure 5.37. The initial conditions of the soil along with the results of testing are shown in Table 5.21 along with the vertical strain versus time of inundation for the tests in Figure 5.38.

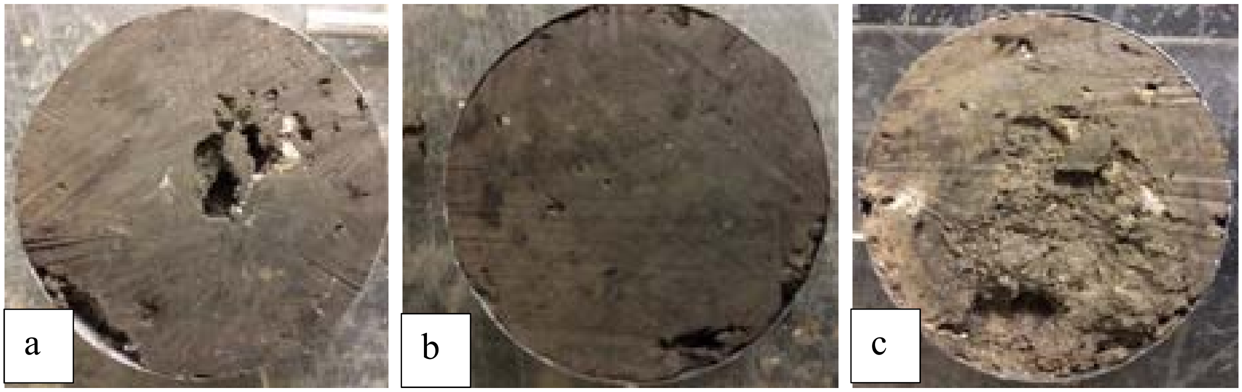


Figure 5.37: B-20 Samples at In-situ (a,b) and Moisture Adjusted (c) Conditions

Table 5.24: Summary of Results for B-20

Date of Testing	ω_i (%)	γ_d (pcf)	Primary Swelling (%)	End of Primary Swelling (hr)	Vertical Stress (psf)
6/20/2014	26.0	94	1.4	17.9	694
6/20/2014	26.8	97	1.1	24.1	698
10/26/2014	23.1	92	1.4	12.0	633

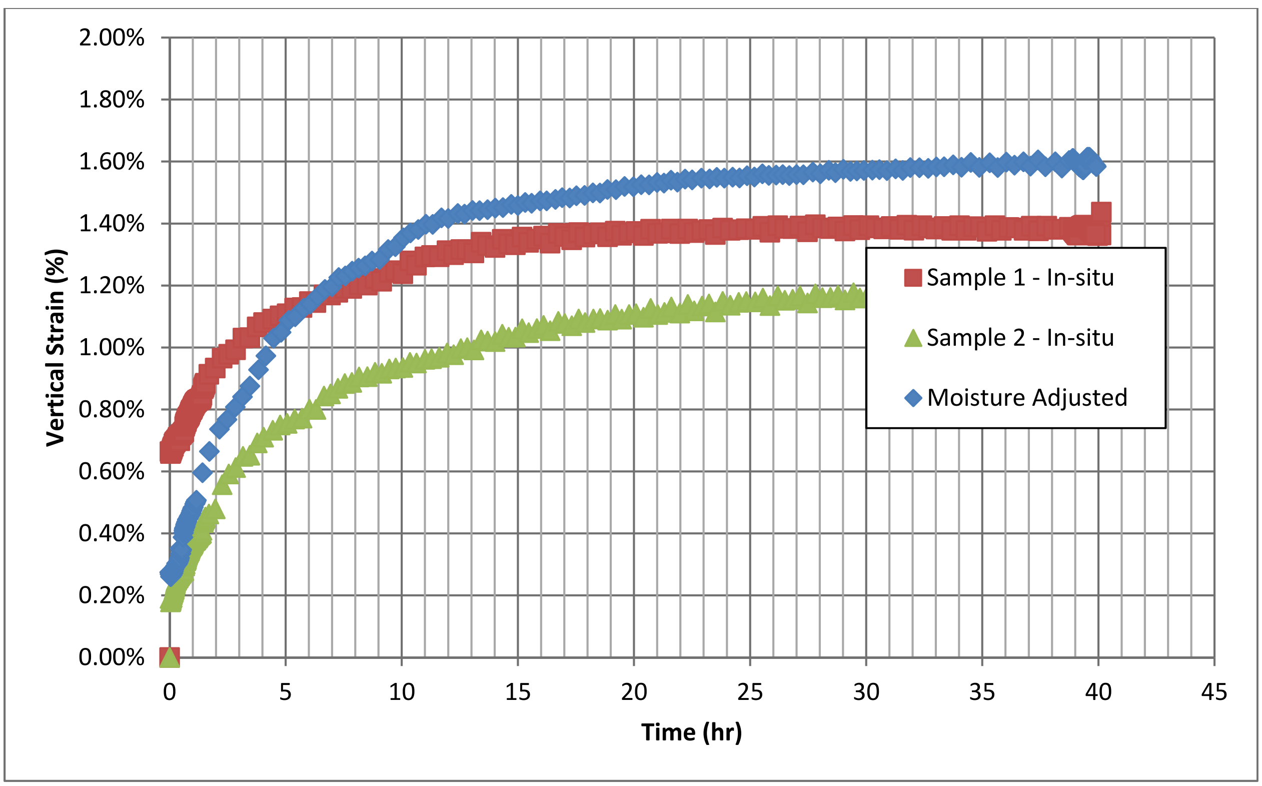


Figure 5.38: Swell-Time Curves for B-20

The results indicate that, for a vertical stress of approximately 650 psf which corresponds to the vertical stress at the top of the boring specimen in the field, the soil does see a fairly significant amount of swelling when compared to the dark brown clay. The moisture adjusted specimen had a similar result to the in-situ specimens, but this can be attributed to the large vugs in the sample which was moisture conditioned that reduced the dry density of the specimen significantly. However, the results still indicate that the soil is not problematic with a single point PVR of 1.75 in for the moisture adjusted conditions.

Bulk samples were taken at Limmer Loop and were typically a lighter brown with portions of calcareous cementations or lime stabilized portions of the soil. The Atterberg Limits do not indicate as high of a liquid limit or plasticity index as the borings and the sulfate content is negligible. The stress-swell curve for the bulk samples is shown below in Figure 5.39. Based on the stress-swell curve and a range of stress consistent with boring B-17 the PVR for the bulk samples would only be 3.09 in. Thus, the borings from Limmer Loop were shown to be expansive in regards to a PVR analysis.

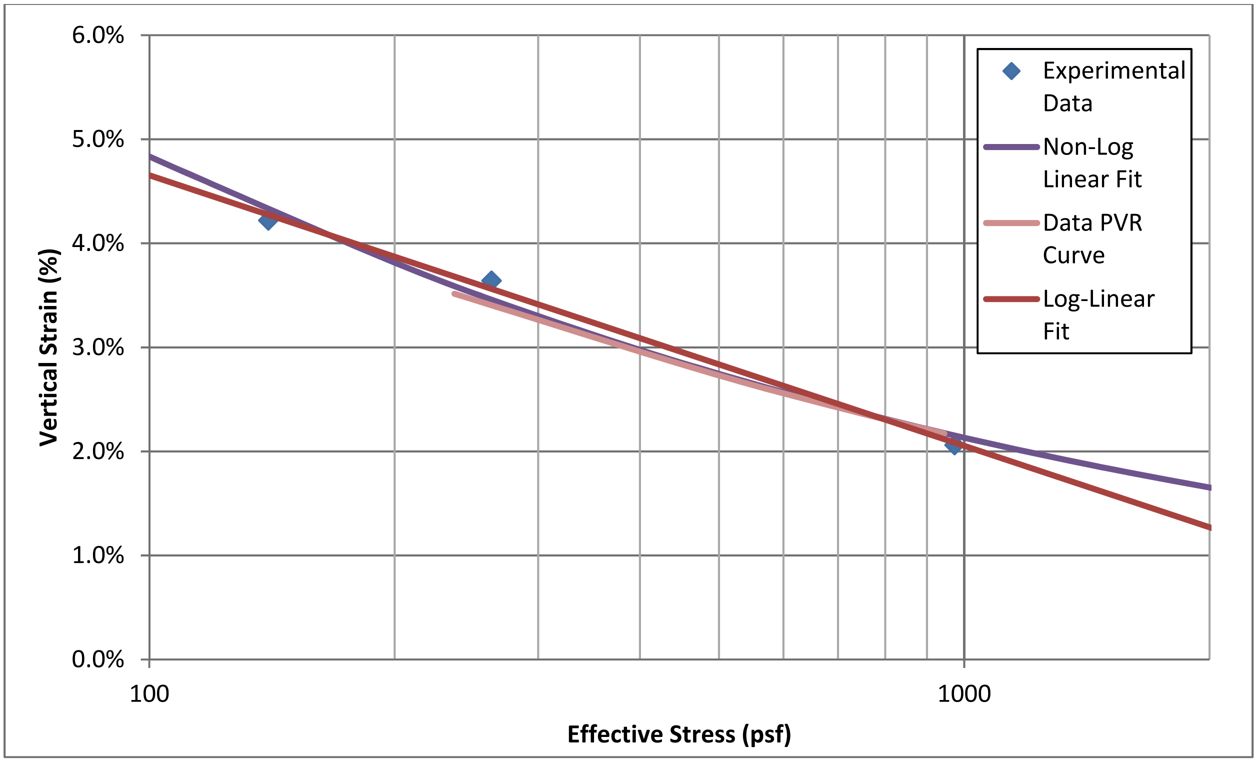


Figure 5.39: Swell-Stress Curves for Limmer Loop Bulk Sample

5.2 Conclusions from PVR Calculations and Field Performance

Table 5.22 summarizes the PVRs calculated by each method for each site as well as the environmental cracking found in the field. The main conclusion from the results is that the newly developed model shows inconsistencies within Tex-124-E. Tex-124-E is neither always conservative, nor under-conservative regarding the PVR as the assumptions made in the development of the method do not take into account heterogeneity in soil units. One thing to note is that the centrifuge method can help show when there isn't an issue with the subgrade under a pavement structure due to environmental cracking. This observation is detailed further in Chapter

5 for the FM 972 site, but sites such as the Crawford site and the FM 672 North site indicate that the Atterberg Limits may overestimate how large of an issue a subgrade will be for Tex-124-E, whereas the sites on FM 672 South and at the Taylor Maintenance Office illustrate the opposite. The main point of consideration is that the soils at each site must be tested in order to determine the actual performance in the field. While Atterberg Limits correlations may be able to give an indication of the expected performance of a subgrade, the soil itself must be tested as there is a significant amount of heterogeneity within soil deposits. This trend is seen in the Crockett soil, as the samples are within a mile of each other but behave extremely differently. Further analysis is needed into using a threshold value of PVR for the centrifuge-based methods, as the field performance is influenced significantly by previous maintenance operations, including repaving of the asphalt surface or sealing of cracks.

Table 5.25: Summary of PVR Methods and Field Performance

Site #	Soil Name	County	PVR [Centrifuge]	PVR [Tex-124-E]	Edge Cracking [ft/100 ft]
1	CR	Travis	1.53	1.49	76
2	TN	Travis	1.34	1.17	31
3	HB – M36	Travis	1.87	0.70	-
4	HB – M127	Travis			-
5	FR	Williamson	2.03	1.01	0
6	HE - LF	Williamson	1.31	1.03	0
7	HB - 971	Williamson	2.30	1.76	43
8	BR - 972	Williamson	0.82	1.40	42
9	BR - 95	Williamson	1.20	0.18	218
10	HB - Taylor	Williamson	3.28	0.93	97
11	BH - 535	Bastrop	1.69	0.93	0
12	BH - 20	Bastrop	0.93	0.54	82
13	CR – 672N	Caldwell	0.18	0.14	75
14	CR – 672S	Caldwell	3.38	0.89	29
15	BU	Caldwell	1.57	1.35	93
16	HE – 1854W	Caldwell	1.42	1.27	136
17	HE – 1854 and SH 21	Caldwell	3.12	1.28	45
18	BR - 685	Williamson	1.85	0.99	-

5.3 Moisture Monitoring of FM685 Site

In conjunction with testing to determine how expansive a soil is, one site was selected for sensor installation in order to verify whether laboratory experiments matched the moisture changes being experienced in the field. Since the sensors needed to be installed at depth beneath a roadway, the research team sought a farm-to-market road that had an expansion under construction, as a completed site would require an auguring. Several sites were examined, including the expansion of FM 487 and FM 972 in Williamson County, but a site near Hutto High School off of FM 685 was selected (referred to earlier as Site 18). The geological and geographic locations are detailed in Section 4.1.17, and the centrifuge testing and PVR calculations are detailed in Section 4.2.17.

The site was selected because the expansion of the roadway also included the construction of a paved entrance into a hay-bundling site that was less well traveled. Due to the reduction in traffic and the ease of sensor installation, the vertical cut near a box culvert that went beneath the future driveway was selected. Using a low-traffic site helps ensure that any cracking near the edge will primarily result from the targeted environmental conditions rather than from traffic loading, and allows for total station surveying of the profile (as detailed in Chapter 5). While the site sits on the Branyon Clay, a fairly expansive deposit derived from previous mudstone erosions that is typically more granular, the PVR from the site was still considered an expansive deposit for roadway construction. This chapter will describe the sensors and installation and compare the field and laboratory results.

5.3.1 Field Sensors

Two types of sensors can be used for these types of field studies: moisture content sensors or total suction sensors.

5.3.2 Volumetric Moisture Content Sensors

In order to examine how the moisture changes with time in a soil deposit, volumetric moisture content (VMC) sensors were placed in the soil deposit. The sensors chosen are Decagon 5TE sensors that record the VMC, electrical conductivity, and temperature. The sensors work by using an oscillator to measure the dielectric permittivity of the soil and back calculate the VMC using Topp's equation. The sensors have an accuracy of ±0.03 m^3/m^3 for the VMC of the soil and are shown in Figure 5.40.

Figure 5.40: Decagon 5TE Sensors (Decagon 2016)

The Decagon sensors were selected over other moisture content sensors (such as time-domain reflectometers, or TDRs) because previous research into TDRs in expansive soil deposits showed significant issues (Dellinger 2011). The 5TE was selected over Decagon's EC-5 because Garcia (2015) found that the VMC readings must be corrected for temperature using Equation 5.2 with *tm* and *θm* being the temperature and VMC from the reading and the reference temperature being 20°C:

$$\theta ref = \theta m - 0.002(tm - tref) \tag{5.2}$$

Values from the 5TEs were logged using Decagon Em50 data loggers with readings being taken every hour. These values were then corrected for the temperature in the soil using Equation 5.2 to remove any influence of the temperature on the measured VMC.

5.3.3 Total Suction Sensors

In order to determine the extent of the capillarity increases during seasonal fluctuations, suction sensors were placed in the soil deposit. Decagon MPS-2 sensors were chosen as they utilize the capacitance of a calibrated ceramic disk in order to read the suction of the surrounding soil. These sensors were chosen instead of tensiometers because they are easy to install and don't need a water reservoir or other extensive support. The sensors have an accuracy of ±25% as well as an additional 2 kPa for the range of -9 to -100 kPa. While this is a significant amount of potential error, the expected range of suction is expected to be within the range of -9 kPa to 50 kPa, meaning the potential error is limited to a range of 4 kPa to 14 kPa. Once a full seasonal fluctuation has been experienced, the results from the field will be compared against results from ASTM D5298, filter paper tests, and using a chilled mirror hygrometer at a similar VMC. The MPS-2 sensors are shown in Figure 5.41.

Figure 5.41: Decagon MPS-2 Sensors (Decagon 2016)

Values from the MPS-2 sensors were logged using an Em50 data logger with a reading schedule of once every hour.

5.3.4 Field Installation of Sensors

The sensors were first tested in the laboratory using a similar soil to that encountered in the field, a granular Houston Black clay, and were found to work during both the wetting and drying cycles. The sensors were installed on November 24 and December 5, 2015. The soil profile is shown in Figure 5.42.

Figure 5.42: FM 685 Soil Deposits prior to Sensor Installation

The soil profile is broken into three portions. The upper 4 inches of the slope was found to be compacted base material installed during construction of the driveway. The base layer was underlain by 5 ft of Branyon Clay that is classified as a fat clay and characterized in previous sections. Beneath that Branyon Clay was found a non-expansive deposit, Krum soil classified as a lean clay. The division between the expansive and non-expansive deposit is delineated by a red line in Figure 4.3 and shown closer up in Figure 5.43.

Figure 5.43: Soil Strata Delineation between Branyon Clay and Krum Soil at FM 685

The sensors were installed over two separate days, and the slope was backfilled after the sensors came to equilibrium. The soil was sampled during the first visit on November 24 and two 5TE and MPS-2 sensors were installed using the first data logger. The second visit consisted of installing two additional 5TE and MPS-2 sensors at depth as well as an additional 5TE sensor to replace one sensor that was broken during installation. The sensors were installed by using a hollow metal pipe and a flat head screwdriver to dig a hole that was 6 inches within the soil strata. The sensors were installed on opposite sides of the hole, and the hole was filled back in by soil from the surrounding area. This installation approach was used to ensure the sensors were reading naturally occurring soil from the strata, not externally sourced soil used during construction. The installed sensors in the slope are shown in Figure 5.44. The sensors were placed 0.5 ft below the base and Branyon Clay interface, 1.5 ft below the interface, and 3.5 ft below the interface in the Branyon Clay, and 5.5 ft below the interface in the Krum soil. The sensors were installed in pairs with both a 5TE and MPS-2 sensor placed at each depth.

Figure 5.44: Installed Sensors at FM 685

After the MPS-2 sensors' ceramic disks came to equilibrium (which took one week), the sensors were buried by the contractor on December 12, 2015. The sensors were protected from construction activities with PVC pipes, placed to prevent any cutting of the wires during burial. In addition, a PVC pipe protected the wires running along the side of the slope to the Em50 data loggers, which were placed away from construction and maintenance activities.

5.3.5 Field Monitoring Results

The data from the data loggers were taken every month and analyzed using the temperature corrections as defined by Equation 5.2. The precipitation was monitored using data from the Weather Underground site, and the temperature for the correction came from the 5TE sensors. Note that the suction sensors were not corrected for temperature fluctuations.

5.3.6 Precipitation at the Site

Precipitation data for the site was initially taken from the Weather Underground location at the Manor Executive Airport in Pflugerville. This location was selected for its historical data as well as data for the sampled range that the National Oceanic and Atmospheric Administration (NOAA) has not released for the Hutto site. Overall, there have been four major rainfall events at the site with a few smaller recorded rainfall events in the logged timeframe. The major events occurred on typically in the spring, though there was a significant amount of rainfall at the end of May towards Veterans Day as well as towards the end of August. All the rainfall events recorded for the site are listed in Table 5.23. During the events, the sensors recorded dramatic increases in the VMC and decreases in the total suction, as shown later in Figure 5.45 and Figure 5.46.

Table 5.26: Precipitation Events at FM 685

Date	Precipitation (in)	Date	Precipitation (in)
1/6/2016	0.26	5/10/2016	0.29
2/23/2016	1.62	5/11/2016	0.62
3/8/2016	0.29	5/14/2016	0.95
3/9/2016	2.55	5/18/2016	0.14
3/10/2016	0.5	5/19/2016	1.77
3/11/2016	0.53	5/22/2016	0.11
4/1/2016	0.44	5/27/2016	0.53
4/12/2016	0.82	5/29/2016	0.1
4/13/2016	0.17	5/30/2016	0.3
4/17/2016	3.24	5/31/2016	2.02
4/18/2016	1.01	6/2/2016	1.63
4/20/2016	0.63	6/4/2016	0.29
4/21/2016	0.22	6/28/2016	0.42
4/27/2016	0.98	7/25/2016	2.6

5.3.7 Moisture Data from Field Sensors

The corrected data from the 5TE sensors as well as the temperature taken per each 5TE is shown in Figure 5.45.

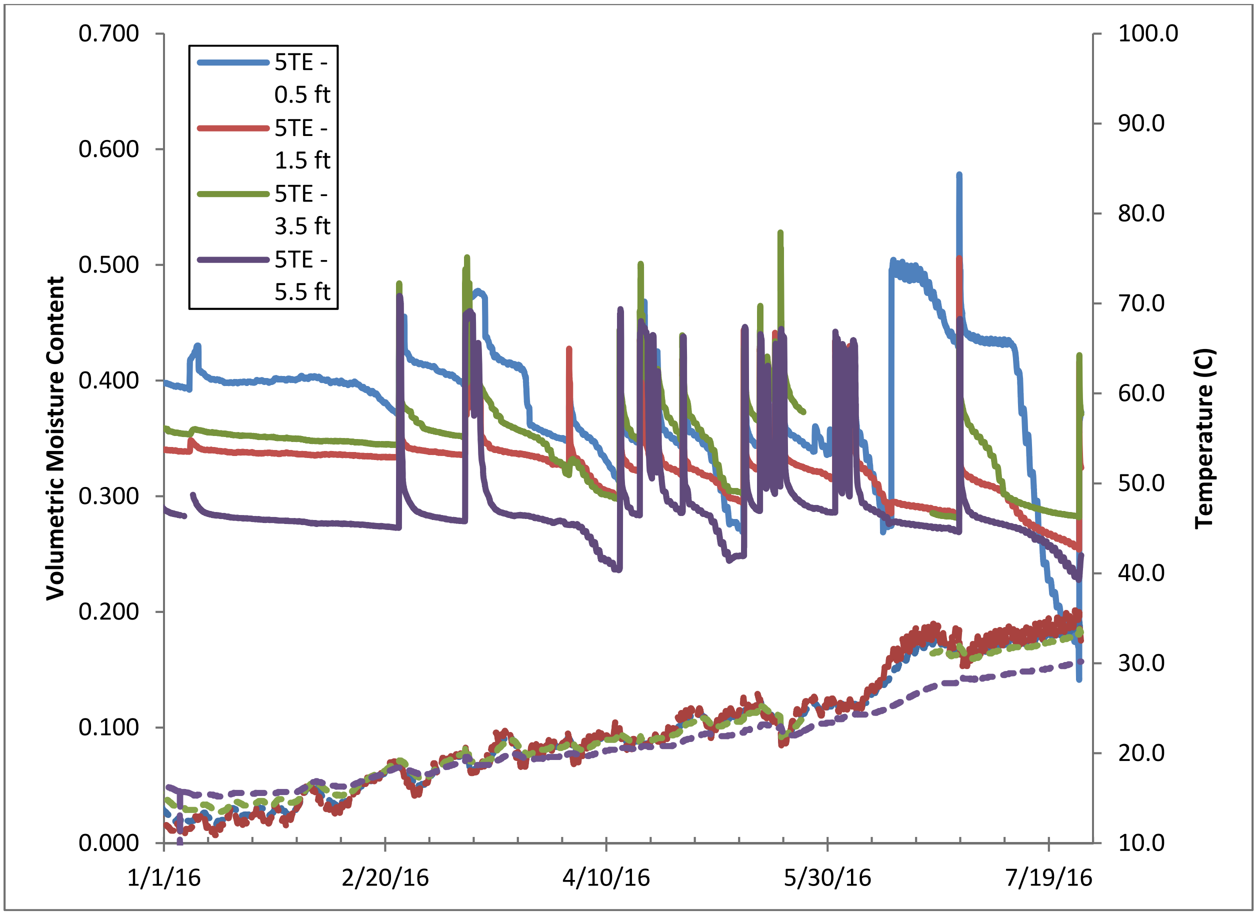


Figure 5.45: VMC and Temperature Data for FM 685 Site

The first thing to note is that the temperature fluctuates much more in the Branyon Clay than in the Krum Series. Further, the moisture trends in the soil deposit are to be expected. In the Branyon Clay, the VMC decreases with depth, and the amount of fluctuation is much higher in the surface when major rainfall events occur. Furthermore, the soil tends to dry more rapidly near the surface as opposed to at depth, which is caused by being closer to the atmospheric evaporation of moisture from the clay deposits from the radiative energy of the sun and relative humidity suction in the air. However, in terms of the initial starting point, the soil VMC typically lies between 0.410 and 0.310 during the season without moisture fluctuations. This VMC is matched by the laboratory testing on reconstituted specimens of the Branyon detailed in Chapter 4, as the initial VMC for testing typically lies between 0.360 and 0.375 after the compaction phase. Thus, the initial conditions tested in the laboratory are somewhat similar with those from the field. These results may indicate that drier specimens may be needed to fully validate the testing procedure, but also indicate that the results from FM487, shown later, may be a good representation of the summer moisture condition. The upper bound for the Branyon Clay after rainfall events tends to be approximately 0.500. In laboratory tests, this moisture condition is below the final VMC in tests, typically between 0.489 and .500. Therefore, while the initial conditions may not match as well as needed, the soil in the field does see the same change in volumetric moisture content towards inundation after significant rainfall events. This revelation is important as it illustrates the need to go to the end of swelling in totality to estimate the volumetric changes in the field due to the climatic conditions east of the Balcones Fault Zone.

5.3.8 Suction Data from Field Sensors

The data from the MPS-2 sensors is shown in Figure 5.46.

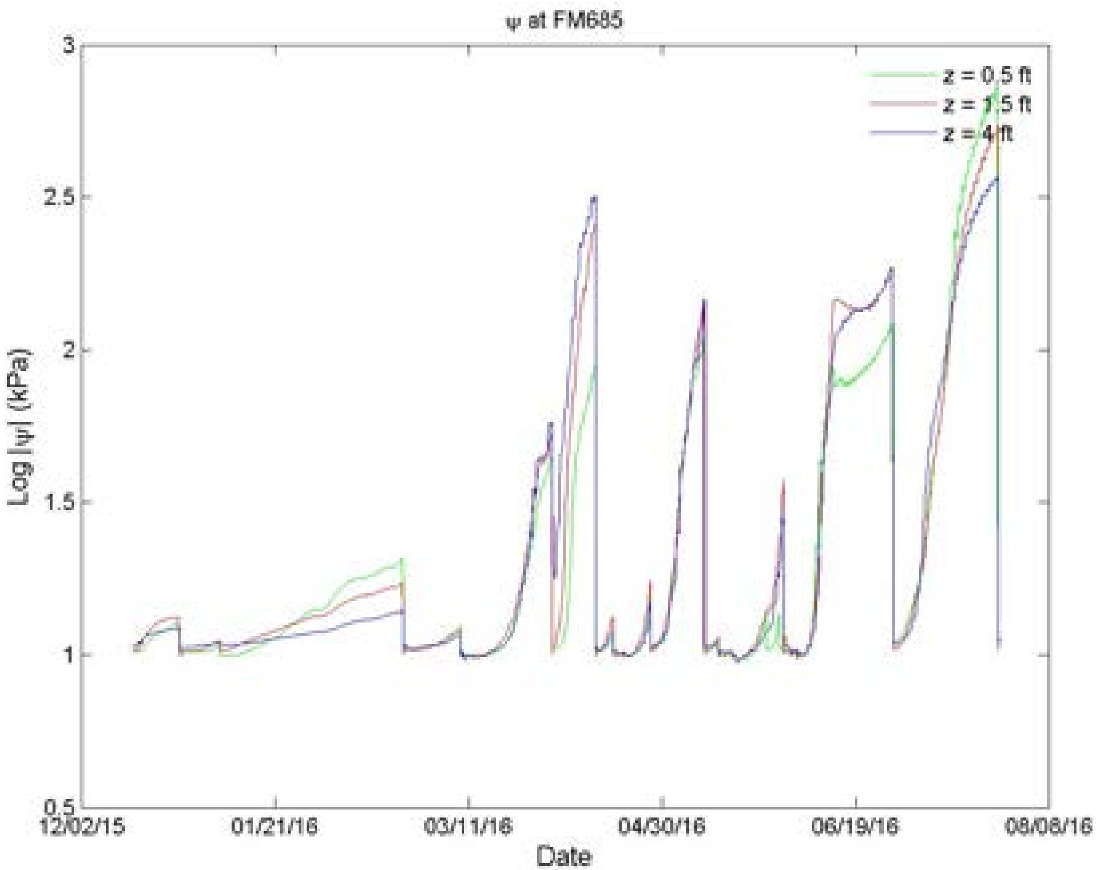


Figure 5.46: Suction Data for FM 685 Site

The results from the MPS-2 sensors are less clear than those from the 5TE sensors. The general trend is that the suction will the highest for the top sensor, and then decrease with depth. Further, the MPS-2 sensors in the Branyon Clay tend to have a higher suction than the sensor in the Krum soil. One thing to note is that these results from the field do not match those from the chilled mirror hygrometer, which sees a suction of 570 kPa for the soil's initial conditions in the laboratory. Data from this site is being used to analyze the wetting and drying paths of the soil, thereby looking at the differences between the soil water retention curves generated form these sites in relation to those from laboratory methods. Further analysis will be performed on this site over the next few years. Overall, the first field site illustrated the benefit of instrumentation in the field to validate the moisture changes seen in the lab as well as to validate the unsaturated soil characteristics from laboratory methods. These methods are further being examined at an inundation site, the Taylor Maintenance Office, as well as an additional site in Williamson County to continue to provide support in terms of field performance versus laboratory performance.

5.4 Inundation Project

As part of the project, an inundation project in which the heave under moisture fluctuations was designed and installed. The site was selected to be the Taylor Maintenance Office previously seen in Chapter 4 and was selected due to the relatively high PVR as well as fenced in area to prevent tampering with the data acquisition equipment. The site location and soil survey is shown below in Figure 5.47. The site sits on a homogenous layer of Houston Black clay from the C horizon, a well-known soil throughout this study and for its agricultural use. The location has a small amount of base on the top portion of the soil, thereby adding a non-expansive surcharge to eliminate issues with low stress conditions which become problematic in the calculation of PVR. This depth of base ranged from approximately 0.5 to 1 ft above the expansive soil itself. The project consisted of a circular geomembrane with a diameter of 20 ft surrounding 5 augured holes that contained instrumentation. An additional hole was placed outside of the geomembrane to monitor the moisture fluctuation.

Figure 5.47: Soil Survey for Taylor Maintenance Office and Location of Inundation Project

The sensors selected for the site included the 5TE sensors from the FM685 site as well as Acclima TDR-315 for comparison between field performances of soil sensors and an updated version of the suction sensors, MPS-6, which uses a higher amount of factory calibration for an increased accuracy. These sensors are shown in Figure 5.48.

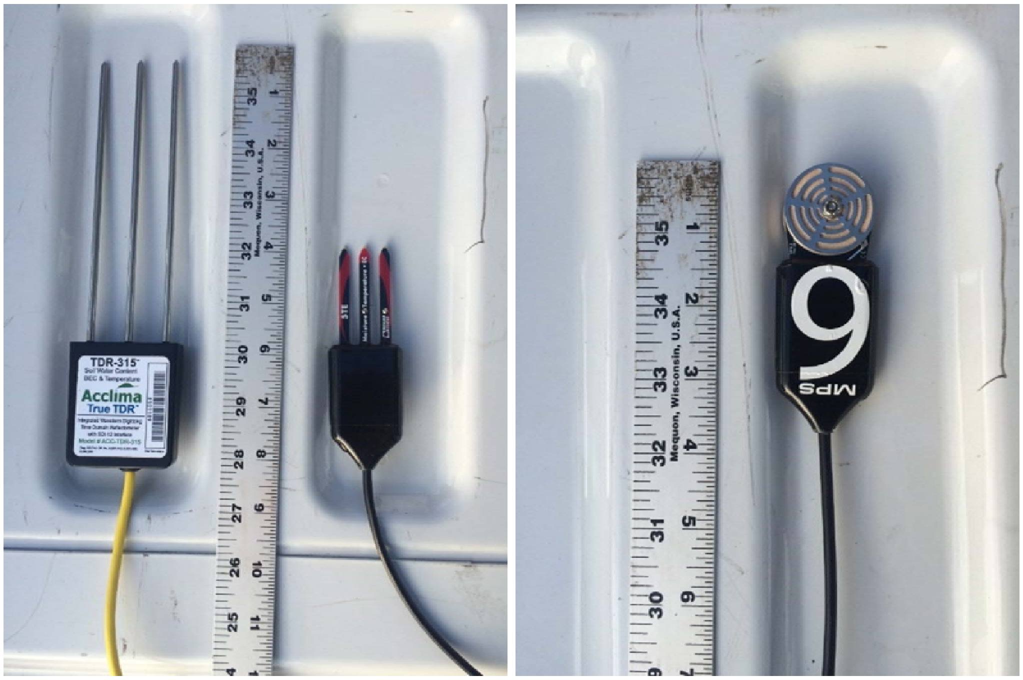


Figure 5.48: Acclima TDR-315 and Decagons 5TE and MPS6 Sensors for Inundation Project

The project also incorporated settlement sensors from Geokon to monitor the swelling and shrinkage at depth. The sensors were placed in the holes at specified depths beneath the surface in the central hole and were stabilized in the hole by use of a geogrid. The geogrid used as well as the sensor placed in the field is shown in Figure 5.49. The sensors work by measuring the pressure at the depth of the sensor, as shown in Figure 5.50. Since the sensors were placed at the same depth as the moisture content sensors, the volumetric moisture content fluxes could be seen at said depths to tie to the change in height. Two sensors were placed at a depth of 1 and 2 ft below the base and soil interface.

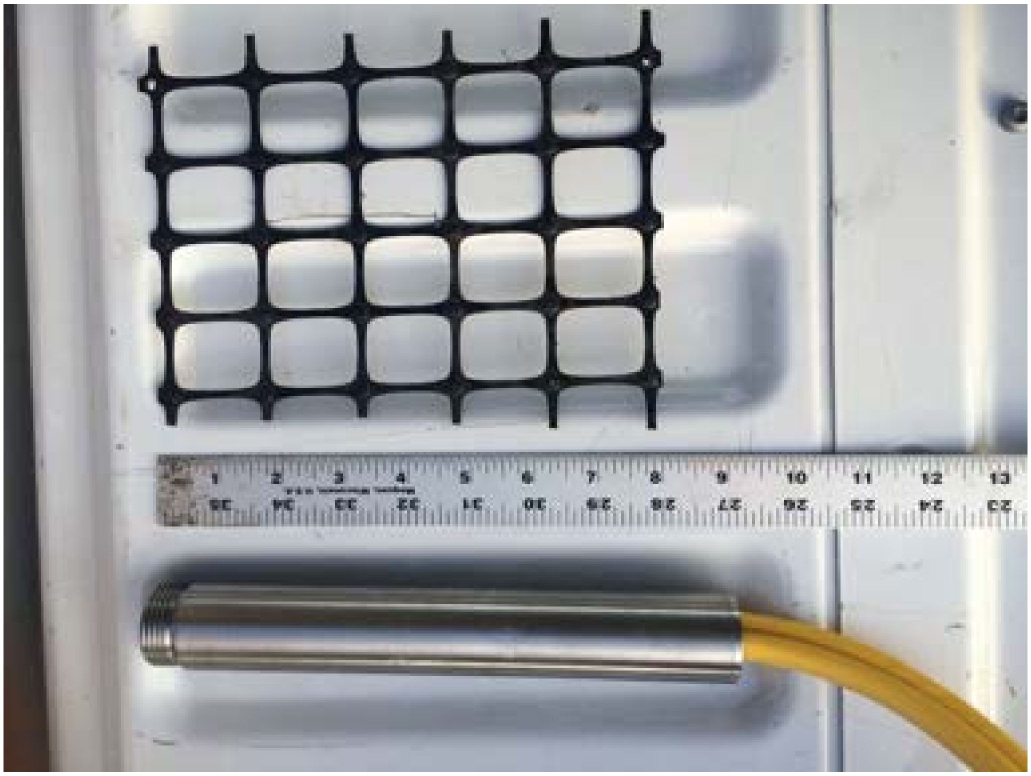


Figure 5.49: Geokon Settlement Sensor and Geogrid for Inundation Project

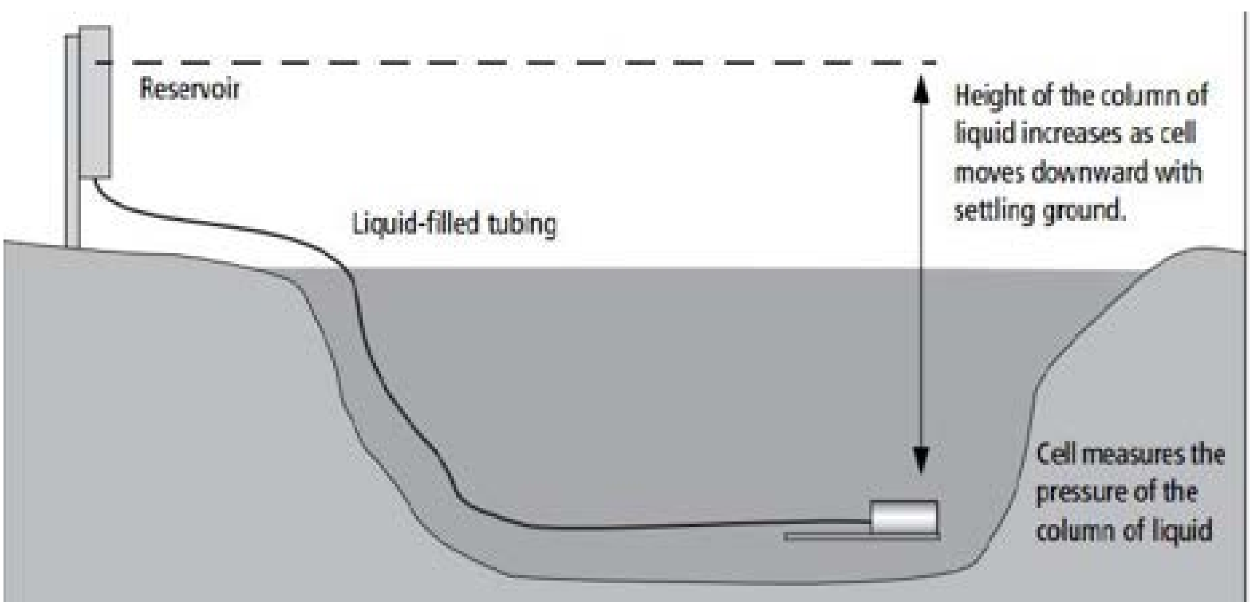


Figure 5.50: Geokon Settlement Sensor Diagram

Installation of the sensors involved the use of a towable auger with a 12” drill bit to dig a hole that was approximately 3 ft. A single hole was drilled for the outside holes in the circle, and a double hole was drilled using an extension for the middle hole to go down to a depth of 4 ft below the ground surface. The sensors were installed by digging a hole using gardening tools in the side of the trench and placing the sensors in the hole. The hole were then covered with augurings and hand compacted using a falling weight tool. The digging of the holes and an inner view of an instrumented hole is shown in Figure 5.51.

Figure 5.51: Installation of Field Sensors at Inundation Project

The final instrumented site is shown in Figure 5.52. Due to the extreme climatic conditions, the geomembrane has yet to be installed as well as the data acquisition system for the Geokon settlement sensors. Further work will be done past the end of this project to monitor the site and compare the results to those from the laboratory testing.

Figure 5.52: Instrumented Inundation Site

5.5 Total Station Monitoring of FM972

A total station instrument is used to monitor the shape of the roadway at various times. The total station used uses a prism-less system that can be operated remotely, thereby increasing the safety of the workers monitoring the pavement structure without sacrificing accuracy. Measurement are taken at 1-ft intervals. An additional system was developed that runs a rope across a roadway to prevent marking along a high-traffic roadway. An example of a marked roadway is shown in Figure 5.53.

Figure 5.53: Total Station Marking

The data is saved in the total station, and the coordinate data is taken from the surveying equipment and plotted by using the Pythagorean theorem and assuming a linear line across which the points were taken. These assumptions convert the surveying points from a 3-D system into a 2-D system that can be plotted using an Excel Workbook. Example results from a total station survey are shown in Table 5.24 and Figure 5.54. Note that the main site examined was FM 972 in conjunction with another research project. However, the technique used at FM 972 is to be expanded at two future sites: FM 487 in Bartlett, TX and FM 685 in Hutto, TX. The site in Hutto, TX is the field monitoring site from Chapter 4 and will be used to examine the change in shape of the pavement over time with the change in moisture fluctuations. The site in Bartlett, TX will be used in conjunction with Shelby Tube samples to validate the results of in-situ vs. reconstituted specimens and determine whether the difference in PVR is seen in the cracking and deformation on a recently reconstructed roadway.

Table 5.27: Example Results from Total Station Monitoring

FM 1979-Westbound Readings-Section#8-Visit #1							FM 1979-Eastbound Readings-Section#8-Visit #1						
	Total Station Coordinate Reading Relative to center line			Modified coordination for cross section of the road				Total Station Coordinate Reading Relative to center line			Modified coordination for cross section of the road		
	Z	X	Y	Z	X	Y		Z	X	Y	Z	X	Y
Pt.	m	m	m	mm	mm	mm	Pt.	m	m	m	mm	mm	mm
1	0	0	0	0	0	0	1	0	0	0	0	0	0
2	-0.00762	0.217932	0.224028	-7.62	313	0	2	-0.0061	-0.19812	-0.22403	-6.096	299	0
3	-0.01981	0.428244	0.44196	-19.812	615	0	3	-0.02286	-0.39472	-0.4511	-22.86	599	0
4	-0.02134	0.641604	0.673608	-21.336	930	0	4	-0.02896	-0.61265	-0.67208	-28.956	909	0
5	-0.01524	0.839724	0.897636	-15.24	1229	0	5	-0.01829	-0.80467	-0.89002	-18.288	1200	0
6	-0.01372	1.042416	1.124712	-13.716	1533	0	6	0.001524	-1.03327	-1.11404	1.524	1519	0
7	-0.02591	1.260348	1.344168	-25.908	1843	0	7	0	-1.23292	-1.33807	0	1819	0
8	-0.03658	1.444752	1.56972	-36.576	2133	0	8	-0.01372	-1.43256	-1.54991	-13.716	2111	0
9	-0.04877	1.674876	1.7907	-48.768	2452	0	9	-0.02743	-1.6322	-1.7907	-27.432	2423	0
10	-0.05334	1.868424	2.013204	-53.34	2747	0	10	-0.03505	-1.83032	-2.01168	-35.052	2720	0
11	-0.05944	2.112264	2.25552	-59.436	3090	0	11	-0.03048	-2.04673	-2.24028	-30.48	3034	0
12	-0.05334	2.289048	2.465832	-53.34	3365	0	12	-0.03505	-2.2479	-2.4704	-35.052	3340	0
13	-0.05486	2.526792	2.692908	-54.864	3693	0	13	-0.03505	-2.45212	-2.68834	-35.052	3639	0
14	-0.06706	2.706624	2.912364	-67.056	3976	0	14	-0.05486	-2.67919	-2.91084	-54.864	3956	0
15	2.717292	-12.7559	3.003804	2717.292	13105	0	15	-0.0701	-2.87426	-3.14554	-70.104	4261	0

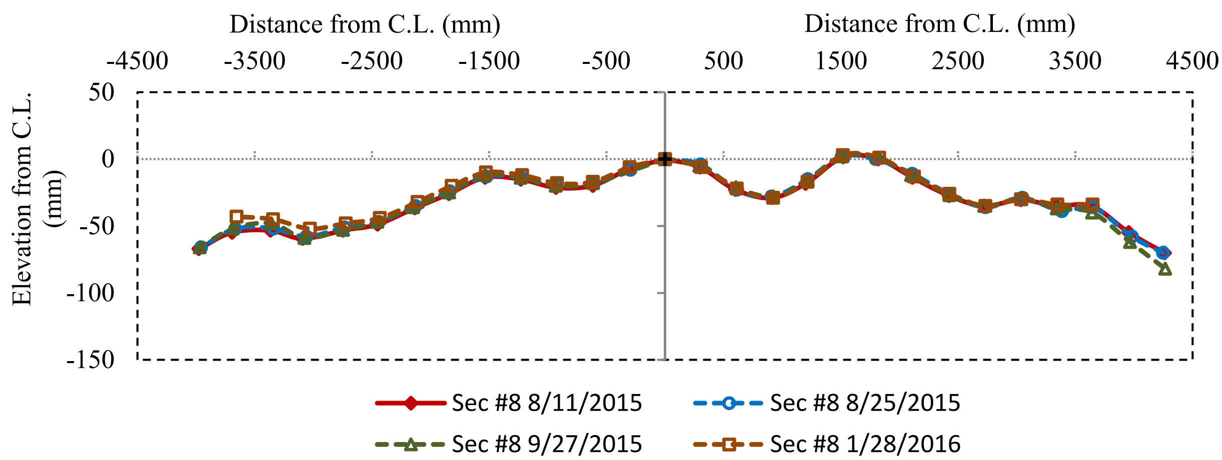


Figure 5.54: Example Results from Total Station Monitoring

5.5.1 FM972 Total Station Monitoring

Site Description

Site 8 is fully characterized previously in Sections 4.1.7 and 4.2.7.The results from those sections indicated that soil at FM 972 sits on the Branyon Clay, with a liquid limit of 66, a plasticity index of 41, and a fines content of 44%. These results categorize the soil as a Sandy Fat Clay (CH) according the ASTM D2487 and the USCS classification. The PVR from Tex-124-E was calculated to be 2.41 in., but the measured PVR from centrifuge testing was calculated to be 0.42 in. This difference, as well as the significant longitudinal cracking (found in 42 ft per 100 ft of roadway), was a cause for concern, which led to a more extensive study of the site. A significant crack was found near the sampled location on the south side of the road, as shown in Figure 5.55. The sealed crack is approximately 12 mm thick. Notably, this site has two steep slopes that lead to the drainage ditch on each side of the road. Due to the differences in PVR, two sections were marked at this location (referred to below as Sections 6 and 7) in order to monitor the change in pavement shape over time.

Figure 5.55: Longitudinal Crack at FM 972

Results from Total Station Monitoring

Figure 5.56 provides the total station monitoring results for Section 6, where the crack is located, and Figure 5.57 provides the results for Section 7, a site further to the west that does not lie on the crack, to analyze the change in pavement shape over the course of the drying and wetting seasons.

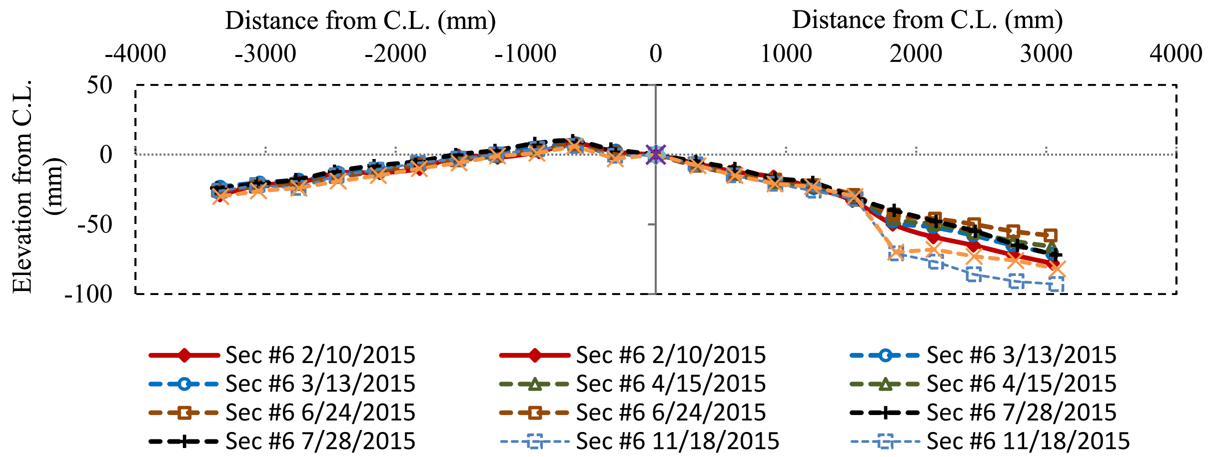


Figure 5.56: Total Station Monitoring Results from FM 972 Section 6

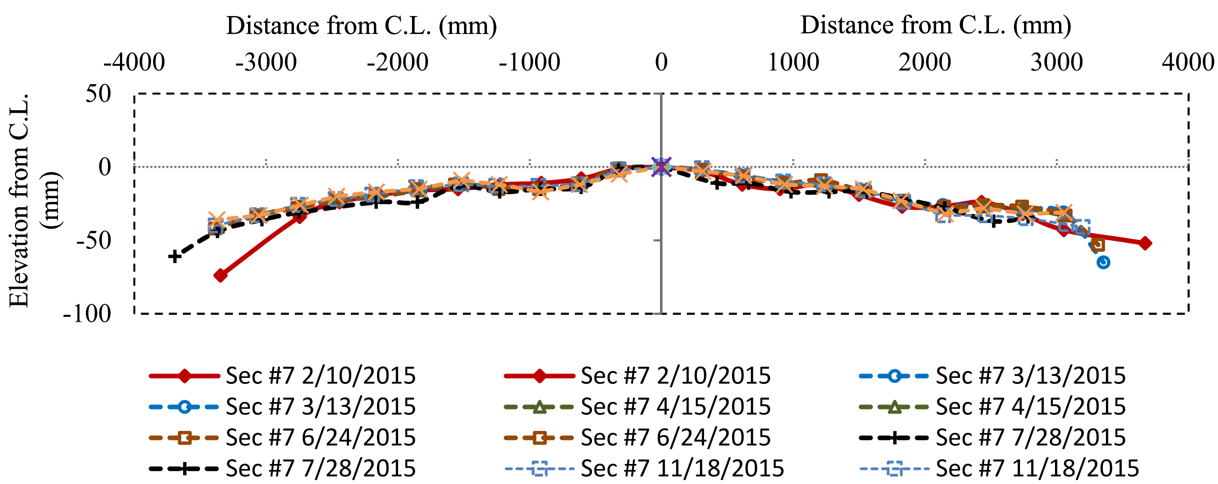


Figure 5.57: Total Station Monitoring Results from FM 972 Section 7

The results from these locations indicate that the soil is not undergoing typical movement that is found at an expansive subgrade. The crack in Figure 5.55 is seen on the right portion of Figure 5.56 and shows a decrease in height over the course of the wetting season. This change in height during wetting indicates that the soil is moving downwards during the period when the slope is expected to be swelling. This difference can be caused by a slope failure due to the high plasticity clay that sits beneath it, which typically has a low shear strength and resilient modulus that will tend to decrease as the soil becomes saturated during the normal wetting months. Further, the low PVR from the site is validated by Figure 5.57. The slope does not see a significant amount of change during the wetting and drying seasons. Due to these changes, the results are consistent with those taken from centrifuge testing.

<u>Summary of Results from Field Monitoring at FM 972</u>

The results from FM 972's total station monitoring illustrate the benefit of field monitoring pavement underlain by a potentially expansive subgrade. The cracking found in the field was much higher than that calculated by the centrifuge results, indicating that the underlying subgrade may be problematic for the shrink/swell cycles. However, the results from Figure 5.56 illustrates that the decrease in the height during the wetting season runs counter-intuitive to that from a pavement sitting over an expansive subgrade. Further, the results from Figure 5.57 show how the total station monitoring can validate the results from centrifuge testing.

5.5.2 FM685 Total Station Monitoring

Site Description

The site at FM685 is previously characterized as well as explained earlier in Section 5.4.The results from those sections indicated that soil at FM 685 sits on the Branyon Clay, with a liquid limit of 65, a plasticity index of 34, and a fines content of 93%. These results categorize the soil as a Fat Clay (CH) according the ASTM D2487 and the USCS classification. The PVR from Tex-124-E was calculated to be 0.78 in., but the measured PVR from centrifuge testing was calculated to be 1.38 in. This small PVR was not known prior to the selection of the site as the site sits on an expansive subgrade that only has 5 ft of expansive soils. However, since the site is instrumented, the site was also selected to be monitored and the section is shown in Figure 5.58. The roadway was just built, and the site sits over a driveway by a drainage facility. During the course of the project, cracks began to form at the side of the project, indicating that while the PVR was small, the moisture fluctuations were damaging the site prematurely. The crack is approximately 3 mm thick.

Figure 5.58: Painted Lines at FM685

Results from Total Station Monitoring

Figure 5.59 provides the total station monitoring results for FM 685.

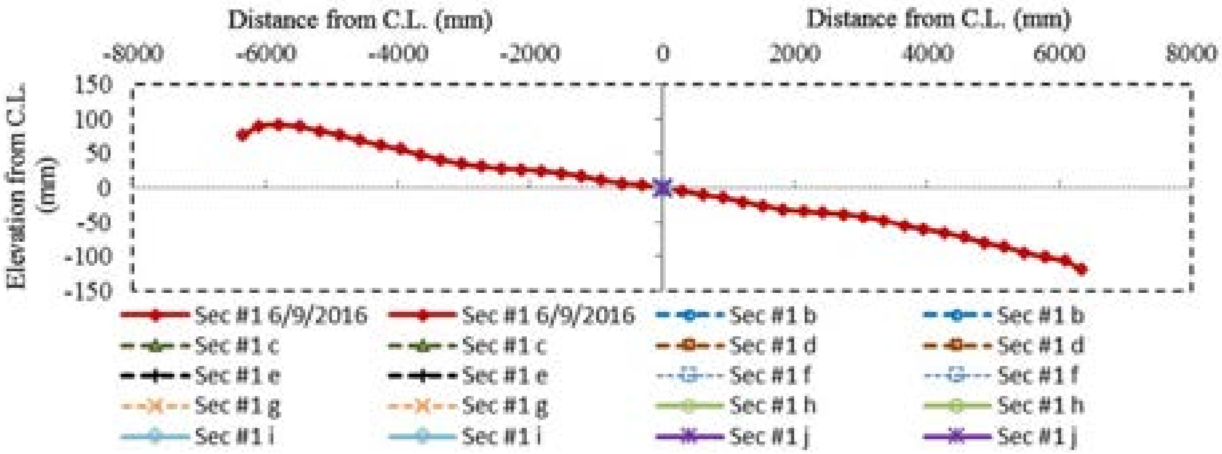


Figure 5.59: Total Station Monitoring Results from FM 685

The results from these locations indicates that the road is sloped towards the instrumented side. Due to this slope, the accumulation of moisture is expected to be present there and may be the cause of inundation. Further studies at this site and the monitoring of the deformed shape will be important as the thickness of the base layer and asphalt was measured during construction, giving the exact overburden pressure seen over the sensors. The slope does not see a significant amount of change during the wetting and drying seasons. Due to these changes, the results are consistent with those taken from centrifuge testing. This monitoring will continue past the end of the project and be reported on further in order to validate the change in the structure of the soil over multiple cyclic shrink-swell cycles.

5.5.3 FM487 Total Station Monitoring

Site Description

The site at FM487 has been fully characterized, and the subgrade has been identified as the Branyon Clay, with a liquid limit of 74 and a plasticity index of 46. These results categorize the soil as a Fat Clay (CH) according the ASTM D2487 and the USCS classification. The PVR from Tex-124-E was calculated to be 1.70 in., but the measured PVR from centrifuge testing was calculated to be 6 in. This difference was very important to examine as the difference in this test came from the moisture adjustment of the moisture content of the sampled soils. Due to the differences in PVR, two sections were marked at this location (referred to below as Sections 1 and 2) in order to monitor the change in pavement shape over time.

Results from Total Station Monitoring

Figure 5.60 provides the total station monitoring results for Section 1, closer to the intersection of FM301 and FM487, and Figure 5.61 provides the results for Section 2, a site further to the south which lies on the same soil conditions, to analyze the change in pavement shape over the course of the drying and wetting seasons.

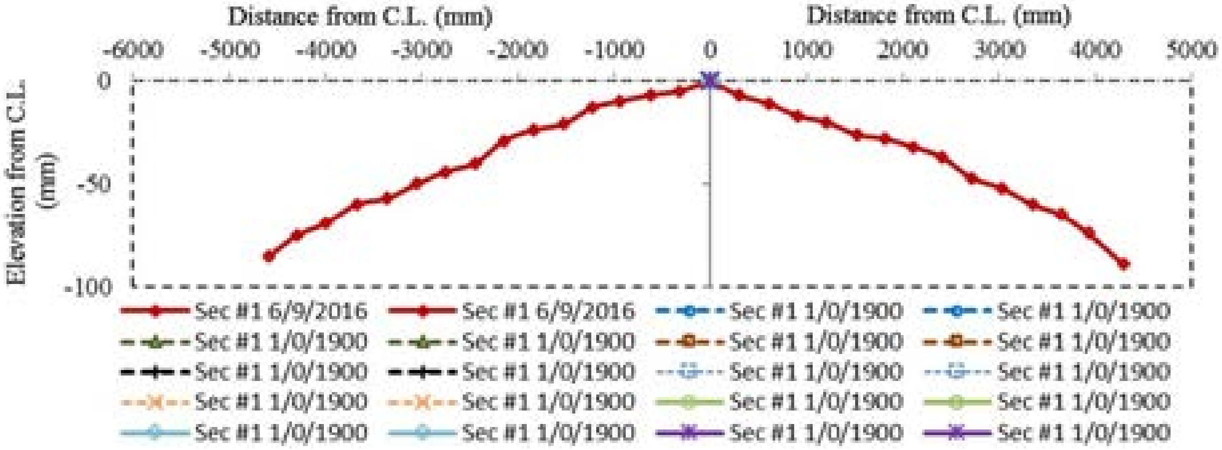


Figure 5.60: Total Station Monitoring Results from FM 487 Section 1

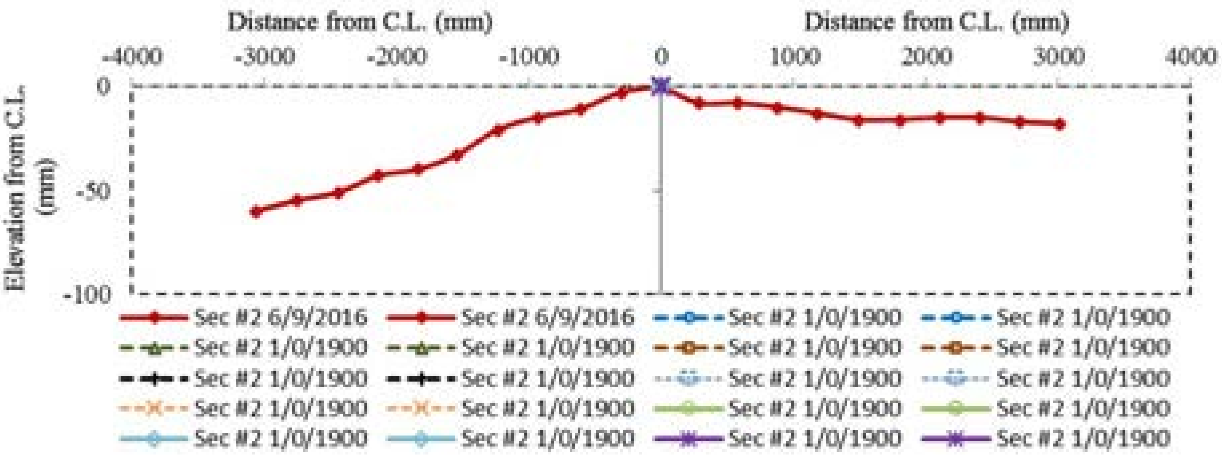


Figure 5.61: Total Station Monitoring Results from FM 487 Section 2

The results from these locations indicates that the roadway has a significantly different slope on one portion of the road as opposed to a few hundred ft down. The cause for this difference may come from the drainage ditch that comes up as you move further south on FM 301. Further analysis at this site will be completed past the end of this project to further analysis the deflected shape of the roadway over time.

5.6 Testing of Undisturbed Specimens from FM487

In order to validate the newly developed PVR method, the heterogeneity between borings was examined for the samples taken from FM487 explained in Section 2.2.2. The heterogeneity of the site was examined as the borings were approximately 100 ft away from each other at approximately the same elevation. A map of the borings is shown in Figure 5.62 with B-1 being the closest to FM487 and with all borings taken from the A horizon of the Branyon clay. The samples were dried using the environmental chamber to a gravimetric moisture of approximately of 20% immediately after sampling in 2014 as well as later while revisiting the testing in 2016. Note that there was some scatter in the gravimetric moisture content due to the natural heterogeneity in the moisture content and vugs, but the volumetric moisture content ranged between 0.290 and 0.310 for the samples. The sample data and the curve fits are shown below in Figure 5.63.

Figure 5.62: Sampling Locations for FM487 Soils

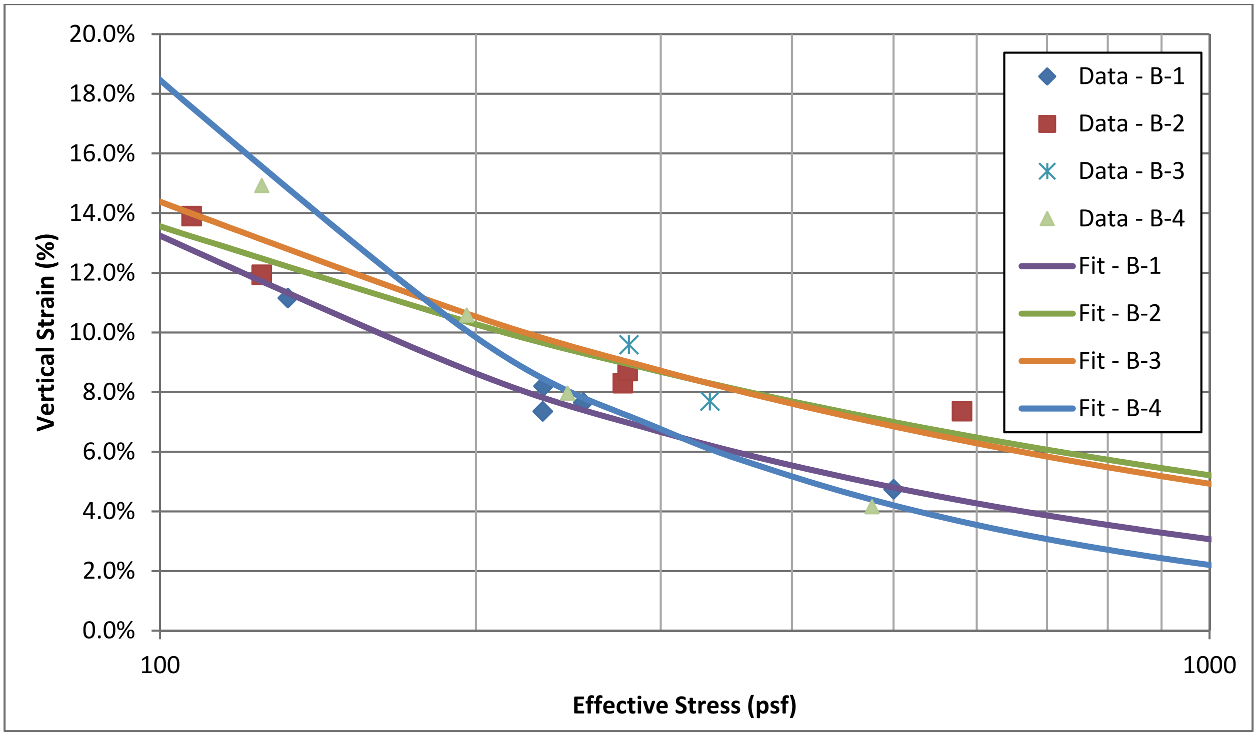


Figure 5.63: Comparison of Curve Fits for FM487 soils

While the samples did have some scatter within the results, the curve fitting indicates that the stress-swell curves between the borings is very consistent over the typical range of stresses in the active zone of approximately 200 psf to 1200 psf. Note that while the curve fitting for the two borings with the most amount of tests, B-1 and B-2, slightly differed at higher stresses, the vital portion of the stress-swell curves comes at lower stresses in which the difference between the two curves, approximately 2%, is relatively close. As such, the results indicate that the method will work well for samples taken from the same site and moisture adjusted. This application of the sampling, testing, and curve fitting has shown to work well and will be an important tool for TxDOT in the future.

In order to further understand the relationship between push samplers versus bulk samples of soils, the results of the bulk specimens taken from the borrow pit at FM487 were compared with the results from the borings. While both of the sampled soils came from the A horizon of the Branyon clay, the differences in the liquid limits were significant as the borings had a single point liquid limit of 75 whereas the bulk specimens had a liquid limit of 55. With this in mind, the difference in the stress-swell curves and experimental data is shown in Figure 5.64.

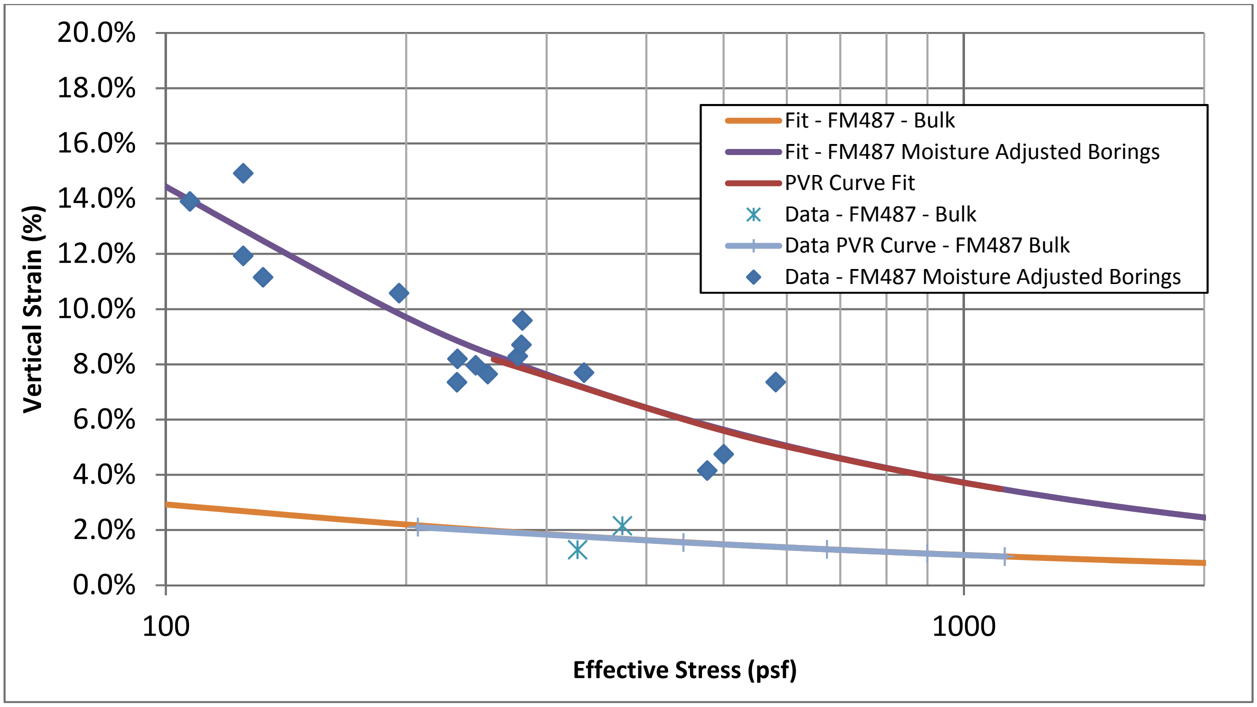


Figure 5.64: Comparison of Bulk vs Boring Specimens for FM487

The results are indicative of a massive amount of difference in swelling based upon the sampling location. Due to the nature of a borrow pit with the amount of vehicular and machine traffic, the bulk samples were likely contaminated with non-expansive parts of the soil, particularly from the non-expansive, non-clay soil that underlain the soil at the location. From these results, the importance of not contaminating the bulk specimens is shown to be vital for a proper characterization of the site. Further, differences in the unit weight of the soil becomes key in the swelling of the soils. The moisture adjusted specimens typically saw their dry unit weight increase past 100 pcf, an increase which is approximately 10% higher than the dry unit weight for the reconstituted specimens. This increase in the density is important as, when the clay particles are drawn closer together, the potential to swell increases as well. With this result and the results from the FM685 field monitoring site, the volumetric moisture content can reach the values of those from the moisture adjusted specimens from a high dry unit weight and low gravimetric moisture content which will increase a soils potential to swell significantly.

5.7 Conclusions from Field Validation of Results

In conjunction with the laboratory testing of soils, the results from typical field sites show the importance of sampling and monitoring of moisture in the active zone. The results from CAPEC sampled soils as well as the development of 6048 Method B from the project contributed to comparing results from the same soil series to soils taken from depth. The results from the moisture content monitoring of FM685 showed that the volumetric moisture contents reached in the active zone during the months of high temperature and low rainfall can exceed the baseline for testing in the laboratory experiments and drastically increase the potential of a soil to swell. The results from field monitoring are a way to monitoring the change in the shape of the deflected roadways, though further work needs to be done to monitor the shape of the roadway immediately after significant rainfall events. Finally, the results from FM487 are indicative that the contamination of bulk specimens can significantly skew the results from the field and demonstrated the differences between bulk and reconstituted specimens. As a whole, the field portion of the project was a success to validate the results taken from the lab.

Chapter 6. Conclusions and Recommendations

Over the course of the implementation project, the database of expansive soils and their properties has been significantly expanded, and the implementation of a newly developed PVR method was validated. The database of soils now includes multiple, previously undocumented soils, including those that are found west of I-35, near the fault zone itself. Further, with previously identified soil groups, the amount of soils tested has been increased significantly, providing designers with tools to examine the changes in soil behavior based on standard geotechnical characterization in order to better understand what soils will be present on-site. These tools have been combined to expand the newly developed PVR method to give designers a preliminary estimation of the PVR at a site without the need for indirect correlations for testing on unrelated soils. The method has also demonstrated the need for testing of soils on critical projects, as soil sampled in the field may vary significantly between small locations, as seen in FM487. Overall the laboratory testing program has grown significantly, with a new experimental method, an expanded database, and methods to better characterize the PVR of a site without the need for correlations from the 1950s.

Monitoring of the field validation portion of this project will continue, and it has been indicated that the methods used in the laboratory are suitable for field characterization. The sensors at FM685 indicate the soil desiccates significantly in the summer months, and sufficiently so that initial conditions for testing are suitable for calculating the PVR. These sensor methods are to be continued at FM685, and an inundation project involving the response of a subgrade to rain will be monitored. Results from samples taken from the field indicate that previously tested soils are suitable for the estimation of PVR parameters, especially those from the CAPEC sites. However, the FM487 site revealed the need for testing of soils from the field, as the moisture-adjusted specimens give a better, more accurate representation of the desiccated structure of a clay during months with lower rainfall. Finally, the total station monitoring of soils on expansive sites will further validate the bending and movement of pavements over expansive subgrades, with the results from field tested sites of FM487 and FM685 being used to validate the new PVR method. Total station monitoring has been important: the results from monitoring indicated that the PVR calculated using the new method— a value which indicates that expansive soils will not be problematic—was validated, as the pavement does not fluctuate like one underlain by an expansive subgrade would.

Overall, the implementation of the previously developed centrifuge-based method for testing of expansive soils has been a success. Further work will be done to expand the database with additional soils to give designers more choices in preliminary estimations of the response from soils on expansive subgrades, to further validate the model based on results from FM685 monitoring and the inundation projects' multiple swelling cycles, and to move toward the remediation techniques to be used when building on a site with an expansive subgrade.

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Appendix A. Database of Expansive Soils

This appendix provides 5-6048-03-P1: *Spreadsheet with Data of Swelling Curves for Clays in TxDOT Austin District*.

Soil	Soil Location	USDA Soil	Liquid Limit	Plastic Limit	Plasticity Index	USCS Classification	Fines Content (%)	Clay Content (%)	Organic Content (%)	Specific Gravity (Gs)	OMC (Std. Proctor) (%)	Max. γd (Std. Proctor) (kN/m^3)	3V-vG ε_0	3V-vG "A"	3V-vG "B"	3V-vG"C"
EF	I-35	-	88	39	49	CH		64	0.07632	2.74	24.3	15.25	28.9%	0.32	100	0.24
BR	Hutto	BrB	62	27	35	CH		58	3.6741	2.701	25.5	14.72	5.7%	26.19	20	0.05
HB	Manor	HnA	55	28	27	CH		52	3.6747	2.712	23.3	15.34	25.0%	0.61	100	0.38
TT	US-71	-	69	21	48	CH		73	-	2.76	22.5	15.68	7.7%	0.00	100	0.19
CM	SH-21	BeC2/BeB	58	17	41	CH		40	-	2.784	20	15.42	7.6%	0.38	100	0.17
BR	FM 487	BrA	74	28	46	CH		-	-	2.729	26	14.3	4.8%	0.44	100	0.31
HB	Taylor Maintenance Office	HuC2	57	26	31	CH		-	-	-	-	-	7.2%	0.42	100	0.29
HB	Kelly Ln	HnA	70	27	43	CH	77	54	-	-	-	15.79	20.4%	1.91	19	0.66
BR	Limmer Loop	BrA	55	19	37	CH		-	-	-	-	16.21	7.3%	0.37	100	0.27
HB	Turnersville Rd	HnB	56	19	37	CH		-	-	-	-	16.19	10.1%	0.38	100	0.28
HD	La Frontera	HeC2	63	21	42	CH	85	69	-	-	16.5	16.5	3.4%	0.39	100	0.28
FR	SH-45	FaA	59	25	34	CH	60	37	-	-	-	-	7.0%	0.00	11	1.11
TN	Greenlawn	Tw	69	24	45	CH	96	71	-	-	-	-	6.1%	0.83	30	0.45
CR	Parmer	CrB	71	28	43	CH	50	35	-	-	-	-	9.6%	0.70	100	0.41
HB	FM 971	HuB	72	25	47	CH	86	61	-	-	28.5	13.7	11.0%	0.60	100	0.38
BR	FM 972	BrB	66	25	41	CH	44	27	-	-	25.5	14.8	6.5%	0.00	100	0.85
BR	SH-95	BrA	60	34	26	CH	92	68	-	-	24.5	14.7	4.3%	0.50	100	0.34
HB	Taylor TxDOT	HuC2	55	23	32	CH	93	66	-	-	-	-	7.0%	0.66	100	0.19
HB	Manor -36	HnA	52	24	28	CH	87	52	-	-	22.5	15.7	10.5%	1.36	22	0.58
HB	Manor - 127	HnA	55	23	32	CH	83	52	-	-	21	15.2	7.5%	0.41	100	0.29
BH	FM 535	BeB	53	21	32	CH	78	55	-	-	21.6	14.94	4.8%	0.42	100	0.30
BH	SH-20	BeC2	50	21	29	CH	82	55	-	-	18.5	16.3	4.6%	0.60	100	0.38
CR	FM 672N	CfB	40	22	18	CH	72	36	-	-	16	16.3	2.8%	0.00	0	132.36
CR	FM 672S	CrC2	54	23	31	CH	85	76	-	-	19	15.7	8.7%	0.39	100	0.28
BU	FM 1854E	BuB	62	23	39	CH	90	53	-	-	23.5	15.1	6.1%	0.53	100	0.35
HE	FM 1854W	HeC2	65	27	38	CH	91	-	-	-	-	-	8.2%	0.00	100	0.73
HE	FM 1854 and SH-21	HeB	63	22	41	CH	91	67	-	-	22.5	15.1	13.0%	0.33	100	0.43
BR	35N Georgetown Beach	HtB	52	22	30	CH		-	-	-	-	-	1.6%	0.00	32	1.12
BR	FM685	BrB	65	31	34	CH	93	55	-	-	25.5	13.4	16.7%	2.94	100	0.19
KR	FM685	KsB	36	19	17	CH	80		-	-	-	-	-	-	-	-
HB	Gregory Manor	HnB	74	22	52	CH	64	50	-	-	-	-	16.6%	0.67	36	0.40