TECHNICAL MEMORANDUM

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Date: 11/15/2021	**Study No:** 0000027674-1
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From: Research Team	**Revision No:**
Subject: **Evaluation of the Pavement Condition in SS5 in Houston**	

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Introduction

A request was made on October 5, 2021, under the inter-agency contract (IAC Contract No 0000027674) by the Houston District, to conduct field testing and analyze data to identify the causes of distresses on SS 5 frontage road, as well as to provide recommendations on optimum rehabilitation strategies as well as whether the frontage road should be added to the upcoming project. The frontage road on SS 5 was completed in August 1999 (CSJ: 0178-09-025), and the pavement structure consists of 8-in CRCP + 1-in ASB + 6-in CTB + 6-in LTS (lime treated subgrade).

The condition of the subject section was evaluated at the project site, and undulations on the pavement surface and longitudinal cracking were two major distresses observed. Those two distress types are not typical distresses observed in Texas. Undulations on concrete slab surface occur only when there are volume changes taking place in subgrade soil. Volume changes in subgrade soil also cause longitudinal cracking in CRCP.

In pavement design and construction, expansive or reactive soils pose major issues in the pavement performance. Some clay minerals within reactive soils experience volume changes when exposed to water as well as when they are exposed to prolonged periods of drying. This condition could lead to surface undulations of the concrete pavements, resulting in issues on serviceability and safety as well as distresses such as uncontrolled cracking.

To identify the causes of the pavement distresses observed in the subject project, field testing was conducted on October 8, 2021. The field testing conducted included deflection testing with FWD, coring, dynamic cone penetrometer (DCP) testing, and MIRA testing. Soil samples were obtained from cored locations and various soil testing, including Atterberg limits, moisture content and shrink/swell, was conducted in the laboratory. Figure 1 illustrates the project location as well as the locations where 6 cores were taken. FWD testing was conducted to determine the deflections at regular intervals throughout the section. Six locations were selected for coring based on the deflection values. DCP testing was conducted at 6 core locations to estimate the modulus of the subgrade soil at various depths. Lastly, soil samples were extracted from the subgrade for laboratory testing for upper-mentioned soil properties. Figures 2 through 7 illustrate FWD testing, coring, DCP and soil extraction activities conducted in the field. Based on all the information gathered, potential rehabilitation strategies were developed.

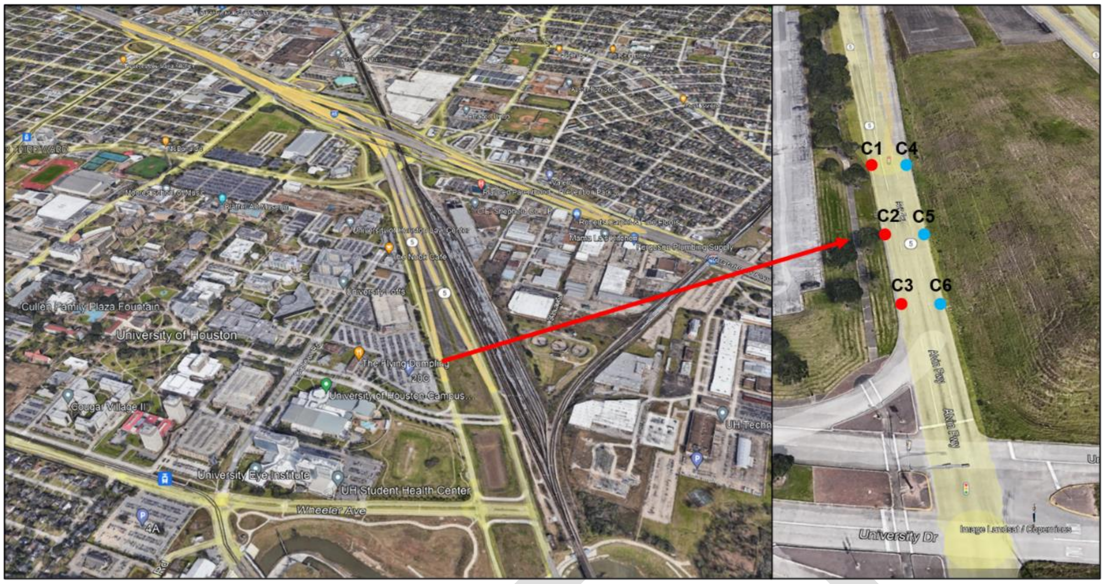

Figure 1. Field testing location

Figure 2 FWD testing

Figure 3 Coring operation

Figure 4 Thickness measurements

Figure 5 DCP testing

Figure 6 Soil extraction

Figure 7 Extracting soil sample

The following sections present the results of the testing conducted during the field testing as well as the laboratory testing to identify the soil characteristics. Structural capacity of the in-situ CRCP system evaluated by deflection testing as well as DCP testing is presented first, followed by the characteristics of the subgrade soil evaluated at the laboratory, and finally rehabilitation options identified based on the testing results.

Slab Deflections

Deflections on the slab were evaluated at outside and middle lanes, but not in the inside lane. Figures 8 and 9 present the deflections at 9,000 lb loading evaluated along the outer lane and the middle lane of the subject section, respectively. It is shown that in the outside lane, deflections were about 4 mils except DMI between 200 and 300 feet where deflections were much larger. For 8-in CRCP, the statewide average deflection is about 4 mils, and accordingly, the structural capacity of this section appears to be satisfactory, except for DMI between 200 and 300 ft. In Figure 8, 3 locations selected for coring are shown (C1, C2 and C3). As can be seen, 3 locations were selected where deflections were large and small in order to evaluate the slab support condition, more specifically subgrade modulus values, at those locations. As will be discussed later, soil modulus values were the smallest and PI was the highest at C3, even though the deflection at C3 is near average while that at C2 is the largest. This large deflection at C2 is not necessarily due to lower modulus of soil; rather, it is due to void between concrete slab and/or the edge loading condition (due to wide longitudinal cracks). There are deflections indicated by orange color between DMI of 200 to 250 ft. Those deflections were obtained at the other side of the longitudinal crack from the original drops. The difference in deflections between the original drops and the drops on the other side of the longitudinal crack is not that large, indicating that voids, if exist, are on both sides of the longitudinal crack. On the other hand, deflections in the middle lane are smaller than those in the outside lane. The pavement condition in the middle lane was also better than that in the outside lane. The pavement condition in the outside lane at DMI between 200 and 300 ft was quite poor, with longitudinal cracks with large crack widths, as shown in Figure 10.

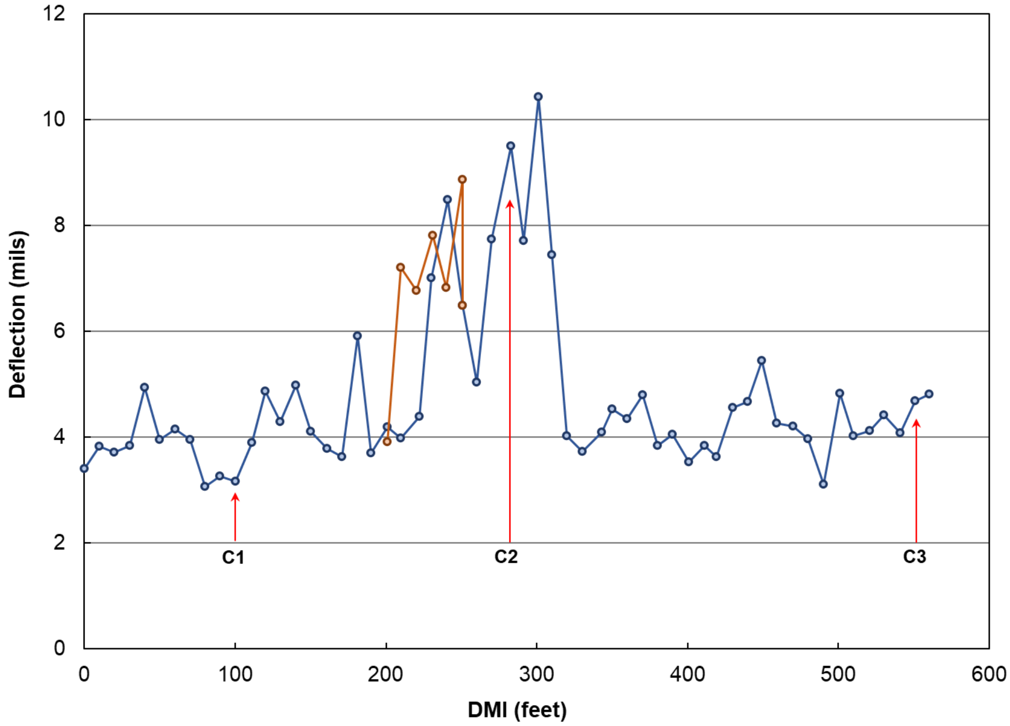

Figure 8 FWD deflections in the outside lane

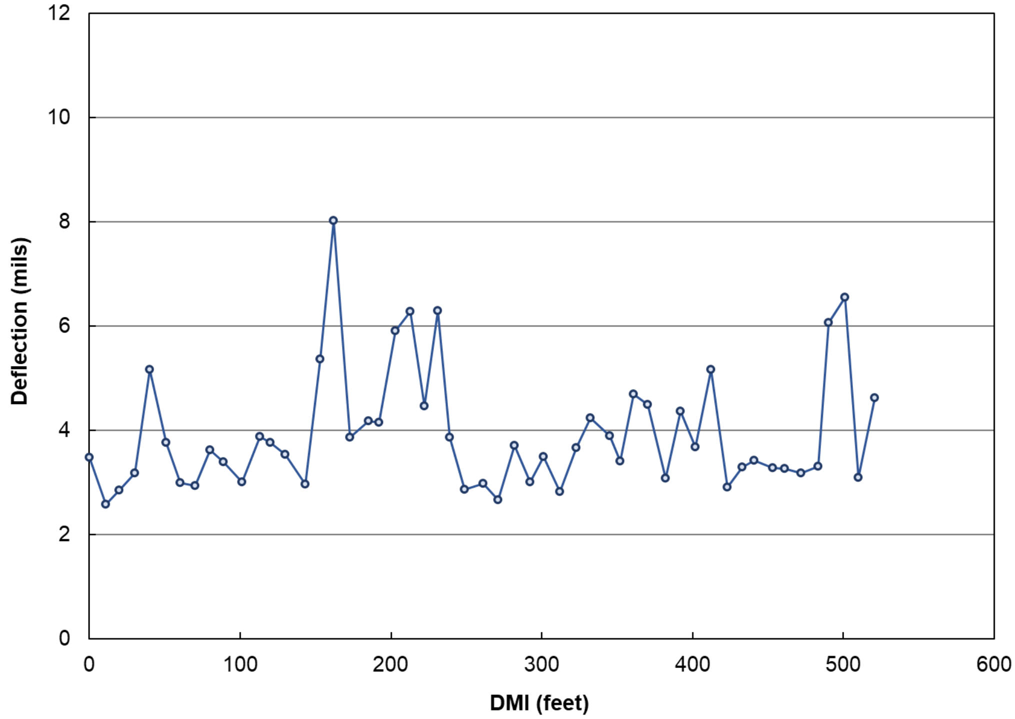

Figure 9 FWD deflections in the middle lane

Figure 10 Longitudinal crack observed between 200-300 feet (DMI) in the outside lane

Subgrade Modulus by DCP

Modulus of subgrade soil was estimated from DCP testing conducted at 6 locations (3 in the outside lane and the other 3 in the inside lane). Figure 11 presents DCP data at all 6 coring locations. It is interesting to see that the DCP curves for C1 to C3 (outside lane) are quite similar, while the curves for C4 to C6 (middle lane) are different from those for C1 to C3. The slope of the DCP curves indicates the penetration rate per one blow, and steeper slope means larger penetration rate or lower modulus of soil. The difference in the penetration rates at outside and middle lanes indicate different soil stiffness between the two lanes, with soil stiffness in the outside lane is lower than that in the middle lane. This difference in soil stiffnesses explains larger slab deflections in the outside lane compared with those in the middle lane. Modulus of subgrade soil obtained at 6 cored locations at various depths was estimated and Figure 12 shows the results. Here, the depth is from the bottom of LTS (lime treated subgrade). It shows that the modulus values are quite low, especially in the outside lane. In general, the modulus values about 9,000 to 10,000 psi are considered average or acceptable values. Figure 13 shows subgrade modulus values obtained from DCP at 10 locations in FM 1938 project in the Fort Worth District. The modulus of cement treated subgrade soil (indicated as "CTG") is larger than 30 ksi, while the modulus values of natural soil varied from 12 ksi to 22 ksi. Compared with the subgrade modulus values in FM 1938 project, the subgrade modulus values in this project are quite small. As will be discussed later, large moisture content as well as the soil type are responsible for the low subgrade modulus values.

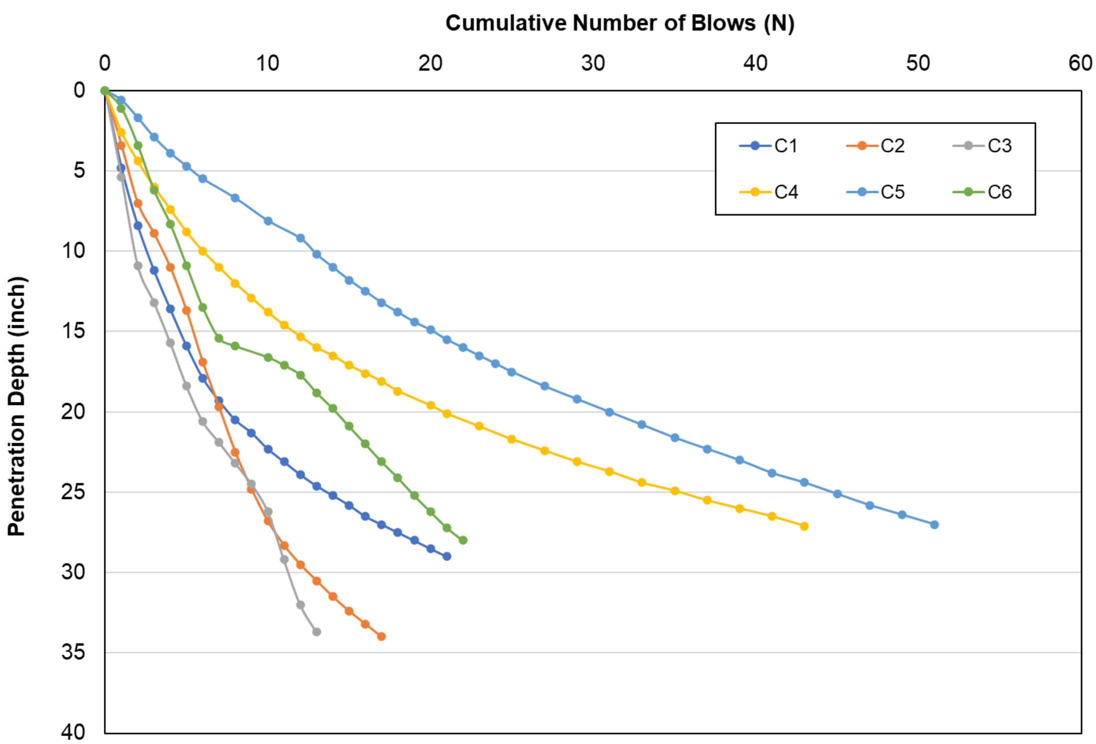

Figure 11 DCP data in the coring locations of the test section

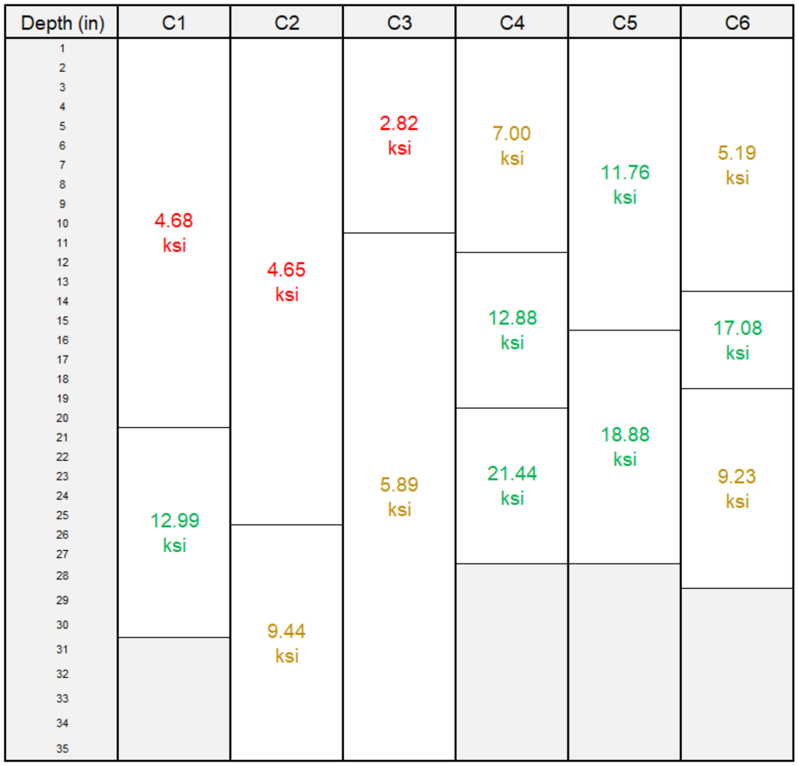

Figure 12 Calculated modulus of the subgrade (Red: less than 5 ksi, Yellow: between 5 to 10 ksi, Green: larger than 10 ksi)

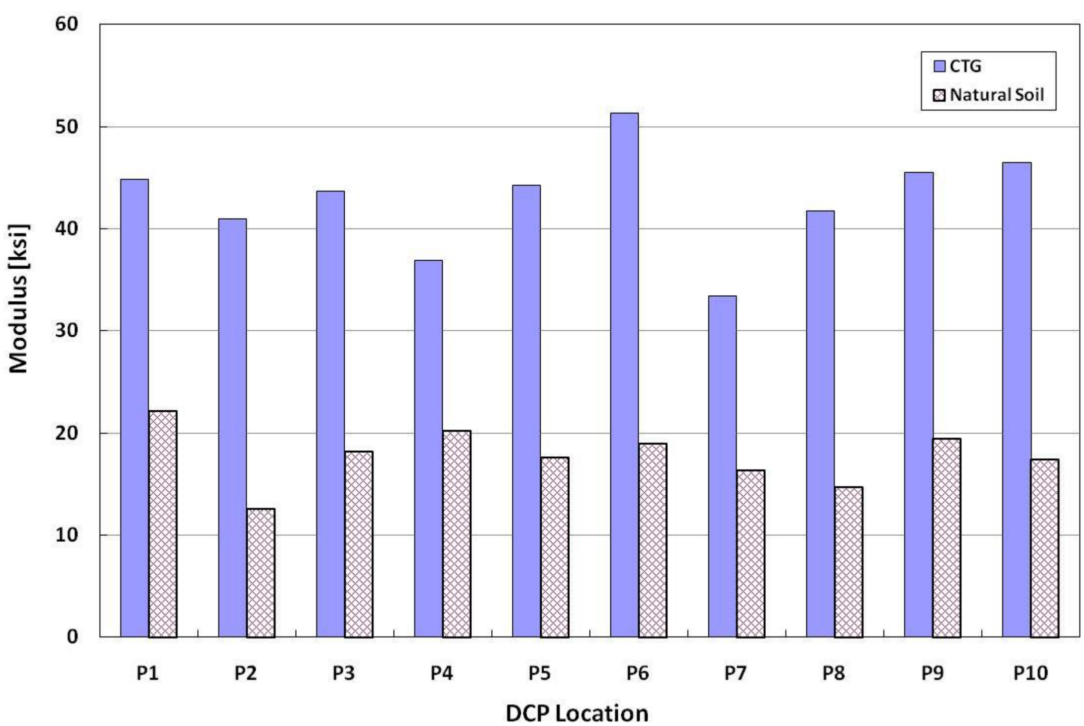

Figure 12 Modulus of subgrade soil from DCP in FM 1938 in the Fort Worth District

Subgrade Soil Characterization

Subgrade soil properties were evaluated in the laboratory. The soil properties evaluated were:

- Particle Size
- Atterberg Limits
- Shrink/Swell Potential

Heaving by the reactions between sulfate and lime was not considered as a potential cause for the distresses, as sulfate heaving has not been reported in the Houston area, and sulfate content was not evaluated.

The following testing procedures were followed:

- Tex-101-E Preparing soil and flexible base materials for testing
- Tex-103-E Determining moisture content in soil materials
- Tex-104-E Determining liquid limits of soils
- Tex-105-E Determining plastic limit of soils
- Tex-106-E Calculating the plasticity index of soils
- Tex-110-E Particle size analysis of soils
- Tex-142-E Laboratory Classification of Soils for Engineering Purposes
- The Shrink Swell Test (Fityus et al, 2005/ASTM)

No discussions are provided for the testing procedures since they are standardized testing, except for the shrink/swell testing.

<u>The Shrink Swell Testing (Fityus et al, 2005)</u>

A volume of soil contains solid particles and void space. The void space can be filled with air and water. When the void space of a soil is completely filled with water, the condition is said to be saturated. However, when the void space is only partly filled with water, then it is in an unsaturated state.

The mechanics of the way in which the soil behaves in an unsaturated state is driven by the inter-particle forces, the air and water pressures within the void space, and the surface tension arising from the interactions of the water and air within the void spaces.

In order for the expansive soil to manifest volume changes, the soil shall be subjected to a prolonged periods of wetting and drying. If the moisture content within a soil is constant, a stability is reached. However, when the layer is subjected to seasonal fluctuations of soil moisture, the volumetric changes could take place, depending on the characteristics of the soil.

The shrink swell test is one of the simple tests to investigate the reactivity (volume change) potential of soils. This test has not been adopted into TxDOT test procedures. However, it provides valuable tool for the evaluation of soil's potential for volume changes as the moisture content in the soil changes. The test procedure used in this investigation was developed by Fityus and is described below:

Shrinkage Test: A shrinkage core is of up to 2 inches in diameter and a length of 1.5-2 times the diameter as prepared from the collected soil sample. Figure 13 shows a shrinkage core specimen. The core specimen is prepared such that it is free of any defects or loose material. Initial dimensions and mass are recorded. Small pins are added to each end as reference points to facilitate consistent measurements of sample length as drying proceeds. The shrinkage core is firstly air-dried. Measurements of length and mass are recorded until shrinkage ceases. The core is then oven-dried to a constant mass at 221-230ºF and final length and mass are recorded. The data recorded facilitates the calculation of the initial and final water contents and the axial strain.

Swell Test: This involves a simplified oedometer test in which the sample is installed in a steel ring (usually around 2.5 inches in diameter and 1 inch high) and placed in a consolidation apparatus. An LVDT sensor is installed to monitor the sample height. A load of 3.6 psi (25 KPa) is then applied for 30 minutes to record any initial settlement or seating adjustment. This displacement is used to correct the initial sample height for the determination of the swelling strain. After re-zeroing the displacement gage, the sample is then inundated with distilled water and allowed to swell until the swelling increment is not more the 5% of the total recorded swell. The initial water content is determined from the sample trimmings and final water content is measure from the extracted soil sample at the end of the test. Figure 14 illustrates the sequence of the swell testing and Figure 15 shows the close-up view of the last phase of the swell testing.

Shrinkage and swell strains, measured in the respective tests, are then combined to obtain a shrink-swell index (I_{ss}). This is given by the following equation:

$$I_{ss} = \frac{\varepsilon_{sh} + \frac{\varepsilon_{sw}}{2}}{1.8}$$

where:
ε_{sh} is the shrinkage strain in %
ε_{sw} is the swelling strain in %

Shrink-swell index (I_{ss}) will be used to estimate ground surface movements, as will be discussed later.

Figure 13 Shrinkage core specimen

Figure 14 Swell testing procedure

Figure 15 Last phase of swell testing

Presentation of Testing Results

Soil Index Properties

Figure 16 shows the in-situ moisture content of the soil samples. Here, each layer is 6-in. Layer 1 is from the bottom of LTS to 6-in, Layer 2 from 6-in to 12-in and Layer 3 from 12-in to 18-in deep. It is observed that, in general, moisture contents are quite high, and there is not much variation among layers, except that for inside lane (C4, C5, and C6), where moisture content actually went up with depth. This high moisture content indicates the soil was in saturation at the time of testing. Moisture content in soil, especially clay, has a large impact on soil modulus, and an analysis was made to obtain the correlations between moisture content and soil modulus from DCP. Figure 17 presents the results, showing a strong correlation between moisture content and soil modulus, which is as expected.

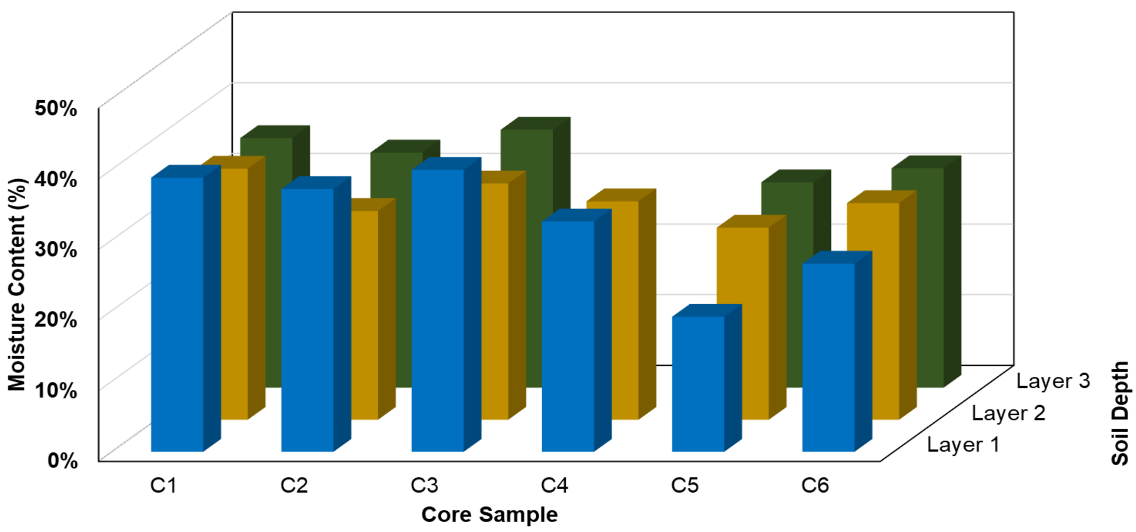

Figure 16 In-situ moisture content of the soil samples

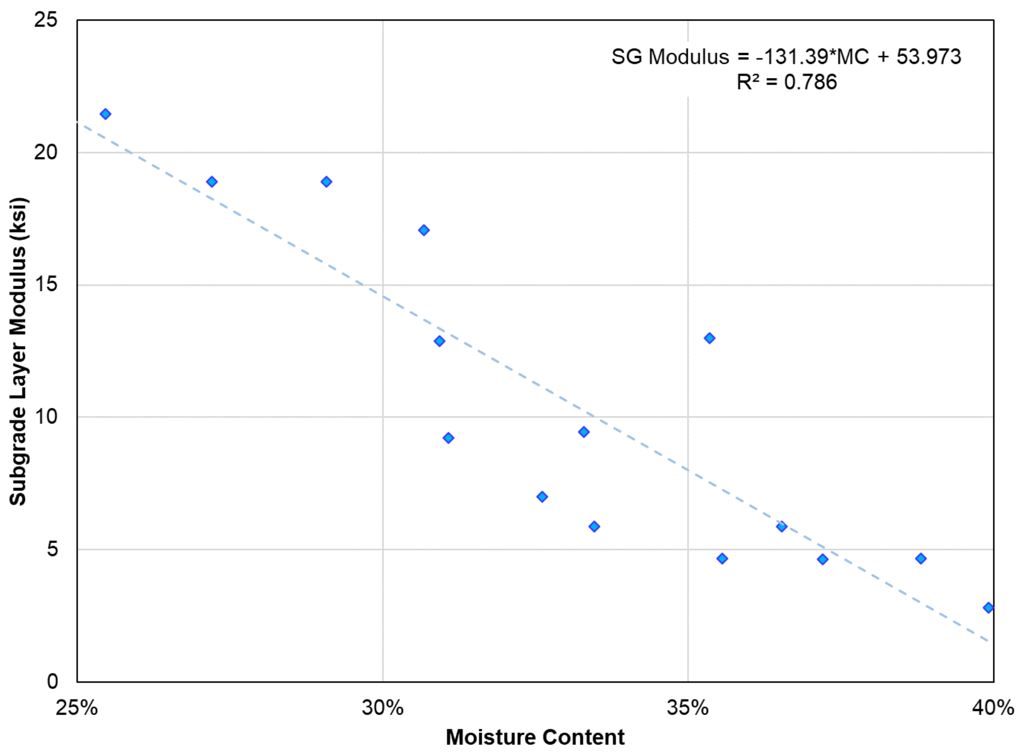

Figure 17 Relationship between in-situ moisture content and subgrade modulus

Figure 18 illustrates gradations of soils from all 6 coring locations, showing quite similar particle gradations.

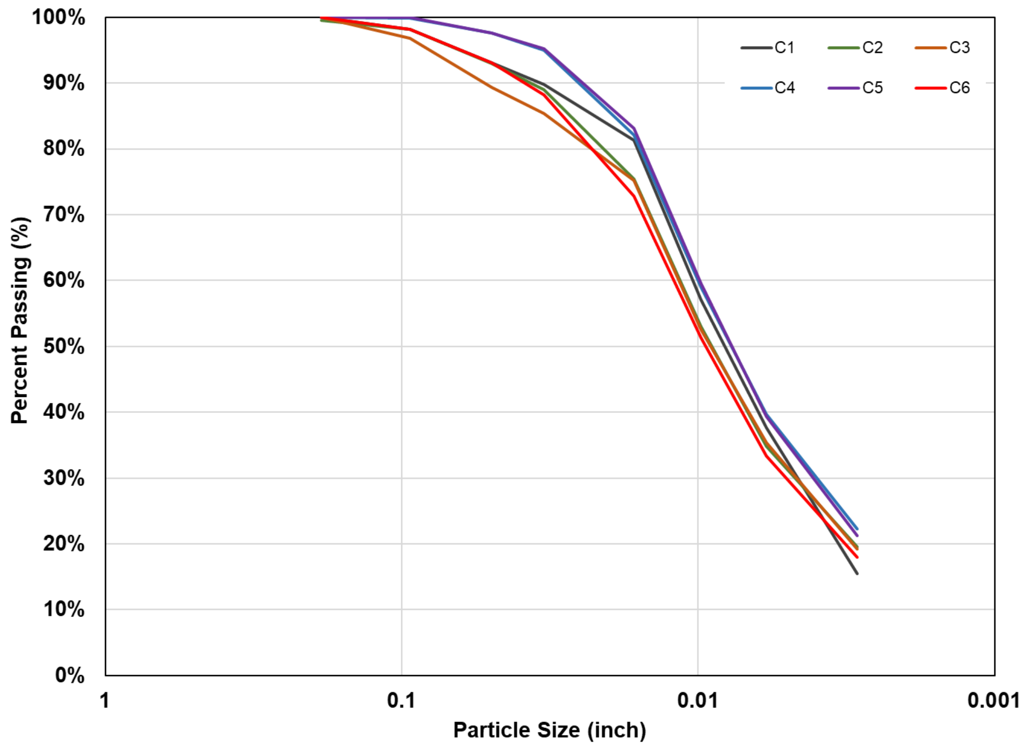

Figure 18 Particle size gradation in all coring locations within the test section

Figures 19 to 21 show the testing results of the Atterberg limits. Figure 21 shows that PI values are quite high, and PI values went up as the layer depth increases. These high PI values along with high moisture contents explains low soil modulus values, even though slab deflections were close to state-wide average, which illustrates the benefits of good quality cement treated base in providing adequate slab support.

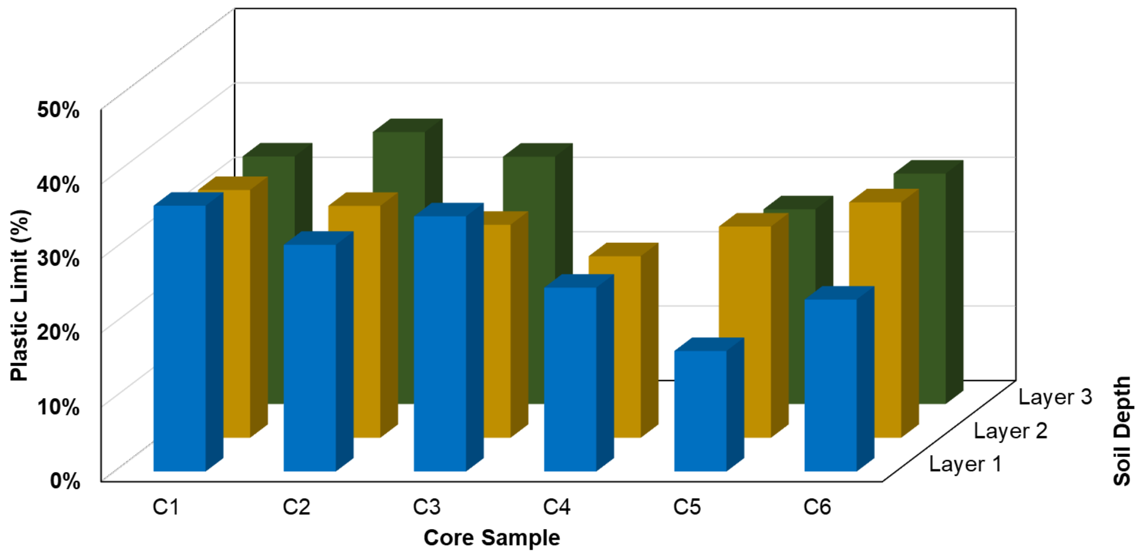

Figure 19 Plastic limit of soil samples

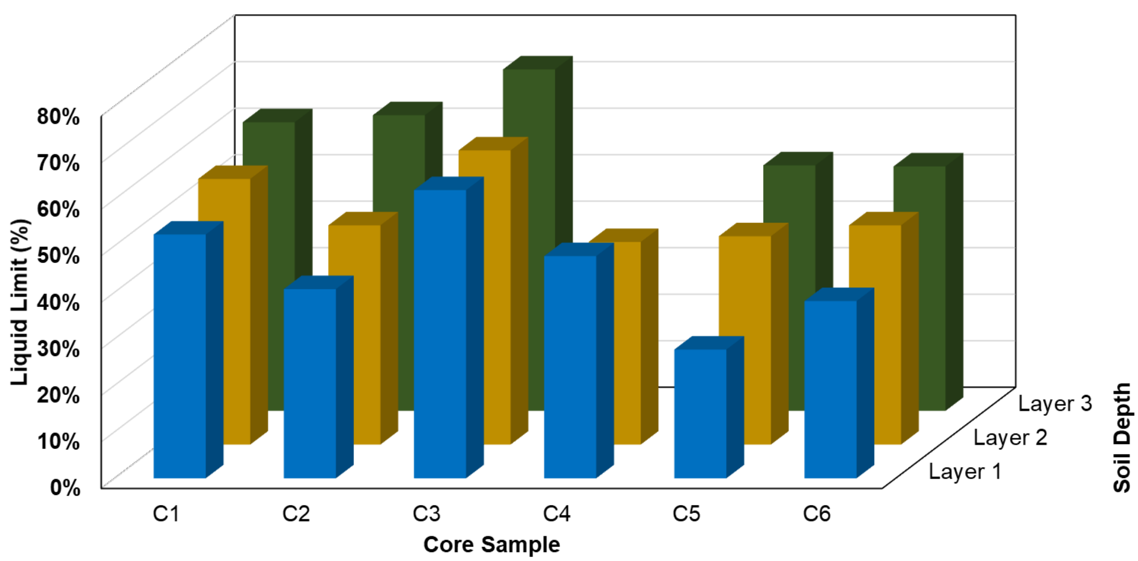

Figure 20 Liquid limit of soil samples

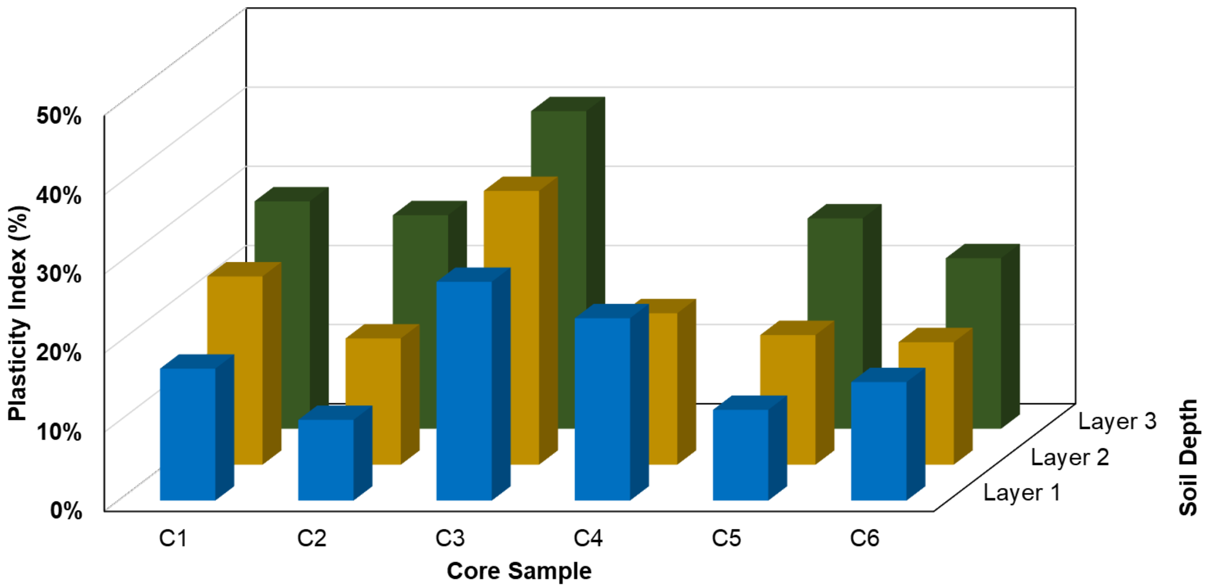

Figure 21 Plasticity index of the soil samples

Shrink Swell Test

For the shrink swell testing, in addition to the soil samples obtained from SS 5, soil samples obtained in Lubbock were also tested as a reference. In Lubbock, no undulations have been observed in CRCP, even when CRCP slab was placed directly on subgrade, which imply that the soil in Lubbock has very little shrink/swell potential. In addition, silica sand was also used as a reference sample, since silica sand has almost no plasticity and shrink/swell potential. For shrinkage testing, soil samples in Layer 2 was used while Layer 3 soil samples were used for swell testing. It is because the amount of soil samples in each layer was not sufficient to conduct both testing. Figure 22 shows that moisture contents before and after the swell test. It is recalled that the soil sample is inundated with water during the swell testing. Figure 22 shows little variations in moisture contents before and after the swell testing, except for C6 and silica sand, indicating that the water absorbed during the testing was minimal, even though there was swelling in the soils from the subject project, which will be discussed next.

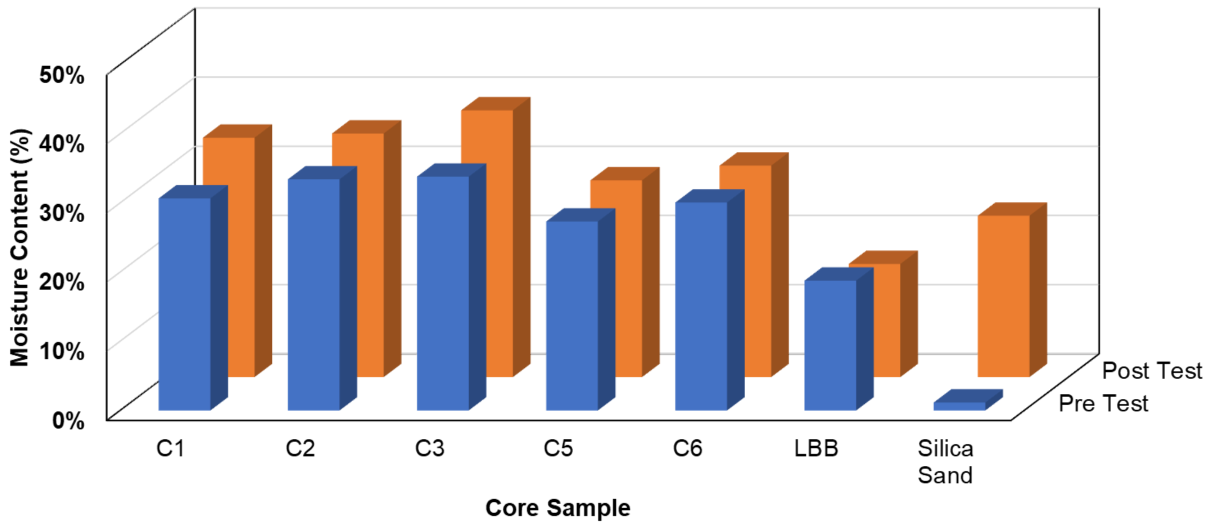

Figure 22 Moisture content before and after swelling test of soil samples

Figures 23 shows the results of the swell test. Time zero (0) represents the time when the soil specimen was inundated with water. It is observed that the silica sand does settle nor swell during the entire testing period, even though it absorbed water. The water just filled the voids among silica sand particles without changing overall volume. It is recalled that the samples were under 3.6 psi compression. Since soil has compressive strength of roughly 10 psi plus/minus few psi, 3.6 psi pressure is not small. Still, all the soils from the subject project swelled even though the absorbed water during the testing was minimal. The swelling varied from about 3 mils to 11 mils. Meanwhile, soil sample from Lubbock actually experienced consolidation, by expelling water under the 3.6 psi overburden pressure. This data clearly indicates the nature of the soil in the subject project as far as soil swell potential is concerned.

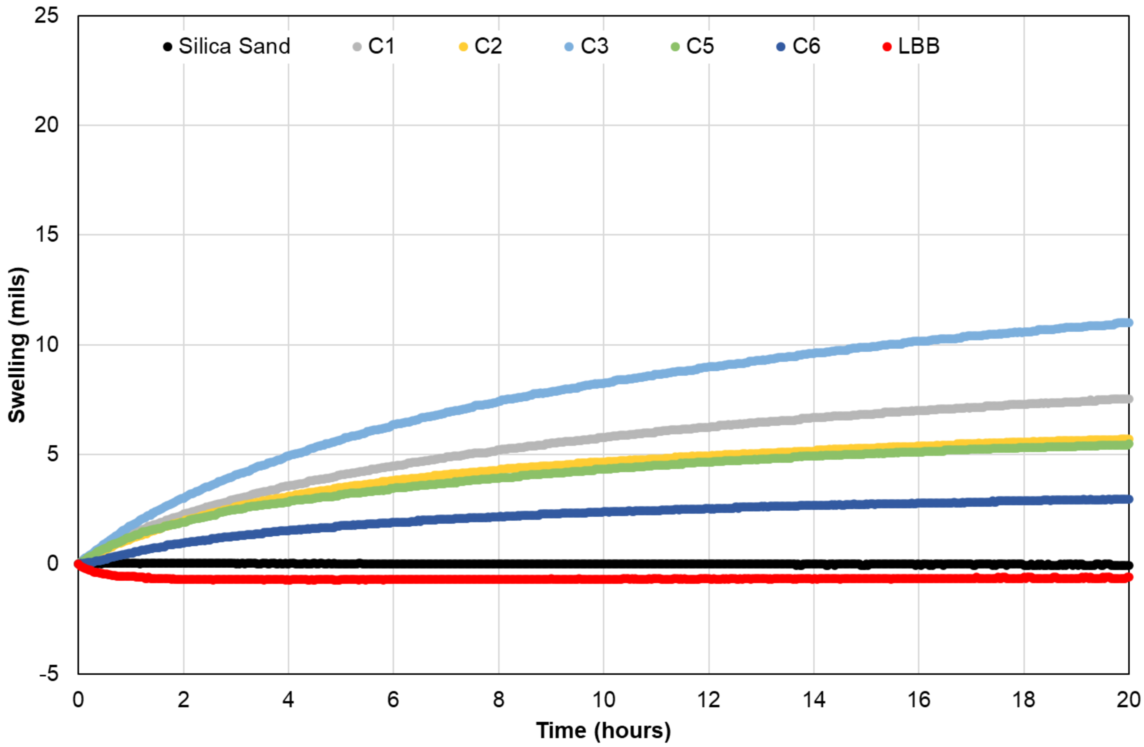

Figure 23 Swelling test results for Layer 1 soil samples

Table 1 below summarizes the swelling and moisture content variations between pre-test and post-test. It attempts to quantify the swelling potential per unit change in moisture content (1%). It is observed that Lubbock soil lost water during the testing, and was consolidated, indicating that swell potential of this soil is almost zero. Meanwhile, for soil samples from the subject project, it is observed that moisture content increased during the swelling test. The rate of swelling per change in moisture is from 190 to 581 mils/(0.01*moisture change in %). Soil sample C6 did not have significant change in moisture during the test but the magnitude of swelling is relatively high. Figure 24 presents the information in Table 1 in a graphic format. It clearly indicates, compared with Lubbock soil, the soil in the subject project has much higher potential for swelling when absorbing moisture.

Table 1 Vertical rise and moisture content variation during swelling test

Soil Sample	ΔMoisture Content (%)	Swelling, δ (mils)	Swell/ΔMC (mils/.01MC)
C1	3.95	7.52	190.6
C2	1.80	5.72	317.1
C3	4.77	11.00	230.5
C5	1.14	5.48	482.1
C6	0.51	2.96	581.7
LBB	-2.41%	-0.60	24.8

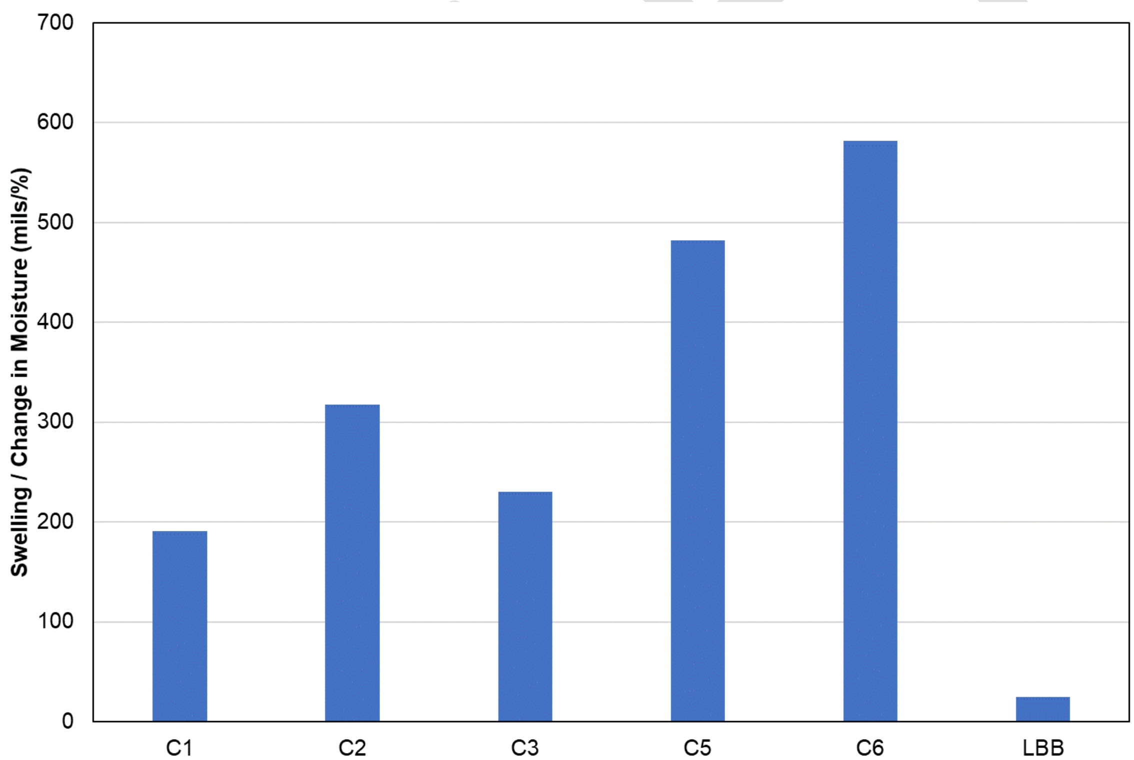

Figure 24 The rate of swelling per change in soil moisture in soil samples

Shrinkage testing consisted of measuring moisture contents as well as lengths of the soil samples at in-situ, air-dry and oven-dry conditions. Figure 25 shows the moisture contents of soil samples at in-situ and air-dry conditions. The amounts of moisture loss in C4 to C6 are slightly larger than those in C1 to C3 by 2 to 4%. However, the amounts of shrinkage after air dry in C4 to C6 are larger than those in C1 to C2 by 1 to 2% as shown in Figure 26. The shrinkage potential of the soils at the subject project is larger than that of the soil in Lubbock.

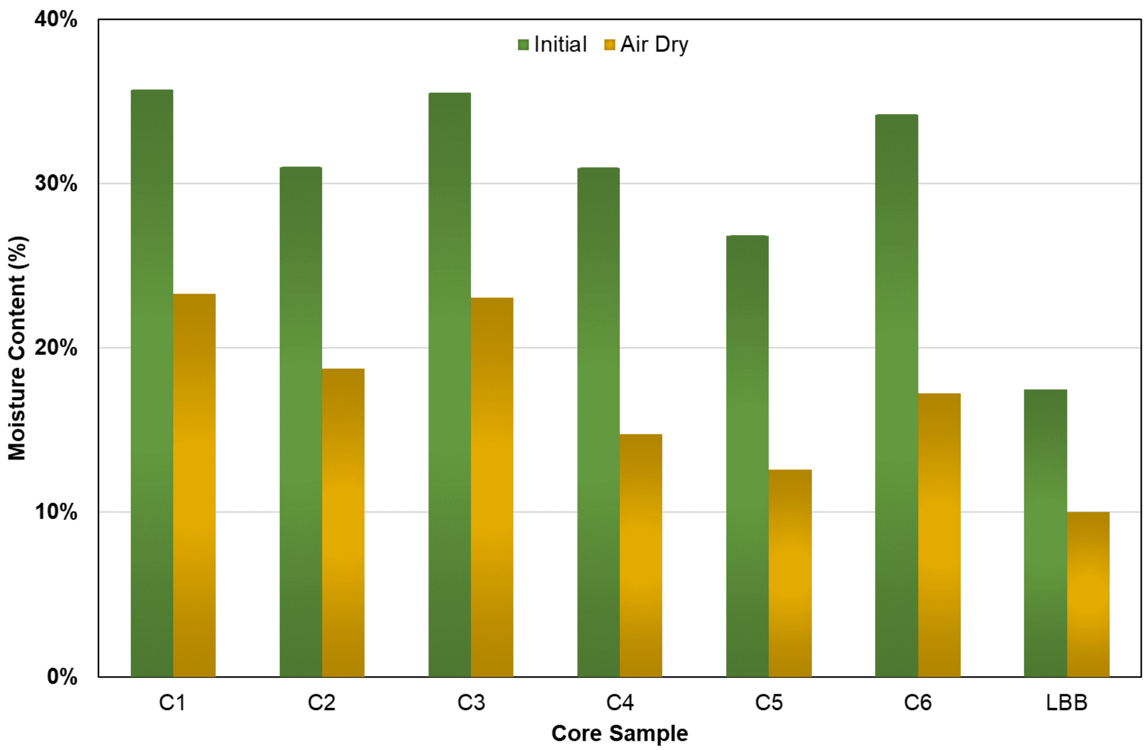

Figure 25 Moisture contents in soil samples during shrinkage test

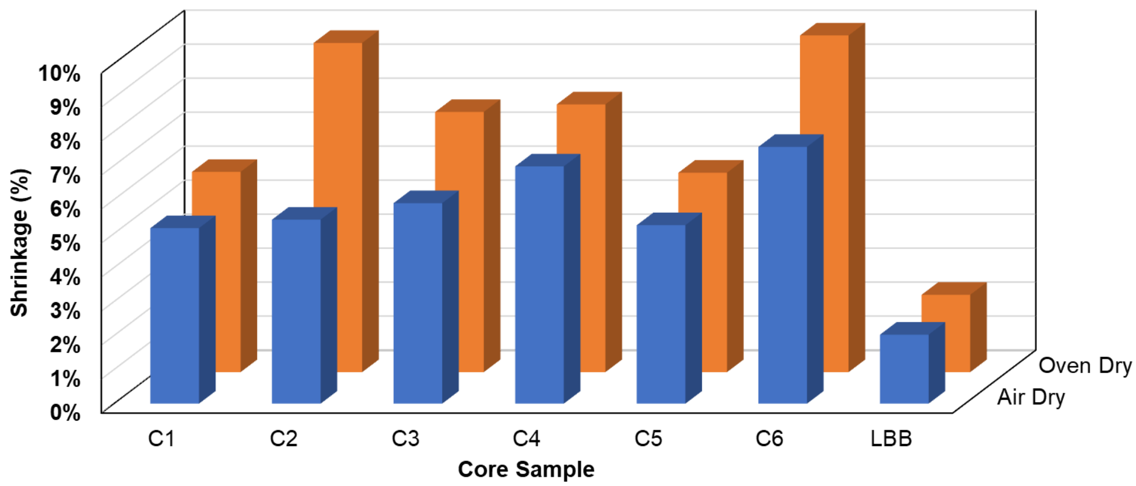

Figure 26 Shrinkage of soil samples after air dry and oven dry

Estimation of Ground Surface Movement

Fityus et al (2005) stated that the shrink swell index provides a quantitative measure of the vertical strain that will occur in the soil per unit change of suction. Aitchison (1973) developed an equation (1) to estimate the net ground surface movement using the shrink swell index. In addition, the Australian Standard AS2870 uses the same equation to predict the characteristic surface movement that a particular soil may produce under seasonal moisture variations. The equation (1) is given as:

$$y_s = \sum_{i=1}^{n} \propto \cdot I_{ss,i} \cdot \Delta pF_i \cdot \Delta z_i$$

where:

y_s = net ground surface movement

$I_{ss,i}$ is the shrink swell index of the soil layer

$\propto = 2$, constant multiplied to I_{ss} to account for the effects of lateral restraint

ΔpF_i is the magnitude of suction change assumed to be equal 1.8 pF units (6.3 KPa)

z is the layer depth

Figure 27 presents the net ground surface movements for soils from the subject project and Lubbock. It shows much larger ground surface movements in the subject project than in Lubbock.

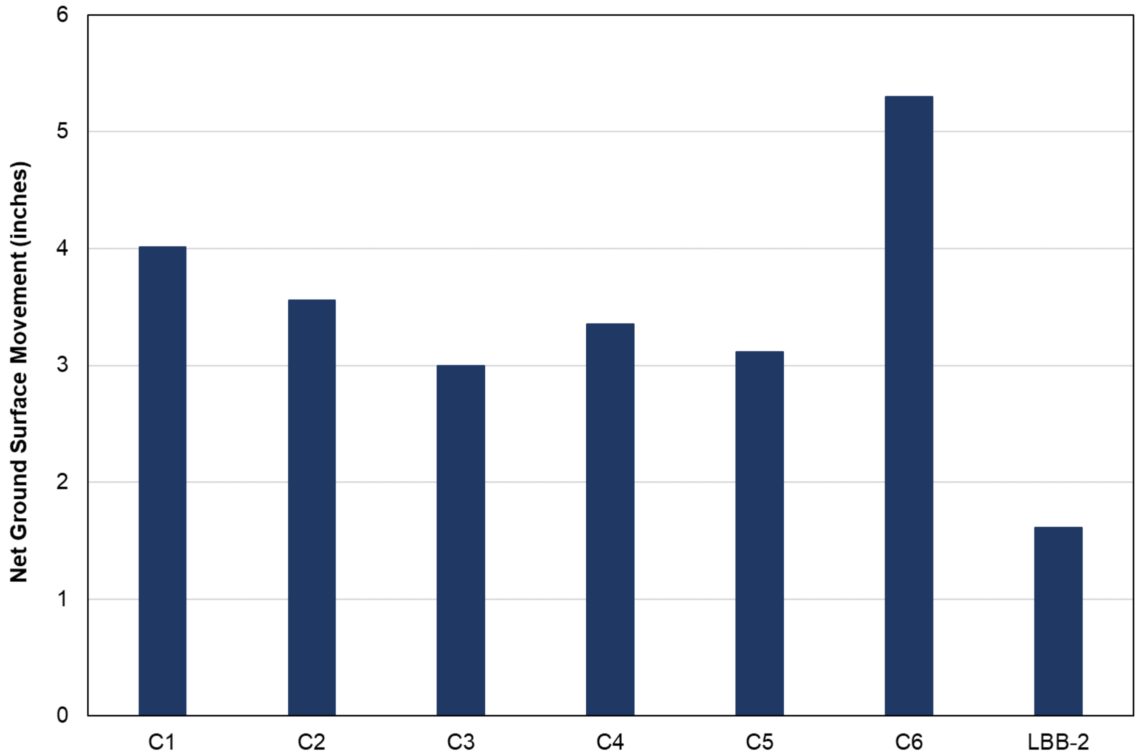

Figure 27 Characteristic soil movement of the soil samples

Summary

In this evaluation, pavement structural capacity was evaluated with FWD, while soil modulus, moisture contents/Atterberg limits, and volume change potential (shrink and swell) of the subgrade soil were evaluated with DCP and laboratory testing. The findings are summarized as follows:

- Slab deflections are comparable to state-wide average value for 8-in CRCP, indicating the pavement is in adequate condition from a structural standpoint.
- In-situ moisture contents of the soil in the subject project are quite high, and soil modulus values evaluated by DCP are low. It appears that CTB layer contributed to enhancing the structural capacity of the pavement system.
- PI values of the soils in the subject project are high.
- Compared with soil sample obtained in Lubbock, subgrade soil in the subject project has much higher potential for shrink and swell during moisture variations.
- It is concluded that the high volume change potential for the soil in the subject project was responsible for the distresses observed in the project – undulations of pavement surface and uncontrolled cracking, especially longitudinal cracking.

Recommended Rehabilitation Strategies

This investigation identified with a good confidence that the primary cause for the distresses in the CRCP slab is volume changes in the subgrade. This finding is further supported by the low level of traffic on the subject project. This project was completed in 1999, with a design life of 30 years. Since there was no fatigue cracking observed, it appears that the concrete still has many years of structural life before fatigue failure in concrete deteriorates the pavement to the terminal condition. Based on the fatigue life of CRCP sections in Houston, which is much longer than 40 years or more, structurally, this pavement section could provide another 20 or more years of structural life. Accordingly, a decision on what needs to be done should be based solely on the effects of subgrade volume changes on future performance of this section. There are several options regarding rehabilitation strategies. They are (1) remove and replace, (2) repairs/maintenance and (3) asphalt overlay. The table below outlines the advantages and disadvantages of each option:

Rehabilitation Option	Advantages	Disadvantages
Remove and Replace	• *Potentially* eliminate the existing problem.	• Cost ($$$) • User delay cost • No good information on how much soil needs to be removed and replaced or on treatment options to reduce shrinkage/swell potential
Repairs/Maintenance	• Less cost • Shorter user delay cost compared with Remove/Replace	• The same types of distresses could develop in the future.
Asphalt Overlay	• Less cost • Shorter user delay	• This may not be a permanent solution as same types of distresses may develop in concrete, which will reflect through the asphalt layer.

Other factors that could have an impact on the optimum rehab strategies include what will be done to this section during the construction of main lanes, such as adding drainage structures under the pavements in this section. Without that information, it is somewhat challenging to make a definite recommendation.

However, with little information available on what works best to control soil volume changes (little information available on this issue are still on research stage, and no long-term effectiveness has been verified in routine applications of those technologies), repairs/maintenance might be an option to further explore, since there are many tools available to address concrete pavement issues due to volume changes in the subgrade soil, such as undersealing, full-depth repairs with thick concrete (as much as 20+ inches) and heavier steel reinforcements, and milling/diamond griding. However, if repairs are not done properly or excessive soil volume changes continue, further repairs might be needed in the future, even though it is not known how long it will take before next round of repairs will be needed. This “future repairs” might be applicable to all the options, including remove/replace and asphalt overlay.

The research team will be happy to participate in the de-briefing meeting to present information on the effectiveness of various CRCP repair methods and routine maintenance techniques.

References

Fityus, S.G. (2005), Cameron, D.A., and Walsh, P.F. *The shrink swell test***,** Geotech Test J, 28, pp. 92-101

Atchison, G.D. (1973), *Quantitative Description of Stress-Deformation Behavior of Expansive Soil*, Proceedings of the 3rd International Conference on Expansive Soils, Haifa, Vol 2, pp. 79-82