Forensic Investigation of Pavement Premature Failure Due to Soil Sulfate-Induced Heave

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ABSTRACT

A series of nondestructive testing and lab tests were performed in determining the cause of extensive longitudinal cracking and severe roughness in the vicinity of the Baylor Creek Bridge on US 287 near by the city of Childress, Texas.

A sulfate-induced heave problem was identified from site observations and verified based on the lab test results. One site where the greatest distress was observed contained a high sulfate concentration exceeding 35,000 ppm (parts per million), far above the 8,000 ppm limit recommended for use of calcium-based stabilizers. Reactions of sulfates at concentrations at this level with calcium-based stabilizers can cause severe heaving. The second site had a moderate sulfate concentration of about 6800 ppm, a level where special considerations must be used when applying a calcium-based stabilizer. Swell potential at this site is lower than the location with higher sulfate concentrations.

It is not recommended that the soil with the high sulfate content be stabilized using traditional calcium-based stabilizers alone because of high swell potential and low residual unconfined compressive strength demonstrated in the 10-day capillary soak evaluation. The combination of lime and fly ash proved to be the best stabilizer for the soil with high sulfate content, provided that special precautions are given during the mix design and construction process. The effective stabilization for the soil with moderate sulfate content could be achieved using cement, lime-fly ash, or lime-slag. The lime-slag seemed to be the best stabilizer in terms of residual unconfined compressive strength and three-dimensional swell measurements. Although cement can be used to stabilize this soil, special precautions are needed to avoid sulfate-induced heaving problems.

Key Words: Pavement Nondestructive Testing, Sulfate-induced Heave, Sulfate Concentration, Conductivity and Colorimetry
INTRODUCTION

Sulfate-induced heave problems have been observed for years (1–5). Recently, it has become a more recognized serious problem in soil stabilization using traditional calcium-based stabilizers such as lime and cement throughout the state of Texas. The Texas Department of Transportation (TxDOT) has developed two lab tests (conductivity and colorimetry) to measure the sulfate concentrations in soils. These two tests are currently under state-wide implementation. A series of precautionary measures for construction should be taken based on the level of sulfate concentrations, once the sulfate problem has been identified and verified.

The Materials & Pavements Section of TxDOT was requested to assist in determining the cause of extensive longitudinal cracking and severe roughness (swells) in the vicinity of the Baylor Creek Bridge on US 287 near by the city of Childress of Texas in February, 2005. Both the northbound (NB) and southbound (SB) lanes were reconstructed approximately 5 years prior to this investigation. The old pavement structure was completely removed and the new structure placed in a phased construction process with that included salvaging the existing base. The new structure consists of a two-lift (1 ½ inches each) of Type D hot mix asphalt (HMA) using PG 76-22 binder, followed by a seal coat on the top of a 10 inches Type D fly ash (FA, Class C, 6% by weight, same as below) stabilized crushed stone base, underlain by a 7 inches salvaged flexible base and then a 9 inches of lime (3%) treated subgrade. The section near Baylor Creek began experiencing roughness, extensive fatigue cracking primarily in the right wheel paths, and longitudinal edge cracking on the shoulders within two years following this reconstruction. About 2 years ago the northbound (NB) lanes over the fully paved width were milled and inlaid in the proximity of Baylor Creek.

An initial site visit was conducted on March 1, 2005. The distress in the mill and fill section was generally limited to the NB direction, south of the bridge, with large longitudinal edge cracks confined primarily to the outside shoulder, and swells appearing across the entire NB paved width (Figure 1). Some of the cracks are more than 1.0 inches wide and some show faulting. Crack sealing has been attempted more than once, but cracks continue to open wider. A vein of gypsum (calcium sulfate) was clearly visible, exposed by erosion in the ditch line paralleling the NB outside shoulder, south of the Baylor Creek Bridge (Figure 1). The gypsum deposit was not observed at any other location on the job site.
FIELD EVALUATION TESTS AND RESULT DISCUSSION

Following an initial site visit for evaluating the extent of damage, a series of nondestructive testing was performed for further field evaluation. Data collected from these nondestructive tests were used as a basis for making a strategic rehabilitation plan. The Ground Penetrating Radar (GPR) survey using the non-contact 1GHz antenna was performed for evaluating the uniformity of the thickness of each layer, and identifying any abnormalities including air voids and/or moisture trapped beneath the hot mix asphalt (HMA) pavement or within the base and subgrade soils. The Falling Weight Deflectometer (FWD) Tests were conducted, with deflections collected roughly every 150-ft. in the right wheel path of the outside NB and SB lanes to evaluate the overall structural capacity and back calculate the stiffness (modulus) of each layer of the pavement structure. The Dynamic Cone Penetrometer (DCP) Tests were also carried out at the same sites from which HMA cores were taken for evaluating the bearing capacity by measuring the resistance of a cone penetration.

Ground Penetrating Radar Survey

The GPR analyses were run to evaluate anomalies within the pavement structure related to moisture and/or low density. Depending upon signal attenuation, the 1 GHz antenna has the capability to penetrate about 24 inches beneath the surface. In this project there appears to be much higher moisture in the base material in all passes in the NB direction compared to the one pass in the SB direction as evidenced by the darker red trace at the 3 – 4 inches depth for the NB passes. Two small areas in the SB pass also have a darker red trace – these areas appear to correspond to the milled/patched areas in the inside lane. There are no other pronounced traces detected during these surveys, which may indicate moisture/density...
conditions do not change appreciably beyond this depth, or at least to the depth where the signal becomes too attenuated.

**Falling Weight Deflectometer Testing**

The FWD deflection measurements offer an indication of the structural capacity at discrete locations. Raw data was interpreted using the indices of: maximum deflection (W1), W1 is an indicator of overall pavement stiffness; Surface Curvature Index (SCI), the SCI is an indicator of “upper” strength (0 ~ 8 inches) and is the difference of the W1 and W2 sensor measurements; Base Curvature Index (BCI), the BCI is an indicator of “lower” strength (8 ~ 16 inches) and is the difference of the W2 and W3 sensor measurements; and deep subgrade strength where the W7 deflection is an indicator of deep subgrade strength (>24 inches). Locations with raw deflections W1 > 15 mils appear to be problematic. Notably, there are weak locations in both the NB and SB directions. At locations where these high deflections occur, the fault appears to be attributable to a combination of weaknesses in the “upper” and “lower” structure (high SCI & BCI).

**Dynamic Cone Penetrometer Testing**

Evaluations were conducted using the US Army Corps of Engineers correlation of the rate of penetration (Penetration Index) from DCP testing to the California Bearing Ratio (CBR). The CBR is an index that relates a soil’s resistance to penetration to that of a high quality crushed stone base. As the rate of penetration increases, the soil strength decreases. CBR’s below 10 can be considered moderately weak material (typical of untreated subgrade), and below 5 as very weak. Flexible base materials should be at least 100 CBR. For some locations, most radically in the SB lane, testing indicates that there is a softer region at the bottom of the fly ash stabilized base/ top of the reclaimed base layer. It is possible this material is moisture susceptible.

**LAB EVALUATION TESTS AND DISCUSSION**

In addition to the field testing, six hot mix asphalt (HMA) cores from the NB outside lane were pulled and two raw subgrade soil samples both from south of the bridge were taken to the lab for further evaluation and verification. HMA core locations were selected based on interpretations from the radar survey. All lab tests were conducted according to TxDOT Manual of Testing Procedures (6), except the Dynamic Shear Rehometer test for asphalt (7).

**Evaluation of HMA Cores**

Visually, none of the cores had obvious voids and the top lift was well bonded to the bottom lift in all cases. The underseal appeared to be intact on all cores. The underseal was removed and each top and bottom portion of the core was evaluated for absorption and density (Tex-207-F). All samples had low absorption (<1.0%), and relatively good density. The poorest density hovered between 90.2 ~ 90.5%. Penalty under SS 3022 (TxDOT Special Specification) begins at in-place air voids > 9.0%. The tensile strength was evaluated using the indirect tension test (Tex-226-F); all evaluated samples had breaking strengths in the 174
~ 244 psi range at temperature of 77°F (25°C), which largely complies with the newer TxDOT 2004 specification recommendations of 85 ~ 200 psi (93% density). Values higher than 200 psi might be considered more brittle, which could be a problem for HMA surfaces less than 4 ~ 6” thick over a mediocre substructure. Aggregate gradations of HMA field cores (top and bottom lifts separately) were evaluated (Tex-200-F). The top lift gradations differ significantly from the original job mixes in the 12.5, 9.5, 2.00, 0.180, and 0.075mm factions for all core locations.

The asphalt content (AC) binder was evaluated using the extraction method (Tex-210-F) for volumetric content and viscosity. Target AC by JMF (job mix formula) for the original job ranged between 4.5 and 4.7%. Viscosity of the recovered AC binder was evaluated using the dynamic shear rheometer, (AASHTO T315-04). Ideally, if knowing the viscosity reading from the virgin binder, the aging properties of the recovered binder could be evaluated using the “aging ratio”. An aging ratio above 3.5 indicates an overly aged or hardened AC binder. No test results of the original binder are available, but typical virgin binders generally have a viscosity in the neighborhood of 1.1 kPa using the DSR. This would place two of the tests samples in the borderline category and one in the exceeded (overly aged) category.

Evaluation of Subgrade Soils

The soils sample taken from the NB shoulder right of way (ROW) south of the Baylor Creek Bridge, where the greatest distress was observed, were evaluated at sulfate concentrations exceeding 35,000 ppm (parts per million), far above the 8,000 ppm limit recommended for use of calcium-based stabilizers. Reactions of sulfates concentrations at this level with calcium-based stabilizers can cause severe heaving. The rate or appearance of this heaving is highly dependent upon the coarseness of the sulfate compounds within the soil, and the influx of sufficient moisture. For this particular location, the area had experienced a prolonged period of dry weather, followed by much higher than normal rainfall. Although the underlying soil in its natural state was not found to be highly expansive, moisture transfer at the pavement edge combined with the high swell potential due to the lime stabilized sulfate-rich subgrade is likely responsible for the shoulder cracking. Swells in the interior of the pavement are most likely a more limited reaction since the moisture transfer is not as great, and because sulfate concentrations can be highly variable over a relatively small area, perhaps concentrations are lower than found at the pavement edge.

The second sample taken from the SB shoulder ROW south of the bridge had a moderate sulfate concentration of about 6800 ppm, a level where special considerations must be used when applying calcium based modifiers. Swell potential is lower than locations with higher sulfate concentrations. The series of tests using lab-molded specimens of material from the two sites (high sulfate and moderate sulfate content) south of the bridge were run to determine optimum stabilization for these materials.

Lab Testing Plan for Soils

A series of lab tests were performed to characterize the unmodified soil engineering properties, including soil particle size distribution, Atterberg limits, bar linear shrinkage, conductivity and colorimetry tests, moisture susceptibility, free-free column resonant seismic
modulus/stiffness, swelling, and unconfined compressive strength after a 10-day capillary soaking period. Addition testing on samples modified with hydrated lime (5% by weight, same as below), Portland cement Type I/II (3%), lime with fly ash (Class F) (2% lime + 12% fly ash), and lime with slag (Grade 100, ground granulated blast furnace slag) (2% lime + 6% slag) for dry strength and retained strength following a 10-day soak were then conducted. In the following discussion, these two soil samples are referred to as the High Sulfate (HS) and the Low Sulfate (LS) soils, respectively.

Soil Particle Size Distribution (Tex-110-E) A particle size distribution (PSD) constitutes a fundamental soil property correlated to many other soil properties that are used to predict soil performance. PSD information can be of value in providing initial rough estimates of a soil’s engineering properties such as permeability, strength, swell/shrinkage, etc. The particle size distributions of samples of HS and LS soils are plotted in Figure 2.

![Particle size distributions of HS and LS soils.](Figure 2)

From Figure 2, it is seen that the LS soil is coarser than the HS soil. Based on the Unified Soil Classification System, the HS soil is classified as a fine-grained soil while the LS soil is defined as a coarse-grained soil. From Figure 2, it can be seen that the HS soil has 17.8% of clay-sized material (smaller than 0.002 mm) while the LS soil has 12.1%. This high clay content in the HS sample can be an indicator of weak strength and high potential swell when wet. It also indicates that stronger and faster reactions between soil and stabilizers will occur when it is stabilized with calcium-based stabilizers.

Atterberg Limit (Tex-104 – 106-E) and Bar Linear Shrinkage Tests (Tex-107-E) The objective of determining the Atterberg limits is to obtain basic index information about the soil as an estimate of strength and settlement characteristics. Fine-grained soil is tested to determine the liquid and plastic limits, which are the moisture contents that define boundaries between material consistency states. The liquid limit (LL) and plastic limit (PL) define the water content boundaries between non-plastic, plastic and viscous fluid states. The plasticity index (PI) defines the complete range of the plastic state.
The bar linear shrinkage test is an in-house TxDOT test in determining the potential swell or shrinkage of soils. It uses the same materials as that used in Atterberg limit tests. This test can also be used as an alternative test for estimating the PI for soils that do not have enough fines for evaluation of the Atterberg limits.

Atterberg limits and bar linear shrinkage test results are plotted in Figure 3. The PI for the HS and LS soils are 13% and 5%, respectively. The bar linear shrinkage is 10% for the HS and 3% for the LS soil. These data are in conformity with the particle size distribution data analysis.

![FIGURE 3 Atterberg limits and bar linear shrinkage for HS and LS soils.](image)

It is concluded from Figures 2 and 3 that neither the HS soil nor the LS soil is considered a very highly expansive soil. This conclusion is also supported by the data from the direct swell test in which the compacted samples were placed in water and soaked through capillary rise for 10 days, with subsequent measurement of the three dimensions, the volume expansions of the samples were 3.9% and 0.2% for the HS and LS soils, respectively. More detailed discussion will be found in the following sections.

**Conductivity (Tex-146-E, draft) and Colorimetry Tests (Tex-145-E, draft)**

Conductivity and colorimetry tests are currently used in TxDOT for measurement of sulfate concentrations in soils. Electrical conductivity is related to sulfate content and can, therefore, be used to determine whether substantial amounts of sulfates are present and whether they will cause a problem when lime or cement stabilization is introduced. The conductivity test measures how well a soil sample solution can carry an electric current via soluble ions. The conductivity of a soil solution is the sum of the conductivities of all the ions present in a soil; therefore, it is not a direct test and can not distinguish the contributions of conductivity from different types of soluble ions. However, it is a relatively quick test, and can be used as a blanket screening for sulfates of in-situ soil. Another technique for sulfate detection is the colorimetry method. Colorimetry is a quantitative chemical analysis by color spectrum. It measures the degree of absorption of light transmitted through the sample solution by a colorimeter.

The conductivity and colorimetry test results for determination of sulfate concentrations for the HS and LS soils are plotted in Figure 4.
FIGURE 4  Conductivity and sulfate concentrations of HS and LS soils.

From Figure 4, it can be seen that the sulfate concentration in the HS soil is more than 5 times of that in the LS soil. Even the sulfate concentration in the LS soil exceeds the limit of 3000 ppm, which is the recommended provisional upper limit for soil stabilization using calcium-based stabilizers without taking special precautions. Therefore, the HS soil is not recommended for any calcium-based stabilization. The LS soil may still be stabilized using calcium-based stabilizers but special precautions must be used, including evaluating a proper mix design prior to construction, adding excess water during mixing, mellowing and curing. Mixing water should be at least 3% to 5% above optimum moisture content for compaction. Lime slurry should be used in lieu of dry quicklime or hydrated lime to get better uniformity, if at all possible. The mellowing (curing) period should typically be at least 72-hours, but may need to be longer depending upon previous experience.

Moisture-Density Relationship (Tex-114-E) The relationships between moisture and dry density were established for both raw soil samples and stabilized soil samples in order to prepare standard specimens for other property evaluations. The moisture-density relationships for the HS and LS soils are shown in Figures 5 and 6, respectively.
FIGURE 5  Moisture-density relationships for HS soil.

From Figures 5 and 6, it is noted that lime stabilized specimens for the HS soil did not show the typical trend in which a lime stabilized specimen normally has higher optimum moisture content (OMC) and a lower maximum dry density (MDD) relative to raw soil. For the LS soil, however, the stabilized specimens followed this general rule. This phenomenon could be caused by the adverse reactions between the sulfate and lime, which interfered with the typical cation exchange/flocculation and pozzolanic reactions between lime and soils and causes heaving problems in pavements.

It is worth pointing out, in general, the maximum dry densities of the LS soil specimens were higher than those of the HS soil specimens, and the optimum moisture contents of the LS soil specimens were lower than those of the HS soil specimens.

Unconfined Compressive Strength Test The unconfined compressive strength (UCS) test is the simplest and most widely used method for evaluation of base and soil chemical stabilization. UCS is commonly used as a key design parameter for stabilized pavements, and
reduction of strength of UCS from the as-cured to soaked state may indicate loss of strength caused by excessive moisture.

Figure 7 is the unconfined compressive strength after a 10-day capillary soaking for both the HS and LS soils. It is defined as residual UCS in this study. It is clearly seen that calcium-based stabilizers do not work well for high sulfate-bearing soils. For both the HS and LS soils, the best stabilizer is lime-slag in terms of the residual UCS. The addition of lime made HS soil’s strength even worse. Cement stabilization is also not good for the HS soil unless a large amount of cement is added, which may result in other problems such as shrinkage cracking and cost increasing. For the LS soil, the lime, cement and lime-FA can still significantly improve the residual UCS, although not as much as that provided by the lime-slag combination.

**FIGURE 7** Unconfined compressive strength for both HS and LS soils.

For the HS soil, as seen in Figure 5, all types of stabilized specimens have lower OMC’s and higher MDD than the raw soil. However, as shown in Figure 7, the 5% lime stabilized specimen has an even lower residual UCS than the raw soil, indicating that the dry density does not necessarily correspond with the residual UCS, which is normally assumed that higher dry density will result in higher UCS. The 3% cement stabilized specimen has a moderate high residual strength (16 psi); while the lime-FA and lime-slag stabilized specimens have higher residual UCS (66 and 82 psi).

For the LS soil, except the lime stabilized specimen, the stabilized specimens have lower OMC and MDD than the raw soil as seen in Figure 6, but all stabilized specimens have significantly higher residual UCS compared to the raw soil specimen as shown in Figure 7. Lime-slag enhances the residual UCS of soil the most, while other stabilizers perform similarly. This again supports the conclusion that there is no clear relationship between dry density and residual strength in the chemically stabilized specimens.

In general, by comparing Figures 5, 6 and 7, it is concluded that the LS soil specimens with higher maximum dry densities and lower optimum moisture contents have higher UCS than the HS soil specimens.

**Moisture Susceptibility (Tube Suction Test) (Tex-144-E, draft)** The Tube Suction Test is a lab test method for evaluating the moisture susceptibility of flexible base or subgrade soil
materials. The test continuously monitors surface dielectric constant or value (DV) with time through the capillary rise of moisture within the specimen for 10 days. The dielectric value is a measure of the “free” or unbound water within the base sample. It is the unbound water that is believed to be directly related to the strength of the material and to its ability to withstand repeated freeze-thaw cycling. Specimens with final dielectric values less than 10 are expected to provide a good performance, while those with dielectric values above 16 are expected to provide poor performance as base materials. Specimens having final dielectric values between 10 and 16 are expected to be marginally moisture susceptible.

Figure 8 shows the maximum surface dielectric values after 10-day capillary soaking for different types of stabilized specimens for the HS and LS soils, respectively.

![Figure 8: Dielectric values for both HS and LS soils.](image)

For the HS soil, Figure 8 clearly shows that lime and lime-slag stabilized specimens have much higher DV than the raw HS specimen implicating that potential moisture susceptibility and UCS reduction problems could occur in these stabilized specimens in the presence of water. The very low residual UCS (4 psi) in Figure 7 for the lime stabilized specimen proved this hypothesis. However, the residual UCS of lime-slag stabilized specimen is much higher than the raw HS specimen. This demonstrates that the DV is not a good indicator strength reduction due to moisture susceptibility for chemically stabilized soils. This is especially true for stabilization of high sulfate-bearing soils.

However, for the LS soil, comparing Figures 7 and 8 shows a clear trend between the residual UCS and DV in which the lowest dielectric value (6) for lime-slag stabilized LS soil is in conformity with the highest residual UCS (420 psi), and the highest DV (24) for the LS raw soil is in conformity with the lowest residual UCS (12 psi). The cement, lime-FA and lime-slag stabilized specimens have similar DV (17, 15, and 15, respectively), they also have similar residual UCS (218, 235, and 208 psi). Therefore, for the LS soil it appears the typical trend between DV and residual UCS is still valid.

**Free-Free Resonant Column Seismic Modulus Test (Tex-147-E, draft)** The free-free resonant column seismic modulus test is a simple laboratory test for determining the modulus of base materials. The modulus measured with this method is the low-strain seismic modulus...
(SM); by measuring the velocity that a wave propagates through a cylindrical specimen and combining those results with other measurable properties, the SM can be calculated.

Figure 9 shows the final seismic modulus after a 10-day capillary soaking for the HS and LS soils, respectively. 

![Bar chart showing Seismic Modulus for both HS and LS soils.](chart)

**FIGURE 9  Seismic modulus for both HS and LS soils.**

For the HS soil, by comparing Figures 7, 8, and 9, it can be seen that the SM are not exactly in conformity with either DV or UCS, but the general rule between SM and UCS is still valid. Both lime-FA and lime-slag stabilized specimens have relatively higher SM (83 and 40 ksi) and UCS (66 and 82 psi), while the raw soil, and lime and cement stabilized soil specimens have relatively lower SM (5, 9, and 21 ksi) and UCS (11, 4, and 16 psi).

For the LS soil, however, by comparing Figures 7, 8, and 9, it can be seen that the SM are in accordance with DV and UCS. The raw soil specimen has the highest DV (24) and the lowest SM (6 ksi) and UCS (12 psi). The lime-slag stabilized specimen has the lowest DV (6) and the highest SM (80 ksi) and UCS (420 psi). The lime, cement and lime-FA stabilized specimens are in between and similar in DV (17, 15, and 15), SM (44, 45, and 48 ksi) and UCS (218, 235, and 208 psi).

**Three-Dimensional Swell of Soils** The swell test is performed to evaluate the expansion of both the raw soil and stabilized soil in the presence of water. The volume of each specimen was measured after compaction and at the end of the 10-day soaking period, respectively. The percent of volume expansion for each specimen was calculated from the measurements. Figure 10 provides the plot of swells for both the HS and LS soils.
FIGURE 10  Three-dimensional swell for both HS and LS soils.

It is clearly seen from Figure 10 that the lime stabilized specimen has the highest expansion of 17.9%, and the cement stabilized specimen has the second highest expansion of 15% for the HS soil. This strongly indicates that lime or cement alone can not be used for stabilization of the HS soil. The volume expansion for the specimen stabilized with 2% lime and 12% Class F FA was only 0.1%. Therefore, lime-FA is the best stabilizer in terms of swelling.

Lime-slag may still be used as long as special precautions are given to using a proper mix design prior to construction including adding excess water during mixing, mellowing and curing. Mixing water should be at least 3% to 5% above optimum for compaction. Lime slurry should be used in lieu of dry quicklime or hydrated lime, if possible. The mellowing period should typically be at least 72-hours, but may need to be longer depending upon previous experience.

For the LS soil, the volume expansions for the specimens stabilized with 5% lime and 3% cement were 7.3% and 4.4%, respectively. Therefore, it is also recommended that lime and cement not be used for stabilization. Lime-FA and lime-slag are better choices for this soil stabilization provided that careful precautions are taken as mentioned above.

Mechanism of Soils Stabilization

Stabilization of soils is an effective method for improving soil engineering properties and pavement system performance. In this investigation, soils were treated with lime, cement, a combination of lime and Class F FA, and a combination of lime and slag.

When adding lime to soils, three major reactions will occur. Cation exchange and flocculation/agglomeration take place almost instantaneously, while pozzolanic reaction will last days, months, or sometimes even years in the presence of water. Formation of cementitious material by the reaction of lime with the pozzolans (AlO₃ and SiO₂) provided by soils in the presence of water is known as a pozzolanic reaction. The calcium silicate hydrate gel (CSH) or calcium aluminate hydrate gel (CAH) can bind inert material together. The pozzolanic reactions that occur during soil stabilization can be summarized as follows:
CaO + H₂O => Ca(OH)₂

Ca(OH)₂ => Ca²⁺ + 2[OH]⁻ (2)

Ca²⁺ + 2[OH]⁻ + SiO₂ => CSH (silica gel)

Ca²⁺ + 2[OH]⁻ + Al₂O₃ => CAH (alumina gel)

However, when sulfate-bearing soils are stabilized with calcium-based stabilizer such as lime or cement, the pH of the soils can be raised to above 12. This high pH causes dissolution of clay minerals and releases alumina into the system. With the availability of calcium released by the stabilizer and sulfur supplied by the sulfate-bearing soil, a new highly expansive mineral called ettringite, which can expand as much as two times of its original size, with a chemical formula of Ca₆[Al(OH)₆]₂(SO₄)₃·26H₂O, can form in the presence of water. Water can be supplied during the stabilization process, occur as precipitation after stabilization, or be supplied from groundwater or adjacent reservoirs.

The following reaction models are summarized by Hunter (2):

Al₄Si₄O₁₀(OH)₈ + 4(OH)⁻ + 10H₂O + => 4Al(OH)₄⁻ + 4H₄SiO₄

(dissolution of kaolinite)

CaSO₄·2H₂O => Ca²⁺ + SO₄²⁻ + 2H₂O

(dissolution of gypsum)

6Ca²⁺ + 2Al(OH)₄⁻ + 4(OH)⁻ + 3(SO₄)²⁻ + 26H₂O => Ca₆[Al(OH)₆]₂(SO₄)₃·26H₂O

(formation of ettringite)

Lime-FA or lime-slag stabilization of soils occurs in a similar manner, except the pozzolanic reactions depend on the siliceous and aluminous materials provided by fly ash or slag rather than by the soil. Fly ash and slag are artificial pozzolans which have siliceous and aluminous materials, which itself possesses little or no cementitious value but, in the presence of moisture, chemically reacts with lime at ordinary temperature to form cementitious compounds that are similar to the products formed by cement hydration. The addition of fly ash or slag enhances the formation of cementitious materials (CSH and CAH) due to pozzolanic reactions, resulting in high residual strength.

Another reason for high residual UCS and low dielectric values observed can be attributed to the higher density achieved due to the effect of filling voids in the soil structure with fly ash and slag fine particles. Therefore the channels for moisture migration within the soil specimens were mitigated.

In the meantime, by adding fly ash or slag, more soluble alumina is also available to combine with calcium and sulfur ions to form ettringite. During construction, the formation of ettringite results in a volume increase in the fresh, plastic soil-lime/fly ash or soil-lime/slag mixture. However, since the soil mixture is in a plastic condition, this expansion is harmless and unnoticed. In this case, essentially more sulfur in the soil mixture is normally consumed to form ettringite within the mellowing period. Therefore, less sulfur will be available for later ettringite formation. It is this later formed ettringite that causes expansion and cracking in the subgrade after the completion of compaction. Since at this stage the subgrade is compacted and rigid, if there are insufficient voids to accommodate the ettringite volume
increase, bulk expansion and cracks can occur. Therefore, by mitigating the formation of later ettringite formation, the swell of the soils can be reduced.

The mechanism of cement stabilization of soils is different from lime stabilization or lime-FA and lime-slag stabilization. The major reaction in cement stabilization is the formation of cementitious materials due to the hydration of cement particles. While the pozzolanic reactions do occur between calcium hydroxide and silica and alumina from the soils, they are minor reactions.

CONCLUSIONS AND RECOMMENDATIONS.

Conclusions

The extensive longitudinal cracking and surface roughness observed on US 287 were primarily from the underlying sulfate-induced soil heave, as a result of stabilization with lime, and migration of moisture through the pavement edge and subsequent penetration of surface moisture through the longitudinal cracking. Cracking most likely originated in the stabilized lime treated subgrade layer and has reflected to the surface.

The sulfate concentrations in the two soil samples varied significantly from 6,800 ppm to 35,000 ppm, respectively. This indicates the high sulfate-bearing soil problem is localized. It is strongly recommended that the soil with the highest sulfate content not be stabilized using traditional calcium-based stabilizers alone (such as lime and cement) due to the high potential for swelling and low residual unconfined compressive strength after a 10-day capillary soaking period. The combination of lime and fly ash appears to be the best stabilizer for the highest sulfate-bearing soil provided that special precautions are taken during the mix design and construction process. The soil with moderate sulfate content can be stabilized using cement, lime-FA, or lime-slag. The lime-slag combination seems to be the best stabilizer in terms of residual unconfined compressive strength and swell measurements. Although cement can be used to stabilize this soil, special precautions need to be taken to avoid a sulfate-induced heaving problem.

Recommendations

In the long term, improving the ride quality and reducing the maintenance effort can best be guaranteed by removing the existing structure within the limits of the greatest distress, properly addressing the sulfate issue, and reconstructing the pavement. This could entail either removing the existing soil and replacing with a suitable select fill, or reworking the existing subgrade using stabilizers recommended in this study. Mechanical soil stabilization using Geogrid in lieu of chemical soil stabilization may be another option.

Milling the surface and applying a thin overlay to the roughest section will restore the ride quality, but the longevity of this fix will be highly dependent upon whether further sulfate-induced reactions will occur (or the extent of these reactions). In pursuing an adequate repair and sealing of the pavement surface cracking to prevent further moisture intrusion/evaporation, some consideration should be made to stabilize moisture transfer through the exposed shoulder front slope by application of compost.
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REFERENCES