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Identification and Stabilization of Problematic Silts

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IDENTIFICATION AND STABILIZATION OF PROBLEMATIC SILTS

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ABSTRACT

Silty soils are a common occurrence and can exhibit low strengths, and minimal bearing capacity, causing widespread construction and performance problems. These soils are highly moisture-sensitive and their stability is greatly influenced by the degree of densification achieved during compaction. The strength and stiffness of silty-subgrade soils is also greatly reduced when moisture infiltrates the compacted soil during post construction period.

This investigation concentrated on further developing the description of the pumping problem for soils with high silt content and the development of guidelines for stabilization of problematic silts. Using documented field experiences of the DOTD districts, the study presents a testing program to investigate the nature of the problem, the character of the silt materials, and their performance with modifying/stabilizing agents. Seven representative silt soil samples were used in the laboratory program. The unique characteristics that identify a soil with a high potential to pump were determined with standard laboratory tests. The response and stability of the silts under compaction and loading with various moisture levels and compaction efforts was also tested. The susceptibility to pumping of the different samples was reviewed in terms of their physical characteristics. In addition to the silt content percentage, the plasticity character was noted as being significant. The potential for the modification/stabilization of the problem silt soils was also studied. The laboratory tests were selected with respect to construction

needs and possible post construction conditions. A limited number of specific additives were proposed with consideration for their ability to dry the subgrade silts sufficiently to be compacted and with the strength to provide a “working table” for the construction of the base and pavement. The additives selected included hydrated lime, portland cement, and class c fly ash. A unitary set of guidelines for stabilizing problematic silts is proposed.

INTRODUCTION

Soils with high-silt contents have low strengths and minimal bearing capacity. When considered for road subgrade, wet silts or those located in areas with a high water table, soil compaction efforts and construction traffic can produce detrimental pumping action, caused by the redistribution of water due to an uplifting effort. A modification of the moisture content produces negative effects on construction parameters such as soils strength and dry unit weight. This can be seen in the field by observing the behavior of the soil immediately under the compactor or the wheels of heavily loaded equipment. When the soil is too wet and the applied compaction energy is too great, pumping or weaving will occur as the wheel shoves the weaker soil ahead of its motion. In several cases reports indicated the fabric separating the subgrade from the subbase, was pushed up by an uplifting effort created by pumping phenomenon. The high-silt soils when wet and located under pavement subgrade or are used as pavement embankments can constitute a real challenge during road construction phase.

Temporary solution for these construction difficulties, like adding drying agents, can not eliminate further performance problems, when the road is in service. Soils with high silt content considered for road subgrade still can be affected by pumping phenomenon due to moisture increase and traffic which can induce detrimental cyclic stress loads. A more significant cause to pumping is represented by the "high moisture susceptibility" of silts. Precipitations raise the water table level, increasing the subgrade moisture content. The capillary phenomenon induces a further ascension of water within the soil matrix. The most negative situation occurs when the subgrade can be completely saturated due to an exceptionally high volume of rainfall.

The problems generated by pumping phenomenon have negative influences on durability and integrity of road base and pavements with direct impact on the economical aspect. Direct financial costs for road repair adds to detrimental financial sides regarding set backs induced by road remediation affecting the public (consumption of extra fuel, time delays).

The approach of the problematic high-silt soils as road subgrade represents the subject of numerous research studies. Chemical stabilization takes in consideration the addition of different agents like Portland cement, lime and fly ash. The guidelines for road construction issued by different states Department of Transportation consider independent protocols for soils mixture. The important aspect of moisture susceptibility is lacking a proper attention.

The present study is focused on the problems encountered in Louisiana due to problematic silts. The Louisiana Department of Transportation and Development standard specifications limits "usable" soils to have a maximum of 65 percent silt content for use in embankments and a maximum of 60 percent silts for chemically stabilized bases (1). When a base course, subbase or embankment less than three feet in thickness is to be constructed on the surface of a cut section, the specifications require the top 36 inches of the cut area to conform to the maximum 65 percent silt requirement. In-place soils that do not meet these requirements must be removed and replaced. This can become very expensive especially when usable soils must be hauled a significant distance, if available. In practice, special provisions are often included in the contract to provide for chemical stabilization in lieu of undercutting.

Although alternative solutions for treating the silt soils are used, the long-term performance is questionable. Treatment with lime has been commonly used in an effort to temporarily dry the soil enough to construct the pavement. However, concern for the long-term performance and the integrity of the base and pavement structure has been expressed and premature failures may occur as a result of an unstable foundation. The current practice of treating the wet, silt subgrade with lime does not address long-term performance. It is used to expedite the construction activities.

Other stabilization efforts using various reinforcement methods and Portland cement have been reported as having mixed results, mainly because the current specifications do not address properly the problematic of subgrades moisture susceptibility with direct effect on long term performance. Section 305.04(2) (1) indicates a maximum silt content of 69 percent for subgrades soils to be treated. Stabilization consists of treatment with 9 percent Portland cement, with the provision that the engineer reserves the option of changing the percent cement based on field conditions (1).

More precise guidelines would increase the effectiveness of the soil stabilization, providing an improved road subgrade as a component of a more cost efficient terrestrial transportation means.

Research Objectives

This study considers the following research objectives:

1. Identification of silts with high pumping potential;

    When does pumping occur and why do some soils with high silt contents seem to pump, while others do not? What are the important characteristics of "pumping" soils?

2. Description of pumping phenomenon with respect to road construction methods;

    A description of pumping in relation to the activities and the conditions that produce pumping will be explored. Observations of pumping in the field and the stability of the soils under test conditions will be reviewed.

3. Opportunities for modifying and stabilization silts with chemical agents;

    Methods for modification of the soil which will eliminate pumping during construction will be explored. The methods reviewed will be limited to chemical stabilization/modification additives. Also, the long term stabilization effects produced and the gain in stability during pavement performance will also be explored.

4. Performances of selected chemical stabilizer;

    A comparative performance of the different modifying/stabilizing agents to identify the compatibility between the raw soil and modifying agent will be analyzed.

5. Evaluation of testing methods for silts moisture susceptibility;

Tube Suction Tests will provide information of soils moisture susceptibility by determining its final dielectric values.

6. Improvements of guidelines for road construction with silt subgrade.

Guidelines will be developed to identify those soils with a high potential for pumping, and a procedure to establish the performance of modifying agents.

LITERATURE REVIEW

General considerations regarding silts

The Bureau of Reclamation defines silt as “the fine-grained portion of soil that is nonplastic or very slightly plastic and that exhibits little or no strength when air dry. Nonplastic soil which passes a No. 200 United States Standard sieve. A soil composed of particles between 1/256 mm and 1/16 mm in diameter. A heavy soil intermediate between clay and sand.” (2)

The shape of the silt particles differentiates this material into rock – flour, which is the least plastic variety consisting of more or less grains of quartz, whereas the most plastic type is referred as plastic silt, with a considerable percentage of flake – shaped particles. Inorganic silts exhibit a smooth texture and are relatively impervious, but in a relative loose state it may rise into a drill hole or shaft like a thick viscous mud. (3)

Silty soils are highly susceptible for frost damage. If the groundwater table is close enough, the moisture is drawn up due to high capillarity, leading to the formation of ice lenses within soils subjected to freezing temperatures. This frost action causes serious damage to highways structures, the losses in U.S. and Canada involve millions of dollars annually. (4).

Aspects related to the study of silts properties

The experimental studies of silts characteristics must take in consideration the aspects related to the samples reconstituting process and its effects on geotechnical properties of the soil considered.

Comparative tests demonstrated the undisturbed silt specimens showed dilative and ductile behavior, whereas the accompanying reconstituted specimens at the same or lower density showed contraction, much lower undrained strength and brittle behavior. It is not sufficient to only satisfy the criteria of correct density and grain size distribution, but the same fabric also must be reproduced at a certain degree.

The specimens for the laboratory tests can be reconstituted dry or wet, by the use of vibration or by sedimentation or by moist tamping in layer. (5) This last procedure provides samples of which the homogeneity is tacitly assumed, even if the variation of void ratio can vary as much as 0.04 over the height of 7 cm. moist tamped loose sand. (6). The Atterberg limits, grain size distribution and mineral composition of reconstituted samples by mixing, wetting and drying process are all very similar to those of undisturbed samples (7).

For the tests conducted in this research, samples were reconstituted by moist tamping using Standard Proctor Compaction Device and Harvard Miniature Apparatus.

Collapsible Silts

Collapsible soils are defined as any unsaturated soil that goes through a radical rearrangement of particles and a great loss of volume upon wetting with or without additional loading (8).

These formations are encountered in numerous areas in the world: Russia, Romania, Argentina, South Africa and India. In U.S., collapsible soils are present but not limited to the Coastal Prairies, Loessial Hills and Mississippi Terraces. In Louisiana, unstable silts exist in a 30 miles wide band, extending from Lafayette westward to Texas and southwest of Lafayette to the Gulf of Mexico. In dry state, collapsible soils exhibit sufficient strength in order to support heavy vehicles, construction equipment, houses and similar construction. However, when it rains, these silts lose the strength and when under pressure, became a soft mud, flow or collapse. (9).

Unstable soils can be situated near stable silts deposits, with a gradational contact between these formations. The difference in behavior cannot be noticed neither by grain size, structure fossil content or Atterberg limits.

According to Thornton (9), collapsible soils in Louisiana contain over 70 percent silt size particles; the occurrence downstream of Mississippi suggests these formations are alluvial redeposited loess. The most extensive deposits of collapsible soils are Aeolian or wind deposited sands and silts (loess). In Mississippi, these deposits exhibit a porosity between 0.43 and 0.54, a plasticity Index between 2 and 12, and with Liquid Limit of 24. Specific gravity is situated in the range of 2.66 – 2.73. Silt size particles represent 89 percent and with clay at 11 percent. Soil particles in loess are loosely bounded by clay or calcareous material, which explain the low density. The sand content in loess is less than 3 percent, comparing with alluvial silts deposits containing more than 5 percent sand. (9).

Collapsible soils have a honeycomb structure of bulky-shaped grains with the particles held in place by some bonding material or force. When the support is removed, the grains move into vacant spaces, sliding over one another. The strength of many collapsible soils is provided by inter-particles cementing agents such as iron oxide or calcium carbonate. (8)

Thornton identifies four criteria for determining collapsible silts:

1. In place unit weight of undisturbed silt is less than 80 lb/ft^3.
2. Maximum dry unit weight corresponding to standard compaction is less than 104 lb/ft^3.
3. The liquor of silt and stock solution (Calgon and distillated water) from the mechanical analysis is black in color after the settlement of solids.
4. The sample undergoes over 15 percent strain from 0 to 16 tons/ft^2 in a collapse test. (9).

Because of lack of data regarding in place unit weight of undisturbed silt, these criteria could not be applied in this research.

Based on engineering experience in Louisiana, any two of these requirements are sufficient for identification of collapsible silts. The results of stabilization tests demonstrate lime and Portland cement can be reliable additives for improving the soils behavior (9).

The Pumping Phenomenon

One of the most important factors affecting the behavior of the road courses is represented by soils compaction control. Requirements for compaction moisture content and dry unit weight are referenced to an optimum water content and maximum dry unit weight corresponding to a particular compactive method (10), which usually for road construction is Standard Proctor compaction (ASTM D 698)

In the field, the specified density is achieved by progressively increasing the compaction effort by using either heavier equipment or more passes. However, soils with a high potential to pump become unstable with the higher compaction energies when the soil is wet of optimum. The conditions where this occurs are demonstrated qualitatively in Figure 1. Higher densities produce greater strength and stability, in general. Thus, in compacting the soil, the moisture governs the density achieved for a fixed compaction effort.

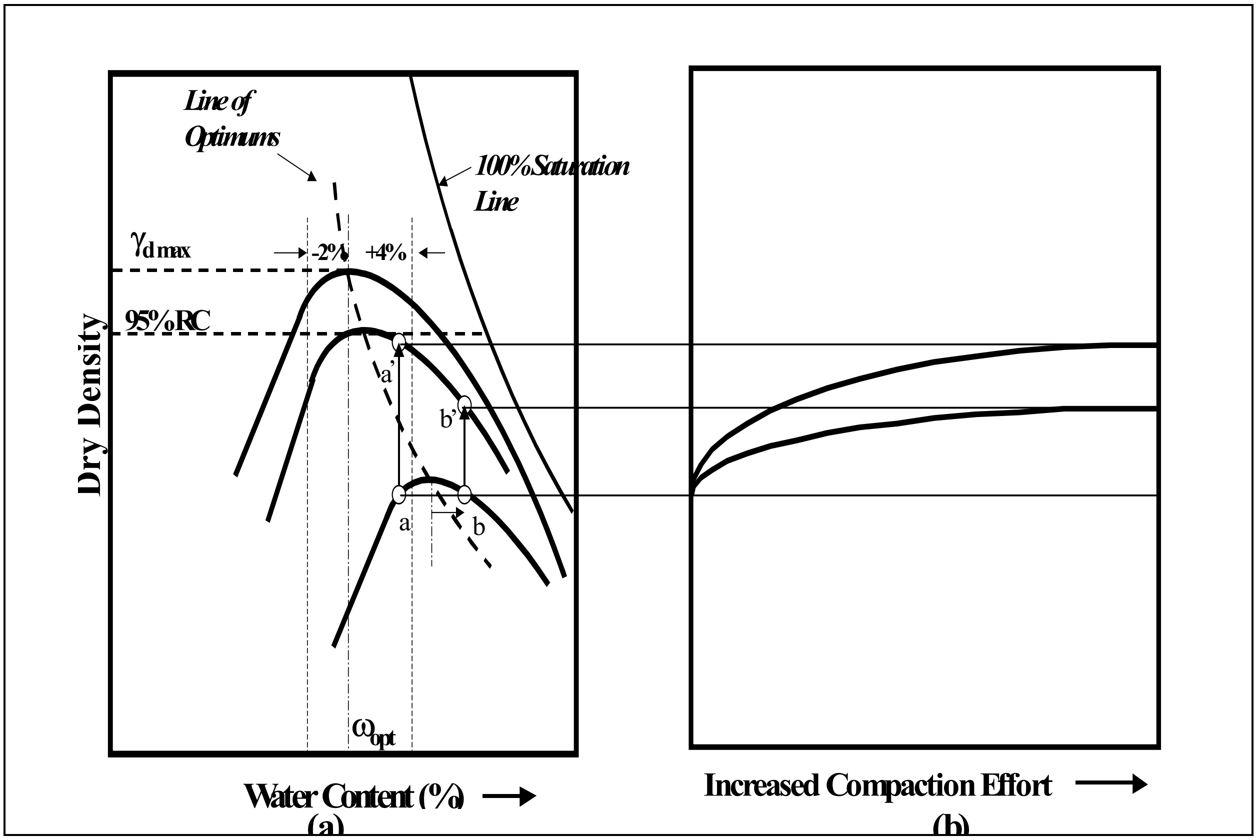

Figure 1

Pumping condition created by excessive compaction effort and moisture wet of optimum (after Sherard, et al, 1963)

In Figure 1a, the moisture content/density curves for the same soil compacted at three different energy levels are shown. Points *a* and *b* on the curve represent two conditions of the soil compacted at the lower compaction effort and to the same density but at different moisture contents. The next higher compaction energy level will generate a more substantial increase in density and stiffness for the drier moisture conditions of soil in condition *a* than that for the wetter moisture content of soil in condition *b.* The wetter material is soft and the shear stresses imposed on the soil during compaction create increased pore pressure and exceed the shear strength of the soil. The water takes up a part of space that would have been occupied by the solid particles, reducing the dry weight (11). The compaction energy is dissipated largely in shearing the compacted material without much additional densification. The soil behavior at this point has been identified as overcompaction. Under these conditions, pumping occurs with some soils (12).

In laboratory pumping susceptibility is indicated by soil's high creep under cyclic loading conducted in an essentially undrained manner for specimens not saturated (13). Specimens of soils susceptible to pump are subjected to cyclic triaxial tests, with drain valves open in order to absorb water if they develop negative pore water pressure. Large values of specimens' axial strain induced by transient load illustrate a loss in elasticity and further loss of strength due to pumping phenomenon. These testing conditions duplicate the conditions from construction site where wet soils with high silt content exhibit pumping phenomenon when subjected to cyclic loads due to traffic of heavy equipment.

Pumping phenomenon can be seen in the field by observing the behavior of the soil immediately under the compactor or the wheels of heavily loaded equipment. If the soil is too wet and the applied compaction energy is too great, pumping or weaving will occur as the wheel shoves the weaker soil ahead of its motion. A sheepsfoot roller will not be able to "walk out" (12).

Boutwell observed several attempts to compact a "pumping" material. Typically, compaction efforts stop when pumping begins. If the soil is allowed to rest undisturbed for several hours, further compaction efforts do not result in immediate pumping and some additional compaction can be achieved before pumping reoccurs. This phenomenon is caused by the dissipation of excess pore-pressure during the rest period. The difference in behavior between the undrained and drained cyclic tests on silts is consistent with this field observation (13).

The pumping phenomenon can occurs even after the construction phase, when the road is in service, if the contributing conditions are in place, especially excess moisture content and excess cycling load. In these conditions, excess pore pressure builds up due to the traffic cycling loading as the silts low permeability impedes its proper dissipation, leading to a loss in roads courses strength. In addition to these factors, the erosion and fractures of pavement will facilitate further development of subbase or subgrades pumping (14).

A more significant cause to pumping is represented by the high moisture susceptibility of silts. Precipitations raise the water table level, increasing the subgrade moisture content. The capillary phenomenon induces a furthermore ascension of water within the soil matrix. The most negative situation occurs when the subgrade can be completely saturated due to an exceptional high volume of rainfall.

The basic conditions that contribute to a “pumping” condition are:

1. The presence of a soil with characteristics susceptible to pumping (high silt content, high moisture susceptibility, low PI).
2. Excess moisture conditions (above optimum) and/or access to water.
3. An excessive compaction effort during construction phase or an excessive cycling loading during roads service (12).

The prevention or control of pumping phenomenon can be ensured by soil stabilization, improvement of courses drainage and / or use of paving fabric interlayer as a pavement moisture barrier. (15).

The problems generated by pumping phenomenon have negative influences on durability and integrity of road base and pavements with direct impact on the economical aspect. Direct financial costs for road repair add to detrimental financial costs regarding the problems induced by road remediation affecting the public (consumption of extra fuel, time delays).

Soil Liquefaction

Liquefaction represents "the sudden drop of shear strength under undrained conditions from the yield strength to the substantially smaller critical state strength", and it can be caused by cycling loading due to earthquakes or by dynamic application of a single large increment of shear stress induced by a sudden toe failure of a slope (3). Liquefaction induces extremely disruptive effects, generating landslides that damage transportation routes, interrupt surface communication lines and damage structures built in their path (16).

Pumping phenomenon as described above, could be considered a particular case of liquefaction, as in both situations soils exhibit a loss in strength due to excess pore pressure, triggered by cycling loading. However, liquefaction is characteristic only for saturated soils and leads to a total loss in strength, as the excess pore pressure becomes equal to the effective confining pressure (16). Pumping phenomenon can occurs at some percentages above optimum moisture content and affects only the road construction and service, compared with much more dramatic negative effects of liquefaction.

One of the decisive factors influencing the liquefaction susceptibility is moisture content, as liquefaction occurs only in saturated soils. At sites where groundwater levels fluctuate significantly, liquefaction hazards may also vary (12).

Initially liquefaction was considered to be limited to sandy soils. More recent field observation and studies had indicated silty soils are also highly susceptible to this phenomenon, caused when excess pore pressure generated during cyclic loading becomes equal to the effective confining pressure. (16). However, the occurrence of this phenomenon was reported for some gravels too. (3), (18). The cycling loading due to earthquakes generates a lesser strength degradation for clays than for cohesionless and slightly cohesive soils. A distinctive case is considered for brittle quick clay where high cyclic stresses could cause sufficient deformations to initiate a liquefaction type flow failure (17).

The liquefaction methods of study must take into consideration the differences between the undisturbed soil samples and reproduced samples for laboratory testing. Sing concludes there is a difference in the strain developments and the pore pressure generation for undisturbed and laboratory prepared samples of silts for testing under cyclic triaxial loading (19). Mitchell analyzes results of liquefaction behavior of some types of sand compacted at the same density using two distinctive methods: dry vibration and wet tamping. The differences in resistance to liquefaction lead to the conclusion that more than simply relative density is needed to characterize the properties of sand and the samples reconstituted in the laboratory ordinarily are not reliable for determination of properties that are representative for undisturbed sand in the field. These findings explain partially the focus on in situ tests such as Standard Penetration and Cone Penetration tests for assessments of liquefaction resistance of sands deposits. (16).

However, the limitations presented above could be addressed by special reconstitution methods that could simulate the mode of deposition of the soil deposit being modeled. Homogenity of laboratory test specimens is mandatory in order to determine the element properties. (6)

Several factors influence the liquefaction susceptibility. Particle size shape affects the volume change behavior due to cycling loading as coarse silts with nonplastic, cohesionless particles with bulky shape are fully susceptible to liquefaction, compared with finer silts with flaky or plate – like particles which exhibit sufficient cohesion to inhibit liquefaction. (17).

Terzaghi evaluated the effects of gradation on sands liquefaction susceptibility, concluding the well – graded sands are less susceptible as a more stable soil matrix is likely to form under natural depositional environment, comparing with poorly graded sands, subjected to much more liquefaction failures, as several field reports indicate. (3)

In the study conducted by Prakash et al. (19) a number of factors were considered as significant with respect to liquefaction behavior of silts with plastic fine content: density or void ratio e_o, plasticity index PI, and number of pulses of dynamic stress for triggering liquefaction, N. Using data from fifty two results of triaxial tests on reconstituted samples of silt, a statistical correlation was suggested between cyclic strength of silty soil and the factors influencing its cyclic stress ratio (CSR):

$$CSR = 0.065 – 0.234\ PI^{0.5} + 0.057\ PI + 0.34\ (e_o / N)^{-0.028} \quad [1]$$

The curves developed by this equation verify the hypothesis suggested by Prakash et al (19) that there is a threshold PI value of the clay fraction below which the liquefaction resistance is the lowest and above which the resistance to liquefaction is directly proportional with the PI values. For the limited set of data analyzed, the critical value of plasticity index is between 4 and 5 percent (19).

Somewhat different results were reported by Sandoval and Prakash for liquefaction susceptibility of a silt (96 percent passing No. 200 sieve and PI = 1.7 percent) with different amount of clay added to increase the plasticity index to 2.6 and 3.4. For these low PI values, the increase in PI lowered the cyclic stress ratio required to generate the liquefaction. (15) Tests conducted by Puri demonstrated for reconstituted silt and silt – clay mixtures an increase in liquefaction resistance for PI of 10 and 20 percent, which confirm the hypothesis suggested by Prakash et al. (12).

Different criteria were considered for assessing the soils liquefaction susceptibility. Based on sieve analysis of the soils that did or did not liquefy during a number of earthquakes, Tsuchida suggested a grain size distribution boundary curves (3). Wang proposed a set of criteria based on soil geotechnical characteristics:

1. percent finer than 0.005 mm ≤ 15 %
2. liquid limit, LL ≤ 35 %
3. natural water content ≥ 0.9 LL
4. liquidity index, $I_W \leq 0.75$ (12).

Soils susceptible to liquefaction or significant strength loss should satisfy all these criteria. Certain limitations were considered, such as the Chinese determine the liquid limit using a fall cone rather than Casagrande device, generally used in North America.

Thus, the following changes were established to the above guidelines:

- decrease the fines content by 5 percent
- increase the liquid limit by 1 percent
- increase the natural water content by 2 percent.

These criteria may be used when more specific data are not available (21).

Moisture Susceptibility

Water in soils occurs as 1) adsorption water or hygroscopic water, 2) viscous water or capillary water and 3) free water. The water molecules adsorbed by a soils particle from the air by means of surfaces forces form hygroscopic water. This water layer is influenced by factors such as the airs relative humidity, temperature and pressure. Viscous water or capillary water is not bound to mineral grains as hygroscopic water and is controlled by the soil texture and structure, organic matter and gravity. Free water is attracted to the soil particles so loosely that it may respond to the pull of gravity (22).

Water is attracted to soil particles and can develop a surface tension. In this manner, capillary menisci form between particles in a partially saturated soil mass. This curved air-water interface is a result of pore water tension, which generates an effective compressive stress between particles. Capillary stresses are usually considered responsible for an apparent and temporary cohesion in soils (17).

Moisture susceptibility is related to the notion of soil suction considered as the attraction that the soil exerts on the water (23).

The total suction of a soil consists of two components: the capillary or matric suction and osmotic suction (3). Terzaghi defines the matric suction as the difference between the pore – air and pore – water pressure. Osmotic suction is given by "the difference in concentration of the cations in the electrical double layer surrounding the particles and in the free water farther from the particles generates an osmotic suction." (3).

Moisture susceptibility represents the potential of a soil to develop or hold capillary water and produces detrimental or unstable conditions under load. Moisture susceptibility is an important factor that affects the mechanical properties of pavements and subgrade materials. The magnitude of these effects depends on the material's physical and chemical properties, moisture content and even the saturation history (24).

The Tube Suction Test (TST) was developed by the Finnish National Road Administration and the Texas Transportation Institute for evaluating the moisture susceptibility of granular base materials (24). In this test the evolution of the moisture conditions is evaluated with a dielectric probe. A graph of surface dielectric values versus time provides the basis for performance classification. The poorest-performing materials exhibit final dielectric values higher than 16, which is considered to be a threshold value. Scullion and Saarenketo suggested this value as a maximum permissible dielectric constant for granular materials, based on research study, using electrical properties to classify the strength properties of base course aggregates (25).

The water rising due to capillarity transforms the soil's relative dielectric value, which is measured by the surface probe. Adsorbed water molecules are arranged in layers around soil particles as the electrical attraction diminishes with the increasing distance from the soil particle. The water molecules beyond the electrical capture are considered unbound and depending on permeability, changes in pressure and temperature can migrate further.

The dielectric value is a measure of the unbound water within the soil sample. The strength of the material and its ability to resist repeated freeze-thaw cycling are considered to be directly influenced by the unbound water. The Tube Suction Tests reveal the state of bonding of the water within soil particles and should not be considered as a simple measure of the moisture content. (25).

The importance of soils moisture susceptibility as one of decisive factors in road design starts to be acknowledged by an increasing number of experts from geotechnical community. One of the objectives of this dissertation is to demonstrate the necessity of incorporating this concept in the highway design procedures.

Overview of road construction specifications regarding subgrades stabilization

In order to control the pumping phenomenon and to provide the workability of the road bases during construction phase and to ensure reliable results regarding the long term behavior of road courses, stabilization with cement, lime, fly ash or different combination of these additives is largely used.

The overview of some manuals for highway construction revealed the fact that the departments of transportation (DOT) of each state consider different criteria and solutions for subgrades soil stabilization. It is noticed the difficulties raised by problematic silts are better addressed in states like Louisiana where the negatives effects of these aspects may be larger than in other areas.

Louisiana DOT

In this particular case, Section 305 from Louisiana Standard Specifications for Road and Bridges, 2000 Edition, (1) contains clear criteria regarding the soils subgrade layer to be treated: content of sand less than 79 percent, content of silt less than 69 percent, Plasticity Index less than 25. Treatment of these soils should consist in mixing with 9 percent Portland cement. However, the engineer can change the percentage of additive.

The mixture of soil and Portland cement will be compacted after 72 hours curing period at a moisture content which it shall not vary from the optimum moisture content by more than ± 2 percent at the time of compaction.

Section 203.06 refers to the embankment specifications indicating criteria for usable soils such as maximum silt content of 65 percent, maximum Plasticity Index of 25 and organic content of 5 percent or less (1). Selected soils are natural soils with a maximum PI of 20, maximum Liquid Limit of 35, a maximum organic content of 5 percent, and a maximum silt content of 65 percent. Usable soils are used for construction of embankment less than 8 feet high; if the PI of these soils is between 25 and 35, lime is used for treatment (minimum 6 percent by volume). When embankments exceed 8 feet (2.4 m) in height, the soils below 8 feet (2.4 m) shall be usable soils; soils with a PI greater than 25 and less than 35 and silt content of maximum 65 percent will be permitted without lime treatment The soils above 8 feet (2.4 m) shall meet the requirements for embankments less than 8 feet high.

When soils with a PI less than 10 exists on cut slopes, the contractor shall undercut 12 inches and place a plastic soil blanket, consisting of soils with values of PI between 12 and 35, pH between 5.5 and 8.5 and a minimum organic content of 3 percent.

Embankment material shall be placed in layers and compacted to a minimum of 95 percent of maximum dry density before the next layer is placed. The moisture content at the time of compaction shall be within a range of -2.0 percent and +4.0 percent of optimum established in accordance with DOTD TR 418. (1).

Construction Experience Survey

McManis et al. (12) had developed a survey with the Louisiana Transportation Research Center as a fact finding effort and to address the experience of DOTD engineers and contractors in the construction of embankments using high – silt soils. This survey, consisting of five major sections or questions, was sent to all DOTD district offices and to the American General Contractors Association (AGCA) for solicitation of their members with experience in these types of projects (12).

Twenty-five responses to the survey were received, including all of DOTD district offices and three contractors with offices located in Alexandria, Baton Rouge, and Minden, Louisiana. The response to the questionnaire is presented as follows:

1) Current DOTD specs definition of usable soil are adequate? - 56% Agree

2) Silt content is a good indicator of a usable soil? - 76% Agree

3) Construction experience with soils >65% Silt? - 72% Had Experience

3.2) Problems Encountered soils >65% Silt ? - 56% Had Experienced Problems

3.5) Are you aware of any long-term problems for soils >65% Silt? – 52% Say No

4) Construction Problems with soils <65% Silt? - 60% Had Experienced Problems

4.5) Are you aware of any long-term problems with soils <65% Silt - 36% Say No

5) Should soil with silt > 65% Silt be allowed? - 36% Say Yes, 56% Say No

5.1) If yes, are current embank. specs. adequate? - 24% Say Yes, 36% Say No

5.2.1) Should equipment be specified for high silt soils? - 20% Yes, 52% No

5.2.2) Should vibrating compaction equip. be allowed? - 16% Yes, 44% No

5.2.3) Are current specs/moist & QC adequate for silt? - 64% Yes, 20% No

5.2.3.1) Do you have any recommendations for modifications of specifications?

- 24% Yes, 24% No, 52% No Answer

5.2.3.2) Even with proper construction, moisture infiltration in embankments will cause long-term performance problems? - 68% Yes, 20% No,

5.2.4) Could additional specifications or design improve long-term performance?

– 56% Yes, 28% No

The detailed questionnaire is presented in Appendix A.

The results of the survey revealed that only a small majority (56 percent) consider the DOT's definition of an usable soil as adequate. The silt content was considered a good indicator for identifying a usable soil by most of the respondents, as shown in question 2. However, a number of participants considered that using the percent of silt alone as an indicator is not sufficient and suggested other factors such as plasticity needed to be considered. This point of view was shared by Louisiana DOTD and addressed in the updated specifications for road constructions: usable soils for embankments should exhibit a maximum Plasticity Index of 25 and for selected soils the value should be 20, with a maximum value for Liquid Limit of 35. (1).

Question 3 established that construction projects involving soils with high silt content represent a common experience in Louisiana and 56 percent of respondents had also experienced problems using these types of soils. The problems involved were: 1) pumping during compaction efforts, 2) post-compaction problems with continuing construction activities, 3) post-construction problems during the service of the pavement, and 4) stabilization problems on some jobs.

In question 4, the question was asking if anyone had experienced any moisture, pumping problems, and/or density problems with construction projects that used soils with silt contents less than 65 percent. Construction problems were reported by 60 percent.

A slight majority of the respondents considers soils with more than 65 percent silt should not be used in construction projects. When asked what modifications to the specifications should be made to assure proper construction, the suggestions made referred to the use of filter fabrics, a strict control of moisture content, new range of values for PI, etc. The DOTD specification's allowance for four percent moisture content above optimum moisture without limitations of soil types was cited as an example of where specifications applied broadly do not always solve but may create problems.

Moisture infiltration into a properly constructed embankment of a soil with high-silt content is considered as a cause for long-term performance problems by 68 percent of the participants. Suggestions for possible control of this situation included chemical stabilization and drainage systems. (12) Those opinions reinforce the importance of moisture susceptibility of road courses.

Alabama DOT

The Standard Specifications for Highway Construction, 2002 Edition, issued by Alabama Department of Transportation (26), does not contain criteria as rigorous as Louisiana's DOT uses with respect to characteristics of subgrades soil. For example, Section 210.02 specifies "Materials furnished for the improved roadbed shall be any stable material meeting the requirements of Soil Classification A-1, A-2, A-3, or A-4, as determined by AASHTO M 145" The only limitations are :"Materials in the A-2 or A-4 Classification shall have a CBR value of not less than 10". Another material used for roadbed is industrial waste, defined as "a residue from manufacturing operation", taken from "cold" piles and approved by the Materials and Test Engineer. The required characteristics of this material are: unit weight not less than 100 pounds per cubic foot, maximum values of 25 for Liquid Limit, and 6 for Plasticity Index. Minimum values for CBR should be minimum 12.

The stabilization of a roadbed is described in Section 231. The only reference to stabilizing material states those agents "shall be an approved local or commercial material of the type provided by the plans or proposal." Section 232 presents details regarding Lime Stabilized Roadbed. Quantities and percentages of lime are determined based on preliminary soil investigation and dry laboratory sample tests. The mixtures are allowed to mellow for a minimum period of 3 days at a moisture content that shall not vary more than ± 2 percent from the specified optimum moisture content. In place density requirements are 100 percent of maximum dry density determined by Standard proctor Compaction test (according to AASHTO T 99). The Specifications indicate "Moisture content at the time of the in-place density test shall be to the satisfaction of the Engineer, but in no case will be more than ± 5% of the established optimum moisture, unless authorized in writing by the Materials and Tests Engineer." In order to protect the compacted roadbeds layers, no traffic will be permitted for 7 days on the finished structures.

The suitable materials for roads embankments are selected by the projects engineer. Strict moisture control will not be required for the compaction process, the only mention being “it will be the Contractor's responsibility to maintain the moisture content necessary to satisfactorily compact the material. Compaction in a semi-dry condition will not be permitted.” It is noticed there are no specific indications regarding subgrades stabilization with Portland cement or lime + fly ash. (26).

Mississippi DOT

The Standard Specification for Highway Construction, 2003 Edition, issued by Mississippi DOT (27), does not mention specific criteria regarding the characteristics of subgrades soil. Lime stabilization is covered in Section 307. Incorporation of additive is conducted by dry application or slurry application, for the last method the proportion of lime is in such a way that the “Dry Solids Content” shall be at least 30 percent by weight. Those two methods are common procedures used in other states, with different approaches regarding lime quantities. The mellowing period of the sealed mixture is between 5 and 20 days, slight different from previous DOT’s specifications.

Determination of acceptance of mixtures compaction will be performed on 2500 feet / layer lots divided into five equal sublots with one density test taken at random in each sublot. The average of the five density tests shall not be less than 93 percent with no single density test below 89 percent. Curing period of the sealed subgrades layers can be extended up to 21 days, upon contractor’s decision.

A similar protocol is described in Mississippi DOT specification (27) for lime + fly ash with specific indications regarding construction requirements: in a first step fly ash shall be spread at a specified rate and lightly disked or mixed into the soil prior to spreading the lime. This last additive shall be incorporated into the course of soil and fly ash. At the completion of moist mixing and during the compaction operations the moisture content shall be that necessary to obtain the required density. Determination of acceptance of mixtures compaction is the same as described for lime stabilization. After the lime-fly ash course has been finished, the surface shall be covered with a bituminous material for protection and curing. If the finished course is to be covered with the subsequent paving course within 48 hours, the curing seal could be omitted. It is noticed the curing period is much shorter than the 7 to 21 days interval for lime stabilization.

Section 308 covering Portland cement treated courses indicates the percentage of additive is determined on preliminary soil investigation and Department's tests. The tolerance for cement quantity spread on a section is maximum 5 percent of the quantity ordered. Compaction shall be conducted immediately after mixing in order to achieve the mixtures maximum density. The acceptance of this parameter is based upon same procedure as for lime stabilized courses. The moisture content at the time of compaction shall be situated between optimum and +2 percent of this value. The stabilized material will be covered with a bituminous curing seal and protected for at least 7 days (27).

Virginia DOT

Virginia DOT presents in Section 306 (28) the same procedures for subgrades lime (and lime + fly ash) stabilization with some differences. The mixture shall be compacted at a density of 95 percent of mixtures maximum density and at a moisture content situated between mixtures optimum and + 20 percent of optimum. Curing period for the stabilized subgrade is only 7 days, compared to the 21 days maximum period considered by Mississippi DOT. (25).

Section 307 presents the specifications regarding stabilization of roadbed material, being similar with the requirements considered by other states DOT. A particular detail is presented in subsection 307.05.2 (a). For mixing aggregates subbase and base materials using Central plant method, the determination of cement content will be made by titration method. (28).

Texas DOT

The Standard Specifications for Highway Construction, 1993 Edition (29), issued by Texas Department of Transportation considers the same procedures as presented above for subgrades stabilization with lime, lime + fly ash or Portland cement. Important differences are noticed for the last additive (Item 275), where the percentage of cement is determined based on mixtures compressive strength tests. Minimum Design Compressive Strength of 500 psi and 750 psi should be ensured by an allowable cement content of 3-9 percent respectively 4-9 percent. The same approach regarding cement percentage is considered by Louisiana DOT, however, the specifications refer only to cement percentage, without any correlation with mixtures strength values.

As it was mentioned before, specifications for road construction are standardized for all the states; though silts are not considered as a distinct case except by Louisiana DOT as noticed in the above overview of subgrade stabilization procedures. Also, some differences are noted regarding especially control of moisture content, mellowing and curing period, with a significant influence on subgrades strength. A special consideration to this parameter is given by Texas and Louisiana DOT as the percentage of cement for subgrades stabilization is selected based on strength values of soils mixture (29).

Classical Methods for Subgrade Stabilization

Subgrade stabilization was extensively studied as road construction was determined to adapt and to face the continuous challenges imposed by the development of terrestrial transportation in the last century. As one of the symbols of technology progress, the explosive number of new vehicles had requested better roads constructed. Different solutions were considered for improving the characteristics of road foundation. Mixing lime, Portland cement fly ash or lime + fly ash and cement + fly ash with soil subgrade became the most used stabilization methods.

Lime Stabilization

Lime stabilization can improve significantly the subgrades engineering properties in two forms: modification and stabilization. Modification can occur in some extent with almost fine grained-soils, but the most substantial improvement is present in clay soils of moderate to high plasticity. This process occurs due to exchange of calcium cations supplied by the lime ($Ca(OH)_2$) with the present cation adsorbed on the surface of the clay mineral. Hydrated lime reacts with the clay mineral surface in the high pH environment created by the lime – water system, generating cementitious products. (30).

Stabilization occurs when the proper amount of lime is mixed with the soil. In this situation a significant level of strength gain is developed through a long – term pozzolanic reaction. Calcium from the lime reacts with aluminates and silicates solubilized from the clay mineral surface, leading to the formation of calcium silicate hydrates and calcium aluminate hydrates. Research has shown that the pozzolanic reaction can continue for a long period of time, even years, as long as the pH remains high (above 10). As a result of this reaction, soils treated with lime exhibit substantial strength gain, improvements in shear strength and long – term durability. Field demonstrations indicate that the improved soil properties are maintained over 20 to 40 years (30), (31).

The mechanisms of lime – clay reactions are presented furthermore by Boardman et al. The physicochemical properties of soils are transformed by the lime addition. When this stabilizer is added to a soil-water system in the form of CaO, the exothermic formation of $Ca(OH)_2$ improves the soils workability cause of dewatering. The modification reactions are generated by the exchange between ions of Ca^{2+} and hydroxide ions $(OH)^{-}$ and negative charges on the clay mineral lattice. This process is complex and is influenced by the location of the charge on the mineral structure, the ionic potential and the relative ion concentration (i.e. the concentration of the replacing ion in relation to the concentration of ions at the exchange sites) (32).

Based on these considerations, Dallas Little (33) established a set of guidelines for lime stabilized subgrades. The first step is to determine the percentage of lime to be mixed with the soil in order to satisfy initial soil – lime reactions and still provide enough residual lime to drive pozzolanic reactions. The appropriate amount of additive will induce a pH of at least 10 for the mixture.

Once the percentage of lime is determined, specimens of the mixture are compacted at optimum moisture content to maximum dry density and cured for 48 hours at 49^{0} C. Unconfined Compression tests are conducted and strength is compared for natural soil and soils + lime. If the gain in strength is more than 350 KPa, the mixture is suitable for structural layer application.

Prepare another set of specimens with different lime percentages, compacted at the same conditions as mentioned above and conduct Unconfined Compression tests. From the analysis of strength results determine the percentage of lime above which further increase do not produce significant additional strength. Design Field Lime percentage, adding 0.5 to 1 percent to account for construction losses, uneven distribution, etc. (33)

Portland cement Stabilization

ASTM C 150 defines Portland cement as "hydraulic cement (cement that not only hardens by reacting with water but also forms a water-resistant product) produced by pulverizing clinkers consisting essentially of hydraulic calcium silicates, usually containing one or more of the forms of calcium sulfate as an inter ground addition." (34)

The efficiency of Portland cement in soil stabilization is given by the new products resulted from the reaction of cement and soil-water system such as: cementing compounds of calcium-silicate-hydrate, calcium-aluminate-hydrate and excess calcium hydroxide. This lime generated from the cement hydration will further interact with soil alumina and soil silica within the fine grained soils, leading to the material stabilization (33). If enough residual lime is generated, long term pozzolanic reaction will occur, with continuous gain in strength and long – term durability.

Many state department of transportation (DOT) consider strength criteria for mix design specifications, based especially on many years of experience with soil – cement (35).

Long term performance of the stabilized element is influenced primarily by the soil – cement shrinkage which generates cracks that can reflect through the bituminous pavement. The techniques used to minimize this problem include reduced cement content, delayed placement of surface hot mix, reduced cement content and use of interlayers to absorb crack energy and prevent further propagation. Based on these considerations, the strength criteria for bases stabilized with Portland cement are reconsidered. Already most districts are not requiring such high compressive strength values (minimum 5170 KPa or 3450 KPa) (29). A common value now used is 1380 KPa (200 lb/in^2) or others specify a threefold increase over the raw material strength (36). Fundamental research is still needed in order to better understand cementitious reactions and their short and long term effects on the stabilization process. (35)

Fly Ash Stabilization

Fly ash results from the combustion of bituminous, anthracite and some lignite coals; it is pozzolanic but not self-cementing. In order to generate cementitious products, an activator like Portland cement or lime must be added (35)

Mixtures of lime and fly ash represent a good stabilizer, providing reliable road base with adequate strength and durability. Reports indicate quality mixtures were obtained with lime content between 2 and 8 percent by weight and fly ash content between 8 and 15 percent. Typical proportions are from 3 to 4 percent lime and 10 to 15 percent fly ash. In order to accelerate the initial gain in strength, a quantity of 0.5 to 1 percent Portland cement can be added. (35)

Pozzolanic reactions within the soil-lime-fly ash mixture are highly influenced by curing time, moisture content and temperature. Studies of variation of strength with time for mixtures of soil and Class C fly ash indicate early self-hardening characteristics of additive. In order to obtain maximum stabilizing effects of fly ash, the soil should be quickly mixed and compacted. (37). Dallas Little indicates a maximum compaction time delay of 1 hour for stabilization of pavement bases or subbases when maximum potential strength is targeted. (35).

Maximum strength is achieved at moisture content below that corresponding to maximum dry density. Studies revealed the strength of stabilized material is reduced by 50 percent or more for samples compacted at moisture content that exceed optimum (35).

McManis and Arman (37) evaluated the possibilities of replacing the Portland cement or lime with Class C fly ash. Tests results and analysis demonstrated this additive could be used alone or as a partial replacement of cement in sands, based on the dual role of fly ash as a matrix filler and cementing agent. In the case of silty sand or sandy silt, the improving effects of this stabilizer are dependent on the quantity and reactive properties of the fine grained material, that is the matrix quality. The findings of the study indicate the superiority of lime over fly ash in the treatment of clays (37).

Alternative stabilization methods

The progress of technology opens new chapters in engineered materials as competitive solutions to the transportation problems. Classic stabilizing agents can be replaced by innovative products like geosynthetics, chemical systems or recycled and waste products.

Geogrid reinforcements

Geogrid is "a geosynthetics material consisting of connected parallel sets of tensile ribs with apertures of sufficient size to allow strike-through of surrounding soil, stone or other geotechnical material." (38).

The beneficial mechanisms attributed to inclusion of the geogrid to a soft and/or compressible foundation soils are:

- An increase in soils subgrade strength
- An improved load distribution to the soil subgrade due to load spreading
- A tensioned membrane effect. (38)

The degree of strengthening using geogrids is influenced by vertical and horizontal spacing, the number of composite layers and the depth above the first layer of the footings. (38)

However, the extent of using the geogrid to reinforce the pavement base course and to enhance the performances over a soft subgrade road is still limited, as a better definition of mechanistic design procedures, life costs, and material properties is needed for these products (38).

Industrial products

Innovative products are emerging as the continuous progress of science moves further the technology boundaries.

Soil Stabilization Products Company, Inc. (39) provides new product technologies with successful applications in soil stabilization, channel protection, sediment control, slope protection, etc. A four step review conducted by Texas Transportation Institute demonstrates the product marketed under the label "EMC Squared" can be successfully used for bases stabilization.

The first step of this review consisted in preliminary investigation of laboratory and field service history, followed by the two year laboratory study at the Texas Transportation Institute. The main focus at this stage was placed on comparing the effectiveness of lime treatment to the EMC SQUARED System in reducing the moisture susceptibility of a highly expansive clay soil.

For the third step of the study, a field test pad was considered. An EMC SQUARED System treatment was used for a two acres expansive clay soil, which was later compacted to create an eight inch thick working platform. This platform supported extensive use by heavy equipment (cement trucks, large front end loaders, etc.). The treatment used proved to be highly effective and cost efficient when compared to cement or lime treatment.

With success in the materials testing laboratory and the field test pad, the fourth and last step consisted in using EMC SQUARED System treatment for some major project subgrade stabilization applications.

Tests results showed this product can be effective in increasing the strength, stiffness and swell resistance and can be the appropriate solution for subgrade treatment for sulfate bearing expansive clay soils (39).

This concentrated liquid stabilizer (CLS) demonstrates further advantages by reducing the courses moisture susceptibility Specimens treated with EMC Squared and subjected to Tube Suction Test exhibit final dielectric values less than 16 and much lower than corresponding values for raw soil and soil mixed with lime.

Stabilization with silica fume or microsilica

U.S. Department of Transportation defines microsilica as “a byproduct of the reduction of high-purity quartz with coal in electric furnaces in the production of silicon and ferrosilicon alloys”. This product consists of very fine vitreous particles, 100 times smaller than the average cement particle. The extreme fineness and high silica content make silica fume highly effective pozzolanic material, used mainly as additive in concrete in order to improve its properties (40).

Bagherpour and Choobbasti (41) studied the effects of adding microsilica in lime or cement for fine grained soils stabilization. A secondary effect of lime or Portland cement addition to sulfate bearing soils is the formation of deleterious products such as Ettringite (Hydrated Calcium Aluminum Sulfate Hydroxide). The addition of microsilica leads to the formation of calcium aluminate hydrates in the resulting pozzolanic reaction (41).

The geotechnical aspects of a silty clay stabilized with lime and cement mixed microsilica were considered. The comparative tests results demonstrate silica fume increases the Plasticity Index for cement mixtures only, but the lime mixtures exhibit a more significant increase in strength. Also, by adding microsilica, the direct shear strength is increased for both lime and cement mixtures. (42).

METHODOLOGY

The method of investigation was focused on four main objectives:

1. Identification of silts with high pumping potential;
2. Chemical stabilization / modification of these problematic silts;
3. Study of silts moisture susceptibility;
4. Recommendations for a set of guidelines in order to improve the actual specifications for roads subgrade constructions.

The initial stage of this study involved the selection and acquisition of samples, and a testing program to investigate the character of the silt materials and the nature of the pumping problem. The major emphasis was placed on efforts to refine a description of this problem and the development of a method for identifying the problematic silts.

1.1) Soil Samples

Soil samples from four of the DOTD districts were used in the final and extended laboratory program. These included the Lafayette District 03, the Lake Charles District 07, the Alexandria District 08, and the Chase District 58. The samples were acquired from different project sites or areas in which pumping problems were occurring, being typical of those commonly encountered with high-silt content. At least two of the eight samples provided were noted as not pumping and were provided as a comparison with the pumping silts.

Approximately six sampling bags (8 pounds each) were provided for each soil sample. The sample received from the Lafayette District 03 was designated as the Acadia (Sta. 109+00) sample. Two samples were received from the Lake Charles District 07 and noted as DeRidder White and DeRidder Brown by virtue of their color shades. The Alexandria District 08 provided three samples noted as Natchitoches K1-1 (Sta. 125+08), K2-1 (Sta. 149+75), and K3-1 (Sta. 161+70). The Chase District 58 submitted two samples noted as Chase White (Sta. 295+00) and Chase Brown (Sta. 408+00), again by virtue of color shades. The sets of samples from the four districts included soils that had been observed to be pumping under compaction efforts in the field. Two of the soil samples, one from the Alexandria group (K2-1) and one from the Chase group (Chase Brown) were noted as not pumping during compaction in the field. Pumping or non-pumping was not identified with the two soils from the DeRidder site. To this number of eight samples, another soil was added later and noted as HW 171 (from Lake Charles District 07), identified as pumping during construction activities.

1.2) Testing Program

1.2.1) Classifications Tests

In order to determine the characteristics of the soil samples, an extended range of classification tests were considered. Gradation curves were determined (ASTM D422) and Atterberg tests (DOTD TR428) were conducted on all samples. Multiple tests were performed on most with representative values identified as being characteristic. The soils were classified according to the AASHTO (DOTD TR 423 and ASTM D3282) and the Unified Soil Classification System (ASTM D2487). The specific gravity of the samples was determined according to ASTM D854 (AASHTO T100).

1.2.2) Compaction Tests

The Proctor compaction method (ASTM D698) was used in order to establish compaction curves, optimum moisture content (OMC), and maximum dry density of the soils. Different compaction energies were also used to develop a family of compaction curves for all silts. This included the modified effort (AASHTO T180 and ASTM 1557, 56,000 ft-lbf/ft^3 or 2,700 kN-m/m^3), a modified plus effort (78,750 ft-lbf/ft^3 or 3,750 kN-m/m^3), and a reduced standard effort (7,425 ft-lbf/ft^3 or 350 kN-m/m^3). Table 1 illustrates the elements of the different compaction efforts.

Table 1.

Elements of compaction

	Reduced Compaction	**Standard Proctor Compaction**	**Modified Compaction**	**Modified + Compaction**
Hammer Wt. (lb)	5.5	5.5	10	10
Drop Height (in.)	12	12	18	18
Number of Blows	15	25	25	35
Number of Layers	3	3	5	5

Test specimens of the modified soil with various chemical agents were prepared using the Harvard Miniature Compaction Apparatus (described in ASTM D4609). The apparatus consists of a mold 1.3125 in. in diameter and 2.816 in. long with a volume of 1/454 cu. ft. and a spring-loaded plunger. Harvard compaction utilizes a kneading action in molding the specimens. The specimens were molded at different moisture contents and compaction efforts by varying the number of tamps per layer with the spring tamping plunger. The Harvard Compaction Miniature Apparatus is presented in illustration 1.

Illustration 1

Harvard miniature apparatus

The Harvard Miniature Compaction Apparatus represents an ideal means to create soil samples for moisture density studies or strength tests as Unconfined Compression or triaxial tests. The compaction method closely duplicates the kneading action of a sheepsfoot roller. For a given moisture content, the weight in grams of the molded specimen is numerically equal to the wet unit weight of the compacted soil in lb/ft^3 as described in Section A 2.2.6, ASTM D 4609 (43).

Consider K1-1 as a typical example, compacted at optimum moisture content (14.5%), using 15 tamps. The resulting specimen weight is M=126 grams (0.275 lb.) and 1.31 in. diameter (D) by 2.86 in. length (H). Its density is:

$$\gamma_{moist} = M / Volume = 0.277 / (\pi \times 1.3^2 \times 2.85/4) = 0.073 \text{ lb/in}^3 = 126.4 \text{ lb/ft}^3$$

The moist unit weight of the soil compacted at OMC using Standard Proctor energy is 126 lb/ft^3. So the state of the compacted specimen using Harvard Miniature Apparatus can be considered similar to the state of the soil compacted at the same moisture content using Standard Proctor Compaction energy.

1.2.3.) *Mineralogy Study*

A mineralogy study was conducted for the natural soils by the University of New Orleans' Geology laboratory in order to determine the mineral presents and their distribution. Also, the soil mixtures with the chemical additives were investigated to identify any cementitious or other products that may have formed.

Two methods were used in analyzing the mineralogy of the natural soil and the soil with chemical additives. The first employed an AMRAY 1820 digital scanning electron microscope in digitizing the images of the soil particles and to collect the energy dispersive spectra. A software page "Iridium," (IXRF Systems, Inc.) was used. In the second method, pulverized samples were scanned with a SINTAG XDS 2000 X-ray diffractometer. Scans were plotted and compared with ICDD patterns of the common minerals as well as potential reaction products.

1.2.4.) Undrained Strength Tests

An extended set of unconfined compression tests was conducted for the natural soils, compacted at various moisture contents and using the Harvard Miniature Compaction Apparatus. Different compaction efforts were used, including one similar to the Standard Proctor compaction energy. In this manner, compaction curves for each soil were produced.

By trial and error, the Harvard compaction effort required to produce a γ_{dry} equal to the Proctor $\gamma_{d\ max}$ and at a moisture content equal to Proctor optimum moisture content was determined. This Harvard compaction effort was used to produce test specimens at a dry density and moisture content equal to the Proctor $\gamma_{d\ max}$ and respectively optimum moisture content.

The tests permitted an analysis for strength – moisture relationship. As expected, the soil's strength generally decreased at moisture levels that exceeded the optimum value for compaction. The extent of the strength loss corresponding to increased moisture content for the different samples demonstrates the increase in potential for the different samples to pump or not pump, i.e. low strength and greater instability. This is a “static” load test, however.

1.3) Evaluation of pumping phenomenon

Soils with high silt content considered for road subgrade located in areas with high moisture content are susceptible for pumping during construction due to the compaction effort or equipment traffic. The water is subjected to an uplifting effort which causes a modification of the moisture content within soil matrix with negative effects on subgrades strength and dry unit weight.

The pumping phenomenon is examined in the laboratory by creating the conditions from the construction site which lead to this negative situation. A study conducted by Soil Testing Engineers in Baton Rouge considered a cyclic triaxial testing program using soil samples prepared with the Harvard Compaction Miniature Apparatus, compacted with a Standard Proctor compaction effort and at moisture contents 2 to 4 percent higher than the optimum moisture content. The samples were placed into the triaxial chamber in the “as-molded” condition. The specimens were not saturated but the drains valves were open so that the samples could absorb water if they developed negative pore water pressure. A lateral pressure of 2 psi was applied and a set of 300 cycles of loading unloading was initiated. One cycle of load is the alternation between a maximum load of 600 psf, followed by a release of stress up to 100 psf. These values were considered in an effort to duplicate an on-and-off 18 kip wheel load at the subgrade depth. The sinusoidal load was applied at a frequency within the range of 0.1 to 2 Hz.

(ASTM D 3999). The resulting load – axial strain history demonstrates the stability of the soils under cyclic loads.

2) Chemical Stabilization

The LDOTD has allowed the contractor to dry the subgrade soil with lime to prevent the pumping during construction. An evaluation of the lime and other chemical additives potential to modify the pumping was conducted.

The study reviewed the influence of three chemical additives considered on some soil's characteristics like strength and moisture content, especially in the construction phase. Chemical stabilization or modification of the problematic silts is a technique used to construct a working table, prevent pumping and to achieve the relative compaction requirements for the subgrade. The effort attempts to dry the wet silts, allowing them to be compacted, providing a stabilized support platform to construct the pavement.

2.1) Test Series 1. General Stabilization Character

The influence of lime, lime plus fly ash, and Portland cement on the compaction character, and strength of the eight different soils were investigated. The percentages of additive by dry weight selected for the compaction and strength tests with molded specimens from the eight samples were arbitrarily selected as follows: 1) four-percent lime-soil mixtures, 2) two-percent lime plus eight-percent fly ash-soil mixtures, and 3) four-percent Portland cement-soils, as presented in Table 2 and Appendix.

The specimens were molded at different moisture contents using the same compaction effort with Harvard compaction apparatus. In order to generate the compaction curves for the first series of tests, four sets of specimens were molded for each sample type at different moisture contents with the 4-percent lime, alone, and the 2-percent lime plus 8-percent fly ash mixtures.

One of the four molded lime and lime-fly ash specimens produced in a sample set was tested immediately for its unconfined compressive strength, moisture content and unit weight (“direct” or “as molded”). The three remaining sets of specimens were set aside and cured under different conditions. Two were cured under accelerated conditions for 24 hours at 50^0 C (rapid cure or RC). The last sample set was cured in a 100-percent humidity room (HR) with ambient temperatures for a period of two weeks. At the end of the curing periods the specimen-strength was determined in the unconfined compression strength test. One of the set of specimens from the accelerated cure group was subjected to a vacuum saturation period of one hour prior to determining its strength.

Three specimens of the Portland cement-soil mixture for each of the A-4 soils were molded at a moisture content approximately equal to the Proctor optimum moisture content plus 6-percent. One specimen was tested immediately (“direct” or “as molded”). The other two were cured in a humidity room at ambient temperatures for 14-days before testing. One of the specimens with the two-week cure was subjected to one hour of vacuum saturation before being tested in unconfined compression, for the analysis of the long-term stability and durability of the stabilization efforts of road subgrades subjected to flooding as worst case scenario leading to saturation.

Table 2 illustrates the summary of the testing program described above.

Table 2

Testing Program for Chemical Stabilization Series 1

	Additives										
	Lime 4%				Lime 2% + Fly Ash 8%				Portland Cement 4%		
	Direct	Rapid Curing (RC)	Humidity Room (HR)	Vacuum Saturation	Direct	Rapid Curing (RC)	Humidity Room (HR)	Vacuum Saturation	Direct	Humidity Room (HR)	Vacuum Saturation
Soil	As molded	24 hours @ 50^0C	2 weeks	2 weeks	As molded	24 hours @ 50^0C	2 weeks	2 weeks	As molded	2 weeks	2 weeks
	4 samples	4 samples	4 samples	4 samples	4 samples	4 samples	4 samples	4 samples	1 sample	1 sample	1 sample

The procedure described in this section and the moisture contents of the specimens tested are presented in table 3 for the representative case of Acadia. The rest of the results are illustrated in Appendix D.

Table 3

Testing methodology and corresponding moisture contents for Acadia

Soil	Moisture %
ACADIA	9.55
	11.76
	15.42
	16.52
ACADIA + Lime4% DIRECT	10.07
	13.18
	14.21
	17.03
ACADIA + Lime4% RAPID CURING (RC)	8.18
	11.68
	13.26
	15.8
ACADIA + Lime4% VACUUM SATURATION (VS)	10.62
	13.84
	15.04
	17.7
ACADIA + Lime4% HUMIDITY ROOM (HR)	10.62
	13.84
	15.04
	17.7
Acadia+ 4% Cement	17.12
Acadia + 4% Cement HR	17.1
Acadia +4% Cement VS	17.1

Table 3. Continued

Soil	Moisture %
ACADIA	9.55
	11.76
	15.42
	16.52
ACADIA + Lime2%+FA 8% DIRECT	9.64
	12.92
	14.41
	17.92
ACADIA + Lime2%+FA 8% RAPID CURING (RC)	8.57
	11.86
	13.5
	16.75
ACADIA + Lime2%+FA 8% VACUUM SATURATION (VS)	10.41
	13.68
	15.02
	18.88
ACADIA + Lime2%+FA 8% HUMIDITY ROOM	10.41
	13.68
	15.02
	18.88
Acadia+ 4% Cement	17.12
Acadia + 4% Cement HR	17.1
Acadia +4% Cement VS	17.1

2.2) Test Series 2. Modification/Stabilization of Wet Silts

This series of tests for unconfined compression strength used four soils: Chase White (A-4/ML) and Brown (A-6/CL), and the DeRidder White (A-4/ML) and Brown (A-6/CL) as an attempt to simulate a situation where a wet, high-silt soil is encountered.

Using the soil in a state significantly wet of optimum, four set of three samples each were prepared for each soil mixed with Portland cement, lime, fly ash and lime + fly ash. The corresponding percentages of additives are presented in Table 4. The initial moisture content of the soils exceeded their optimum moisture contents by several percentage points. An additional set of specimens were molded using natural soils wet of optimum for comparative purposes.

Table 4.

Additives Percentages

	Natural Soil	Portland Cement	Class C Fly Ash	Lime	Lime + Fly Ash
Chase Brown	-	10%	10%	6%	3%+10%
Chase White	-	8%	10%	6%	3%+10%
DeRidder Brown	-	10%	10%	6%	3%+10%
DeRidder White	-	8%	10%	6%	3%+10%

All the specimens were molded wet of optimum with the Harvard compaction equipment approximating the standard compaction effort.

One of the natural soil specimens and one of the specimens molded with the different admixtures were tested for unconfined compression strength “as molded” or a “direct” test. This was done as a measure of the potential for modifying and/or drying the soil. The other two specimens from each set were allowed to cure. The lime and lime-fly ash specimens were cured under accelerated conditions (50^0 C for 3 –days). A longer rapid curing period was used in the second test series to allow more time for the development of cementitious products. The Portland cement and fly ash (alone) specimens were cured in a humidity room for 2-weeks under ambient conditions. One of the two remaining specimens was tested in unconfined compression at the end of the curing period. The other was subjected to vacuum saturation and then tested for unconfined compression strength, to determine the behavior of stabilized soils in conditions of saturation during the long term road service.

Table 5 illustrates the methodology and the corresponding moisture values for this section.

Table 5

Testing methodology and corresponding moisture contents for Phase 2.

Soil	Moisture %
CHASE BROWN	24.86
SOIL + 10% PC: Direct	20.57
Cured (in HR for 2 weeks)	17.94
VS	17.94
SOIL+Lime 6%:Direct	22.66
Rapid Curing (RC)	21.1
VS	21.1
SOIL + FA10%: Direct	20.58
Cured (in HR for 2 weeks)	20.29
VS	20.29
SOIL +L3% FA10%: Direct	20.79
RC	19.3
VS	19.3
CHASE WHITE	19.67
SOIL + PC 8%: Direct	17.17
Cured (in HR for 2 weeks)	17.59
VS	17.59
SOIL+Lime 6%:Direct	17.5
RC	13.72
VS	13
SOIL + FA10%: Direct	17.13
Cured (in HR for 2 weeks)	16.58
VS	16.58
SOIL +L 3 % FA 10 %: Direct	16.34
RC	11.34
VS	11.88

Table 5 continued

Soil	Moisture %
DERIDDER Brown	19.91
SOIL + PC 10 %: Direct	17.26
Cured (in HR for 2 weeks)	17.4
VS	17.4
SOIL+Lime 6%:Direct	18.06
Rapid Curing (RC)	12.71
VS	12.71
SOIL + FA10%: Direct	16.97
Cured (in HR for 2 weeks)	16.37
VS	16.37
SOIL +L3% FA10%: Direct	16.31
RC	12.07
VS	12.07
DERIDDER WHITE	19.42
SOIL + PC (8% by weight): Direct	16.36
Cured (in HR for 2 weeks)	16.28
VS	16.28
SOIL+Lime 6%:Direct	17.07
Rapid Curing (RC)	14.1
VS	14.1
SOIL + FA10%: Direct	16.24
Cured (in HR for 2 weeks)	17.1
VS	NA, disintegrated
SOIL +L3% FA10%: Direct	15.32
RC	12.43
VS	12.43

3) Evaluation of soils moisture susceptibility

The soils moisture susceptibility represents its tendency to attract sufficient quantities of water to cause changes in its physical properties (44). Silts exhibiting high moisture susceptibility are subjected to an increased influence of water rising due to capillarity. This fact represents one of the complementary factors contributing to the silts pumping phenomenon.

As previously presented, the moisture content plays an essential role in producing pumping conditions (12), (13), (14). Excess moisture and cyclic loads induced by traffic produce pumping, with negative effects on mechanical properties of the subgrades.

Even if the level of moisture is controlled during construction phase by adding drying agents, future variation of the moisture content due to seasonal precipitations and complete saturation by exceptionally high volumes of rainfall can occur.

The introduction of moisture due to capillary action represents an acute problem for silts and increases further redistribution of water within the soil matrix. This is characteristic of the soil's moisture susceptibility and is an important consideration for this research.

The Tube Suction Test (TST) was developed by the Finnish National Road Administration and the Texas Transportation Institute for evaluating the moisture susceptibility of granular base materials (24), (25). In this test the evolution of the moisture conditions is evaluated with a dielectric probe. A graph of surface dielectric values versus time provides the basis for performance classification. The poorest-performing materials exhibit final dielectric values higher than 16, which is considered to be a threshold value. (22), (23).

This value was initially determined from empirical observations. Final dielectric values were measured for soils with acceptable behavior as subgrade materials. Later, these empirical determinations were corroborated by laboratory testing (durability test, CBR test).

The water rising due to capillarity transforms the soil's relative dielectric value, which is measured by the probe. Adsorbed water molecules are arranged in layers around soil particles. The water molecules are distributed and bonded towards the negatively charged mineral surface. The electrical attraction diminishes with the increasing distance from the soil particle. The water molecules beyond the electrical capture are considered unbound and depending on permeability can migrate further (23).

3.1) Description of Tube Suction Test

The soils samples are compacted within a standard concrete cylinder mold (305 x 152 mm) at Optimum Moisture Content and using compaction energy corresponding to maximum dry unit weight, Standard Compaction Test.

The energy of compaction is given by:

$$\frac{\text{(No layers) x (No blows / layer) x (Weight of hammer) x (Height of blow)}}{\text{Volume of the mold}} = \text{12375 ft lb/cu.ft. [2]}$$

The height of the sample is 180 cm and is compacted in 7 layers at 35 blows /layer, using the same compaction energy as that of Standard Proctor. The bottom of the tube was cut and replaced with aluminum foil pierced with a nail of 1.5 mm, on 3 concentric circles, distance between holes approximate 3 cm. The weight of each sample was recorded then placed in the oven at 50^0 until no more significant changes are observed in their weight. After oven drying, the samples are allowed to cool down at room temperature. When their temperature has stabilized, the samples are placed in a dish containing approx. 20 mm of deionised water. The first measurements of the dielectric and electrical conductivity values are taken before placing the tube samples into the water. Once in the water, four measurements are taken. The highest and the lowest reading are disregarded. From the second day on, only one measurement is required per day, until the weight of the samples and the dielectric values become constant. The weight of the tube samples is measured in connection with every dielectric value measurement.

3.2) TST Testing Program

The Percometer is an instrument for dielectric and specific conductivity measurements of soils. The dielectric value of a material is the ratio of its dielectric permittivity to the dielectric permittivity of free space ($8.85 * 10^{-12}$ F/m) and is indicative of its storage capacity in an electrical field.

Consider the soil specimen as a condenser with plane plates. Initially capacitance is:

$$C_0 = \frac{\varepsilon_0 S}{d} \qquad [3]$$

where ε_0 is dielectric value for air, S is the area of the condenser plates and d is distance between plates.

At a specific moment t as the water rose within the soil sample, and the capacitance of the model considered is now:

$$C_1 = \frac{\varepsilon_{soil} S}{d} \qquad [4]$$

where $\varepsilon_{soil} = \varepsilon_0 \text{ x } \varepsilon_r$ is dielectric value for soil, ε_r is relative dielectric value of the soil.

The Percometer probe determines the dielectric value from the change in capacitance measured by the probe due to contact with the tested material according to the following equation:

$$\Delta C = C_a (\varepsilon_r - 1) \qquad [5]$$

where C_a = active capacitance of the probe (in our case initially capacitance , the capacitance of the probe in air)

ε_r is dielectric value of the tested material

Indeed, in our case:

$$\Delta C = C_1 - C_0 = \frac{\varepsilon_{soil} S}{d} - \frac{\varepsilon_0 S}{d} = \frac{\varepsilon_0 \text{ x } \varepsilon_r S}{d} - \frac{\varepsilon_0 S}{d} = \frac{\varepsilon_0 S}{d} (\varepsilon_r - 1) = C_a (\varepsilon_r - 1) \qquad [6]$$

For all the TST described as follows, the soils were compacted at optimum moisture content corresponding to standard Proctor compaction energy and to a 180 mm. height of sample. The first set of experiments was focused on measuring the moisture susceptibility of raw soils. Using the procedure described above, the final dielectric values were determined. The research was extended in order to study the effect of different factors on final dielectric values, such as tubes diameter, compaction energy and the effectiveness of the different chemical additives used for silts stabilization. Due to the limitation of available material quantities, the only silts used were: HW 171, Acadia, Deridder White and K1-1.

The effects of the tube size on the final dielectric values were considered for the next stage of the study. A set of TST was conducted for soil samples compacted at the Proctor optimum moisture content and using the same 180 mm height in smaller tubes, 101.6 mm in diameter. The results were compared with those provided from tests using “classic” tubes. Illustration 2 presents the set up for TST using small tubes.

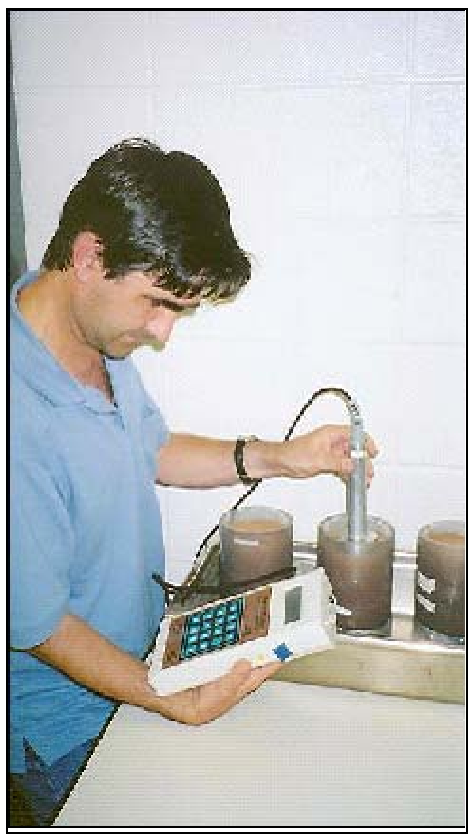

Illustration 2

Tube Suction Test

The effect of compaction energy on the final dielectric values was also evaluated. Two compaction efforts were considered, using smaller tubes (101 mm diameter). The first one corresponded to Standard Proctor energy (12375 ft lb / ft^3) and the second one was approximately twice than the initial compaction effort (26670 ft lb / ft^3).

In order to evaluate the effect of Portland cement on final dielectric values, the silts were mixed with cement, 3.5 percent by weight, and compacted at optimum moisture content in smaller tubes, using Standard Proctor compaction effort. A curing period of 28 days in the humidity room was used in order to allow the formation of cementitious products. After 48 hours in the oven at 50^0 C the samples were subjected to Tube Suction Tests. In order to evaluate the improved moisture resistance of these soils due to stabilizing agents, the final dielectric values were compared for raw and stabilized soils.

4) Guidelines for improving specifications of road subgrade construction

As presented, the pumping phenomenon of soils with high silt content is influenced by moisture conditions, compaction effort and soils characteristics (12). A solution to detrimental problem of silts pumping includes chemical stabilization. Cementitious products generated from chemical stabilization are resulting from the interaction between soils components and additives mixed.

The efficiency of soils stabilization is currently evaluated based on strength criteria. The required percentage of lime will increase the subgrades strength by 350 KPa (around 50 lb/in^2) and the percentage of cement will provide a final strength of approximately 200 lb/in^2 (36). The development of long term pozzolanic reactions in case of lime stabilization will generate a furthermore increase in strength (28). This process explains the differences between the values of subgrades stabilized strength for lime and Portland cement.

Moisture susceptibility is an important factor that affects the mechanical properties of subgrades material and especially their long term behavior (22). The current basis for mix design of stabilized bases should incorporate the concept of moisture susceptibility for a better evaluation of the optimum quantity of additive and for a more efficient stabilization process.

One of the objectives of this research is to contribute to the improvement of current guidelines for silt stabilization. An extended series of tests are considered in the next stage of study, using the current design specifications based on strength criteria and on new technique of investigation, such as Tube Suction Test, in order to address the identification and stabilization of problematic silts.

This testing program used includes:

1. Characterization and classification tests.
2. Chemical Stabilization efforts.
3. Unconfined strength criteria
4. Moisture susceptibility.
5. Deformation character of pumping silts under transient loads.

A brief description of the testing program for developing this set of guidelines included the following:

1. Soil characteristics and classification are determined and pumping susceptibility is assessed using cyclic triaxial tests, as described in section 1.3 of this chapter.

2. Lime and Portland cement are used for chemical stabilization in this section of the study. The percentage of lime to be used provided a minimum pH of 10 for the mixture.

3. Test specimens were compacted at the OMC and the equivalent of Proctor Standard compaction effort using the Harvard Miniature compaction apparatus. Samples were cured for 48 hours at 50^0 C. These conditions are considered as a rapid cure and simulate 30 days curing at 21^0 C (28). Unconfined compression strength was determined and compared with a minimum 200 psi (1380 KPa) strength criteria for cement (45) and a minimum increase of 350 KPa (around 50 psi) for lime.

4. The mixtures meeting the strength criteria were tested for moisture susceptibility as described in Section 3 of this chapter, in order to determine if the final dielectric values are situated below the critical value of 16 as being a critical maximum value (22), (23).

5. The long term effect of stabilization was assessed using Cyclic Unconsolidated-Undrained (UU) test for the mixtures considered in the previous four steps. The samples are placed in triaxial chamber and saturated. Illustration 3 presents the set up of the tests.

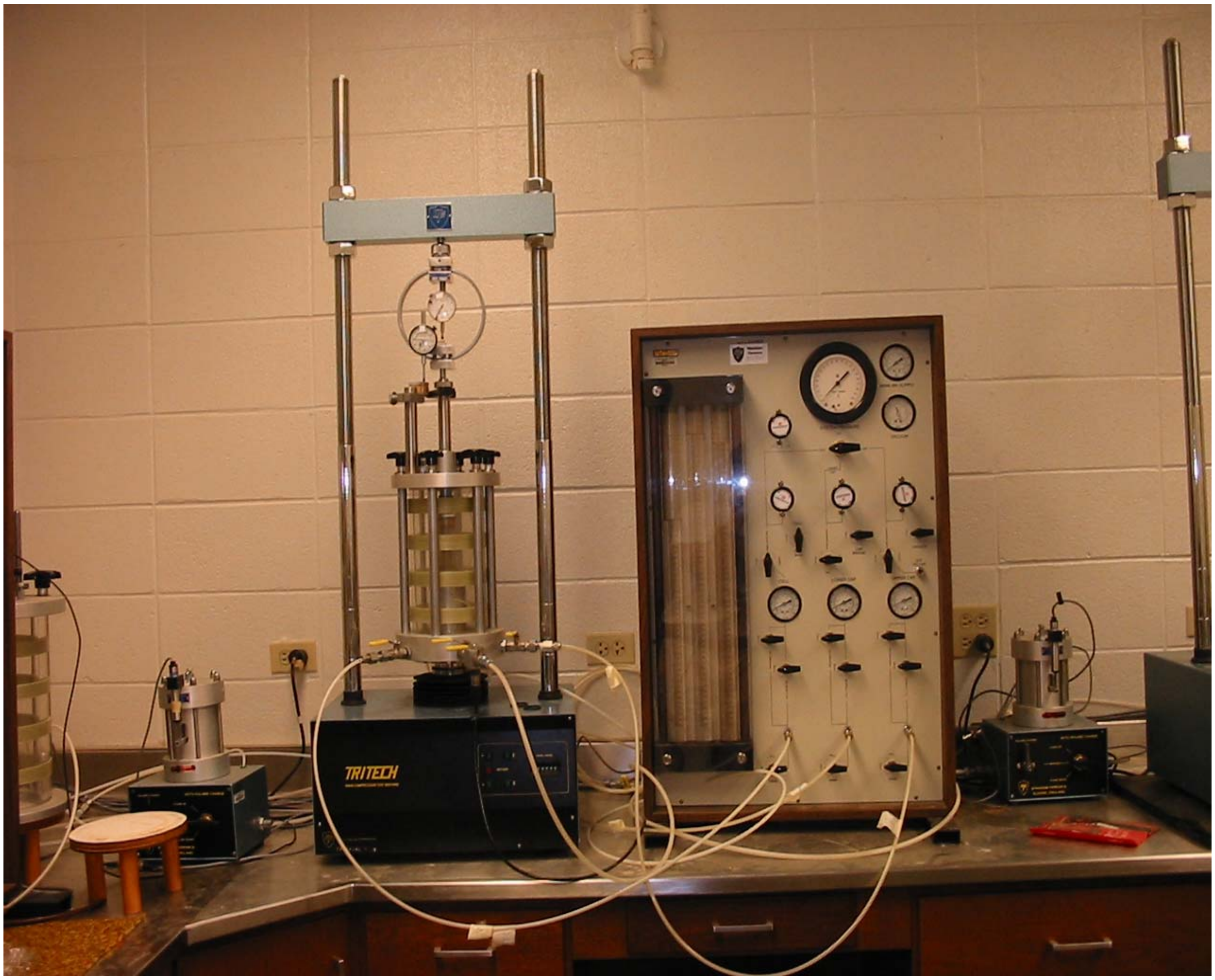

Illustration 3

Set up for cyclic triaxial tests

After the saturation of the samples, the drainage lines are closed and a confining lateral pressure of 2 psi is applied. The cyclic loading ranged from 600 psf (4.2 psi) to 90 psf (0.625 psi) in an effort to duplicate an on-and-off 18 kip wheel load at the subgrade depth. The value of 200 cycles is considered representative for the number of cycles. The axial strain (ε), total change of axial strain (creep) and the stress at specimen's failure are recorded. This procedure is an attempt to duplicate the situation from the road site, subjected to the worst scenario, with the subgrade saturated and subjected to the cyclic loads due to vehicle traffic.

The comparative results of UU tests for raw soil and the stabilized material can illustrate the improvements in silt behavior due to the additives and possibly eliminate detrimental pumping.

TEST RESULTS AND ANALYSIS

The initial phase of the study was focused on the identification of silts with a high pumping potential. At this stage, the research tried to address the important characteristics of pumping soils and why some soils with high silt content seems to pump while others do not.

The initial set of tests identified the soils gradation, Atterberg limits, and specific gravity. Figure 2 illustrates the gradation curve for Chase White.

Figure 2

Gradation curve for Chase White

The set of gradation curves for all the soils is presented in the Appendix B. The results of classification tests are summarized in Table 6.

Table 6

Characterization and Soils Classification

Parameter	Acadia	Chase Brown	Chase White	DeRidder Brown	DeRidder White	K1	K2	K3	HW 171
Sand %	**22**	**14**	**18**	**18**	**22**	**22**	**18**	**18**	**12**
Silt %	**63**	**64**	**72**	**59**	**66**	**64**	**67**	**64**	**75**
Clay %	**15**	**22**	**10**	**23**	**12**	**14**	**15**	**18**	**13**
LL %	**23**	**38**	**26**	**37**	**19**	**24**	**25**	**25**	**17**
PL %	**20**	**24**	**22**	**20**	**18**	**22**	**22**	**23**	**14**
PI %	**3**	**14**	**4**	**17**	**1**	**2**	**3**	**2**	**3**
Activity A	**0.26**	**0.71**	**0.03**	**0.75**	**0.19**	**0.125**	**0.18**	**0.125**	**0.23**
Toughness Index, TI	**0.29**	**0.99**	**0.03**	**1**	**0.79**	**0.39**	**0.4**	**0.31**	**0.51**
AASHTO Classification	**A-4 (1) Silty Soil**	**A-6 (15) Clayey Soil**	**A-4 (0) Silty Soil**	**A-6 (15) Clayey Soil**	**A-4 (18) Silty Soil**	**A-4 (0) Silty Soil**	**A-4 (0) Silty Soil**	**A-4 (0) Silty Soil**	**A-4 (0) Silty Soil**
Unified Classification	**ML Silt**	**CL Lean Clay**	**ML Silt**	**CL Lean Clay**	**ML Silt**	**ML Silt**	**ML Silt**	**ML Silt**	**ML Silt**
F_{200}	**15**	**22**	**10**	**23**	**12**	**14**	**15**	**18**	**13**
D_{10} **(mm)**	**0.0014**	**0.0013**	**0.002**	**0.0006**	**0.0017**	**0.001**	**0.0008**	**0.0007**	**0.0008**
D_{30} **(mm)**	**0.009**	**0.01**	**0.014**	**0.007**	**0.01**	**0.015**	**0.016**	**0.017**	**0.012**
D_{60} **(mm)**	**0.03**	**0.0016**	**0.02**	**0.035**	**0.026**	**0.03**	**0.033**	**0.035**	**0.03**
Uniformity Coefficient c_u	**20.71**	**1.23**	**10**	**58.33**	**15.29**	**30**	**41.25**	**50**	**37.5**
Coefficient of gradation c_c	**1.78**	**23.56**	**4.9**	**2.33**	**2.26**	**7.5**	**9.70**	**11.80**	**6**

DOTD specifications consider a usable soil as having less than 69 percent silt (1). As presented in Table 6, the silt content of the Natchitoches samples varies from 64 to 67 percent. Acadia contains 63 percent silt; K2-1 and Deridder White are marginal with respect of being considered usable soils (silt percentages of 67, and respectively 66). Chase White and HW 171 exhibit the highest silt content (72 and 75 percent), exceeding the maximum value admitted in the DOTD's specifications which can explain the problems exhibited by these last two soils as a subgrade soil.

Atterberg tests conducted revealed very low values for Plasticity Index, as presented in Table 6 for all the silts considered in this study (PI $\leq$ 4), except Chase Brown and Deridder Brown, with higher values (14 and respectively17). Also, these two soils exhibit high values for liquid limit (37and 38, respectively), comparing with the values of the low PI soils (approximately 20-25 percent, even lower for HW 171 - 17 percent).

The low values of Plasticity Index for the soils considered (except Chase Brown and Deridder Brown) suggest a high exposure to moisture sensitivity. Only a small increase in moisture content (1 to 3 percent) will transfer the soil from the plastic state to a state where it flows like a viscous liquid when conducting the liquid limit test.

These results suggest the presence of the clay fraction in higher percentages for Deridder Brown and Chase Brown than the rest of the silts influences their plastic properties by increasing the values for Liquid Limit and Plasticity Index. The viscous state for Deridder Brown and Chase Brown corresponding to Liquid Limit occurs at moisture content of 37 respectively 38 percent, comparing with much lower values of moisture content (between 17 and 25 percent) for the rest of the silts considered in this study. Also, a higher percentage of clay fraction increases the Plasticity Index up to 14 respectively 17 percent for Chase Brown and Deridder Brown, comparing with values in range of 1-3 percent for the rest of the silts considered in this study.

Seven of the samples, representatives from all four DOTD districts, classify as A-4 soils by the AASHTO Classification Method (DOTD TR 423 or ASTM 3282) and ML soils by the Unified Soil Classification System (ASTM D2487). Two of the samples, one from District 07 and another from District 58, classify as A-6 or CL soils (Deridder Brown and Chase Brown). Figures 3 and 4 show the soils distribution on the plasticity chart and on the ASTM D 3282 chart for liquid limit and index ranges for silt-clay materials.

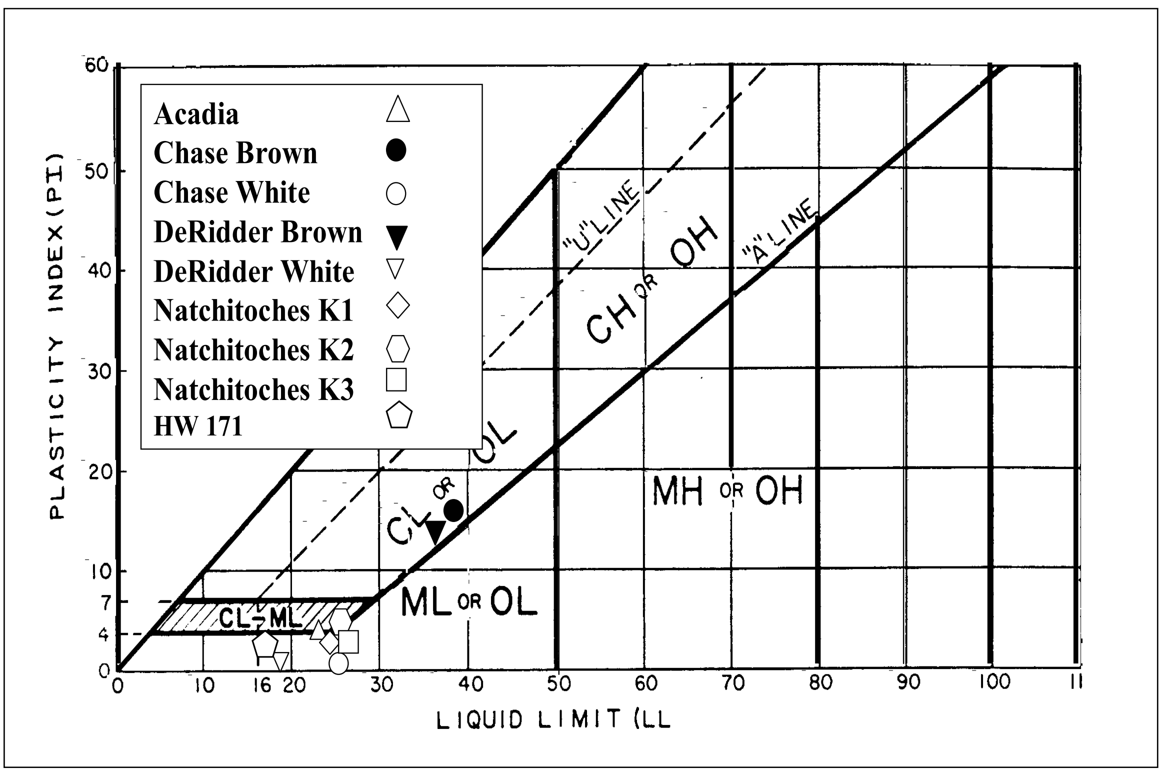

Figure 3

Plasticity Chart, Unified Classification of Soil Samples

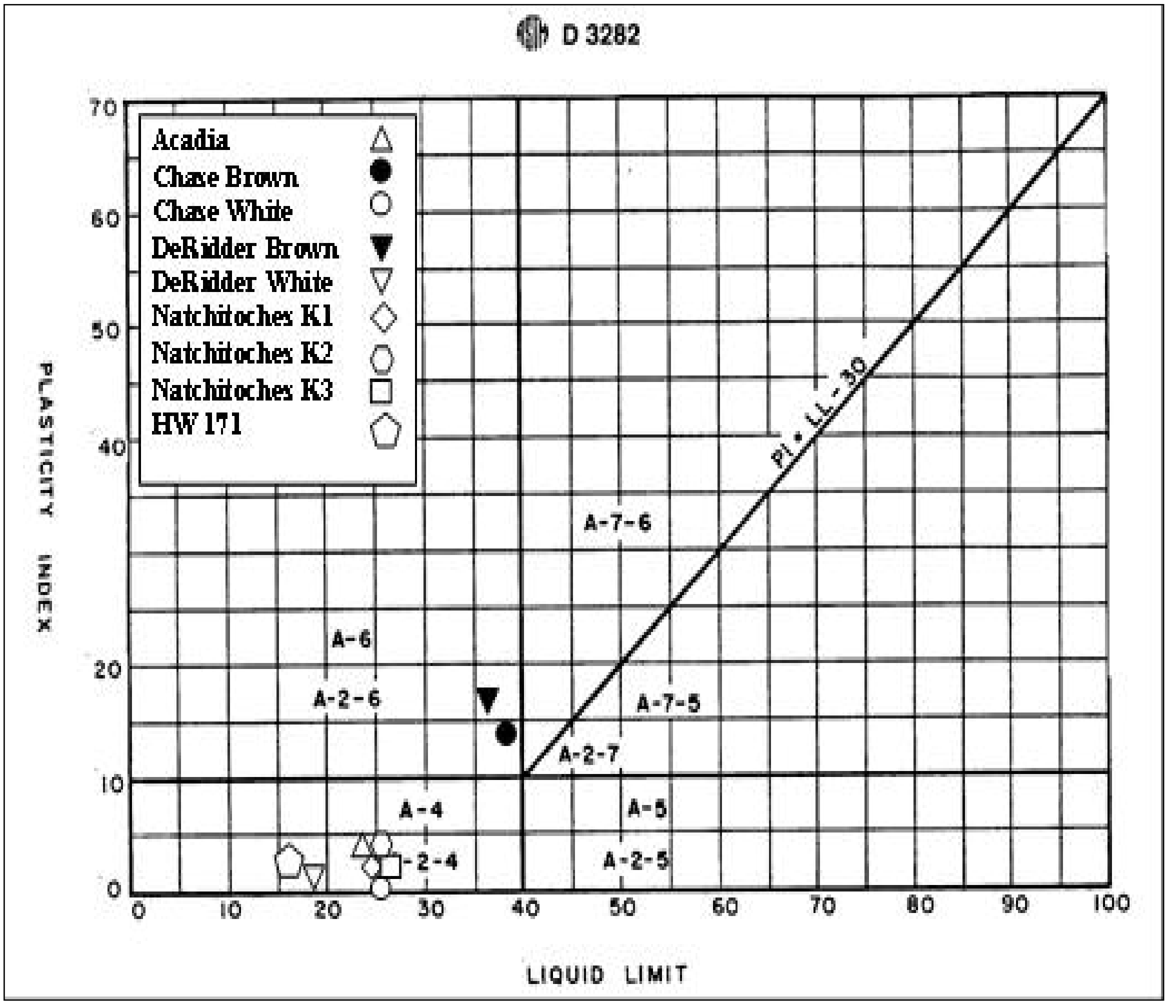

Figure 4
AASHTO Classification of Soil Samples

Textural classification of DOTD TR 423-89 is illustrated in Figure 5

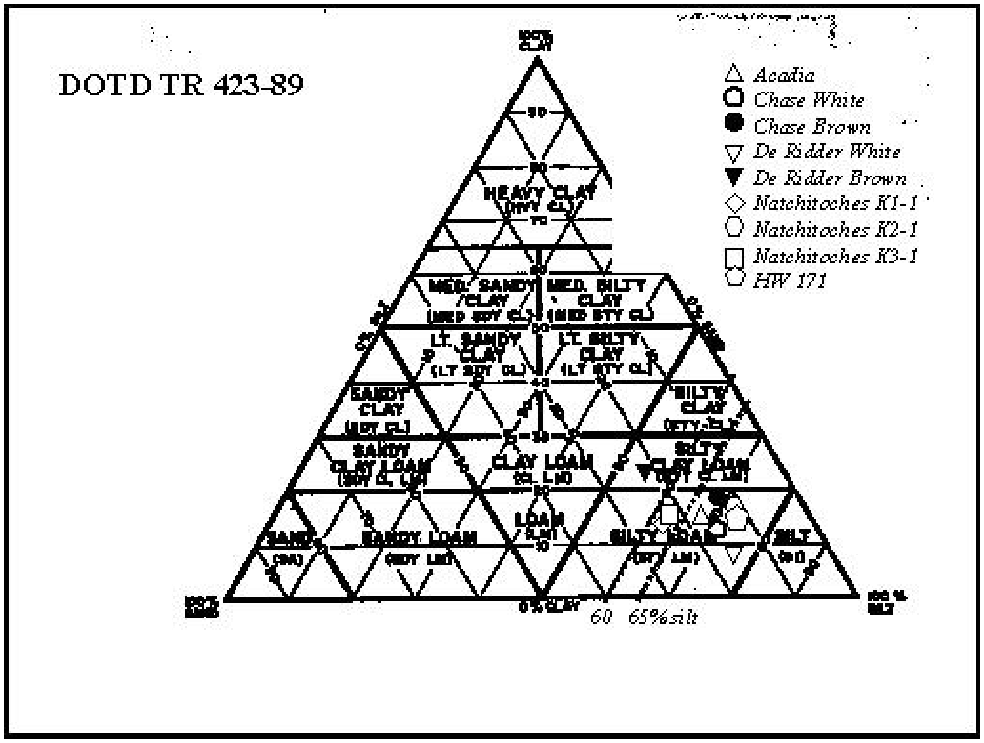

Figure 5

Textural Diagram, DOTD TR 423-89

According to this classification, only Deridder Brown is described as silty clay loam, the rest of the samples are placed in the silty loam area of the textural diagram.

Compaction Tests

Using the Proctor compaction method (ASTM D 698), the corresponding optimum moisture content and maximum dry density were determined for the soil samples. Different compaction energies were also used to generate the corresponding compaction curves. This included the modified effort, noted “Modified Proctor” (AASHTO T180 and ASTM 1557, 56,000 ft-lbf/ft^3 or 2,700 kN-m/m^3), a modified plus effort, noted “Modified +” (78,750 ft-lbf/ft^3 or 3,750 kN-m/m^3), and a reduced standard effort, noted “Standard reduced” (7,425 ft-lbf/ft^3 or 350 kN-m/m^3).

Figure 6 represents a typical family of compaction curves for Natchitoches K1-1, corresponding to each compaction effort.

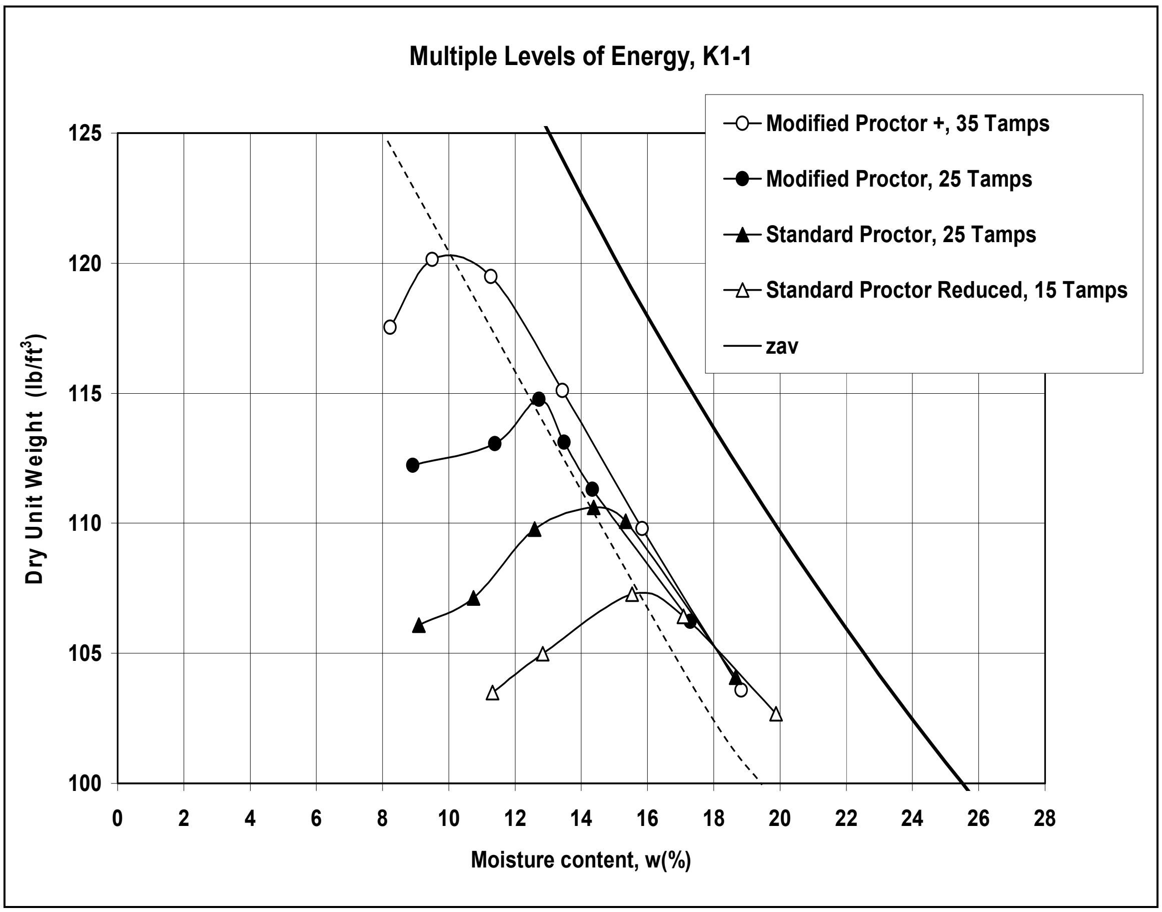

Figure 6

Compaction curves for Natchitoches K1-1

A unique optimum moisture content and a maximum dry unit weight correspond to the different compaction efforts. A higher level of compaction effort induces a higher maximum value for the soil's density and a lower value for optimum moisture content.

Points from each compaction curve with optimum moisture content and maximum dry unit weight as coordinates can be connected to form a line of optimum moisture contents. These points describe a state of soil of approximately equal saturation. The optimum moisture curve and the zero-air-void line can be drawn for each soil.

During the compaction tests, it was noticed that as the moisture increased, a point was reached where the test specimen would appear to heave or pump under the impact of the hammer in the compaction mold. The A-4 samples exhibited the greatest sensitivity and loss of stability with small increments of moisture above the optimum moisture content. The dilatancy character of the silty soils and their reaction to vibrations from the impact of the hammer produced a shiny, wet surface with heaving that was more pronounced in the A-4 samples, compared to the, somewhat, more plastic A-6 samples. However, although the dilatant character and pumping appears to be tempered with an increase in plastic character, there was a perception that all of these high-silt samples in this study could pump or become unstable at moisture levels that exceeds the optimum moisture content.

The degree of saturation can be determined with the following formula:

$$S = \frac{G_s \times w \times \gamma_d}{G_s \times \gamma_\omega - \gamma_d} \qquad [6]$$

Where: G_s = specific gravity of the soil (For K1-1: G_s = 2.7)

w = moisture content

γ_d = dry unit weight corresponding to w

γ_ω = 62.4 lb/ft^3 (unit weight of water)

In this typical case illustrated in figure 6, the Modified Proctor Compaction energy (25 tamps/layer, hammer weight of 10 lb. and drop height of 18 in.) generates a maximum dry unit weight of 115 pcf. and the optimum moisture content is 12.5 percent. Using formula (1), the degree of saturation at this soils state is:

$$\mathbf{S = \frac{2.7 \times 12.5\% \times 115}{2.7 \times 62.4 - 115} = 0.75} \qquad [7]$$

The rest of the values corresponding to each level of compaction are presented in Table 7.

Compaction Energy	Optimum Moisture Content	Dry Unit Weight	Saturation
	(%)	(lb/ft3)	(%)
Standard Proctor Reduced	16	107	0.75
Standard Proctor	14.5	110.6	0.75
Modified Proctor	12.8	114.8	0.75
Modified Proctor +	10.6	120.2	0.72

Table 7

Saturation Calculus for Natchitoches K1-1

Also, further analysis of figure 6 reveals the optimum moisture content corresponding to Standard Proctor Compactions is 14.5 percent. At this state, the soils maximum dry density is not substantially increased by a greater compaction effort: Modified Proctor generates a maximum dry unit weight of 110.5 pcf and Modified Plus generates 112 pcf, all values corresponding to 14.5 percent optimum moisture content (Standard Proctor).

At moisture contents values greater than OMC, the different levels of compaction energies have no effect on dry unit weight. All four compaction efforts considered in this case generate the same dry density of 105 lb/ft^3 at 18 percent moisture content. Similar situations are encountered for the rest of the soils considered in this study. The graphs illustrating the compaction curves for all the silts are presented in the Appendix C.

These aspects represent an indication for the difficulties of silt compaction during construction phase. The control of soils moisture content is very important in order to meet the specifications regarding maximum dry unit weight required for road bases. An excess of compaction effort will not have a significant effect on this parameter and for moisture levels higher than optimum, the effect on densification is null.

For a better illustration, the compaction curves for all the compacted soils using Standard Proctor energy are presented in Figure 7 and 8.

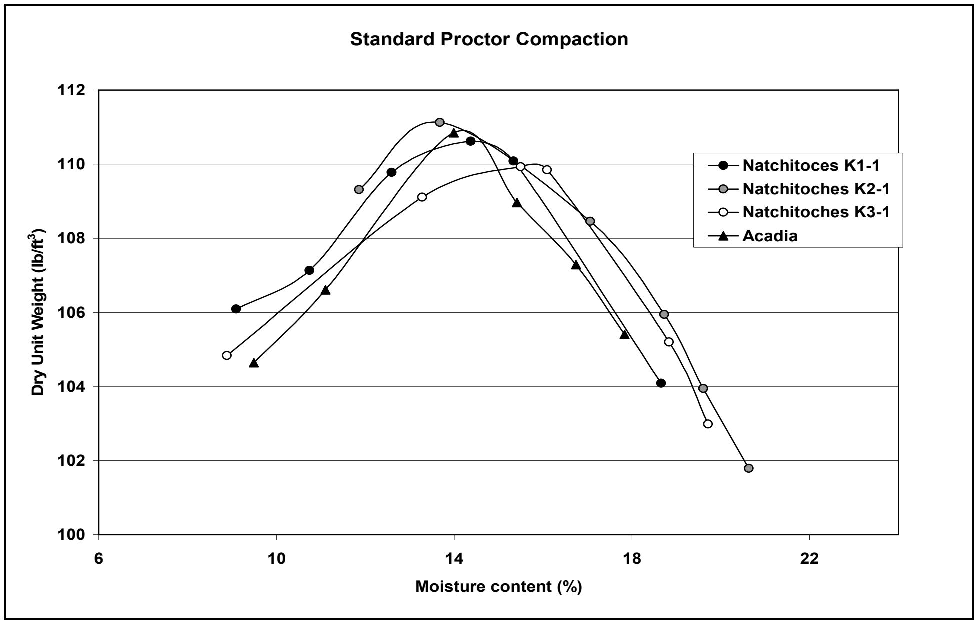

Figure 7

Standard Compaction Curves for Natchitoches and Acadia soils

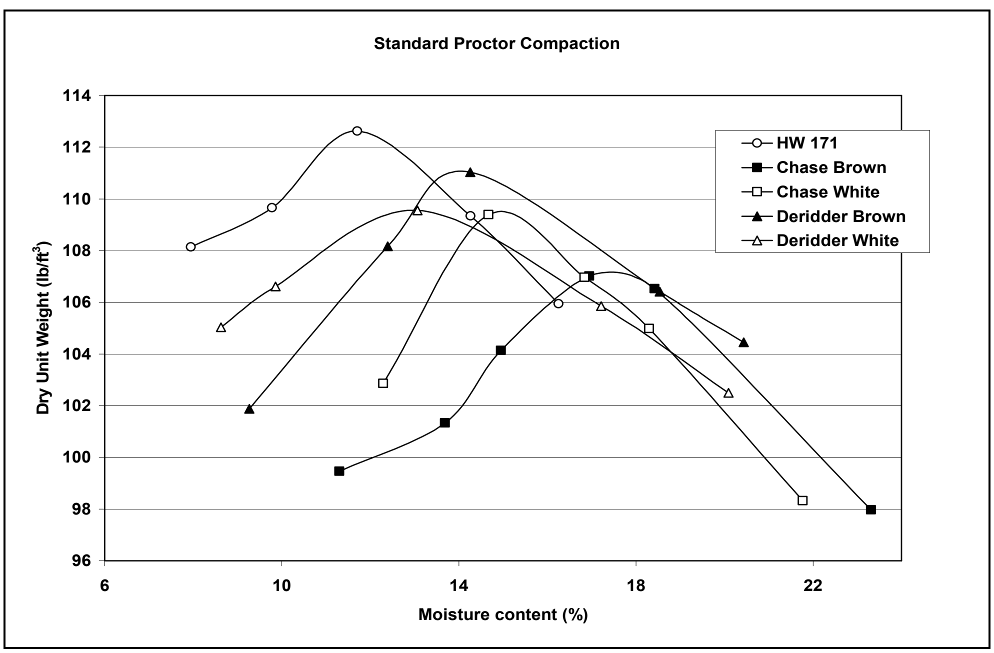

Figure 8

Standard Compaction Curves

The values for Optimum Moisture Content for all the soils are situated in the range of 11.7 - 17.5 percent and the values for dry unit weight are in the range of 107-112.6 pcf.

The study of the effects of compaction effort on soils density was extended by using Harvard Miniature Compaction Apparatus. Specimens were molded at different moisture contents and with different compaction energies by varying the number of tamps. Figure 9 illustrates the variation of different compaction efforts corresponding to 15, 30 and 50 tamps per layer and a comparison with compaction curves from Standard and Modified Proctor compaction for K1-1. Compaction effort generated by using 15 tamps per layer with Harvard miniature apparatus duplicates the soil state created by Standard Proctor at optimum moisture content. All compaction curves converge to a same configuration for moisture contents values higher than OMC.

The Harvard Miniature Compaction Apparatus represents an ideal means to create smaller soil specimens better fitted for moisture-density studies or strength tests as Unconfined Compression or triaxial tests. For a given moisture content, the weight in grams of the molded specimen is numerically equal to the wet unit weight of the compacted soil in lb/ft^3 (43).

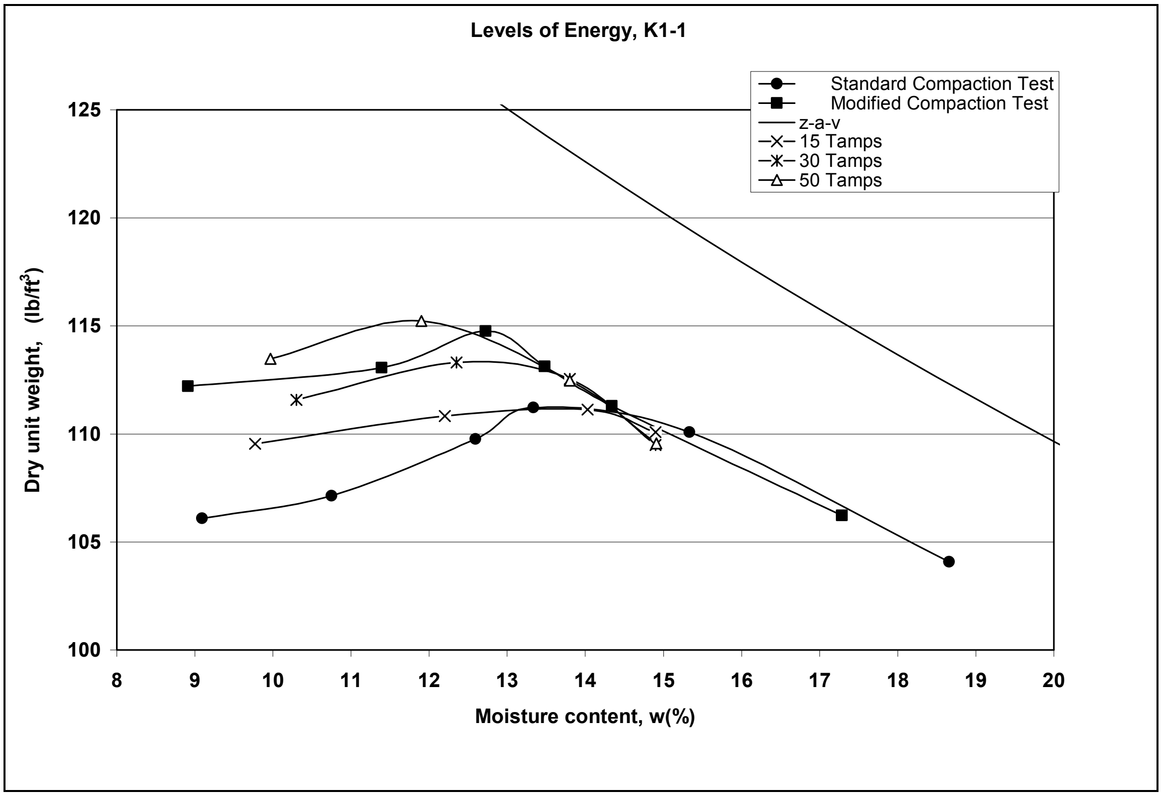

Figure 9

Compaction Curves, K1-1

Mineralogy Study

The mineralogic analysis of the soils revealed the minerals present and their distribution. The results are presented in Table 8. The soil mixtures with the chemical additives were also investigated to identify any cementitious or other chemical products that may have formed.

Table 8

Mineral distribution of the silt samples

Mineral Distribution	ACADIA	K1-1	K2-1	K3-1	Chase Brown	Chase White	DeRidder Brown	DeRidder White	HW 171
Quartz	68	67	72	75	59	60	73	72	82
K - feldspar	6	5	8	5	16	11	11	11	2
Na - plagioclase	9	7	4	5	11	14	5	10	2
muscovite/illite	5	4	5	4		2	1	<1	3
biotite/chlorite	6	7	4	5	4	1	6	8	4
montmorillonite	4	5	4	4			1	1	
kaolinite	0	<1	0	4			0.5	1	
other clay minerals	0	<1	<1	1	7	9	1	1	6
ilmenite	0	<1	0	<1					
hematite or Fe oxide	<1	1	1	1	2		1	0	1
zircon	1	<1	<1	0		1	<1	<1	<1
titanium dioxide	<1	<1	1	<1	1	1	1	<1	<1
calcite/dolomite	0	1	1	2		1	1	1	

The minerals are presented in categories as quartz, feldspar (K-feldspar, Na-plagioclase), clay minerals (muscovite/illite, biotite/chlorite, smectite, kaolinite, and other), and oxides (Fe oxide, zircon, titanium oxides, calcite). The results indicate abundant quartz and evidence for the presence of feldspar minerals.

The clay mineral composition of the samples varied in percentages and in mineral type. The Acadia and Natchitoches (K1, K2, and K3) samples are similar in the types of clays present and the percentages. The quantities of clays minerals present in these samples ranged from approximately 13 to 17 percent, which is identical with the clay percentages determined from gradation curves.

The results of mineralogic analysis of some soil mixtures with Lime and Lime-Fly Ash revealed a limited amount of cementitious products. The one-day (24-hr) rapid curing time was concluded to be insufficient for the full development of the cementitious crystals.

Undrained Strength Test

An analysis of strength – moisture variation was developed based on the results of an extended set of unconfined compression tests for the natural soils. Specimens were compacted using the Harvard Miniature Apparatus with different compaction energies, including one similar to Standard Proctor, and various moisture contents and subjected to unconfined compression tests. Results of the strength variation with moisture were corroborated with the evolution of dry unit weight versus moisture content. Typical results are presented in Figure 10 for K1-1.

At moisture content levels greater than the optimum moisture content corresponding to Standard Compaction effort, the density and strength are significantly reduced. In the case of K1-1, specimens compacted at different energies exhibit almost the same low strength (15.5 psi) for a moisture content of 15 percent, higher than optimum moisture content for this soil. Same aspect was noticed for rest of the silts considered in this study. The extent of the strength loss and the corresponding strain that occur with the different samples can provide an image regarding the relative affinity for the different samples to pump or not pump (i.e. the differences between the different strength values corresponding to different moisture contents and the corresponding axial strain).

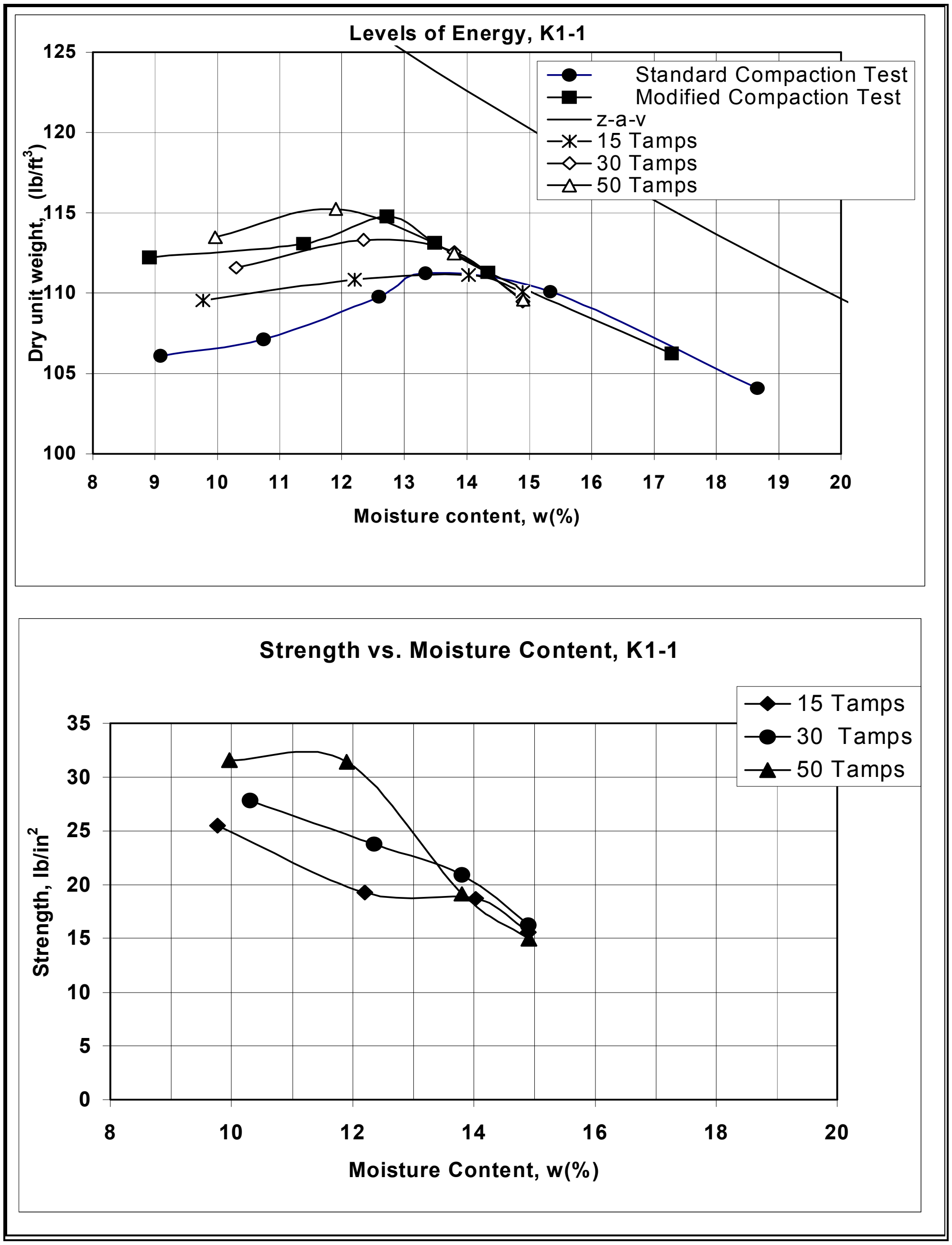

Figure 10

Natchitoches K1-1 compaction effort – strength variation

Evaluation of pumping phenomenon

Among the factors contributing to the pumping phenomenon in silts are high level of subgrades moisture and higher compaction effort. This detrimental situation from the construction site was duplicated in laboratory in order to measure the variations in soils axial strain, strength and saturation. A series of cyclic triaxial tests were conducted with samples molded at optimum moisture level + 2 percent using the Harvard Miniature Apparatus with a compaction effort that produced the Standard Proctor moist density at the optimum moisture content. The test specimens were subjected to300 cycles of triaxial loading with cyclic axial loads that varied from 90 psf to 600 psf. The results are illustrated in Figure 11.

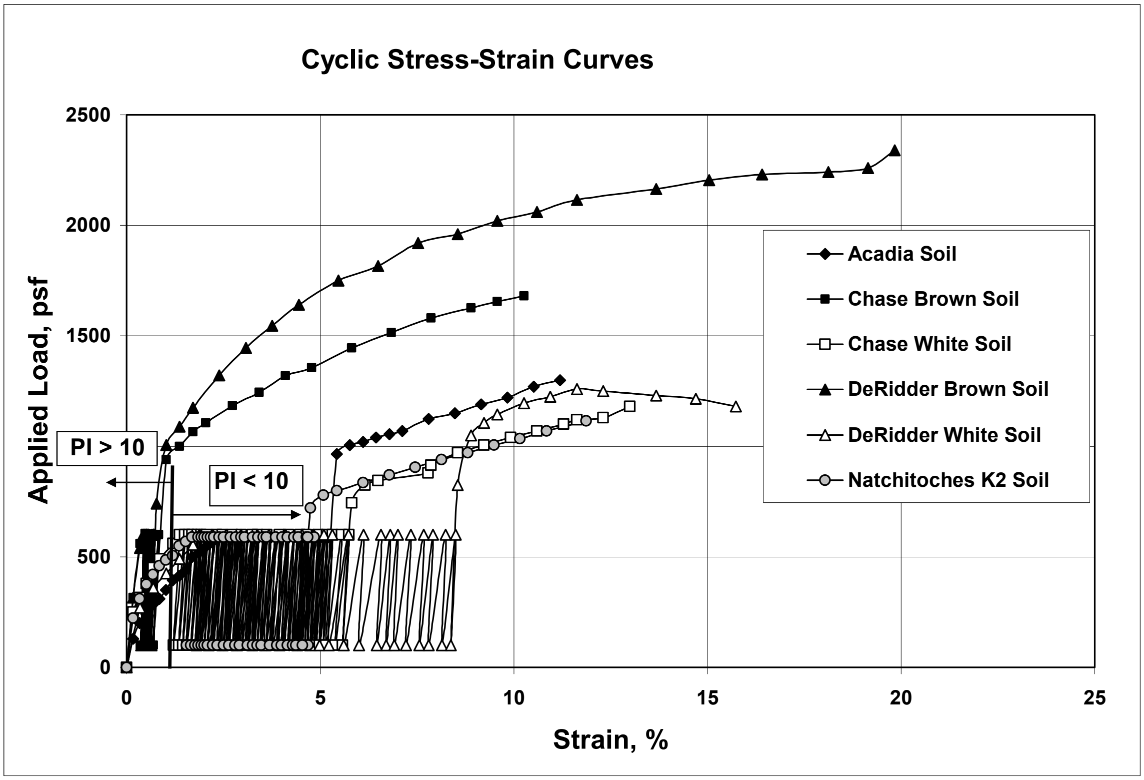

Figure 11

Comparative view of pumping phenomenon

The analysis of figure 11 reveals different responses to cyclic loading of the non – plastic (ML / A – 4, i.e. Natchitoches, Deridder White, Chase White, and Acadia) soils and the plastic soils (CL / A – 6, i.e. Chase Brown and Deridder Brown). These two last soils exhibit the lowest variation in axial strain, approximate 1 percent, compared with all other soils situation (from 3 to 5 percent for Natchitoches, Deridder White, Chase White, and Acadia). The effect of pumping phenomenon on strength is also differentiated according to the soils characteristics: Chase Brown and Deridder Brown exhibit a higher strength after 200 cycles comparing with the rest of the soils (Natchitoches, Deridder White, Chase White, and Acadia). The first two soils (CL / A – 6) continue to provide significant resistance to loading increase beyond the 600 psf maximum cyclic loads, in contrast with the non – plastic soils (ML / A – 4), where the increasing pore pressure corresponding to an increase of strain diminishes the strength of specimens (13).

The comparative analysis of Atterberg limits for the soils considered indicates the two plastic (CL / A – 6) soils exhibit higher values for Plasticity Index (PI > 10) than the non-plastic (ML / A – 4) soils (PI < 10) (See table 6).

The content of clay minerals has an influence on soil plasticity. As presented in Table 6., Chase Brown and Deridder Brown, (CL / A – 6), with 22 and respectively 23 percent clay, exhibit values of 14 and 17 respectively for Plasticity Index, higher than the values situated in the range of 1 to 4 for the rest of the non-plastic soils with clay content in the range of 10 to 18 percent.

Also, the axial strain developed during the cyclic loading testing exhibited a more significant variation for non-plastic soils with Plasticity Index < 10 and silt content > 60 percent comparing with the plastic soils with PI > 10 and silt content < 60 percent.

The analysis of these results lead to the conclusion that Plasticity Index and silt content play an important role in soils response to pumping phenomenon, with most negative effects on non – plastic soils (ML / A – 4). For all these soils, the silt content is higher than 60 percent and the values for Plasticity Index are less than 10. These two findings are relevant criteria in the identification of problematic silts.

Also, the moisture content has an important role in the development of pumping phenomenon. Soils with moisture content higher than optimum moisture content exhibit low values of strength and are more susceptible to pumping action.

Chemical Modification / Stabilization

Chemical stabilization of the problematic silts is a technique used to construct a working table, prevent pumping and to achieve the relative compaction requirements for the subgrade.

The unconfined strengths of the chemically stabilized specimen were measured for samples mixed at different moisture contents with different percentages of two additives: lime and fly ash. Comparative results are presented in Figure 12 for Acadia plus lime 2% and fly ash 8% and Acadia plus lime 4 percent.

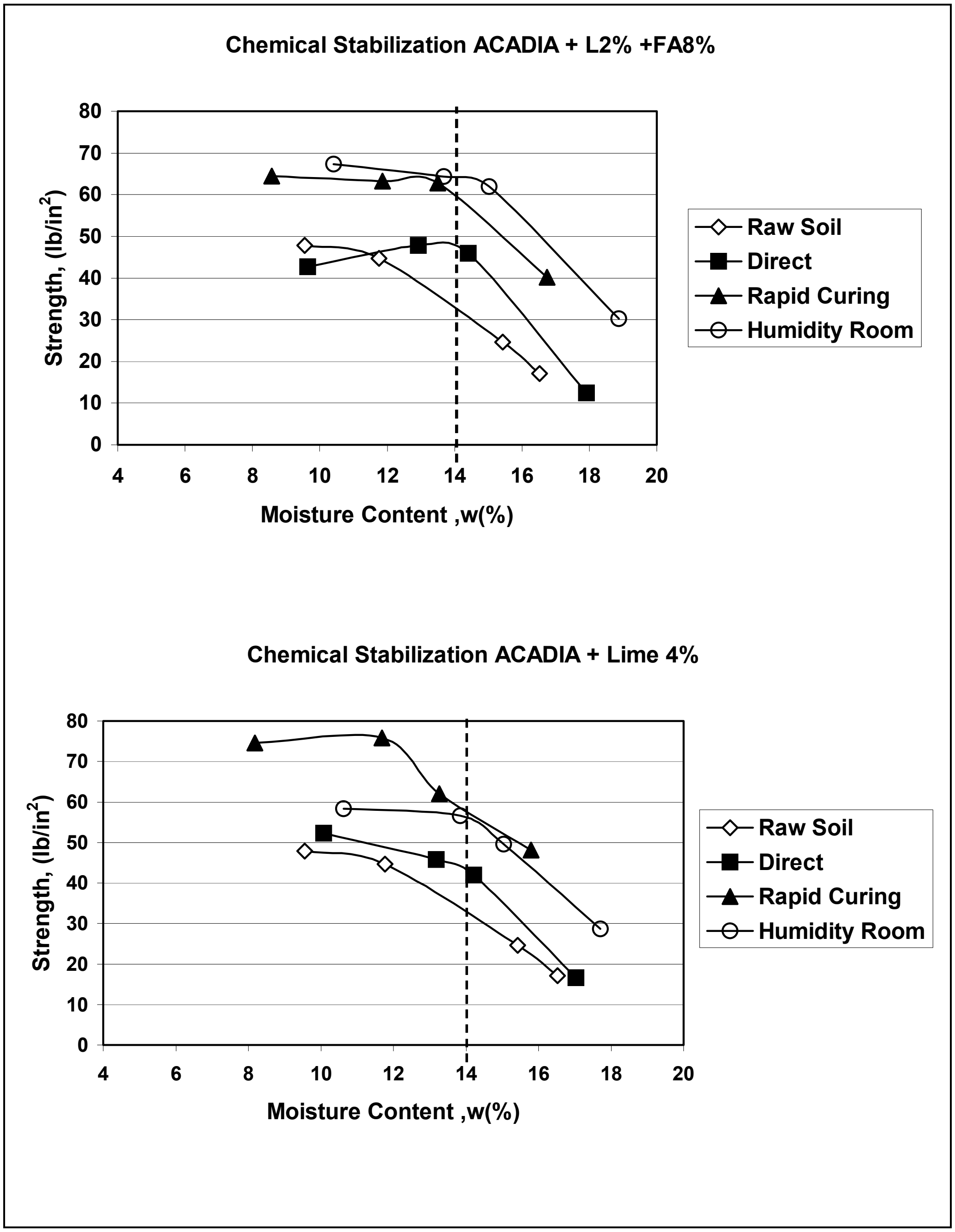

Figure 12

Comparative evolution of strength vs. moisture for

Acadia + L2% + FA 8% and Acadia + L4%

A set of specimens was tested “as compacted” (referred to as “Direct”); a second set was cured for two days at 50^0 C (referred to as “Rapid Curing”) and a last set was cured for two weeks in humidity room. The strength values were compared with those for the raw soil.

The general trend of all the curves in both cases demonstrates a dramatic decrease in strength for moisture contents higher than optimum moisture content (14 percent, marked with the dotted line). Also, further analysis of Figure 12 indicates a very slight improvement of strength for Acadia and additives, tested “as compacted” comparing with raw soil. The curing time determines a significant increase of strength values for mixtures cured at 50^0 C for 2 days or cured for 2 weeks in humidity room.

The samples of Acadia + lime 4 percent and Acadia + lime 2 percent + fly ash 8 percent exhibit approximately the same values of strength and variation of it vs. moisture content. The chemical agents added in the percentages mentioned above have almost the same effect form the stabilization point of view.

A set of samples of Acadia + Lime 4 percent, Acadia + Lime 2 percent + Fly Ash 8 percent and Acadia + cement 4 percent were molded at Standard Proctor conditions and tested after 2 years of curing in humidity room, for determining the stabilization effects on long term for the additives considered. The comparative results for lime stabilization and cement stabilization are presented in Table 9. The rest of the data are presented in the Appendix D.

Soil	Moisture	γmoist	γ dry	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
ACADIA	9.55	116.56	106.4	47.87
	11.76	124.35	111.27	44.68
	15.42	127.61	110.56	24.61
	16.52	126.48	108.55	17.05
ACADIA + Lime4% DIRECT ("As Molded")	10.07	110.42	100.32	52.28
	13.18	118.32	104.54	45.8
	14.21	120.37	105.39	42
	17.03	122.12	104.35	16.57
ACADIA + Lime4% RAPID CURING (RC)	8.18	109.8	101.5	74.55
	11.68	116.61	104.41	75.77
	13.26	118.76	104.86	62
	15.8	121.07	104.55	48.11
ACADIA + Lime4% VACUUM SATURATION (VS)	10.62	109.25	98.762	25.57
	13.84	116.2	102.07	27.82
	15.04	119.23	103.64	30.47
	17.7	121.58	103.3	28
ACADIA + Lime4% HUMIDITY ROOM (HR)	10.62	110.94	100.29	58.33
	13.84	117.74	103.43	56.56
	15.04	120.41	104.67	49.61
	17.7	121.64	103.35	28.65
ACADIA + Lime4% After 2 years curing HR	14	122	107.02	72
Acadia+ 4% Cement	17.12	127.37	108.75	16.31
Acadia + 4% Cement HR	17.1	127	108.45	78.77
Acadia +4% Cement VS	17.1	127.67	109.03	41.82
Acadia + 4% Cement After 2 years curing HR	16.3	125	107.5	350

Table 9

Comparative results for Acadia stabilization with lime 4% and cement 4%

The results indicate the gains in strength for Acadia + lime 4 percent are more substantial for samples subjected to rapid curing (noted as RC) than humidity room (notes as HR). The Vacuum Saturation reduces strength for stabilized samples below the original values of raw soil.

The sample cured for 2 years with a final moisture content of 14 percent, exhibit a strength value of only 72 lb/in^2, with a gain of approximately 12 lb/in^2 compared with the value for the sample with same moisture content subjected to rapid curing. These results suggest the clay content for Acadia is not sufficient in order to constitute a basis for pozzolanic reaction with the lime added. The pH of the mixture is 11.2, which represent favorable conditions for a long term reaction between calcium from the lime reacts with aluminates and silicates solubilized from the clay mineral surface, but the presence of these minerals are not in a sufficient measure.

The comparative results of strength for samples stabilized with lime and Portland cement indicate this last additive is more efficient for Acadia stabilization, providing the highest values for strength.

Modification/Stabilization of Wet Silts

A second series of tests were considered as an as an attempt to simulate a situation where a wet, high-silt soil is encountered. Four soils were used: Chase White (A-4/ML) and Brown (A-6/CL), and the DeRidder White (A-4/ML) and Brown (A-6/CL). Specimens were molded at an initial moisture content of the soils which exceeded their optimum moisture contents by several percentage points. Four test specimens were compacted for each set consisting of the raw soil sample and selected mixture of additives. Using the soil in a state significantly wet of optimum, four set of three samples each were prepared for each soil mixed with Portland cement, lime, fly ash and lime + fly ash. The initial moisture content of the soils exceeded their optimum moisture contents by several percentage points. An additional set of specimens were molded using natural soils wet of optimum for comparative purposes.

The potential for modifying and / or drying the soil was evaluated by comparing the specimen's strength in these new conditions. The typical results are presented in Figure 13 for Chase White. The rest of the data are presented in the Appendix E.

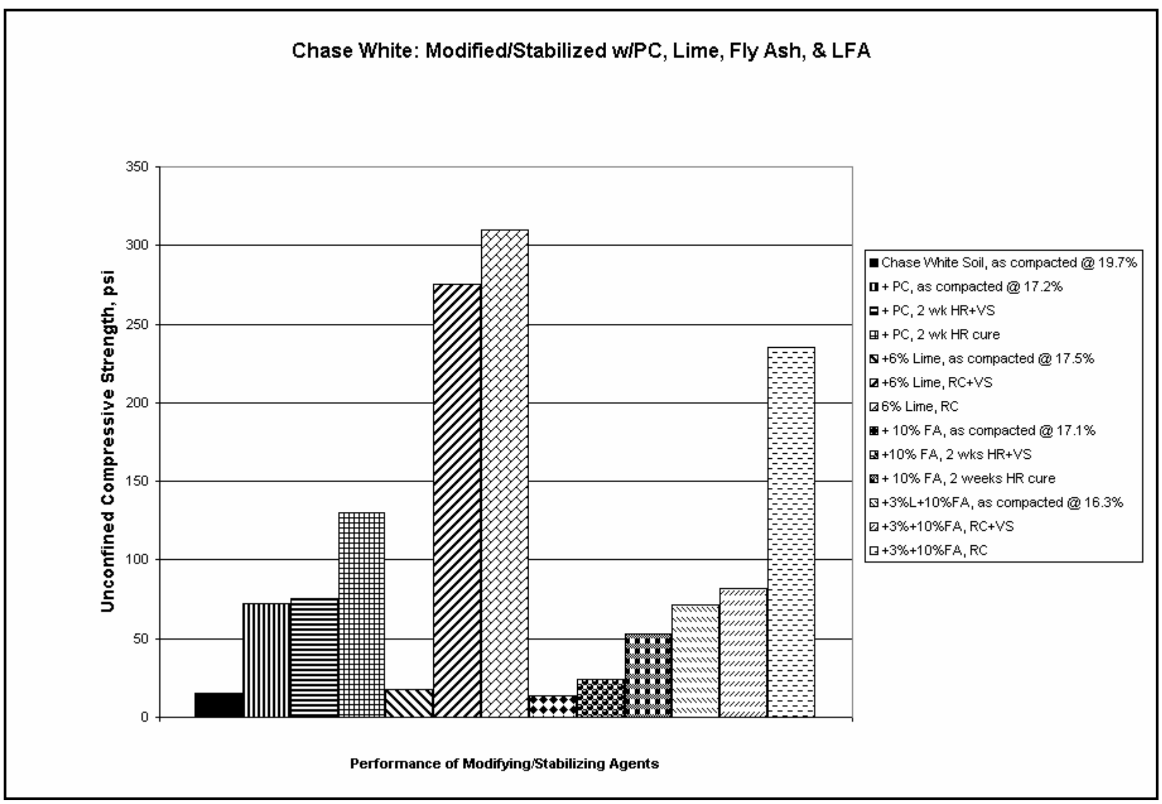

Figure 13

Effect of Modifying / Stabilizing Agents for Chase White

The initial moisture content for Chase White was 19.7 percent, higher than optimum moisture content (15 percent corresponding to Standard Proctor). The percentages of additives for this case (based on dry weight) were as follows:
PC 8%, Lime 6%, Fly Ash 10%, and Lime + Fly Ash 3%+10%.

One of the natural soil specimens and one of the specimens molded with the different admixtures were tested for unconfined compression strength “as molded”. This was done as a measure of the potential for modifying and/or drying the soil. The other two specimens from each set were allowed to cure. The lime and lime-fly ash specimens were cured under accelerated conditions (50^0 C for 3 –days). A longer rapid curing period was used in the second test series to allow more time for the development of cementitious products. The Portland cement and fly ash (alone) specimens were cured in a humidity room for 2-weeks under ambient conditions. One of the two remaining specimens was tested in unconfined compression at the end of the curing period. The other was subjected to vacuum saturation and then tested for unconfined compression strength, to determine the behavior of stabilized soils in conditions of saturation during the long term road service.

The results of unconfined compression tests illustrate the performances of selected chemical stabilizer and also the effects as drying agents. The initial moisture of 19.7 percent for raw soil was reduced with 2.2 up to 3.3 percent by the combinations of additives as described above. The strength of the stabilized samples tested as molded are the smallest compared with the values of cured samples.

For the percentages of chemical additives used, lime appears to be the most effective for this soil, as the values for strength for cured samples are the highest compared with all other results obtained. The drying effect is considerable during the 3 days of curing at 50^0 C, with a reduction of 6 percent of moisture content. Also the long term efficiency of the lime is relevant, as the strength of mixture's samples subjected to rapid curing, vacuum saturation and tested is higher than other mixtures. Table 10 presents the results of this set of tests. The rest of the results are presented in the Appendix E.

Soil	Moisture	γ moist	γ dry	Strength	
	%	lb/ft^3	lb/ft^3	lb/in^2	kPa
CHASE WHITE (OMC = 15%)	19.67	122.5	102.36	15.34	105.766
SOIL + PC 8%: Direct	17.17	124.3	106.09	72.62	500.697
Cured (in HR for 2 weeks)	17.59	124.44	105.83	130	896.318
VS	17.59	123.91	105.37	75.49	520.485
SOIL+Lime 6%:Direct	17.5	119.1	101.36	38.3	264.069
RC	13.72	118.61	104.3	310	2137.37
VS	13	106.5	94.248	275.27	1897.92
SOIL + FA10%: Direct	17.13	124.34	106.16	18.5	127.553
Cured (in HR for 2 weeks)	16.58	122.8	105.34	52.71	363.423
VS	16.58	123.88	106.26	24	165.474
SOIL +L 3 % FA 10 %: Direct	16.34	123.69	106.32	71.07	490.01
RC	11.34	119.61	107.43	235	1620.27
VS	11.88	116.8	104.4	81.7	563.302

RC = Accel curing:3 days in oven @50^0 C
VS= Vacuum Saturation: 30 min deair and 1 hour complete inundate
For the samples subjected to VS, the values for moisture and unit weight are determined before the vacuum saturation procedure

Table 10

Comparative results for chemical stabilization for Chase White

Evaluation of silts moisture susceptibility

The presence of moisture in the subgrade represents one of the key factors for pumping phenomenon. The redistribution of water within the soils matrix is influenced ultimately by capillary phenomenon, which has an important role in determining the soils moisture susceptibility. This last characteristic should be a part of the solutions for the silts pumping problem.

The Tube Suction Test indirectly determines the increased moisture conditions by measuring the soils relative dielectric values. Their variation with time provides a basis of performance classification.

The initial set of tests was conducted for the raw soils. The results verify the highly moisture susceptibility for all the silts considered. The only exceptions are Chase Brown and Deridder Brown with final dielectric values of 11 and 12.5 respectively, less than the critical value of 16 (22), (23). Also, these two soils are characterized by low values of PI (PI < 10), compared with the rest of the silts with PI > 10 and highly moisture susceptibility. Figure 14 illustrates the variation of dielectric values with time.

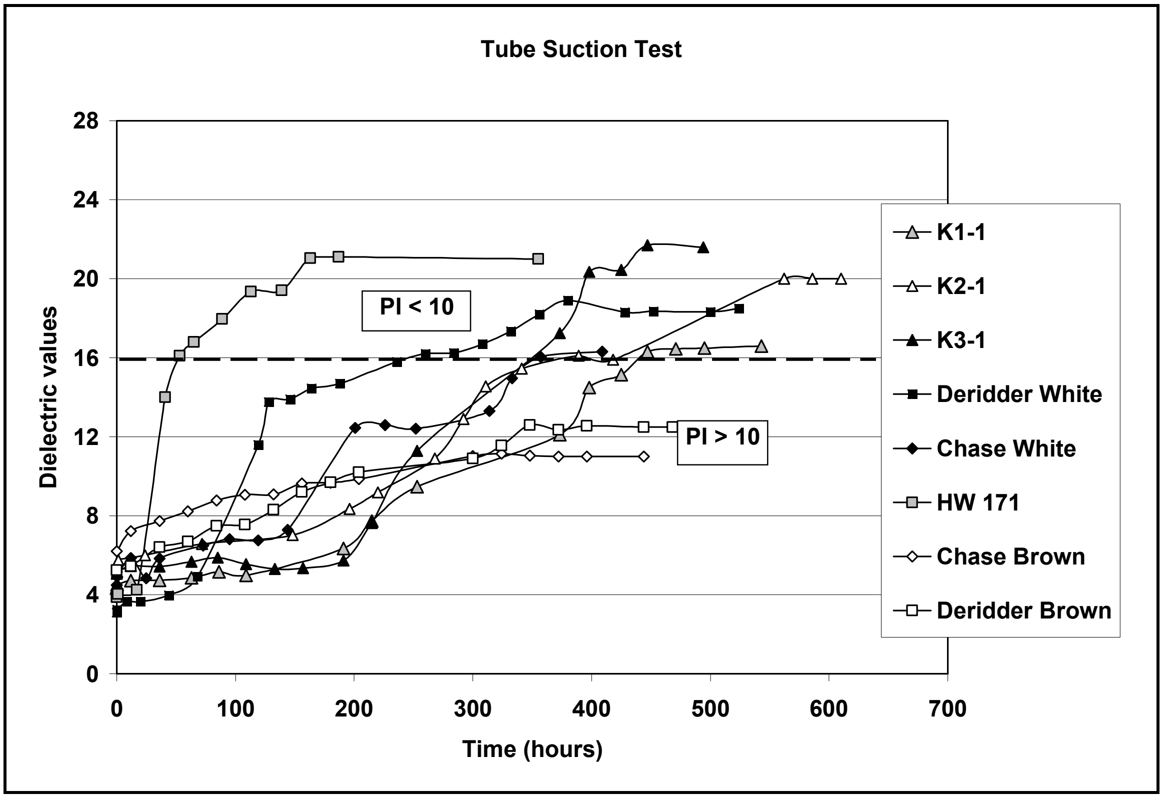

Figure 14

Results of Tube Suction Test

The analysis of figure 14 indicates different variations of the dielectric values, as the final level of constant value is reached at different rates in time. In the case of HW 171 soil, final dielectric values are reached in only 200 hours, compared with K2-1 for which the constant level is reached in more than 500 hours.

The effect of additives on final dielectric values was studied on specimens of soils mixed with 3.5 percent Portland cement and compacted at optimum moisture content in smaller tubes. After 28 days curing period in humidity room, Tube Suction Tests were conducted for these specimens after stabilization and results were compared with those from raw soil. Figure 15 illustrates the effect of Portland cement on TST results.

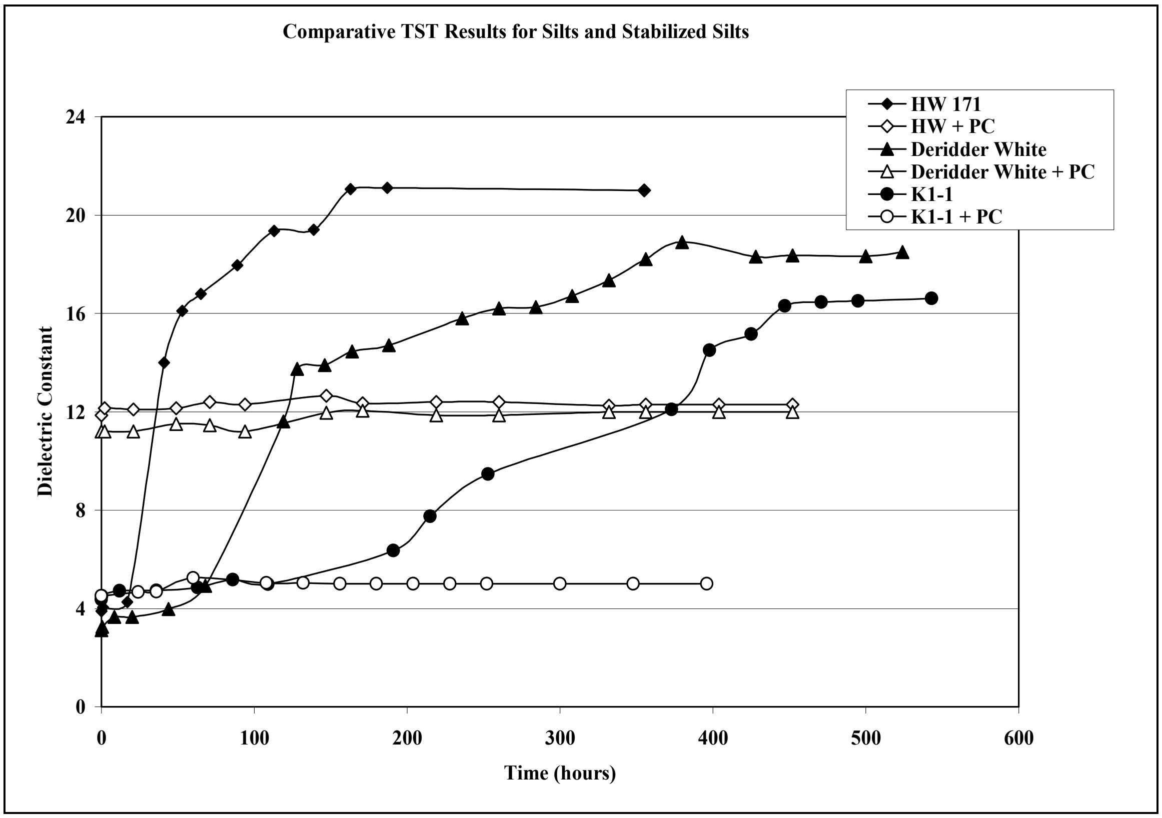

Figure 15

Comparative view on TST results before and after stabilization

The results of these comparative tests illustrate the effect of the stabilizing the soil with Portland cement on the silt's moisture susceptibility. The final dielectric values were reduced from 21 to 12 in the case of HW 171, from 18 to 12 for Deridder White and from 16.5 to 5 in the case of K1-1, lower than 16 for all three soils considered.

By adding 3.5 percent Portland cement, these mixtures pass the criteria of moisture susceptibility and can be considered a reliable material for roads subgrade from this point of view.

A series of tests were considered to determine the influence of different factors on the TST results, such as: tube diameter and compaction energy. Soils used for this stage of study were: HW 171, K1-1 and Deridder White. Specimens were compacted in a tube of smaller diameter of 101.6 mm, using the same Standard Proctor compaction energy. The compared results with those provided from using “classic” tubes (152 mm. diameter) are illustrated in Figure 16.

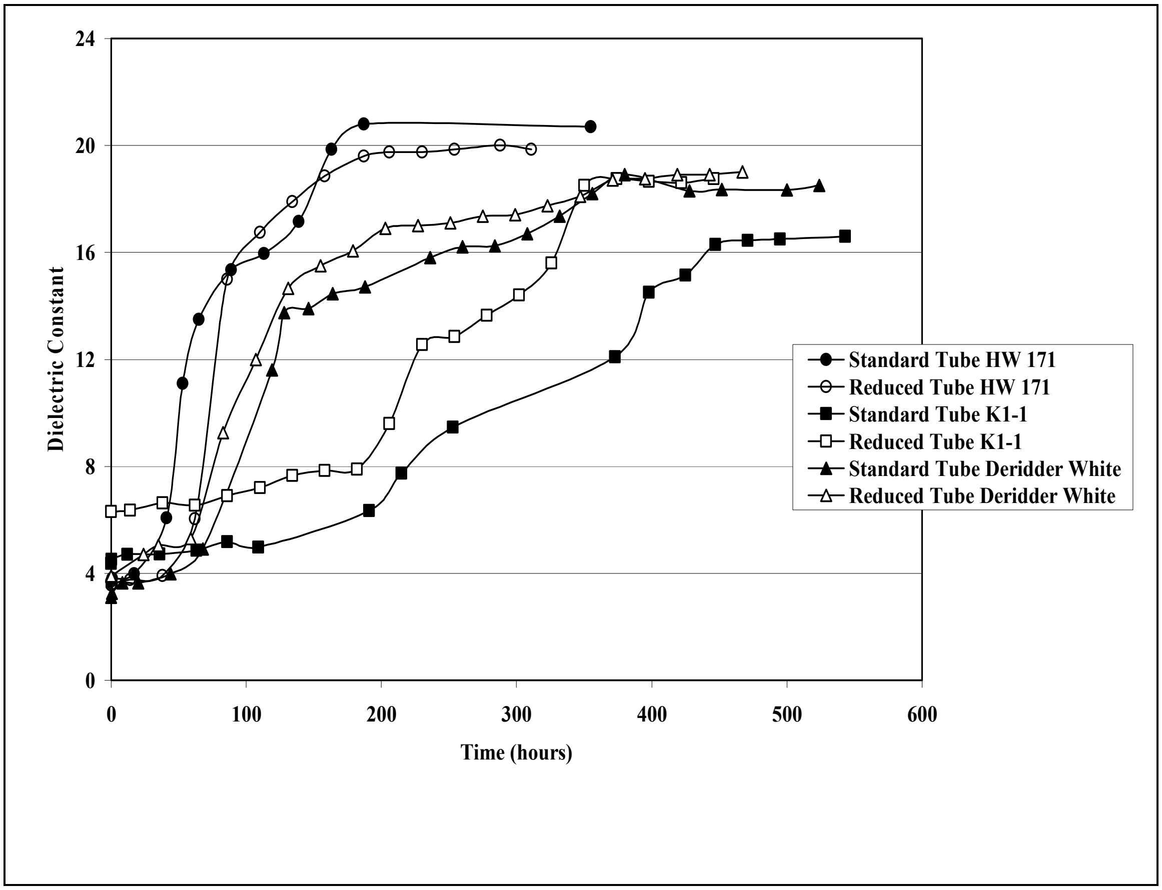

Figure 16

Effect of tube's diameter on TST results

The analysis of figure 16 reveals almost the same evolution for dielectric values for the same samples compacted in the "classic" tubes and the smaller tubes (152 mm and 101.6 mm diameter). The level of final dielectric values and the rate in time of reaching it, is similar for the cases considered: almost 20 in 200 hours for HW 171 compacted in "classic" tubes and smaller tubes, approximate 19 in 400 hours for K1-1 and approximate

17 in 450 hours for Deridder White. These results lead to the conclusion the reduction in tubes diameter has no influence on Tube Suction Test results and smaller tubes can be successfully used in evaluation of moisture susceptibility of silts plus additives.

Specimens of the same soil were compacted using Standard Proctor compaction energy and a modified compaction effort. By varying the number of layers and number of blows per layer, the level of compaction energy was doubled. Typical results of TST obtained in this case are presented in Figure 17 for HW 171.

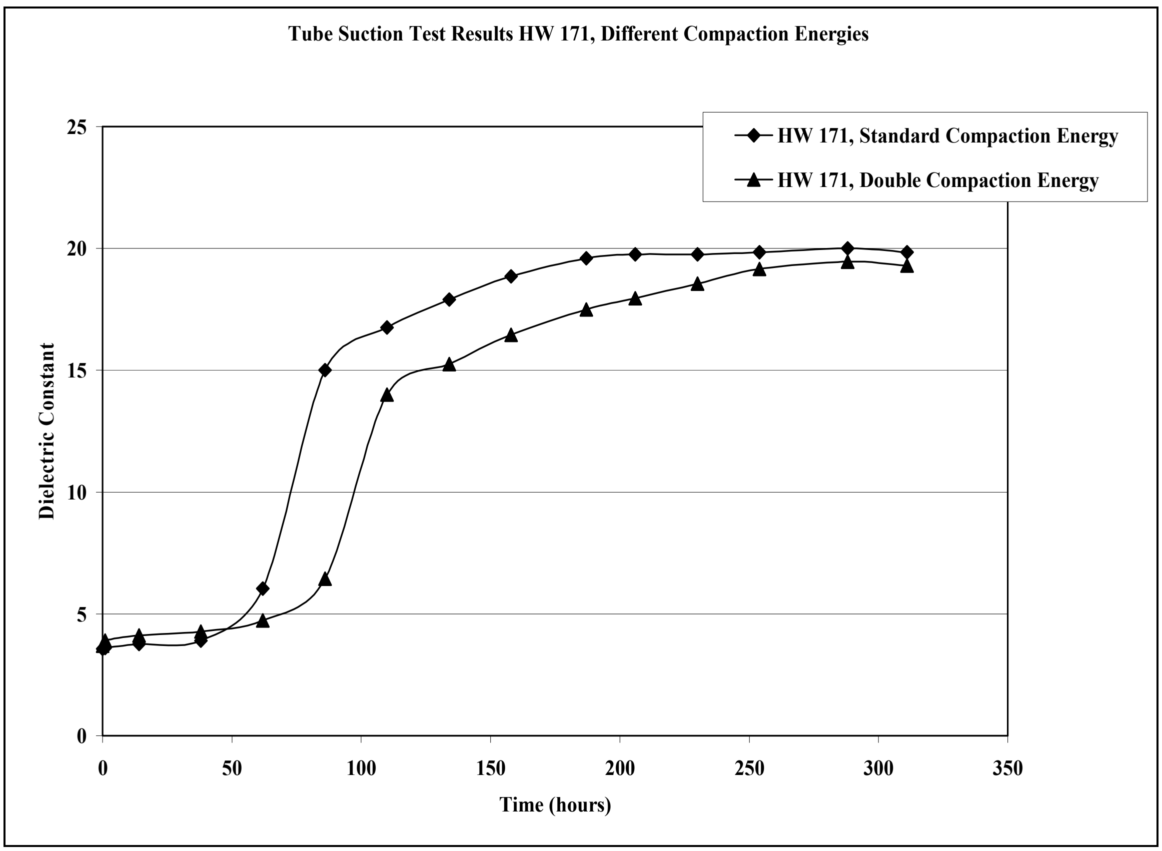

Figure 17

Effect of compaction energy on TST results

The analysis of compaction curves illustrates the fact that for the compaction efforts used in this study, almost the same dry unit weight for this soil at optimum moisture content was produced, which implies the water suction will be developed through almost the same structure of the porous space. In this way the evolution of dielectric values is somewhat similar for samples compacted between the Standard Proctor and the higher energies used in this test.

Moisture susceptibility is an important soil characteristic, which refers to subgrades performance and Tube Suction Tests expand the road design considerations by addressing the problems regarding long term behavior.

Guidelines for silt subgrade stabilization

The last phase of this study consists in an extended series of tests in order to demonstrate the relevance of a proposed set of guidelines for stabilization of problematic silts. The guidelines consist of the following steps:

1. Identification of problematic silts. As presented in the section “Evaluation of pumping phenomenon”, the factors which influence the susceptibility of pumping are: content of 65 percent of silt or higher, a Plasticity Index less then 10 and a minimal bearing capacity reflected by low values of soils strength (approximate 20 psi for the silts considered in this study).

2. Silts stabilization can be achieved by adding different agents like: Portland cement, lime, fly ash. The percentage of additive must provide the right conditions for further development of cementitious products. This can be ensured by a mixture's pH of 10 or higher in case of lime stabilization (30), (31).
3. For lime stabilization the unconfined compression strength of the mixture must surpass the minimal increase of 350 KPa over the original strength of the raw soil (33). For stabilization with Portland cement, the unconfined compression strength of the mixture should be 1380 KPa (200 psi) (36).
4. The moisture susceptibility of the mixture must be reduced in such a measure that the final dielectric values determined using Tube Suction Test should be lower than the threshold value of 16 (25).
5. The evaluation of improving the strength and axial strain for silts before and after chemical stabilization should be illustrated by cyclic triaxial unconsolidated-undrained tests.

This procedure takes into account the concept of moisture susceptibility, which is incorporated with the actual strength criteria for subgrades stabilization. The moisture content for the cases considered in this section is the optimum moisture content corresponding to Standard Proctor compaction energy.

This procedure is demonstrated by presenting a set of typical results with the use of Acadia, Chase White and K2-1. The results for the entire set of silts are presented in the Appendix F.

Stabilization of Acadia

1. Identification of Acadia as problematic silt.

Gradation curve for this soil, presented in Figure 18, revealed a high silt content of 65 percent and the Plasticity Index is only 3, i.e., less than 10.

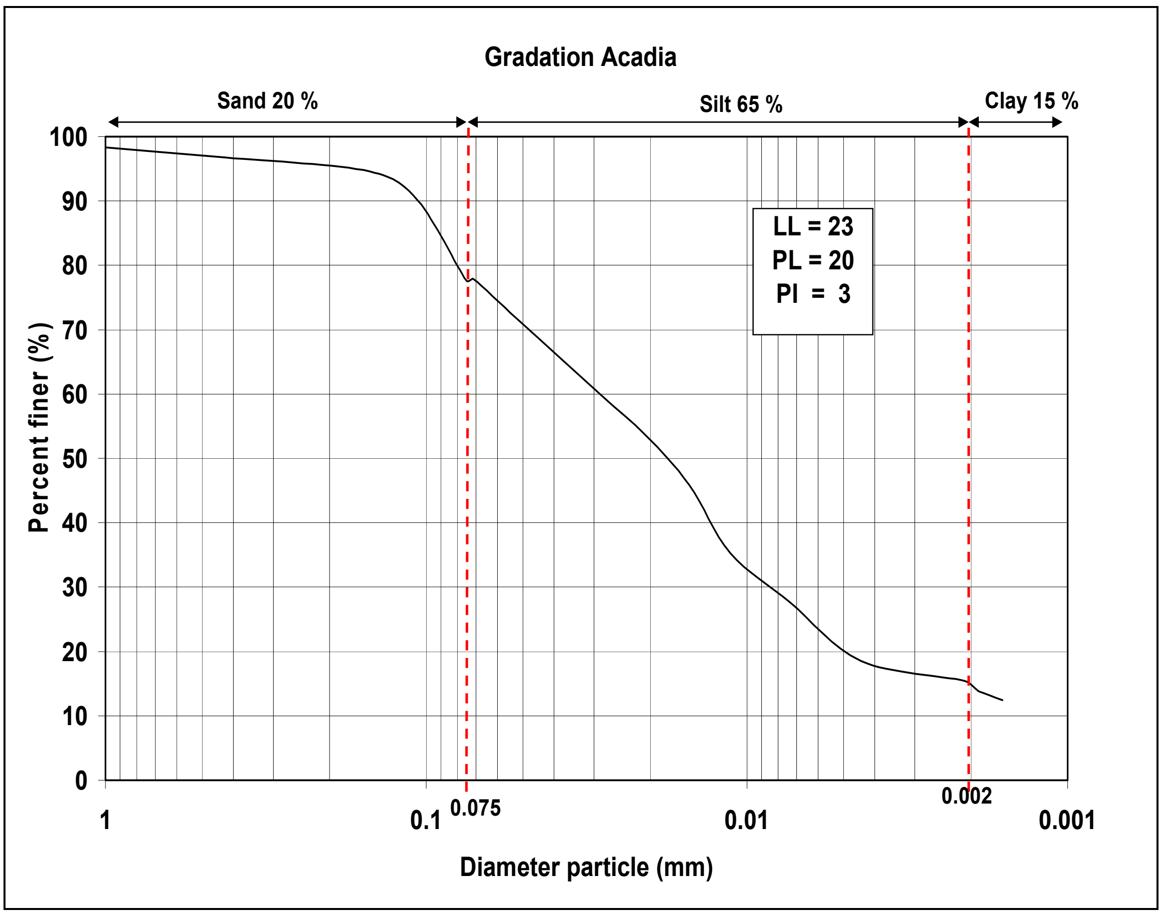

Figure 18

Gradation curve for Acadia

The results of cyclic triaxial tests presented previously in the section “Evaluation of pumping phenomenon” are illustrated in a comparative manner in Figure 19 for Acadia, Chase and Deridder Brown:

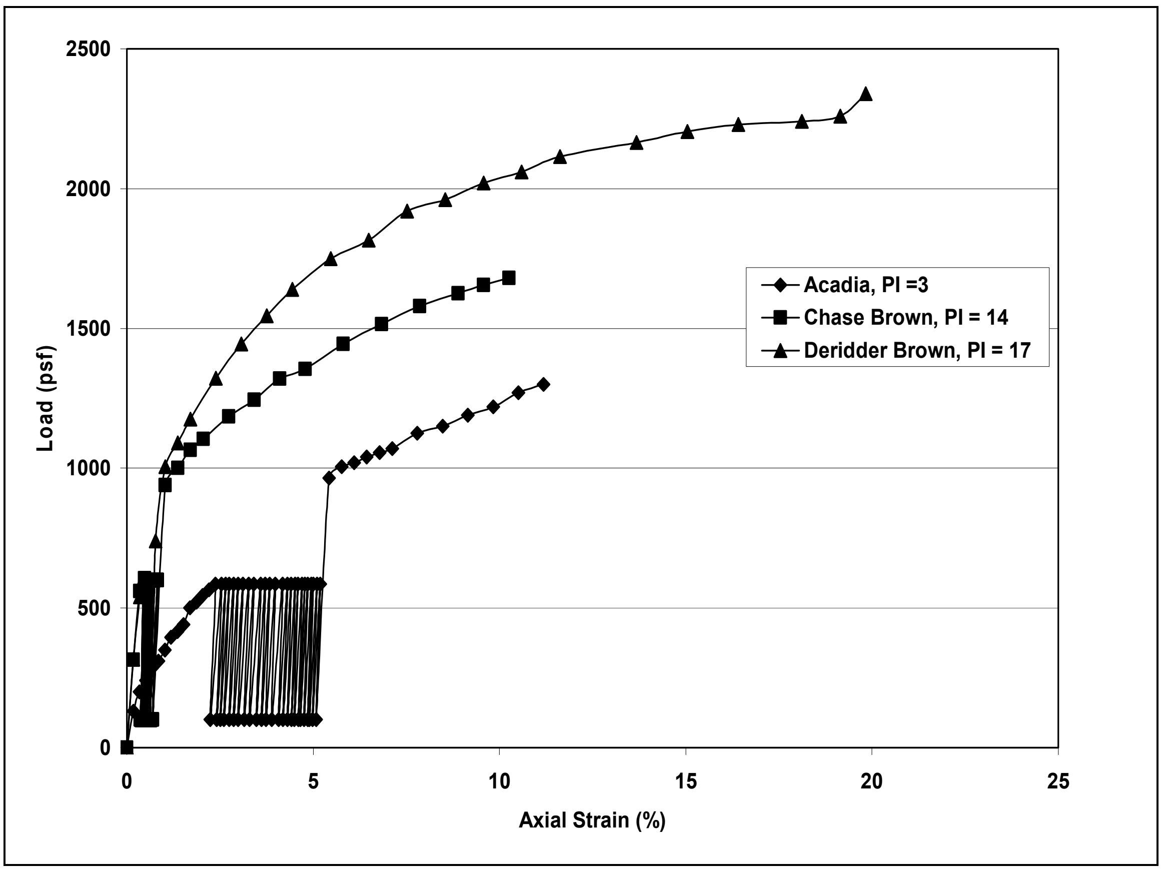

Figure 19

Comparative evolution during cyclic triaxial tests for Acadia, Chase Brown and Deridder Brown

The analysis of Figure 19 reveals the accumulated values of axial strain are larger for Acadia (ML/A-4 with PI = 4) than for the Chase Brown and Deridder Brown, soils with higher plasticity (CL/A-6 with PI = 14 respectively PI = 17).

A specimen of Acadia was molded at optimum moisture content using Harvard Miniature Apparatus. Compaction effort was similar to Standard Proctor compaction. Unconfined compression strength was determined to be 22.2 lb/in^2, as presented in Figure 20.

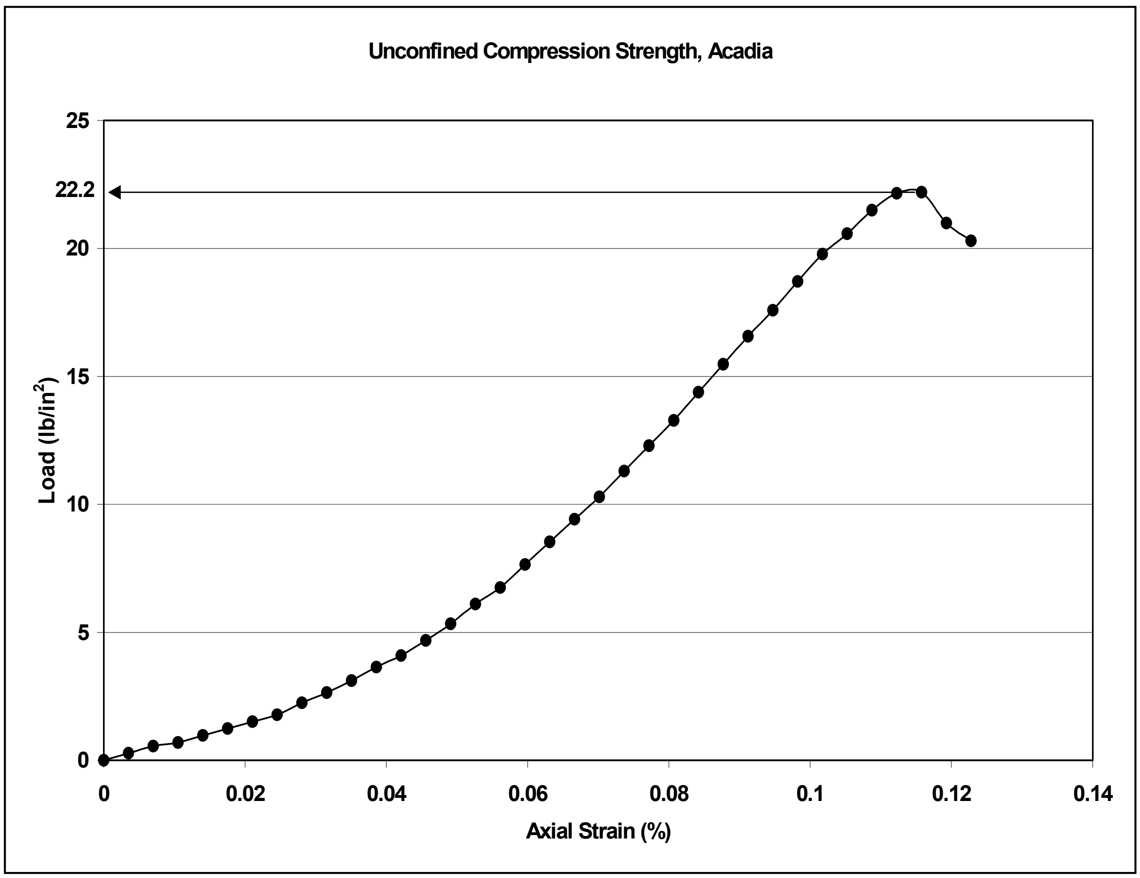

Figure 20

Unconfined Compression Strength, Acadia, raw soil

2. Chemical Stabilization.

For an effective process of modification with lime and further stabilization, a relatively consistent percentage of clay would be required in order to interact with lime and generate substantial amount of pozzolanic products (30). In this case, Acadia contains only 15 percent clay, which leads to the conclusion that Portland cement could be considered a more effective stabilizing agent. Limited quantities of soils available couldn't allow comparative tests between efficiency of cement, lime or lime + fly ash.

Different percentages of cement were considered for Acadia stabilization: 2 percent, 3.5 percent and 5 percent. Determination of cement content for the soil stabilization is not based on pH test, as in the case of lime stabilization (31), (33). Strength criteria govern the selection of cement percentage to be added to the soil, in order to obtain minimum compressive strength of 200 psi for the mixture (36).

3. Strength Criteria

Mixtures specimens are compacted at optimum moisture content using Standard Proctor energy and subjected to unconfined compression test.

The Standard Proctor compaction curves for Acadia and Acadia + 5 percent Portland cement are illustrated in Figure 21. The optimum moisture content and maximum dry unit weight are very close for both cases: 14.2 and respectively 15.2 percent for raw soil and respectively Acadia + 5 Percent cement and maximum density of approximately 111 lb/ft^3 for both cases.

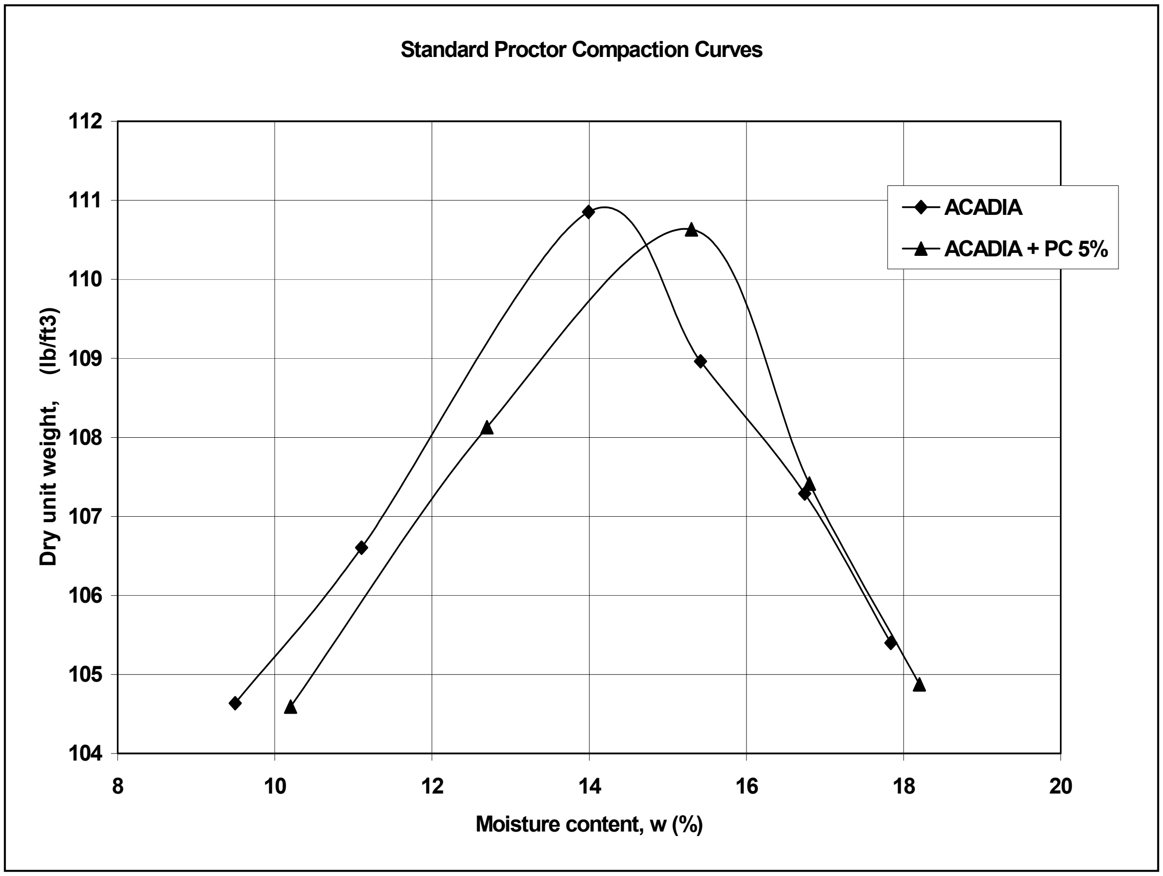

Figure 21

Standard Proctor Compaction Curves for Acadia and Acadia + 5% PC

As it was presented above, the optimum moisture content of the mixture is 15.2 percent and maximum dry unit weight is 110.8 lb/ft^3, which correspond to a mixture's wet weight of 127 lb/ft^3. In order to determine the strength of the mixture in this state, a specimen of Acadia + 5 percent cement was molded using the Harvard Miniature Apparatus.

The comparative results of the unconfined compression test for the raw soil and for Acadia + Portland cement 5% are illustrated in Figure 22.

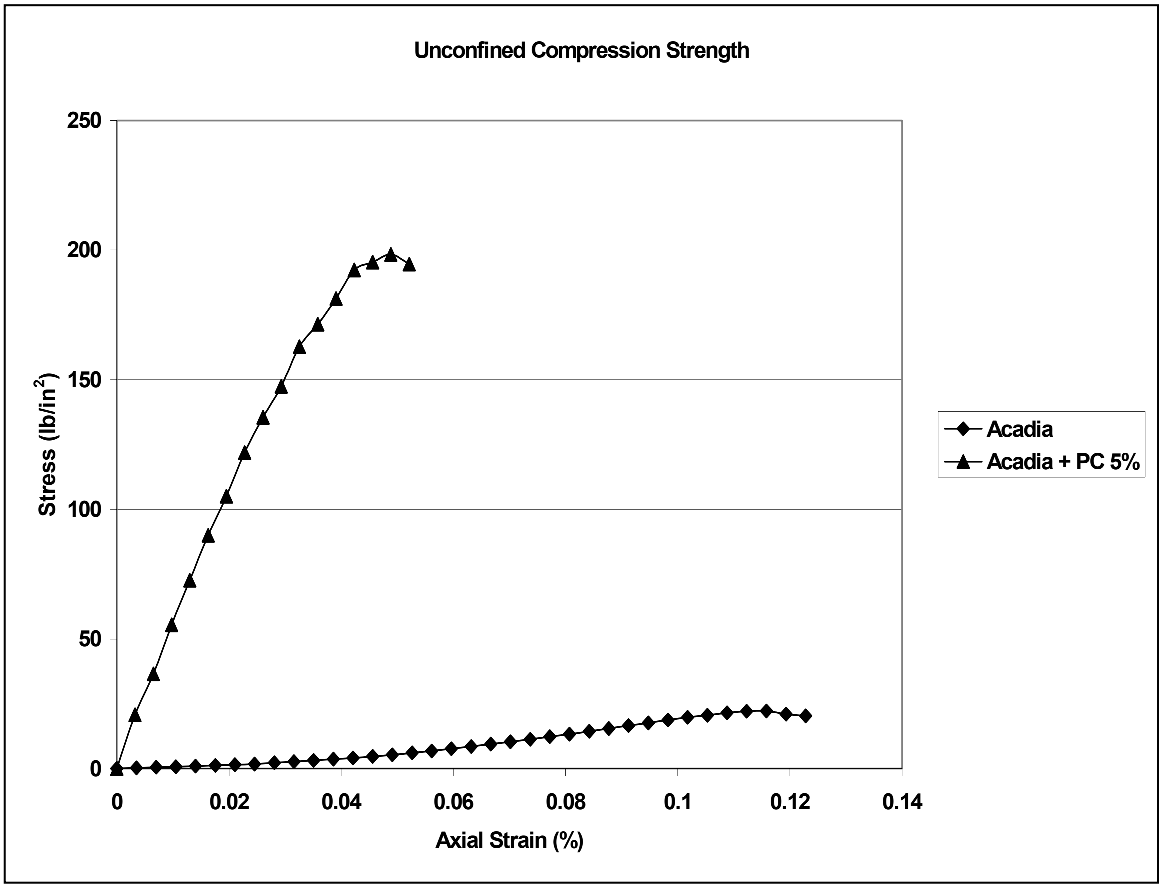

Figure 22

Strength for Acadia and Acadia + PC 5%

In this case, Acadia mixed with 5 percent by weight Portland cement met the strength criteria of 200 psi as minimum compressive strength (The exact value is 198.3 psi, which can be considered sufficient).

4 Moisture Susceptibility

As it was presented before, the Tube Suction Test provides an estimate of the soil's moisture susceptibility. The effect of additives to increase the soils moisture resistance is determined by the reduction of final dielectric values under 16.

Figure 23 illustrates the comparative results for Acadia and Acadia + 5 percent cement subjected to Tube Suction Test. Last specimen was sealed after molding and placed in humidity room for 28 days in order to allow the formation of cementitious products.

The analysis of Figure 23 revealed the reduction of final dielectric values under the value of 16, for the stabilized specimen. In this way, the percentage of chemical agent considered for stabilizing the Acadia soil not only provides the required strength but also decreases the mixture's moisture susceptibility at new levels which can be considered safe for subgrades long term service.

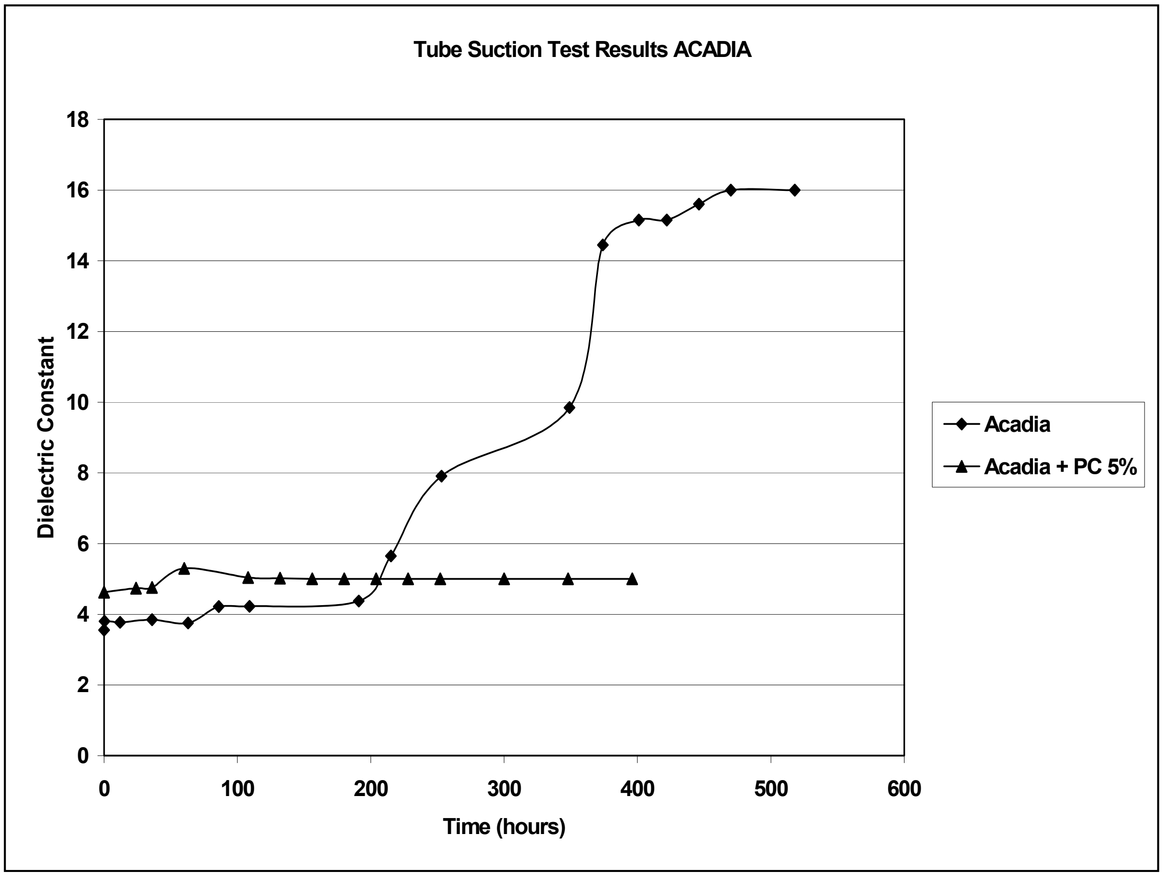

Figure 23

Comparative final dielectric values for Acadia and Acadia + 5% cement

5. Cyclic triaxial tests.

Improvement in the stabilized silts can be demonstrated by cyclic triaxial undrained – unconsolidated tests. The long-term performance can be assessed for cured specimens. The conditions of the tests simulate those encountered by the subgrade during the road's service. Mixtures samples were prepared and cured for 28 days in humidity room, placed in triaxial chamber, saturated and subjected to cyclic triaxial tests during 200 cycles.

The results were compared for the silt before and after the stabilization. The variation of axial strain and strength is illustrated in Figure 24.

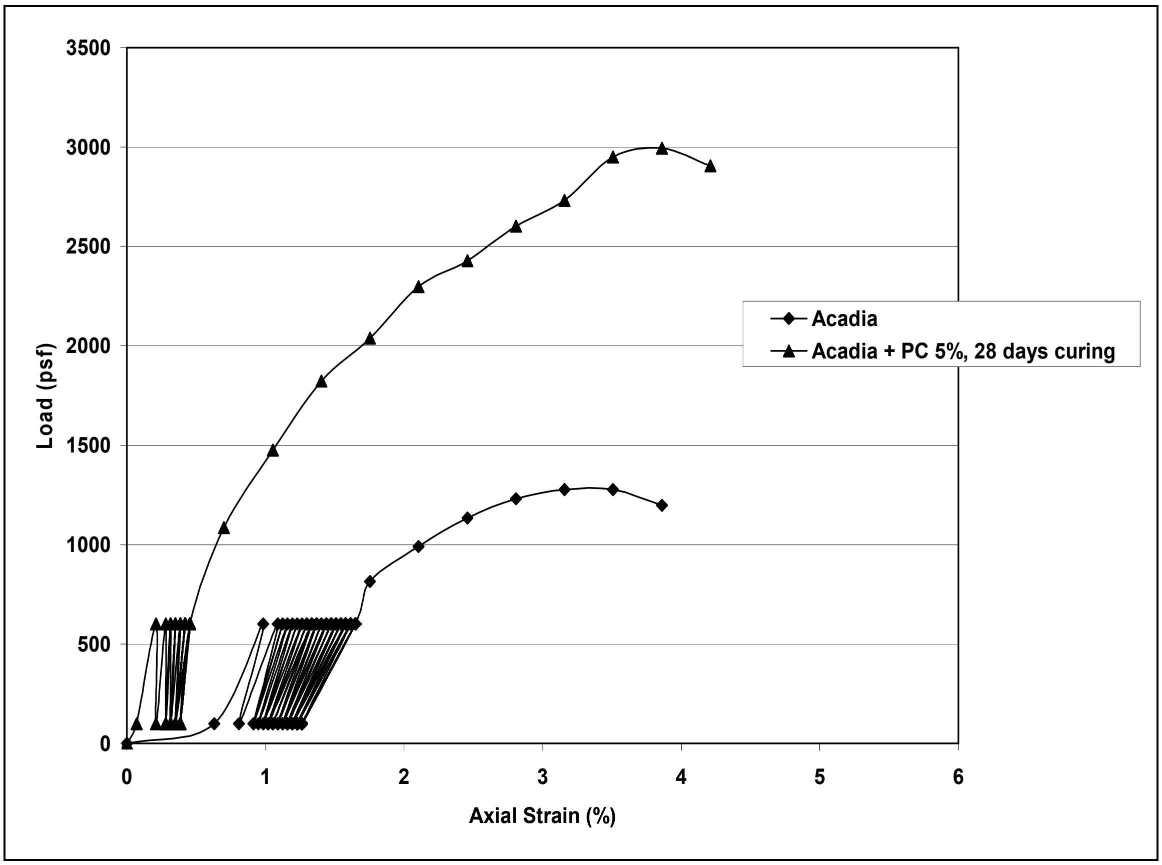

Figure 24

Cyclic triaxial tests for Acadia before and after stabilization with cement

The results indicate a reduction of 268 percent in axial strain from 1.02 percent for raw soil to 0.38 percent for Acadia + cement. The gain in ultimate strength for silt and additive is up to 3000 psf, comparing with 1277 psf for raw soil. The addition of Portland cement leads to a significant improvement of Acadia as a road subgrade, even in the most unfavorable conditions, when the soil is almost completely saturated and subjected to cyclic loading during traffic.

In this case, the percentage of chemical agent added to the problematic silt accomplished the stabilization of Acadia soil by meeting both strength and moisture susceptibility criteria: the unconfined compression strength of the mixture was 200 psi and the final dielectric values determined by Tube Suction Test were below 16 as critical value (36) and (24).

Stabilization of Chase White

1. Identification of Chase White as problematic silt

The soil considered in this case exhibits 72 percent silt which exceeds the indications from Louisiana Standard Specifications for Road and Bridges regarding the gradation of the soils suitable for road subgrades (1). The gradation curve for Chase White is illustrated in Figure 25.

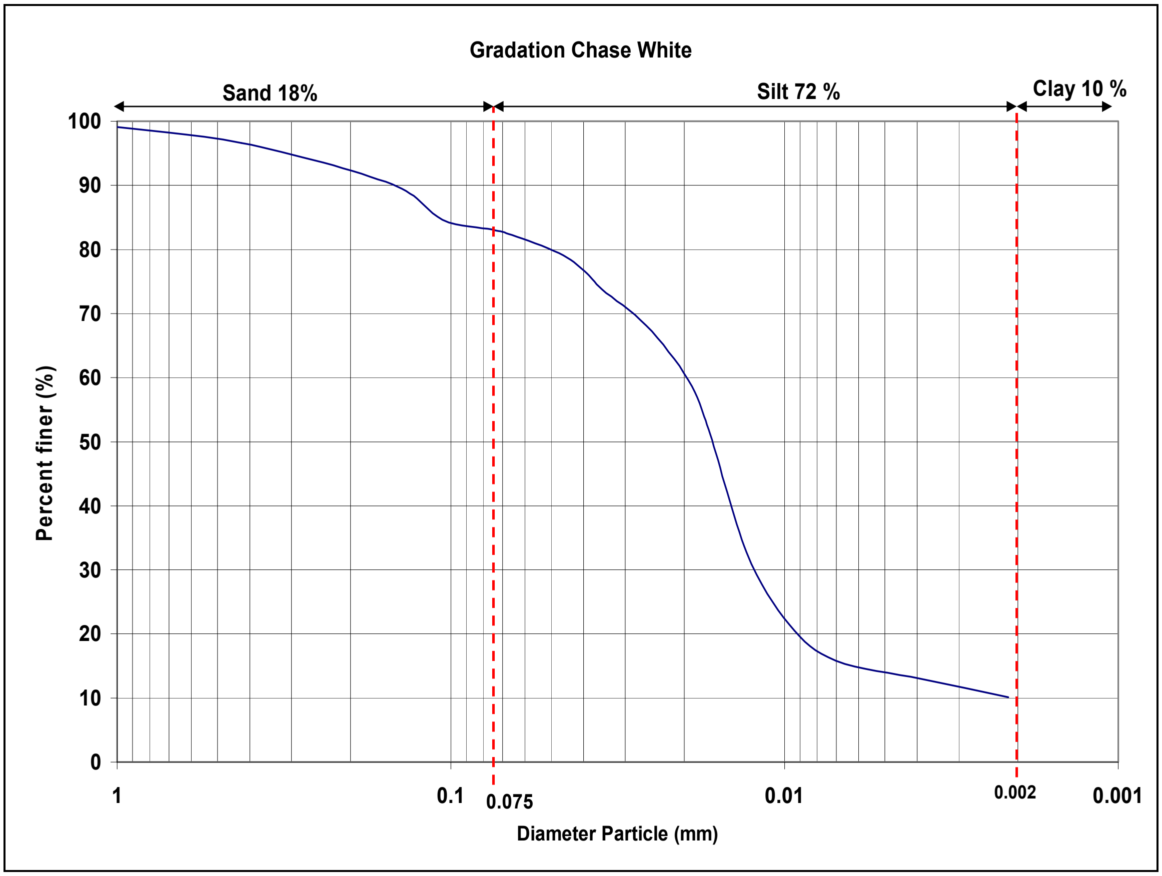

Figure 25

Gradation curve for Chase White

The results of cyclic triaxial tests presented previously in the section “Evaluation of pumping phenomenon” are illustrated in a comparative manner in Figure 26 for Chase White, Chase and Deridder Brown.

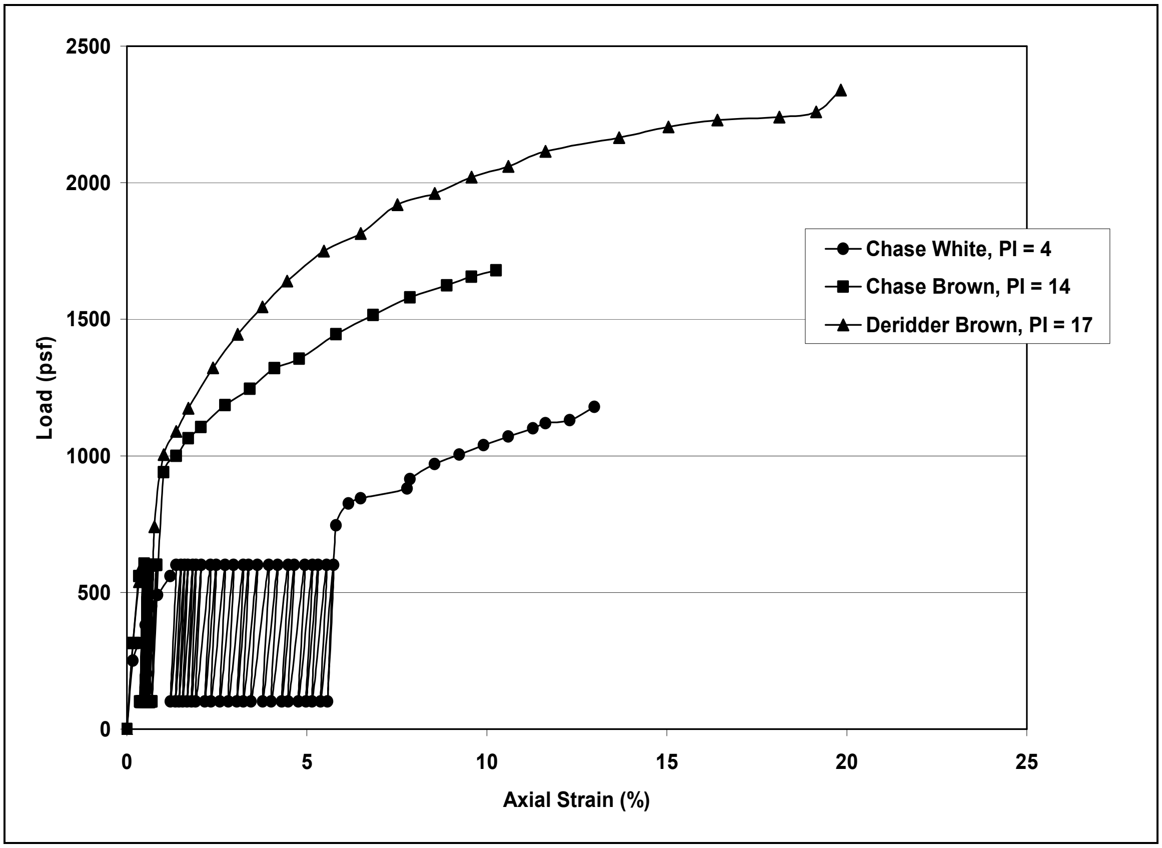

Figure 26

Comparative evolution during cyclic triaxial tests for Chase White, Chase Brown and Deridder Brown

A specimen of Chase White was molded using Harvard Miniature Apparatus at conditions corresponding to optimum moisture content of 15 percent and then subjected to unconfined compression test. The resulting strength was determined to be 24.2 psi, as presented in Figure 27.

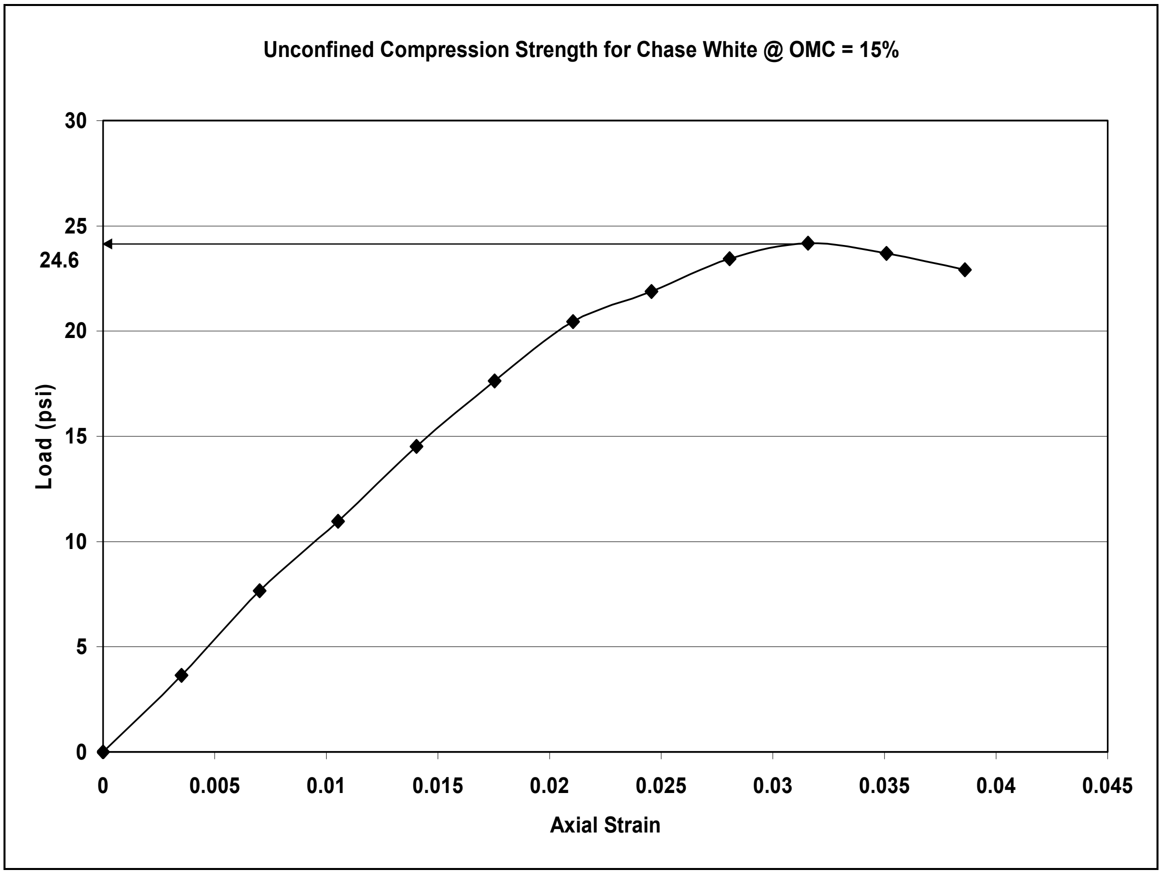

Figure 27

Unconfined Compression Strength for Chase White

The low values of PI (less than 10), reduced strength and extensive axial strain during cyclic triaxial loads lead to the conclusion that Chase White represents a problematic silt which is highly susceptible to pumping.

2. Chemical stabilization

Like Acadia, Chase White exhibits a small content of clay (only 10 percent), which implies lime stabilization is much less effective as cement stabilization. Based on this consideration, 5 percent Portland cement was mixed with Chase White. The Standard Proctor Compaction Test was conducted for this mixture in order to determine its optimum moisture content and maximum dry unit weight. The results are presented in Figure 28.

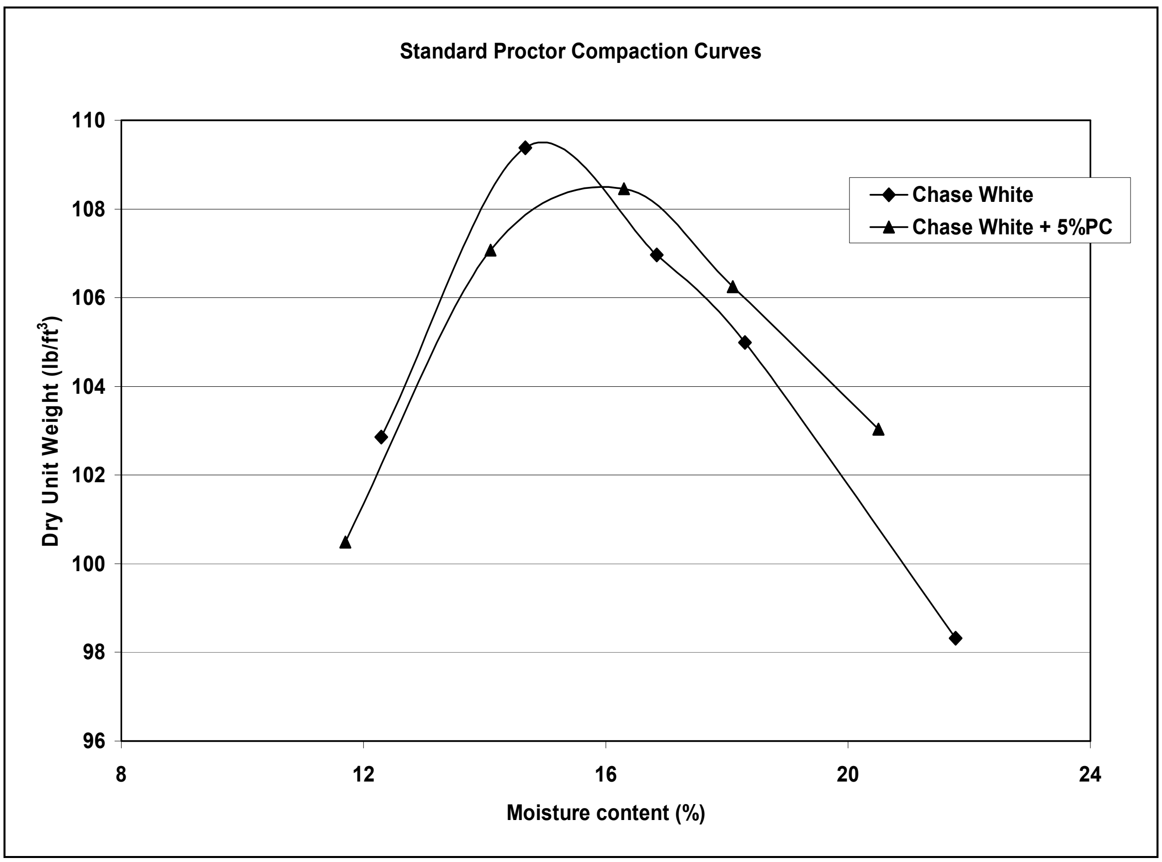

Figure 28

Standard Proctor Compaction Test for Chase White before and after cement stabilization

3. Strength Criteria

The mixtures maximum dry unit weight of 109 lb/ft^3 corresponds to the optimum moisture content of 16.3 percent and the wet unit weight of 126.8 lb/ft^3. A specimen of Chase White + 5% cement was molded at these parameters and subjected to Unconfined Compression Test after 48 hours curing at 50^0 C. The results are illustrated in Figure 29.

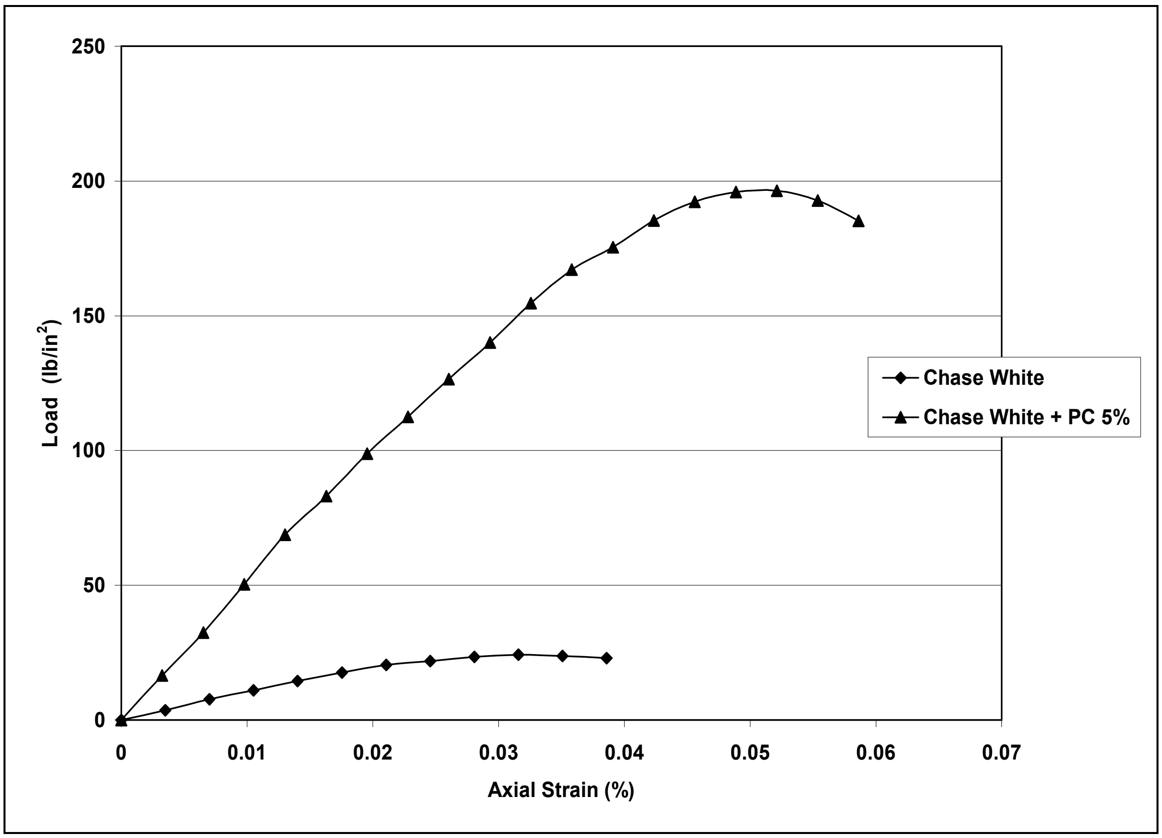

Figure 29

Effects of cement stabilization on strength for Chase White

The analysis of Figure 29 indicates the mixture of Chase White and 5 percent Portland cement met the strength criteria of 200 psi for stabilized subgrade. (The exact value is 196.4 psi, which can be considerate a good approximation).

4. Moisture Susceptibility

A mixture sample of Chase White and 5% cement was subjected to Tube Suction test in order to determine its compliance with the moisture susceptibility criteria. The final dielectric values of the raw soil were compared with those for the stabilized soil. The results are presented in Figure 30.

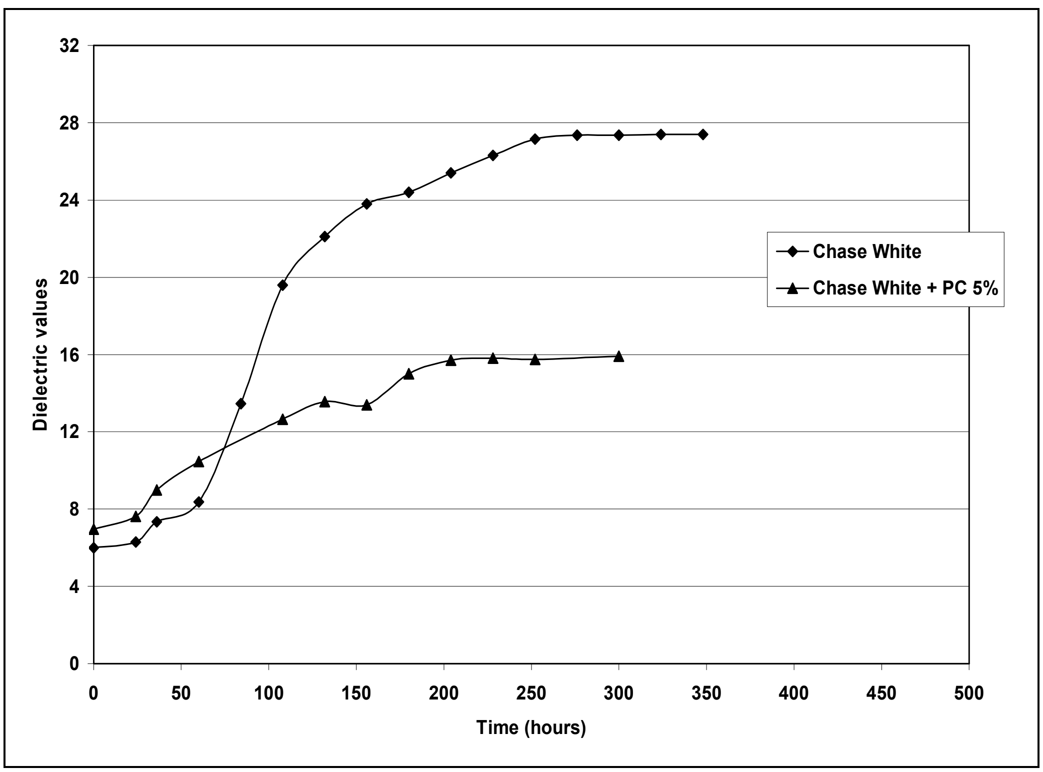

Figure 30

Comparative TST results for Chase White and Chase White + PC 5%

The Tube Suction Tests results for these two cases considered indicate the silt stabilized with 5 percent Portland cement is still highly moisture susceptible. The final dielectric values are reduced from 27.5 for raw soil to 15.8 for Chase White + 5 percent cement, but still this indicates the moisture susceptibility criteria is not met. A further increase in the additive percentage is expected to reduce the final dielectric values under 16, which is considered a threshold value for soils moisture susceptibility (24).

A second set of specimens were prepared for Chase White and 7 percent Portland cement. Unconfined Compression tests and Tube Suction Tests were repeated for this new mixture in the same conditions as previously described. The comparative values for strength are illustrated in Figure 31.

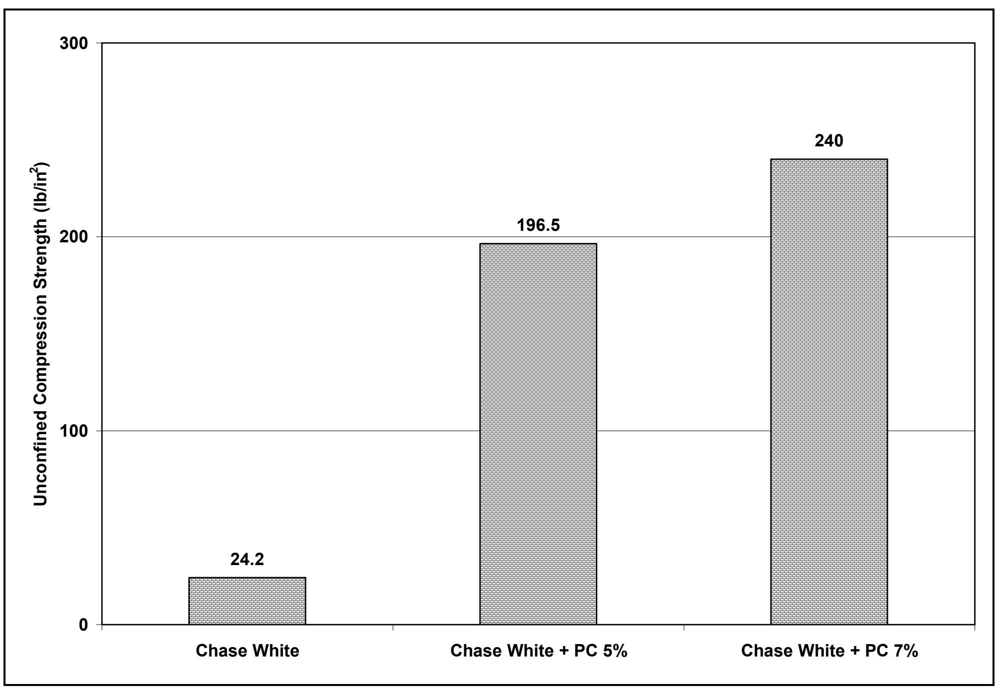

Figure 31

Evolution of strength for Chase White stabilized

The comparative results of Tube Suction Test for Chase White + 7 percent Portland cement are presented in Figure 32.

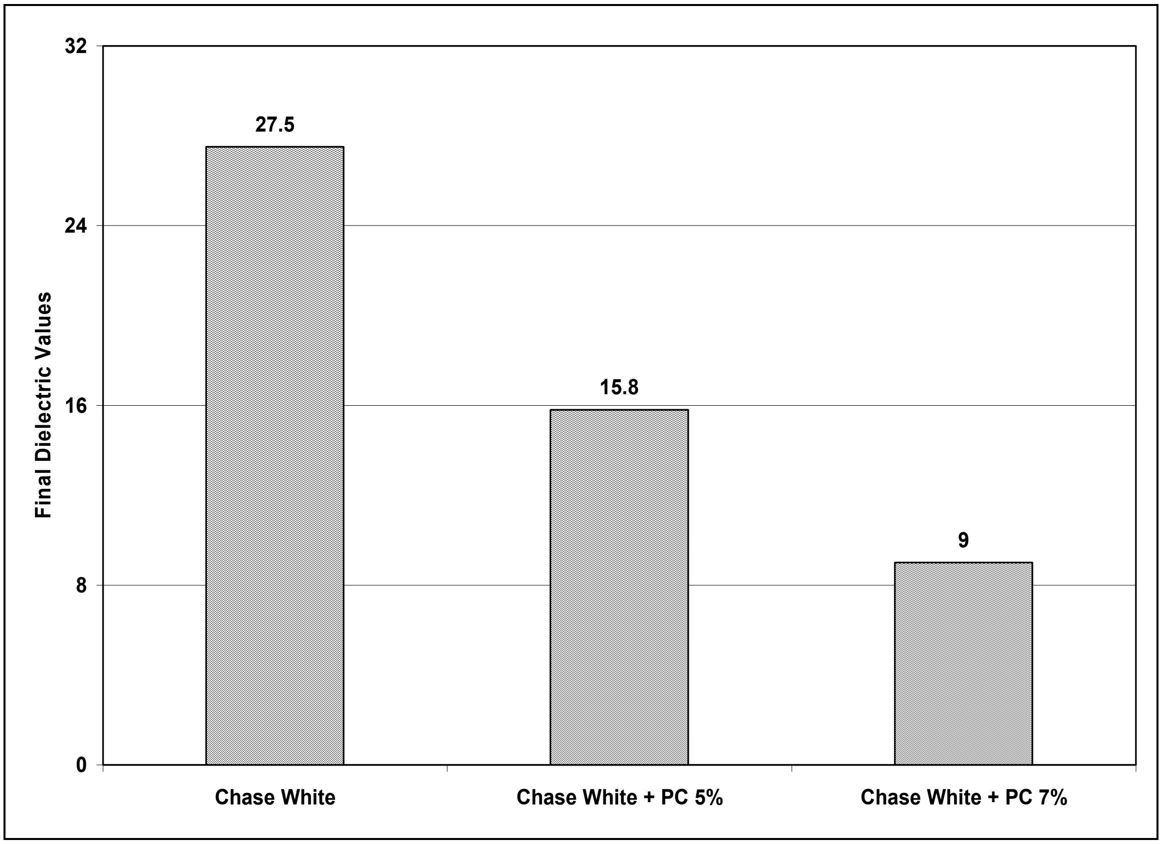

Figure 32

Effect of cement stabilization on final dielectric values for Chase White

In this case, the initial quantity of 5 percent Portland cement provided the appropriate solution for the stabilization of Chase White, based on strength criteria as considered in the current specifications. However, the final dielectric values for this mixture indicate that Chase White + 5 percent cement still exhibit high moisture susceptibility, demonstrating the importance of implementing this new concept in the stabilization design. A higher amount of stabilizer, i.e. 7 percent cement, ensures the compliance of both strength and moisture susceptibility criteria for this soil.

As presented in Figure 25, the silt content of Chase White is 72 percent. The criteria regarding the soils subgrade layer to be stabilized, presented in “Louisiana Standard Specifications for Road and Bridges, 2000 Edition”, indicate the content of silt less than 69 percent (1). However, the results presented for this particular case suggest the stabilization of soils with higher content is possible, with good results from the point of view of strength and moisture susceptibility.

Stabilization of K3-1

As it was presented before, lime stabilization can improve significantly the subgrades engineering properties, especially in the case of clay soils of moderate to high plasticity. The soil stabilization occurs due to the exchange of calcium cations adsorbed on the surface of the clay mineral in a high pH environment created by the lime – water system, generating cementitious products (28). Research has shown that the pozzolanic reaction can continue for a long period of time, even years, as long as the pH remains high (above 10) (30).

From the range of soils considered in this study as problematic silts (exhibiting low plasticity, i.e. PI<10, high content of silt and minimal bearing capacity), Natchitoches K3-1 contains the highest percentage of clay, i.e. 18 percent, which suggests lime stabilization could be effective in this case.

A comparative study of stabilization results for lime and Portland cement mixed with K3-1 is considered. The mixtures met both strength criteria and moisture susceptibility criteria for specimens tested at conditions corresponding to optimum moisture content and maximum dry unit weight. These results, provided by Standard proctor Compaction tests are presented in Figure 33.

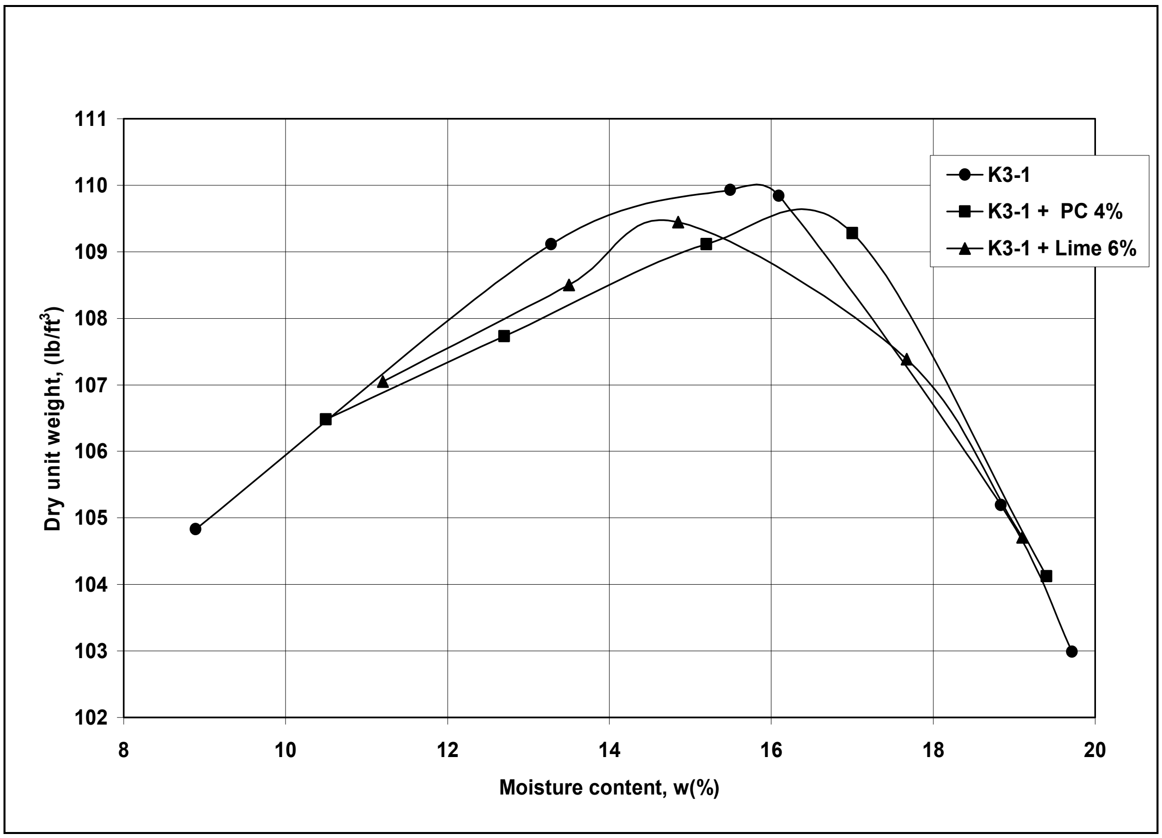

Figure 33

Standard Proctor Compaction curves for K3-1 and K3-1 stabilized with Lime 6% and Portland cement 4%

The compaction curves provide the optimum moisture content and the corresponding wet weight for the specimens subjected to unconfined compression test and Tube Suction tests. The analysis of Figure 33 indicates the addition of 6 percent lime or 4 percent Portland cement does not have a significant influence on the soils maximum dry unit weight or optimum moisture content, these parameters being placed in the range of 109.5 lb/ft^3 and respectively 16 percent.

1. Identification of K3-1 as problematic silt

As it was presented before, K3-1 exhibits a high content of silt, (64 percent), a low plasticity, i.e. PI < 10 and a reduced value of strength. Figure 34 presents the gradation curve.

2. Chemical stabilization

Two solutions are considered for this problematic silt: mixing with lime 6 percent lime and Portland cement 4 percent. The efficiency of these chemical additives is compared. Preliminary analysis demonstrated the mixtures of K3-1 and these percentages of stabilizing agents met both strength and moisture susceptibility requirements, according to the presented guidelines.

The pH for the mixture of K3-1 + Lime 6 percent was measured as 12.4 using pH meter Adekplus. This situation determines favorable conditions for further development of pozzolanic products, as lime stabilization is a long – term process (28), (29).

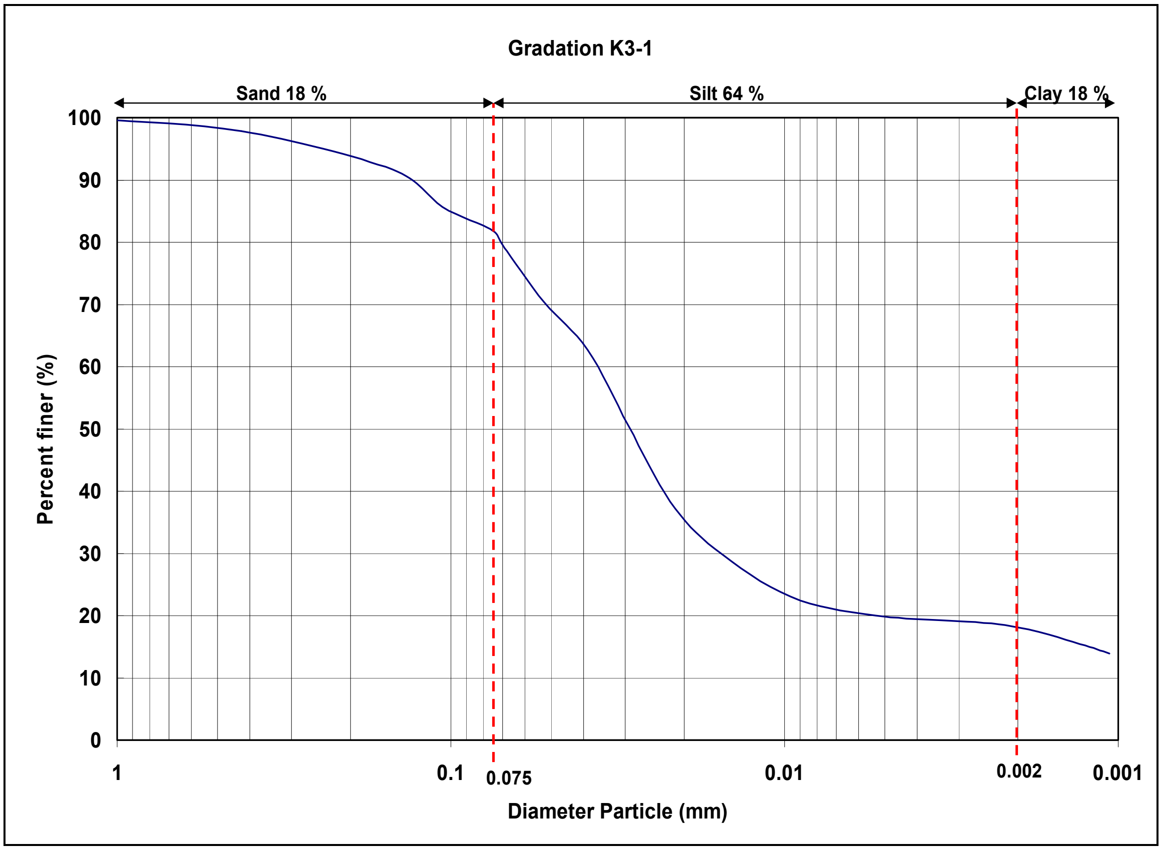

Figure 34

Gradation curve for K3-1

3. Strength criteria

As presented in previous cases, unconfined compression tests were conducted for mixtures specimens, molded at corresponding optimum moisture content, using Harvard Miniature Apparatus. Compaction energy was equivalent to Standard Proctor Compaction. Comparative results are presented in Figure 35 for both mixtures.

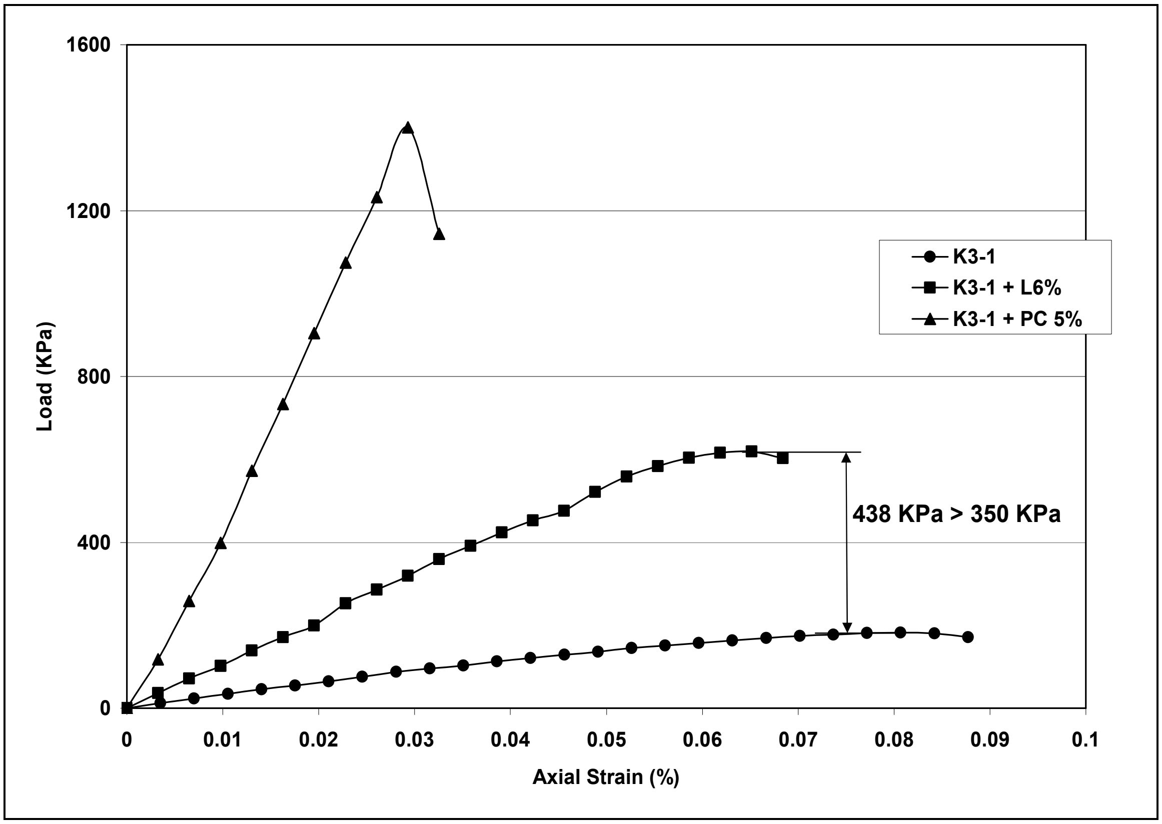

Figure 35

Comparative results of strength for K3-1 stabilized with lime 6% and cement 5%

By adding 6 percent lime, the strength is increased from 182 KPa corresponding to raw soil to 620 KPa for K3-1 + chemical stabilizer. The difference exceeds 350 KPa, which represents a measure of efficiency for lime stabilization (31). Also, the strength for the second mixture, K3-1 + 4 percent Portland cement, is 1450 KPa (210 lb/in^2), which meet the strength criteria for cement stabilization (36).

4. Moisture susceptibility

As the strength criteria were met for both mixtures, their moisture susceptibility was evaluated from Tube Suction Test. The comparative results of final dielectric values are presented in Figure 36.

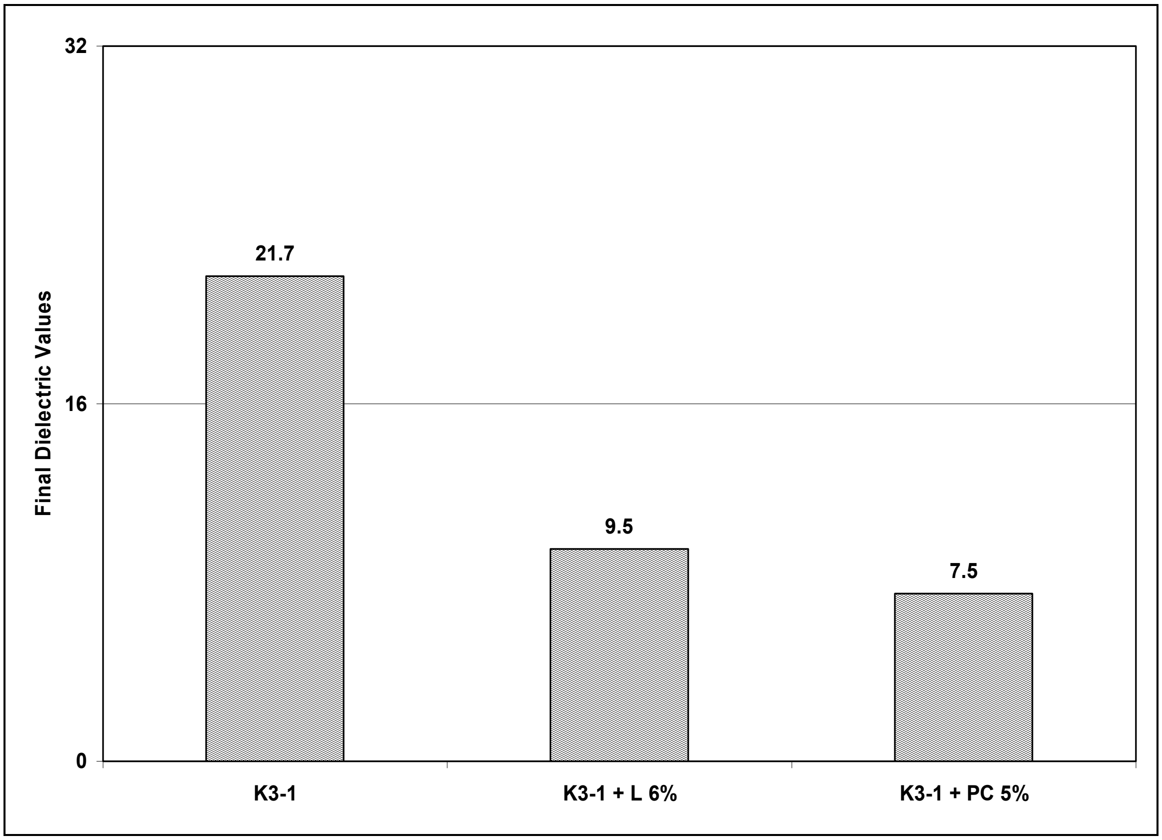

Figure 36

Effect of chemical stabilizers on final dielectric values for K3-1

Both additives decreased the final dielectric values under the critical value of 16 indicating a reduced moisture susceptibility for the mixtures considered.

5. Cyclic triaxial tests.

As presented before, cyclic triaxial tests provide an image regarding the stabilization efficiency, especially with respect to strength increase and reduction of axial strain as a manifestation of pumping phenomenon. The comparative results of these tests for K3-1 before and after stabilization with Lime 6 percent and cement 5 percent are presented in Figure 37

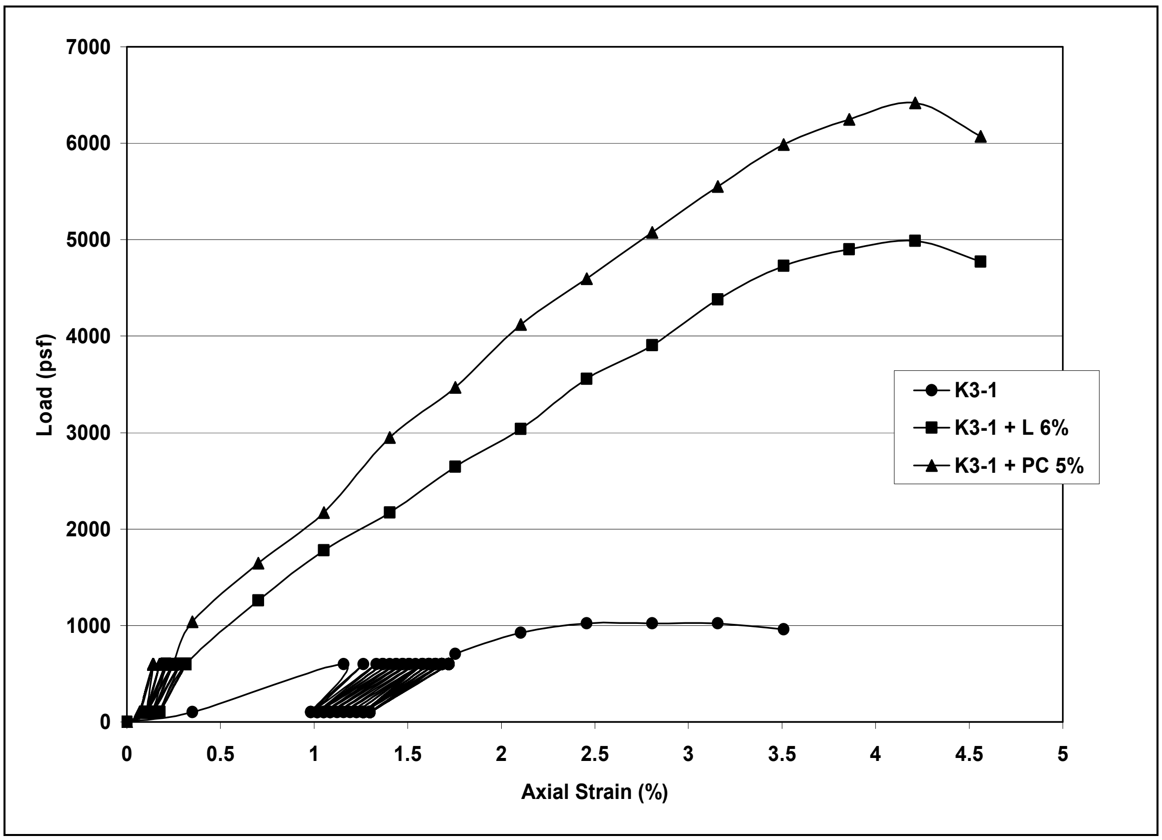

Figure 37

Effect of additives on cyclic triaxial tests for K3-1

The analysis of Figure 37 indicate the substantial reduction of axial strain induced by the cyclic loading for specimens of K3-1 stabilized with lime 6 percent and cement 5 percent, comparing with the specimens of raw soil. The strength of these mixtures reached values of 5000 lb/ft^2 and respectively 6420 lb/ft^2, much higher than the value of 1000 lb/ft^2 corresponding to the raw soil.

These results, corroborated with the data from the steps presented above, indicate the efficiency of lime and cement as stabilizing agents for K3-1.

As it was mentioned before, the effectiveness of soil's stabilization with lime is influenced by its content of clay. Calcium from the lime reacts with aluminates and silicates solubilized from the clay mineral surface, leading to the formation of calcium silicate hydrates and calcium aluminate hydrates as products of a long term pozzolanic reaction in a high pH environment. Natchitoches K3-1 exhibit the highest content of clay (18 percent) from the set of problematic silts considered in this study, with high axial strain due to cyclic triaxial load and low plasticity ($PI < 10$). The long term evolution of strength for K3-1 samples stabilized with 6 percent lime and 4 percent cement was compared in order to determine if lime can be a substitute for cement for this soil, as long as both chemical additives met the strength criteria and moisture susceptibility criteria. Samples of these two mixtures were subjected to unconfined compression tests after 1 year curing in the humidity room. The comparative results are presented in Figure 38.

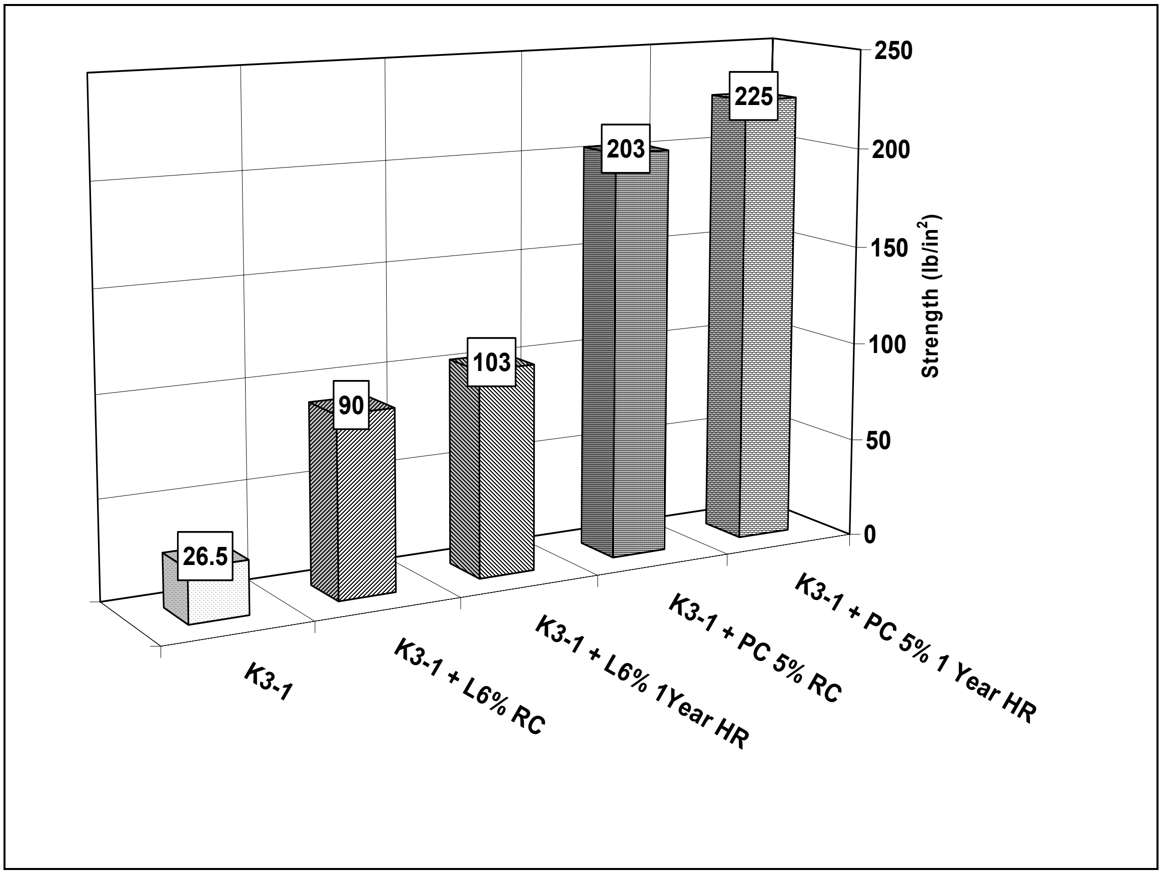

Figure 38

Effect of additives and curing period on K3-1

The notations used in this figure are as follows: K3-1 + L6% RC represents sample of K3-1 mixed with 6 percent lime, and cured at 50 ^{0}C for 48 hours (rapid Curing); K3-1 + L 6% 1 Year HR represents the same mixture cured for 1 year in the humidity room (HR). Same notations are used for K3-1 + 4 percent Portland cement. All specimens tested were compacted using Harvard miniature Apparatus at optimum moisture content and maximum dry unit weight corresponding to Standard Proctor Compaction curve.

The analysis of Figure 38 indicates a reduced gain in strength for K 3-1 plus lime 6 percent, from 90 to 103 lb/in^{2}after 1 year curing. These values lead to the conclusion that the content of clay for Natchitoches K3-1 is insufficient in the reaction with lime. The low level of pozzolanic products generated cannot trigger a satisfactory gain in strength in order to replace the Portland cement as stabilizer for this soil. Based on these considerations, the testing program for the rest of the silts was focused only on cement stabilization.

The results of stabilization testing program for the rest of the problematic silts considered in this study are presented in Table 11. These data indicate that the addition of Portland cement increases the mixture's strength up to 200 lb/in^2, considered as a more representative value for strength criteria (36). Also, the initial moisture susceptibility is drastically reduced, final dielectric values of stabilized soils are smaller than 16.

In the case of K1-1, Chase White, Deridder White and HW 171, the moisture susceptibility criteria is met by adding relatively small percentages of chemical additive (from 3.5 percent cement for Deridder White and HW 171 to 5 percent cement for other two soils). However, these initial amounts of stabilizer are not sufficient for these mixtures to comply with strength criteria. The subsequent supplementation of cement content increases the values of mixtures strength up to 200 lb/in^2 and reduces furthermore the final dielectric values.

Table 11

Cumulative results for problematic silts stabilization

Soil	Stabilizer Agent	Strength (lb/in^2)	Final Dielectric Values
K1-1	-	14	32
	PC 5%	170	5
	PC 7%	205	5
K2-1	-	24.5	
	6%	203	12.5
K3-1	-	26.3	21.7
	PC 5%	203	7.5
ACADIA	-	22.2	16
	PC 5%	198	5
CHASE WHITE	-	24.6	27.5
	PC 5%	196.5	15.8
	PC 7%	216	9
DERIDDER WHITE	-	20	21
	PC 3.5%	89	12
	PC 10%	198	5
HW 171	-	24.5	20
	PC 3.5%	111	12.5
	PC 8%	208	7.5

A distinctive case is represented by Chase White where the initial amount of Portland cement added to the raw soil had increased the mixture's strength up to 196.5 lb/in^2 (almost 200 lb/in^2). At this stage, according to the actual specifications, 5 percent cement represents the solution for Chase White stabilization (36). However, the moisture susceptibility for this mixture is still inappropriate (final dielectric values of almost 16), so an increase of additives amount is considered, up to 7 percent. In this case, both strength criteria and moisture susceptibility criteria are met.

This situation demonstrates the relevance of moisture susceptibility criteria as an important component to be incorporated into the specifications for subgrades stabilization.

As it was mentioned before, "Louisiana Standard Specification for Road and Bridges" indicate a maximum of 69 percent silt content for subgrades soil. The results presented for Chase White and HW 171 suggests soils with content of silt exceeding these specifications still can be stabilized (in this case with Portland cement), with good long term results.

CONCLUSIONS

Soils with high-silt contents have low strengths and minimal bearing capacity. When considered for road subgrade located in areas with a high water table, soil compaction efforts and construction traffic can produce detrimental pumping action, caused by the redistribution of water due to an uplifting effort. The pumping phenomenon involves modification of moisture content, leading to negative effects on construction parameters such as soils strength and dry unit weight. When the soil is too wet and the applied compaction energy is too great, pumping or weaving will occur as the wheel shoves the weaker soil ahead of its motion.

One of the objectives of this study was the identification of silts with high pumping potential. The site conditions generating pumping phenomenon (excessive moisture and excess of load), were considered by a set of cyclic triaxial tests as an attempt to duplicate this phenomenon in the laboratory. The comparative results of soils axial strain and strength were corroborated with their geotechnical characteristics determined in an initial phase. Soils with high silt content, i.e. more than 60 percent and low plasticity index, (PI < 10) were more susceptible to pumping action than other two soils (Chase Brown and Deridder Brown) with a higher plasticity and content of clay.

A significant cause to pumping is represented by the high moisture susceptibility, which can be considered as the potential of a soil to develop or hold capillary water and produces detrimental or unstable conditions under load. The evaluation of soils moisture susceptibility using Tube Suction Tests demonstrates that the soils with higher pumping susceptibility exhibit high moisture susceptibility too.

These findings lead to the conclusion that the basic conditions that contribute to a "pumping" condition are:

1. The presence of a soil with characteristics susceptible to pumping:
    - High silt content (over 60 percent)
    - Low plasticity (PI < 10)
    - Low values of strength (for the silts considered in this study: approximate 20 lb/in^2 for the state of soil corresponding to optimum moisture content)
    - High moisture susceptibility, indicated by final dielectric values determined from Tube Suction Test (over 16)
2. Excess moisture conditions (above optimum) and/or access to water.
3. An excessive compaction effort during construction phase
4. An excessive cycling loading during roads service.

The prevention or control of pumping can be ensured by soil stabilization. This study is focused on chemical soil's stabilization / modification methods, which represent techniques used to construct a working table, prevent pumping and to achieve the relative compaction and strength requirements. The long term stabilization effects produced and gain in stability during pavement performance was explored. Chemical additives used were lime, Portland cement and fly ash.

Different testing procedures used indicate lime has a substantial drying effect on wet subgrades with high silt content, which constitute the main reason lime is used for moisture control. Hydrated lime reacts with the clay mineral surface in the high pH environment created by the lime – water system, generating cementitious products. The problematic low plasticity silts considered in this study exhibit high moisture susceptibility and pumping action. The high percentage of silt (over 60 percent) and especially the low content of clay (between 10 and 18 percent) are not favorable factors for development of pozzolanic products from reaction with lime. The stabilization effect of this additive was limited for the high silt soils considered in this research.

The combination of lime and fly ash exhibited the same moderate effects for soil' stabilization. The gains in strength for samples cured for 2 years were also limited. These results are generated by the insufficient amount of clay as support for pozzolanic reactions.

Portland cement was determined to be the most effective chemical additive for the soils considered in this study. The strength criteria of 200 lb/in^2 for the stabilized mixtures are met for moderate percentage of cement (between 5 and 7 percent) added to the problematic silts. Also, the moisture susceptibility, considered a determining factor for pumping action, is substantially reduced for the soils mixed with Portland cement.

Moisture susceptibility is an important factor that affects the mechanical properties of pavements and subgrade materials. The evaluation of soils moisture susceptibility was conducted using Tube Suction Test and determining the final dielectric values. The comparative results for the set of soils considered in this study indicate the high silts with low plasticity and high pumping action exhibit high moisture susceptibility. This parameter is reduced by stabilizing agents, the most effective one for problematic silts being Portland cement.

The overview of some manuals for highway construction revealed the fact that Department of Transportation (DOT) of each state considers different approaches for subgrades soil stabilization or even different criteria for soil classification.

The findings of this research lead to recommendations for improvements of guidelines for road construction with problematic silt subgrades:

1. Identification of problematic silts. Soils with high pumping susceptibility exhibit content of 65 percent of silt or higher, a Plasticity Index less then 10 and a minimal bearing capacity reflected by low values of soils strength.
2. Chemical stabilization for problematic silts. Different chemical additives can be considered for soil stabilization: lime, fly ash or Portland cement. Lime or combinations of lime and fly ash are not effective for stabilization of soils with low plasticity and low percentage of clay. The results of this study indicate soils with less than 18 percent clay do not develop long term gains in strength.
3. Strength criteria. To evaluate stabilization efforts, the use of a strength criteria is recommended. Those values presented in this study have been used successfully elsewhere. For lime stabilization the unconfined compression strength of the mixture must surpass the minimal increase of 350 KPa over the original strength of the raw soil. For stabilization with Portland cement, the unconfined compression strength of the mixture should be 1380 KPa (200 lb/in^2). To evaluate the stabilization efforts, the use of a strength criteria is recommended. Those values used in this study have been used successfully by several Departments of Transportations

4. The moisture susceptibility of the mixture must be reduced in such a measure that the final dielectric values determined using Tube Suction Test should be lower than the threshold value of 16
5. The evaluation of improving the strength and axial strain for silts before and after chemical stabilization should be illustrated by cyclic triaxial unconsolidated-undrained tests, for saturated specimens as a measure of soil behavior under the most unfavorable conditions.

This procedure takes into account the concept of moisture susceptibility, which is incorporated with the actual strength criteria for subgrades stabilization.

The actual specifications in different states for construction phase indicate the compaction should be conducted for a range of -2 to + 2 percent of soils optimum moisture content. Louisiana Standard Specification for Road and Bridges considers acceptable range of moistures of -2 to +4 percent of OMC only for embankments compaction, which still can be considered a source of problems. A rigorous control of soils moisture at the moment of compaction should consider the range of -2 to + 2 percent of soils optimum moisture content.

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APPENDIX A

Embankment Construction with Soils Containing High Silt Content

Construction Experience Survey

1 Do you believe the current DOTD specifications defining usable soils to be adequate?

Yes _ __**14**_____56% No _____**11**____44%

2 Do you believe that silt content is a good indicator used for the proper identification of usable soils?

Yes ___**19**____ 76% No _____**5**_____20%

2.1 If no, what soil properties or classification would you recommend to specify as a usable soils?

- *(checked yes, but wrote*

1) use current classification for roadway embankment

2) allow higher silt content, PI ,etc. for widening shoulder slopes, levees, canal plugs, etc.

- *(checked yes, but wrote)*

  Increase silt content to 70% (or maybe even 75%)

- *(did not check ans, stated AI don=t know@ and wrote)*

  High silt content soils cannot be used on slopes yet A useable@ allows this choice. Silts erode quickly and need t be contained.

- *(checked yes, but wrote)*

  Max silt is a good indicator, but other factors need to be accounted also, i.e., PI, LL

- *Should have a lower max. Silt content.*
- *(checked yes, but wrote) But current (max silt limit) is too high.*
- *(checked yes, but wrote)*

  CAREFUL; *silt alone not the defining characteristic. A 50% silt/50% fine sand can be one of the worst materials to contain and compact to a stable mass. A 80%silt 20% stable clay soil may give no problems during construction..........................*

- *Liquid Limits 40 - 45, PI 19 - 25, % Silt 70 - 75, Organic 5%*
- *Usable soils shall have a PI of 25 or less, an organic content of less than 5% and a maximum silt content of 80%.*
- *P.I. & L.L.*

3. Have you been involved in a construction project that used soils with a silt content in excess of 65 %?

Yes ____**18**____72% No ____**7**____28%

If yes, (Please use additional pages if necessary to address more than one project)

3.1 Identify the project, date and location.

- *Can't recall*
- *Don=t remember individual projects*
- *Project 455-05-27 (1990) I-49 (LA 498 Interchange)*
- *State Project No. 927-01-0002 - Jennings Airport (Runway 8/26 Extention & Taxiways) Jeff Davis Parish, Final Acceptance July 9, 1990*
- *1. 808-07-0035, June 98 to Dec. ?? , Bossier City, LA*

  2. 742-07-0011, 2/7/94 to 10/6/95, Bossier City, LA
- *S.P. 026-05-0013, 11/97 - Present, LA 15 - Franklin & Catahoula Parishes*
- *SPN 026-06-0018, Wisner - Gilbert, LA 15, Franklin Parish, 9/1/89 - 3/18/91*
- *Humble Canal Bridge, 1997, Terrebonne Parish*

  Bayou Gardens Crossing, 1998, Terrebonne Parish
- *Samples representing a pit were taken for uasable soil. The samples failed for usable soil; however, passed for plastic soil blanket. The material was used on the project for that purpose. S.P. 004-08-0030 Winter*

1998/1999.

- *E. Creswell St. Extension (S.P. No. 056-07-0010)*
- *455-05-0023 I-49 Rapides Parish 1989 - I-49 Mainline was working table (no high silt material used) service road was soil with high silt from Red River, pictures are attached.*
- *455-05-0026 I-49 (Graham Road - Route LA 498) 1989-1992, AHydraulic fill@ pumped from Red River in Rapides Parish - Alexandria, LA {1982 Edition Louisiana Standard Specifications for Roads and Bridges]*
- *80-01-17, U.S. 167 Abbeville-Maurice, Approx. 10 years old*
- *Silty soils are very common in southwest Louisiana. We constantly deal with them in construction.*
- *Cresswell Lane - Opelousas, LA; Studebaker Truck Stop - Duson, LA; Martco - Lemoyne, LA*
- *I-20 Madison Parish, Bayou Macon(Sp?) - Quebec, 1970 B>*
- *SP 804-12-10 Hwy 1011 Assumption Parish 1981*

3.2 Did you have problems during construction? If yes describe.

Yes ____**14**____56% No ____**4**____16%

If yes,

- *Moisture was a problem - pumping existed, method to obtain stable .... was time consuming - using rubber tires only to compact is not sufficient*
- *High silt material is very moisture sensitive, has to be exactly rolled to get good compaction. Can dry and later cause problems. Can get wet later ad cause problems. Has a tendency to Apump@ and be unstable.*
- *This project was constructed by hydraulic fill (pumped from the Red River) the material was very silty. The fill that was pumped did not present problems with density and moisture but a stockpile was made to use for backfilling pipe. The contractor eventually abandoned the silt and ordered sand for his pipe. Silt is too moisture sensitive.*
- *Subsequent to stabilization with cement, the base material just failed to set up. Approximately 1 week after stabilization the airport runway/taxiway fell apart. The base had to be completely re-cut using a higher percentage of cement.*
- *Subgrade was essentially pure silt - tests of in-place materials showed silt content 70 - 90 percent. Subgrade pumped, moved and could not be consolidated and/or compacted. It would not support construction traffic, or stone base course. Obtained stabilization by treating with 10 percent lime by volume - obtained density and built project.*
- *1. Pumping of saturated high silt material requiring undercutting minimum depth of 3 feet.*

  2. Pumping during embankment construction of material with 65 percent silt. However, density was achieved.

- *Material was moisture sensitive. Material would pass density & moisture requirements but would become unstable . Minor construction traffic would cause failures in the zones previously set up.*
- *We used embankment material with silt contents around 70 (some maybe higher). Had some trouble initially getting density (moisture was too low). Contractor was finally able to get moisture up (2 percent above optimum) and we had o more trouble with density.*
- *Existing soil pre-treated with Type D lime treatment performed very well. However, material within shoulders which was notlime treated exhibited pumping and had to be removed within several locations.*
- *Cutting soil cement in river silt; material became dry and brittle and flaked apart, most of soil cement had to be removed and replaced with limestone as based on Parish Road 23.*
- *Minor problems with the silt flaking/ravelling after using cement for a working table. The roadway was broomed and a 2" asphaltic concrete base was placed prior to placing the PCCP surface.*
- *Had asphalt concrete On raw embankment shoulders. Contract used materials w/high silt contents. Shoulders failed before complete, had to plan change to stable shoulders at approximately $400,000 additional cost to project.*
- *Personal involvement was/is from a laboratory, materials standpoint. The in situ materials are generally the worst construction difficulty since materials of this nature cannot be processed deep enough to solve*

instability problems. To undercut is a very expensive solution and finding replacement soil within project areas is sometimes not possible. This type soil always involves moisture sensitivity, high moisture, and instability ehich results in general construction and density difficulties. There are problematic soils when clearing grubbing has occurred due to many roots, stumps, and other debris left in the process that hold moisture.

- *Very high silt content (80 - 95 percent). Very slow processing due to wet soils. Excessive Apumping@ at 95% density & optimum moisture*

3.3 What type of construction equipment was used?

- *Rubber tire roller --- need sheep foot; disk and patrol*
- *Vibra plates on the pipe. Dozers, motor graders, sheepsfoot rollers on the fill.*
- *BOMAG stabilizers, vibratory sheepsfoot roller - pneumatic roller*
- *Standard earthwork equipment*
- *Tractor and dirt bucket, pad foot roller*
- *Sheepsfoot, motor patrol, dump trucks*
- *Standard equipment, sheepsfoot rollers, water trucks, end dump w/trucks, spread w/dozer.*
- *Motor grader, dozer & sheepsfoot roller*
- *Dozer, motor grader, material was used as blanket material for slopes. No density tests were taken.*

- *Typical earthwork equipment (trucks, motor patrol, sheepsfoot rollers, etc.)*
- *Bomag cutters, sheeps foot, motor patrol, etc.*
- *Stabilizers, sheepsfoot roller, motor grader, etc.*
- *Conventional embankment construction*
- *On projects in southwest Louisiana, compaction equipment of various types have and are being used including; sheepsfoot, padded foot, pneumatic, smooth steel, combination steel and pneumatic, waffle wheel, and vibratory. Vibratory rollers are heavily promoted and used. Other equipment includes conventional motor graders, dump trucks (small to large), tractors, discs, water trucks, etc.*
- *Dozers, motor patrol, sheepfoot roller, 9-wheel roller, tractors w/dirt buckets & disc, dump trucks*
- *Standard*
- *rollers*

3.4 What type of quality control was used? (moisture, density, etc.)

- *Moisture very critical - density can ve obtained wasilty; however not stable*
- *Density & moisture checked by DOTD*
- *Moisture and density*
- *Std. QC by contractor as well as Acceptance testing by the department*

(moisture, density, etc.)

- *Moisture & density*
- *Nuclear density, family of curves & be at(?) our own curve*
- *Standard specs on density (95%). We attempted to compact at 2 percent above optimum.*
- *Moisture and density control*
- *Thickness of lifts, moisture, density, suitability*
- *1982 Standard Specifications (Moisture/density tests)*
- *1982 Standard Specifications (Moisture/density tests)*
- *No moisture controll, density control used.*
- *Little QC is conducted by the contractor on raw soils (slowly changing). Most focus on density; the test for pay. QA tests for density are conducted by the department for pay.*
- *Visual, Troxler, moisture, density plotting curves*
- *Standard Moisture Density*
- *yes -------> moisture, density, etc.*

3.5 Are you aware of any long term problems associated with the embankment /pavement structure.

- *If moisture not conform - settlement not conform*
- *Not really, we have not done any monitoring of jobs.*
- *No*

- *No*
- *No*
- *No*
- *No*
- *No problems. In fact embankment is performing very well.*
- *All areas within roadway which were pre-treated with lime are performing satisfactorily. Some areas within shoulders not treated with lime exhibit pumping and/or base/subgrade failures.*
- *No*
- *No*
- *No*
- *Yes there are roadway sections that are experiencing swell.*
- *Historically, early failure experiences have dictated design changes to minimize the effects on future projects of weak subgrade soils. Pavement structures have been continuously increasing in thickness. Concrete pavements have gone from 6" to 9" to 10" to 13" and some 15" have been constructed. Hot mix thickness is progressing rapidly. It is not uncommon to find approximately 12" of accumulated overlays, especially over concrete pavements; with the total pavement structure thickness approaching 2 feet. Failure is still common. I this the result of poor subgrades, poor pavement materials, poor mix design, poor rdwy design, poor construction technique, or ----? In our common design of rehab projects includes lime treatment of the embankment 12" to 15"*

immediately below the pavement structure in an attempt to upgrade the strength of poor soils. This seems to have provided several benefits. The most important is the increase in support value. Another is the diminishing of moisture sensitivity while improving the resistance to water absorption. When lime treated material is subsequently cement stabilized, in most cases, the usual shrinkage cracking is delayed and is of much less magnitude in the long term.

- *No*
- *No*
- *No. The problems I have seen is when you construct embankments with high ADT (or APT?) and too much silt > 65%. Low ADT (or APT?) and low truck traffic allow for higher silt material*

4. Have you been involved in a construction project that used soils with silt contents less than 65% and experience construction problems associated with moisture, pumping and density? If yes explain.

Yes ____**15**____60% No ____**9**____36%

If yes,

- *Moisture on many soils very critical or won=t be stable under heavy equipment*
- *This occurs on numerous projects where the existing underlying soils are too wet. Material may be dried out by processing or may require*

undercutting if it cannot be setup.

- *Heavy wet clays had been used for subgrades which resulted in pumping. Project located in swamp area associated with South Louisiana. Fly ash was urilized as an additive to aide in drying material prior to cement stabilization of base course.*
- *Clays or layers that were extremely wet required some sort of drying before compaction - stabilization could take place. In place soils that were to be built upon is what has given us problems.*
- *No problem placing material if moisture was O.K. When material drew moisture, had trouble with subsequent lifts due to pumpng (in area of pipe backfill). This material was native ---- quit using when started having problems. Went to borrow pit material silt 6%, PI = 0 -- no problems*
- *reinforced sand backfill, 20 ft slopes, a lot of trouble getting density, tried several different variations (vibrations, not vibrations, flood, spray soak.*
- *In numerous projects. These problems could have been associated with excessive rainfall, improper use of construction equipment by contractor, etc.*
- *problems encountered were either related to excessive moisture from the or on the roadway; after proper processing the materials performed satisfactory.*
- *Do not have detail information available on existing soil but was probably less than 65%. This project was thoroughly studied by LTRC and a report prepared. Suggest seeing written report.*

- *There is no recognizable difference in performance between 60% and 65% silt when the other soil fractions are similar. Also, any soil with uniformly sized grains (especially rounded particles) will be difficult to compact and may not perform. There is no significant difference between 65% silt and a 65% fine sand soil.*
- *It was very moisture sensitive and very hard to get 95% compaction.*
- *Usually soils that have a low silt content have a high sand content. This type material when compacted does not seal .. well and water penetrates through after rains. This type material erodes easily and usually has to be confined with clay blanket.*
- *All heavy clays at standard density & moisture*

4.1 Identify the project, date and location.

- *I-49 embankment project in St. Landry, Evangeline, Avoyelles and Rapides Parishes, 30 miles - 13 million yds*
- *Common problem on numerous projects*
- *State Project No. 196-03-0024, Bayou Lacassine - Junction LA 99, Route LA 14, Jefferson Davis Paish, Final Inspection Date: August 29, 1983*
- *Many mostly in Bossier Parish*
- *LA 16 S.P. 262-06-09, 1992-93, Montpelier to Amite*
- *S.P. 053-04-0030 (Lead), 835-06-0011(Actual) , Keyser Ave. LA 494, 1999*

- *158-01-16, LA 546, 1999*
- *Numerous projects*
- *009-01-0059, Pineville (Rapides Parish) US 71/165 Fall 1993-1997*
- *LA 492 009-31-0007, LA 8 (Flatwood) 134-04-0012, 008-30-0037, 009-01-0059*
- *742-07-0095, 8/15/97 - Lakeshore Drive, Mandeville*
- *The two projects listed above are typical of southwest Louisiana. Every construction project is faced with similar problems. Both of these projects have soils that fall on both sides of the 65% silt factor. To add to the problem they are interbedded with other soil types and cannot be effectively separated in place or in the pit as previously discussed.*
- *Ford Street, 1982(?), in Shreveport; 455-06, I-49, 1991*
- *Abbeville Hwy 14*
- *I-20 Madison Parish to Mississippi River*

4.2 If available please identify soil properties. (Gradation, atterbergs etc.)

- *Can not do.*
- *not available*
- *No, the construction section has no available information.*
- *NA*
- *Not available*
- *Percent Silt = 53%, N.P., Sty LM, Percent Organic = 1% => problem soil*

- *Special gradation recommended by FHWA, % Passing 3/4" - 100, No. 4 - 20-100, No. 10 15-85, No. 40 0-60, No. 200 0-15*
- *N/A*
- *granular material*
- *no available*
- *See LTRC report*
- *Soil results are voluminous. If you would like to have copies of all reports contact me and we will copy and forward. (Gradations, Atterbergs limiots, unit wts. In place densities.)*
- *blank ans.*
- *PI>35 (PI up to 85)*

4.3 What type of construction equipment was used?

- *Sheepsfoot, rubber tire, patrol*
- *Tractor w/plow, sheepsfoot rollers, dozers*
- *Standard earthwork equipment*
- *Dozers, vibrator sheepsfoot rollers*
- *vibro-plate; wacker packer - used as pipe backfill only*
- *vibratory steel wheel rollers*
- *all types of equipment on numerous projects*
- *Standard construction equipment, bulldozer, haul vehicles, sheep foot rollers*

- *Haul vehicles, bulldozer, motor graders, sheepfoot rollers*
- *See LTRC report*
- *Best answered by Don Duberville, PE, who was project manager on Moss Bluff-Gillis or Ken Lewis, PE, on the Ragley Overpass.*
- *Sheep-foot roller, steel-wheel roller, scraper, water truck, motor grader, disc*
- *Dozers, motor patrols, pad foot rollers, 9-wheel roller, tractors, dump trucks, trimmer, stabilizers*
- *Standard*

4.4 What type of quality control was used? (moisture, density, etc.)

- *As per spec (DOTD)*
- *Check moisture and density of material*
- *Std. Dept. Soil Testing*
- *Nuclear density device, sampling of borrow pits*
- *Moisture & density*
- *Nuclear, family of curves, bor(?) own curve*
- *Standard Specifications (within 2% of optimum, 95% compaction)*
- *1982 Standard Specifications (moisture/density test)*
- *Moisture/density tests*
- *See LTRC report*
- *The department has a QC (contractor)/QA (department) requirement*

controlling the materials testing. The project managers, were/are responsible and should be contacted for these types of specific details from the project.

- *Standard 1/30 mold, nuclear density testing; achieving 95% compaction or greater*
- *Troxler, visual, moisture, density*
- *Standard*

4.5 Are you aware of any long term problems associated with the embankment /pavement structure

- *No*
- *No*
- *No*
- *No*
- *Locations on this project (LA 16 S.P. 262-06-09 - Montpelier to Amite) have settled within 2 yrs of completion. This seems to have stopped.*
- *No*
- *New construction*
- *Some had long-term problems, but were probably associated with the underlying soils potential for shrink/swell.*
- *No*
- *No*

- *No*
- *On the Moss Bluff-Gillis project some failure is beginning to show through the pavement. Its source is unknown but is certainly premature. Final disposition of the project has not yet been accomplished. The Rageley overpass is now under construction.*
- *embankment sliding*
- *No*
- *Yes - Failure and expansion due to high PI soils compacted at low to optimum moistures.*

5. Do you believe that soils with greater than 65% silt content should be allowed?

   Yes **9** 36% No **14** 56%

5.1 If yes, Are the current embankment construction specifications adequate?

   Yes **6** 24% No **9** 36%

5.2 If no, what modifications should be made to assure proper construction and performance?

- *see attached (Wm. Wayne Marchand statement - $1^1/_2$ pages) construction methods need to be dictated.*
- *Not sure, probably need to be revamped*
- *Moisture control, soil conditioners for high P.I. material*

- *Allow the addition of lime treatment, filter fabric between base curses and subgrade to keep silt from infiltrating the base course, etc.*
- *High silt should be allowed in areas where other is not readily available.*
- *1. Use current classification for roadway & shoulder embankment. If silt/PI too high, pre-treat with lime.*

  2. Allow higher silt, PI, etc. for widening shoulder, for slopes, levees, canal plugs, etc.
- *Some higher levels of silt contents should be allowed since availability of soils is a real concern. There are limits which should be placed on silt but experimentation should be done to ascertain these levels.*
- *No recommendation*
- *No comment*
- *Allow where they will be stabilized.*
- *Stated No to 5.1, but wrote ANone - existing specifications and limitations are appropriate and should be continued - I see no reason to change.@...........a true QC/QA specification must be developed and implemented. We have seen significant performance advancement in HMAC and PCC as a result of improvements in technology and placing Qc responsibility on the contractor. Our soil specs. Have not been updated to the modern world......... no across the board success..........significantly increasing quality, placing total control responsibility on the contracting industry or establishing* ***true*** *QC parameters for the contractor to follow. ..... we are still conducting the*

same tests.....with the same responsibilities. The department is still involved in the control of embankment construction. ... need to develop a new set of parameters for soil selection and embankment construction. .... currently limited to Ain the pit (soil usage: PI, % organic, % silt),@ Aon the roadway (dept approves material in pit, no samples taken from roadway...)@ ......we don=t know or can=t accurate predict soil chemistry, mineralogy, capillarity, moisture retention character, shrink/swell, angularity/sphericity, uniformity, stability/compaction character, support values, destructive nature of construction equipment/techniques (fast const., vibratory rollers, super heavy equip., dry soils compacted with excessive effort vs wet soils compacted with light equipment vs optimum moisture in soils compacted with standard compactors..............one true fact....not all criteria that affect performance can be specified in a method spec.......

- *Raise silt content to at least 75%, raise PI of 20 or less than 35*
- *Do not be concerned with Aslight pumping@ at 95% density and a moisture 2% above optimum.*
- Allowance should be made for type use such as ADT & proximity of bridge approaches.

5.2.1 Do you believe that there should be a specification on the type of construction equipment allowed on an embankment constructed with a high silt content soil? (size, weight, type) Explain.

- *Yes (see Marchand statement attached to survey form)*
- *No. I think AEnd Result@ spec=s are the best.*
- *Yes, if we are going to allow the use of marginal embankment material, we should strictly define the parameters in which it can be used.*
- *No, I believe it is better for the department to set the material specifications and leave the type, size, and weight of the equipment up to the contractor.*
- *Don=t know*
- *I don=t have enough experience to answer*
- *No, but the contractor may be advised of potential problems. Ultimately the contractor will avoid equipment that causes them problems during construction.*
- *No, however there should be classes for inspectors & Project Engineers on the effects that different equipment has on the high silt soils.*
- *Not certain*
- *No - should be as chosen by the contractor which provides for construction of a stable embankment. Let the contractor decide - DOTD will ensure suitable embankment construction.*
- *No. If light equipment is required to set up embankment, later problems may occur when heavier equipment such as concrete trucks and AC haul trucks are brought in.*
- *No comment based on lack of experience in high silt content soil.*

- *I feel that this should be left up to the contractor to determine. He will after acquire the appropriate equipment and then perform whatever work is necessary to achieve densities.*
- *No*
- *Yes, no recommendation*
- *Yes, no recommendation*
- *No - do not use materials*
- *N.A. - should not allow that type of soil.*
- *No - How we would we ever specify in an accurate way what equipment to use in every situation involving soils (not just silts)? This is pitfall in method specs. If this method approach is used it should be done as part of the embankment design and placed in the contract so the contractor is well aware of any special soil conditions and can appropriately bid the project. Too many factors B disaster in the making. The focus on silts being the only problem is a problem in itself.*
- *No, an experienced person in dirt work should be able to determine what equipment is necessary to get proper compaction, moisture and not make sat base start pumping.*
- *Lighter roller with low vibration*
- *No*

5.2.2 Should the use of vibrating compaction equipment be allowed on embankments constructed with high silt content soils? (yes, no, controlled vibrations etc.)

- *No*
- *No*
- *No*
- *I feel we should advise the contractor but still let him make the decision.*
- *Yes, but with specified controlled vibration.*
- *No*
- *Not enough experience*
- *No, but the contractor should be advised of potential problems, etc.*
- *No*
- *Not certain*
- *Allow initial use, but note that vibrations may be controlled or eliminated by P.E.*
- *No*
- *No comment based on lack of experience in high silt content soils.*
- *As a general rule Ano.@ However each situation should be looked at individually to determine whether vibrating is harmful to the establishment of the fill.*
- *No*
- *No comment*
- *No recommendation*
- *No*
- *N.A. - should not allow that type of soil*

- *No - negative results of using vibrations extends beyond silts.....thixotropic characteristics...tend to become liquified.....reorient the soil particles.....high water tables...... **Uncontrolled** vibratory compaction is a real and significant nemesis to good construction and should not be allowed.......personally observed similar responses (significant settling) to traffic vibrations........The equipment is strongly pushed by the equipment industry as being the solution to density problems and is one of the most common compaction rollers.*
- *Yes, with controlled vibrating roller passes   To much vibrating with cause moisture to come to the top of lift if optimum moisture is too high.*
- *Control vibrations*
- *No vibrations*

5.2.3 Are the current specifications for moisture content and control in current specifications adequate for constructing with high silt content soils?

Yes ___**16**____64%   No ___**5**_____20%

If no,

5.2.3.1 Do you have any recommended modifications?

- *see attached (Marchand statement attached to survey)*
- *No*

- *No at this time*
- *No*
- *Treat with lime, filter fabric etc. Encapsulate this high silt content material and keep water out of it.*
- *If pumping occurs during embankment construction, previous lifts should be retested to ensure density control, Finished embankments should be allowed to settle and moisture dissipate prior to roadway construction.*
- *Should make target moisture - 2 percent above optimum - maybe make minimum moisture content be optimum.*
- *(answered yes to 5.2.3, but wrote) However, some allowance for site properties should be allowed.*
- *No*
- *No*
- *Did not answer 5.2.3 , but wrote N.A.*
- *moisture range in specs. Is another example of method spec....which doesn=t solve problems but creates conflict.....we allow 4% above optimum moisture..without limiting it to soil type, etc. If contractor is dealing with moisture retentive sensitive soil that is wet he immediately wants to undercut or take other actions involving taxpayer money rather than processing (his money). Our spec tends to give him support for poor moisture control while constructing embankment lifts even though that is not the intent.....need to develop QC spec with that Amagic number@ for acceptance that will force the contractor to appropriately select, process*

and compact the soil with compatible equipment............

- *Moisture content should be done in field. Determined by 3 point Proctor and plotted on Family of Curves.*
- *indicated yes, but wrote Aup to 80% silt@*
- *No*

5.2.3.2 Do you believe that even if properly constructed, moisture infiltration into the embankment will cause long term performance problems? If no explain.

Yes ___**17**____ 68% No. ___**5**____20%

If no explain

- *Not if material is stable and well compacted*
- *I have found that once the material has the correct moisture and density, and is compacted without allowing water to penetrate, it does appear to perform okay.*
- *(answered yes, however, wrote) water causes silts to behave in an uncontrolled manner. Keep them at constant moisture content and they can be controlled.*
- *(answered yes, however wrote) could cause movement and cracking of pavement structure.*
- *(answered yes, however, wrote) This is true with any embankment.*
- *Sufficient lime treatment of subgrade soils (i.e., Type D Treatment, 15"*

depth, 10% lime by volume) usually corrects high silt problems and results in satisfactory subgrade strength.

- *(did not mark yes or no, but wrote) Some sites will be affected by water infiltration, some may not. Again, it should be site-specific and some sites may not be good candidates for high-silt embankments.*
- *If prperly constructed with suitable drainage provisions should not be a long term performance*
- *(Selected yes but wrote) Who can define <u>proper</u> construction? District 07 conducts approximately 100 miles of subgrade a year, primarily on existing embankments...........The majority of existing embankments sampled and tested are from 5% to 8% above optimum moisture....materials..are generally silty......rainfall and capillarity have significant impact on performance.....exhibit poor internal drainage...... how can they be drained once they become wet?.... How do we prevent the soils....from becoming wet.....if they become wet how to prevent them from becoming unstable?*
- *Maintain ditch drainage so as not to allow Asuper@ saturation of subgrade soils.*
- *no answer and wrote AI don=t know.@*

5.2.4 Are there any additions to the specifications or design that you would that would increase the chances for a successful long term performance? (drainage systems, QC testing, etc.). If yes, explain.

Yes ___**14**____ 56% No___**7**____28%

If yes, explain

- *? Possible chemical like lime......(sp?) & QC*
- *Not sure what combination would work.*
- *Drainage systems do not adequately remove water from silty material because water will not flow through silt like it does in sand (particle size and shape of silt does not allow much water to move around).Better drainage systems and maintenance of systems. However, in Southwest Louisiana that presents a major problem due to high water table, flat terrains and slow run-offs.*
- *Long term performance is an unknown (did not respond as yes or no)*
- *Refers to ans in 5.2.3.1 - if pumping occurs during embankment construction, previous lifts should be retested top ensure density control. Finished embankments should be allowed to settle and moisture to dissipate prior to roadway construction.*
- *(ans. No, but wrote) Moisture already noted.*
- *Lime stabilization. This pretreatment is not moisture sensitive and results in greatly improved strength and workability of existing in-situ soils.*
- *Possibly the use of drainage systems, since our current asphalt specs utilize mixes that are more porous and allow water infiltration from the surface of pavement.*
- *To be determined*

- *To be determined*
- *Stabilization*
- *Most beneficial addition is the provision of a good drainage system*
- *see my lead paragraph(Cryer-Dist. 07). .....The only way that serious improvement will result is that we come up with a set of bona fide acceptance criteria that will ensure a higher level of performance.......................Drainage systems are more practical on new construction.............will not solve the problem of capillarity. Fine soils will load up with water until some level of equilibrium is reached. How is that stopped? .... Compaction of embankments to modified proctor requirements makes practical sense...... increases its support value and diminishes its ability to absorb water................ soil embankments should, where possible, be constructed no less than 3 feet above natural ground..........may not be important in higher relief but where natural ground is flat ....would significantly increase drainage and delay moisterization... Chemical treatment has tremendous possibilities. Lime, cement, flyash, and other chemical modifiers and stabilizers are frequently included...in conjunction with the pavement structure and not general embankment construction. ....cement, even in small quantities, modifies the soil, improving its strength and character. Lime has historically been used.....considered as a modifier in clay type soils....believe positive effects go beyond clay soils.....added problem ...that lime treatment may have life span ...in soil reverting to original condition....personally observed this*

when flyash was used to reduce PI, but not when lime has been used.................... long list of existing products, which are touted to increase strength and durability ......we have research accomplished to solve most problems...... What we need to do now is to implement. Primary hurdle ....is most likely budget...... To treat every lift .....would be costly...... But if we are designing roadways that last half or less their designed life span.......... long term maintenance costs, the costs of reconstruction, the cost of traffic delay, the costs from law suits.....it may be far cheaper than continuing as is.

- *most roads performance would improve with a French drain type system installed; a low percentage soil cement treatment should be used in most clay/silt bases.*
- *Thicker bases (12" - 16") with low % cement (6%)*
- *Application*

6. Any additional comments that you feel are appropriate.

- *Any time we encounter with pumping soil condition, we cured the problems by cutting lime into the soil. This usually solved our problem. We did not the pumping was due to high moisture or high silt content.*
- *Elimination of soils with high silt content in some parts of the state will raise construction cost to the point where cost outweighs the benefit.*
- *Contractors in Grant and Winn Parish have experienced much more*

difficulty in finding soil that meets the organic % specification than the maximum silt content.

- *...Soils are the most complex and variable materials........ The federal government has sponsored spending ....on developing SUPERPAVE in an attempt to extend the life of HMAC....... the PCC industry has developed HIGH PERFORMANCE CONCRETE.....The disparity in funding and efforts are obvious. ....Performance on the roadway will improve when resistance to change is overcome.....with effective use of soils as truly engineered materials. We know that 95% of standard proctor, the corner stone of embankment and base course construction will not give us the performance we need. One last real world comment. LA 3059, a rehab project....typical section is 12 inches of lime treated embankment, 8.5 inches of cement stabilized base, and 5 inches of HMAC........constructed to high standards of quality.........almost catastrophic failure (500 ft section)......because embankment constructed over A-2-4, A-3 water saturated approximately 9 feet thick. ....settlement is quick enough to cause rapid failure up through pavement.......cheapest materials constructed with the least amount of attention (embankment) have created a very expensive problems.*
- *Use a suitable soil to support whatever structure is being supported. No matter the silt content or PI the work effort to get the soil to its proper optimum moisture you will get density and a very stable embankment.*
- *% moisture in density is not as important as density because moisture*

content will vary after construction.

- *I have enclosed an EDSM that addresses P.I. as to the ADT. The same reasoning could be used for silt with a low ADT allowing silt to increase to possibly 70 - 75%. Silty materials need time to construct and drain. If the project allows for that time, then it is wise to increase the silt limit to allow for material that is native to the project and less costly. However, if the time is a real factor in designing the project, then a good draining embankment material is needed and will be priced as such.*

  I recently completed a project Charenton Canal Bridge & Approaches, St Mary Parish, SP 241-02-0040, and our embankment samples were 62% to 70% silt and I could not tell any difference in the workability or visual. We could not use that material due to the range of silt content in the pit. We hauled material from Abbeville. The 1998 ADT was 750. That job should have addressed a silt content up to 72%.

  We have completed projects in the Felicianas with soil samples 68% and could not use. Theses were off-system bridges. The Special Provisions should address the silt requirements depending on the purpose and use of the highway. You cannot limit a silt content in the specifications without giving consideration to the application and design.

APPENDIX B

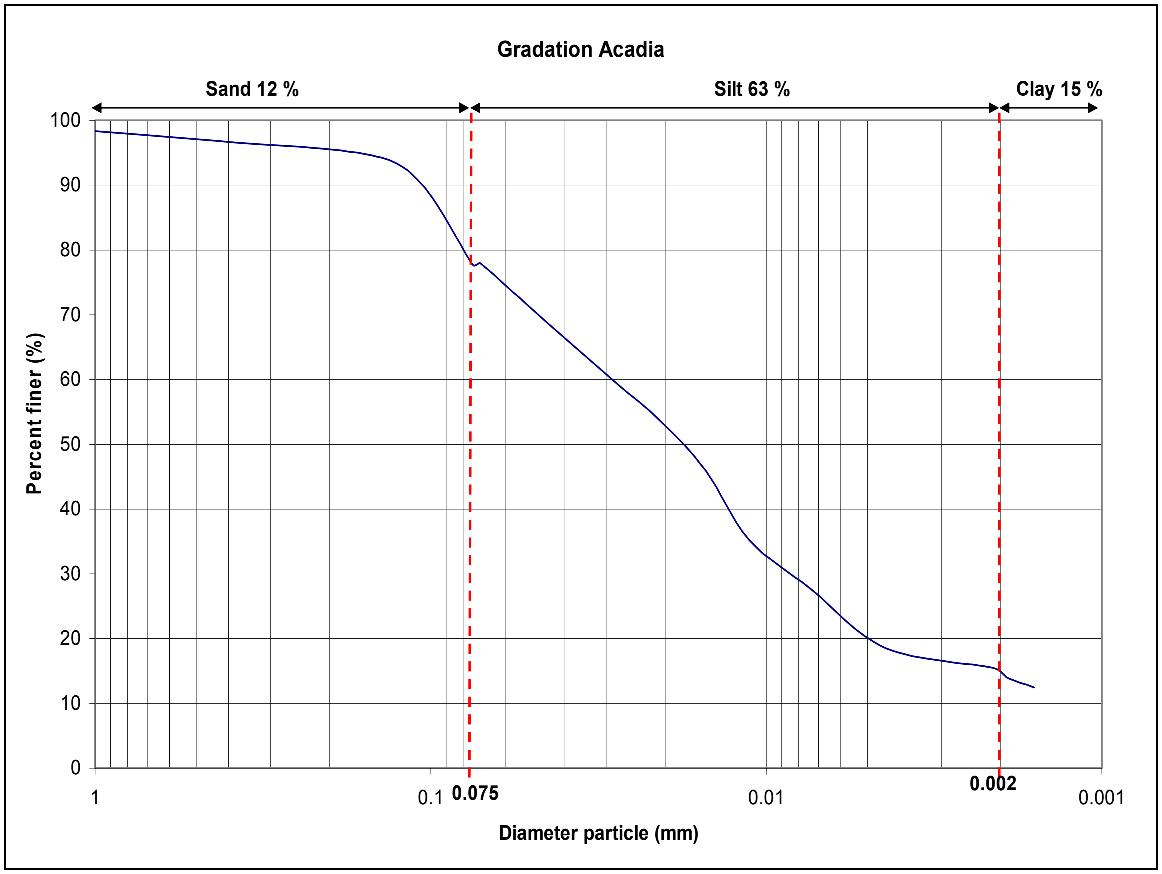

Figure 1 B

Gradation curve Acadia

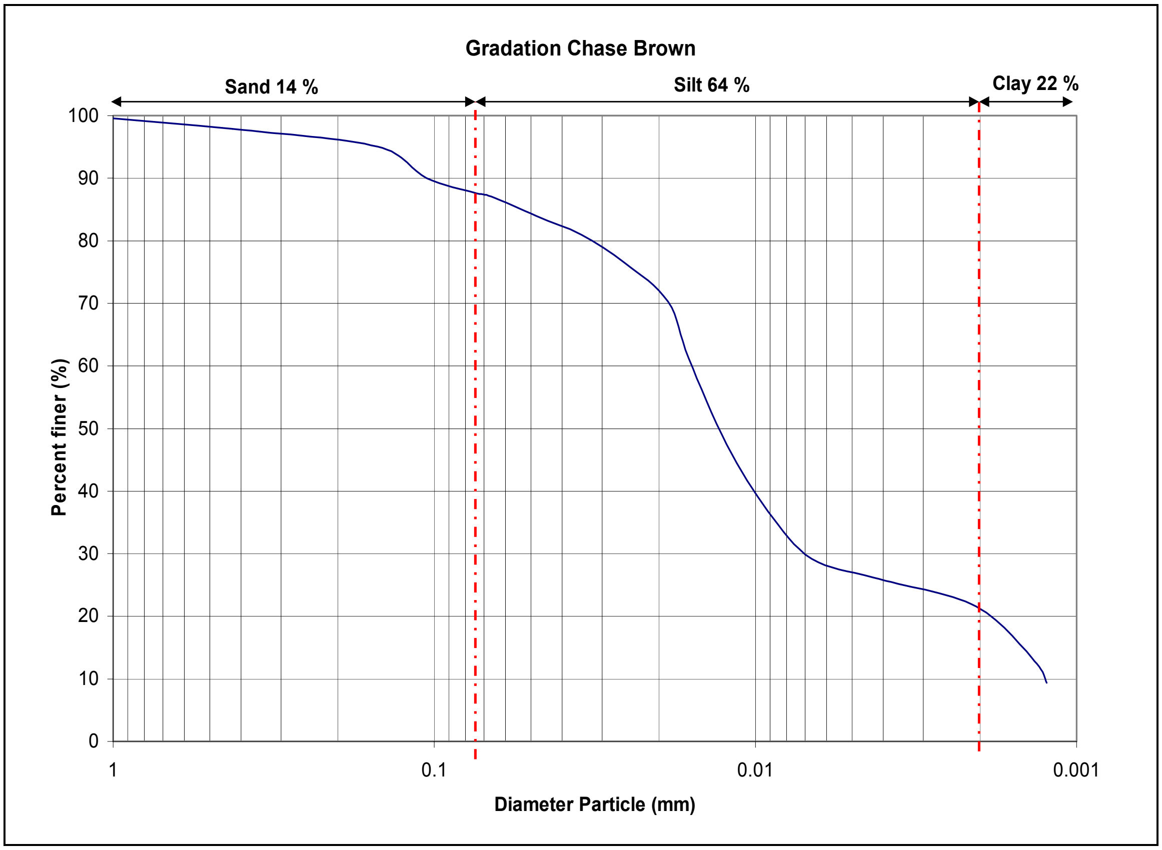

Figure 2 B

Gradation Chase Brown

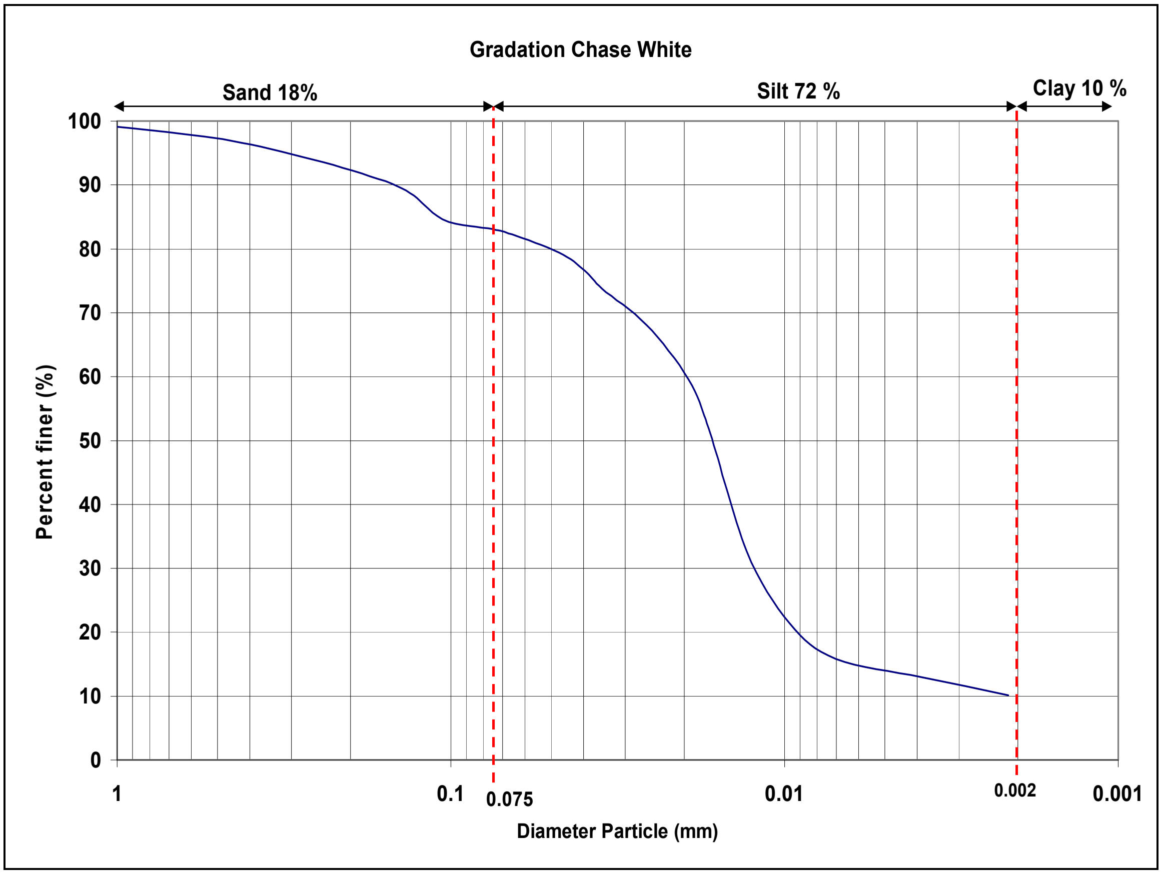

Figure 3B

Gradation Chase White

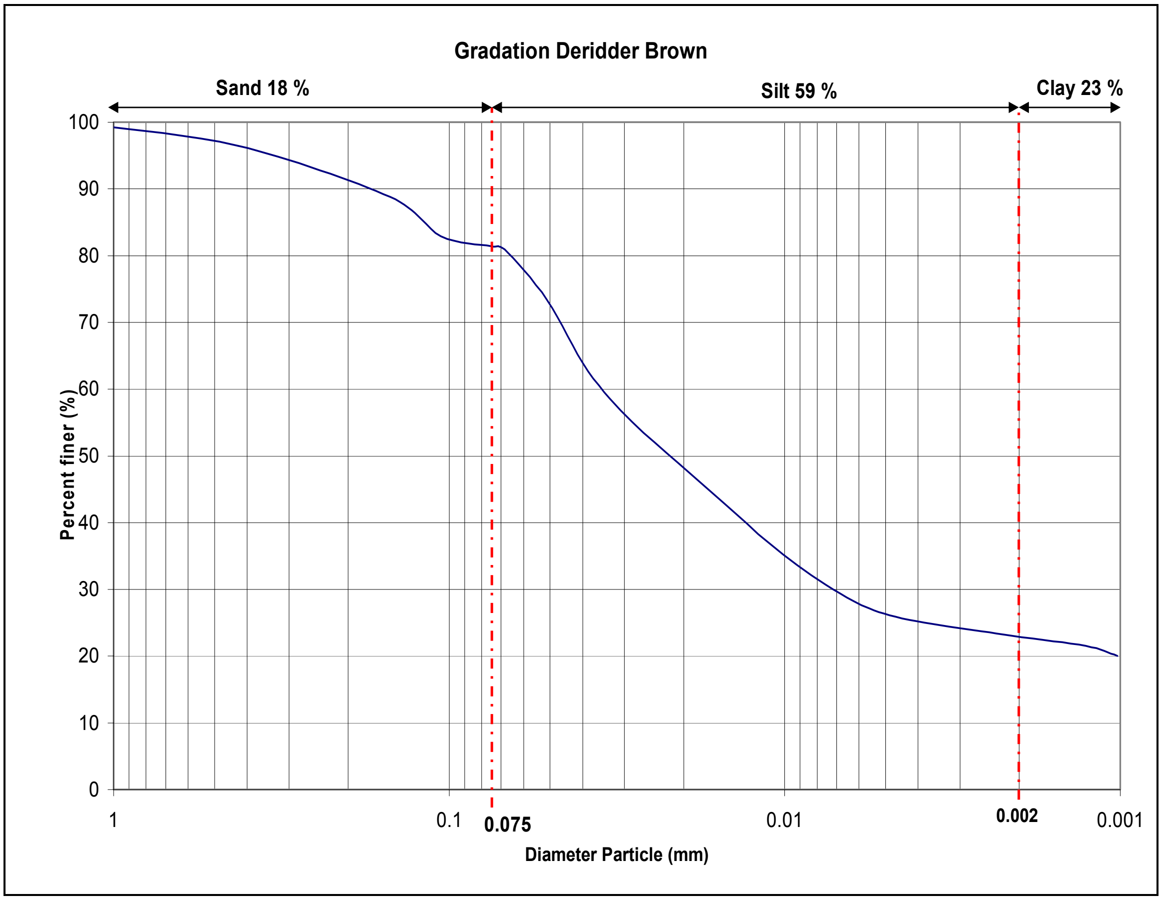

Figure 4B

Gradation Deridder Brown

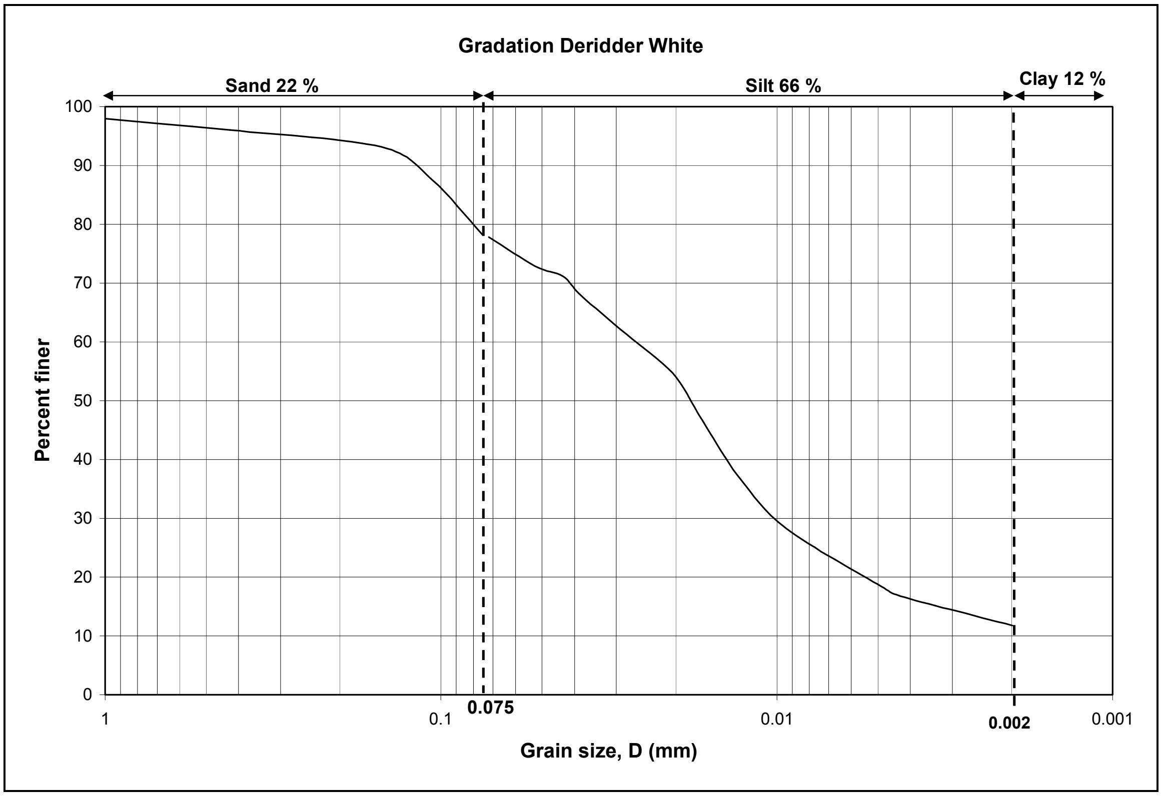

Figure 5B

Gradation Deridder White

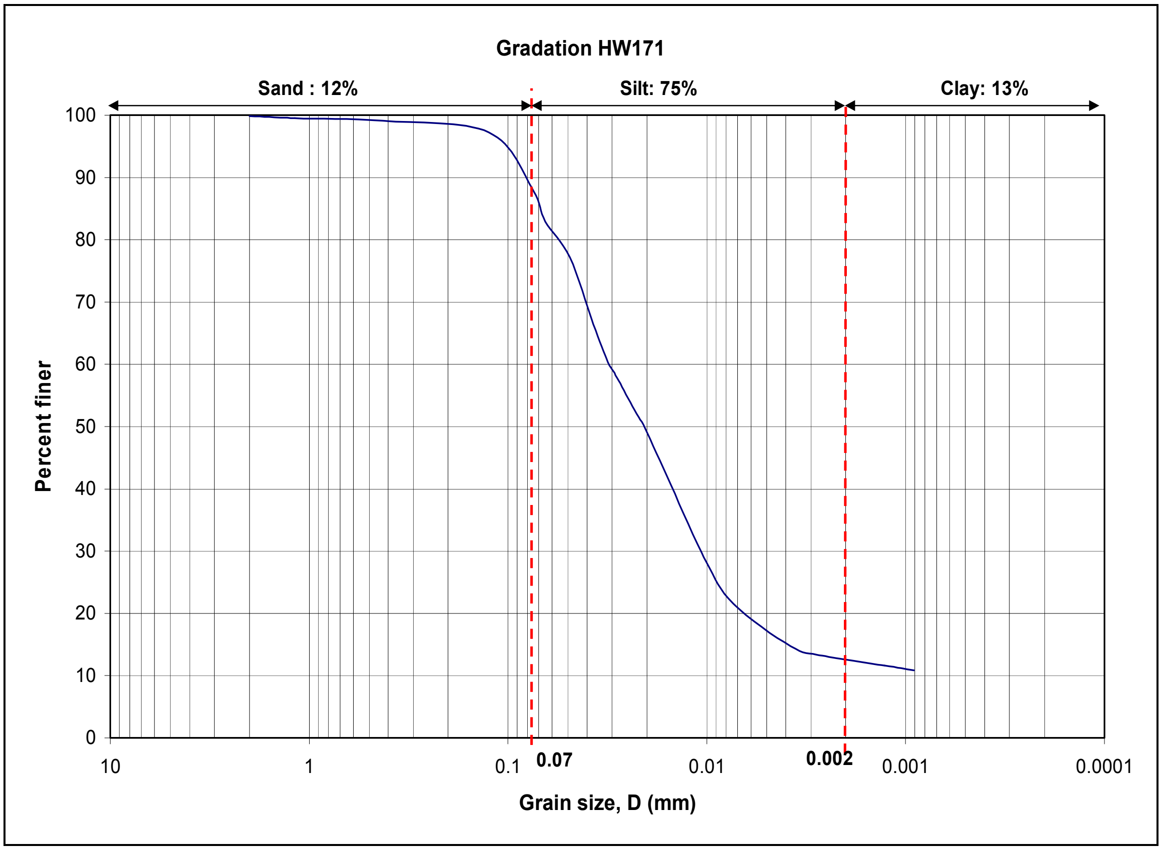

Figure 6B

Gradation HW 171

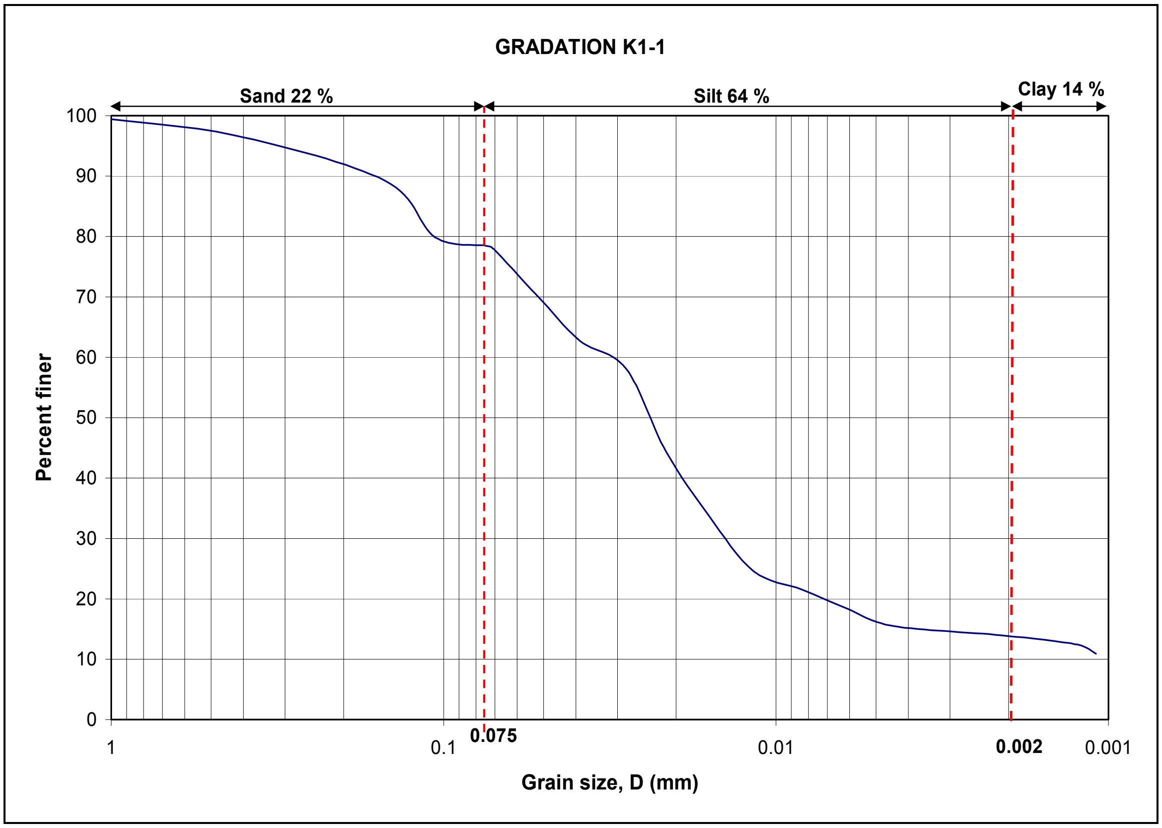

Figure 7B

Gradation K1-1

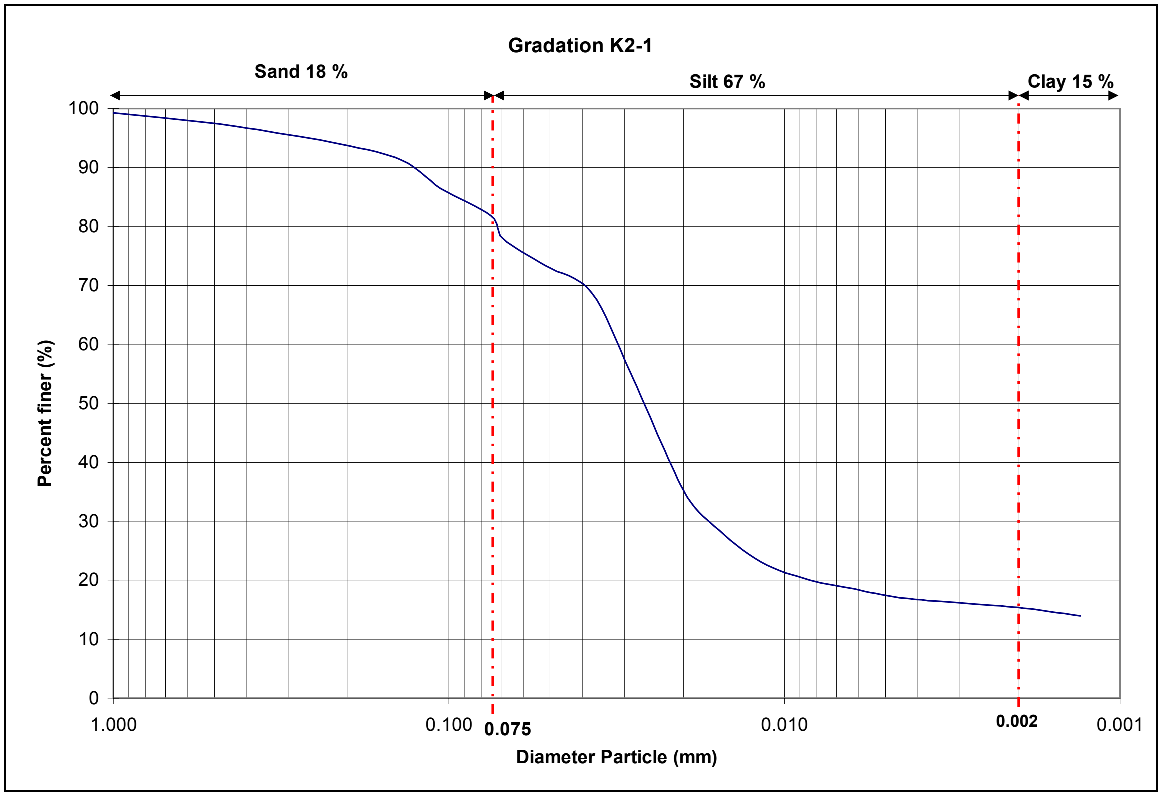

Figure 8B

Gradation K2-1

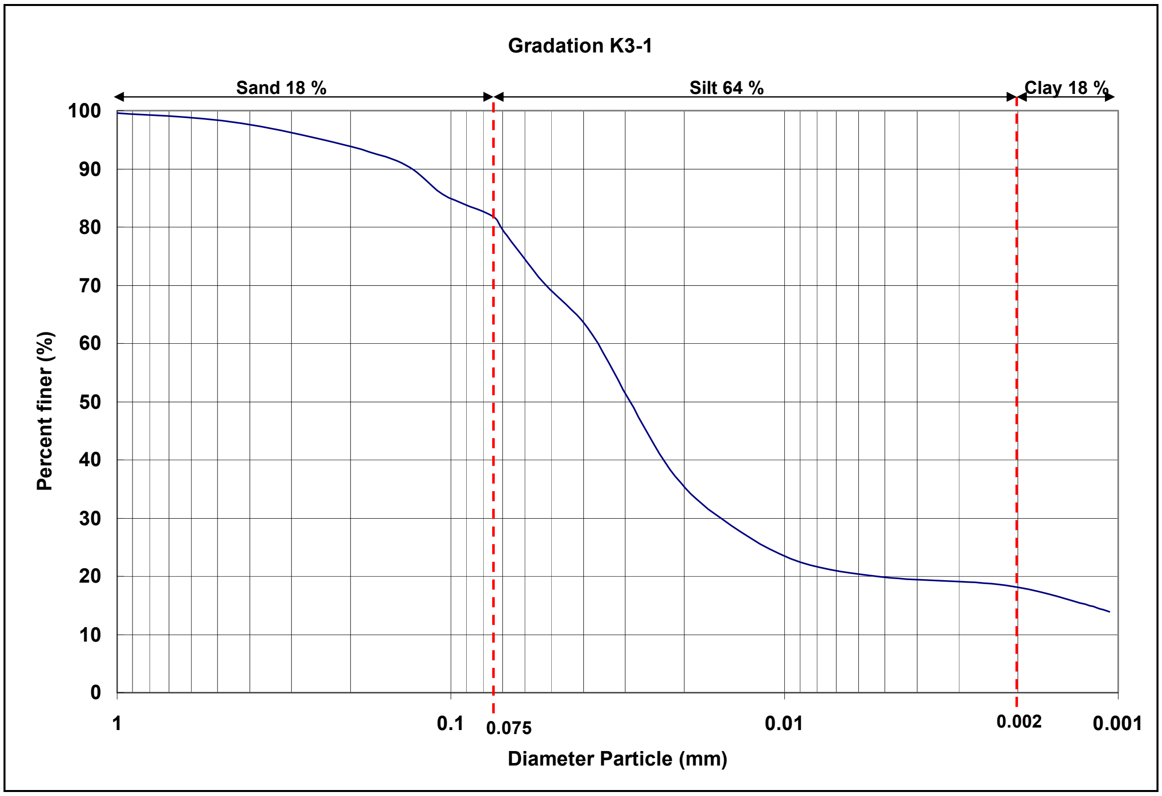

Figure 9B

Gradation K3-1

APPENDIX C

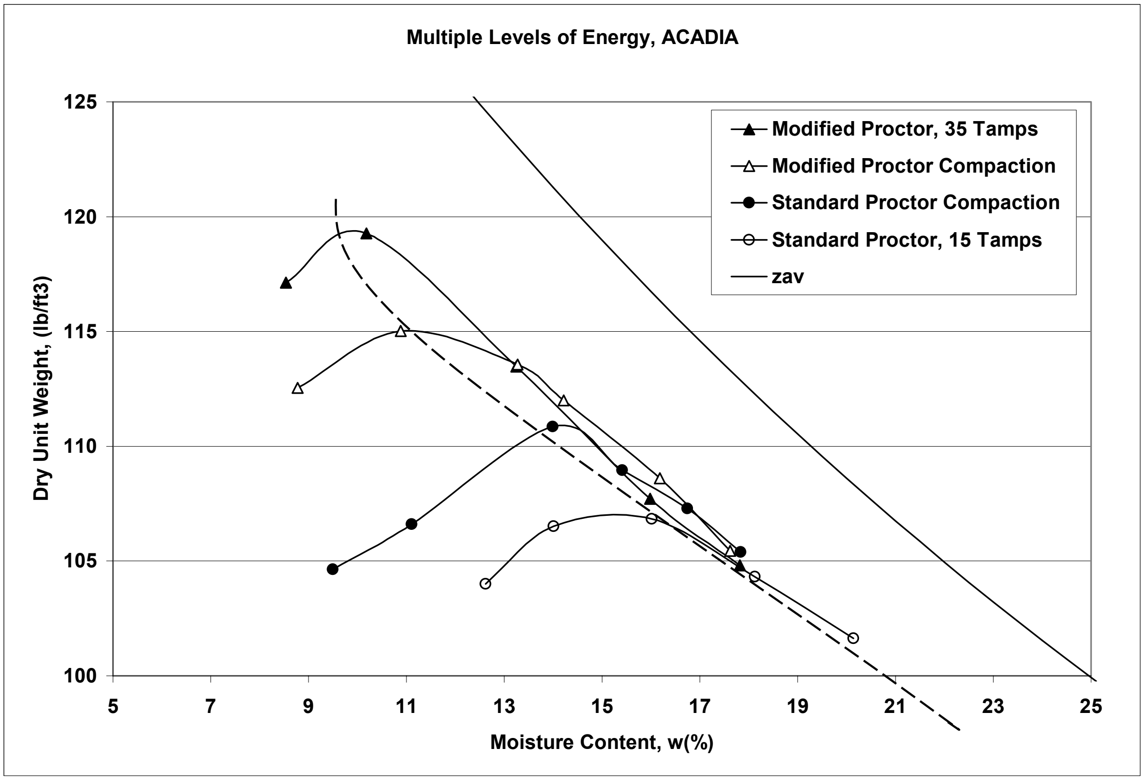

Figure 1C

Compaction curves Acadia

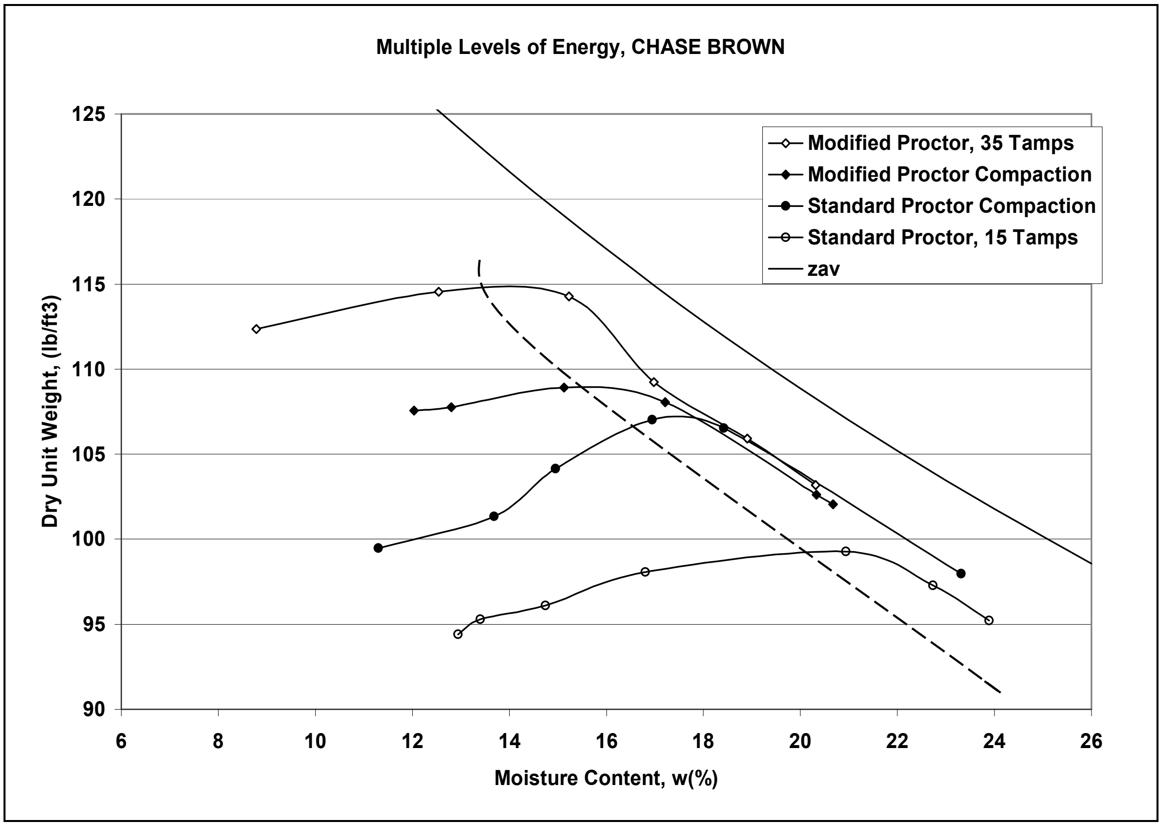

Figure 2C

Compaction curves Chase Brown

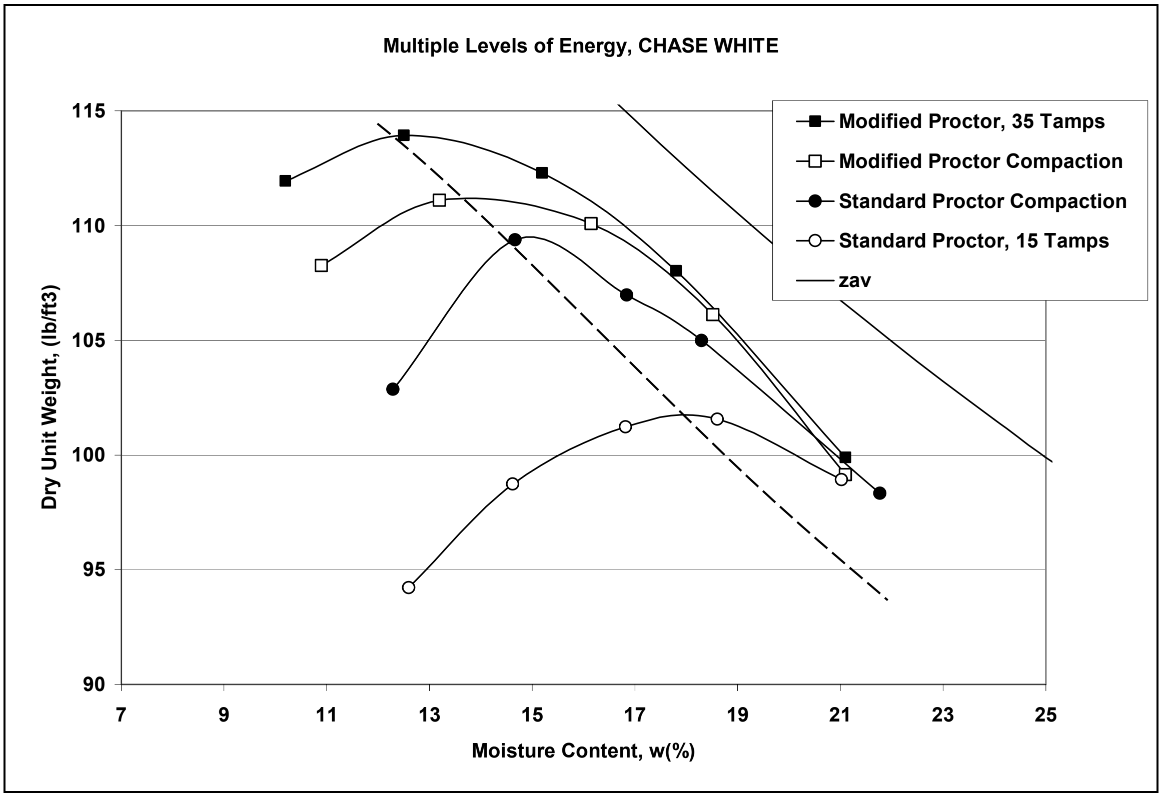

Figure 3C

Compaction curves Chase White

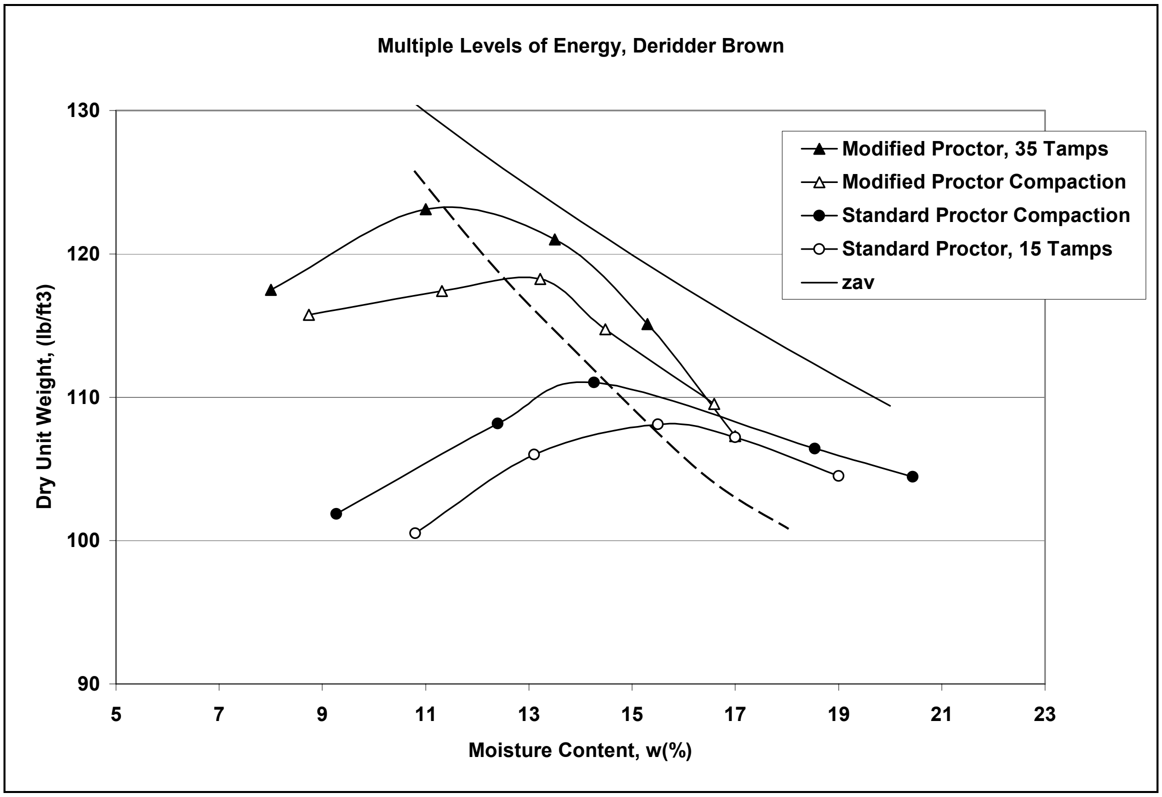

Figure 4C

Compaction curves Deridder brown

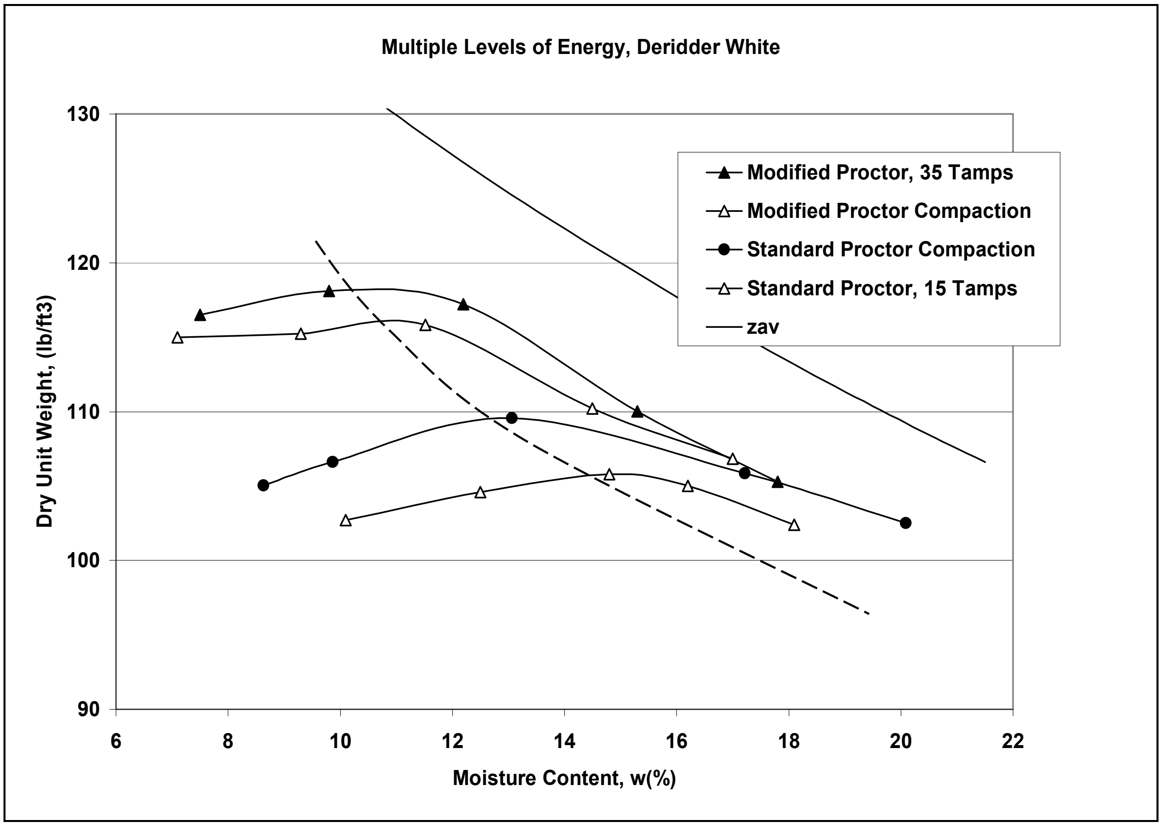

Figure 5C

Compaction curves Deridder White

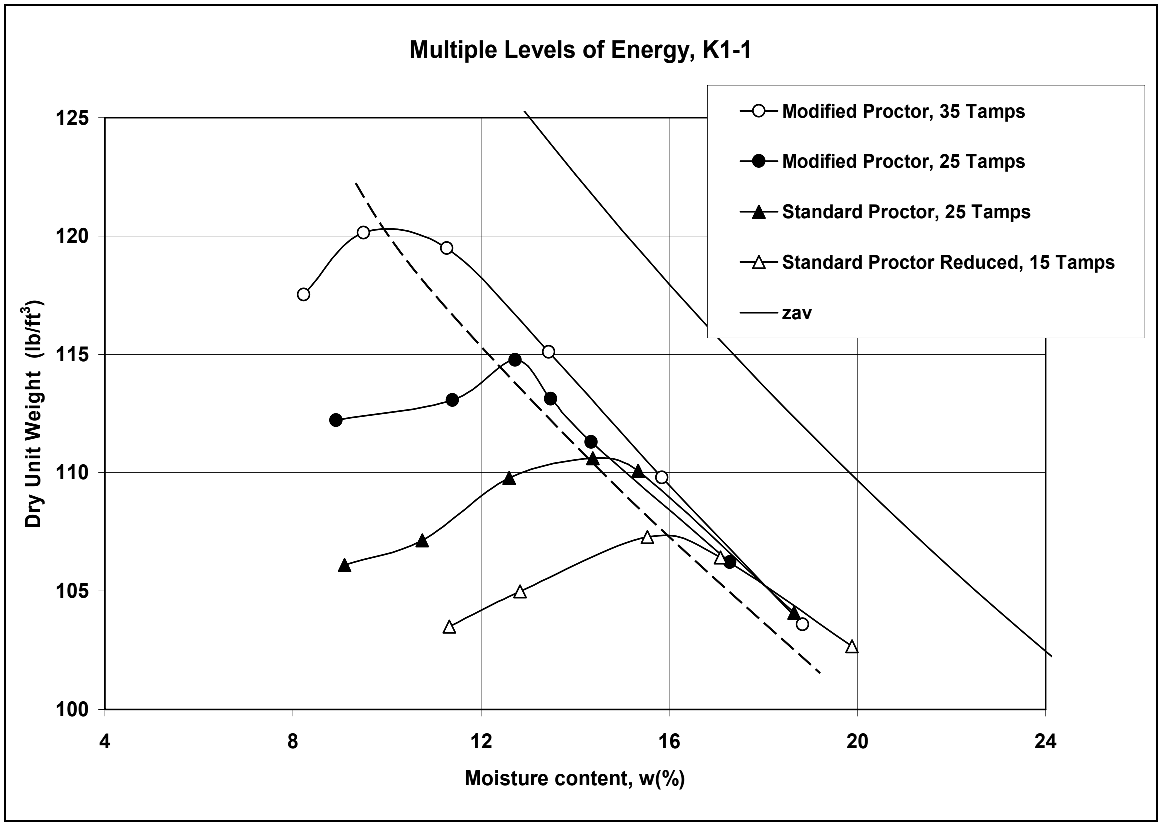

Figure 6C

Compaction curves K1-1

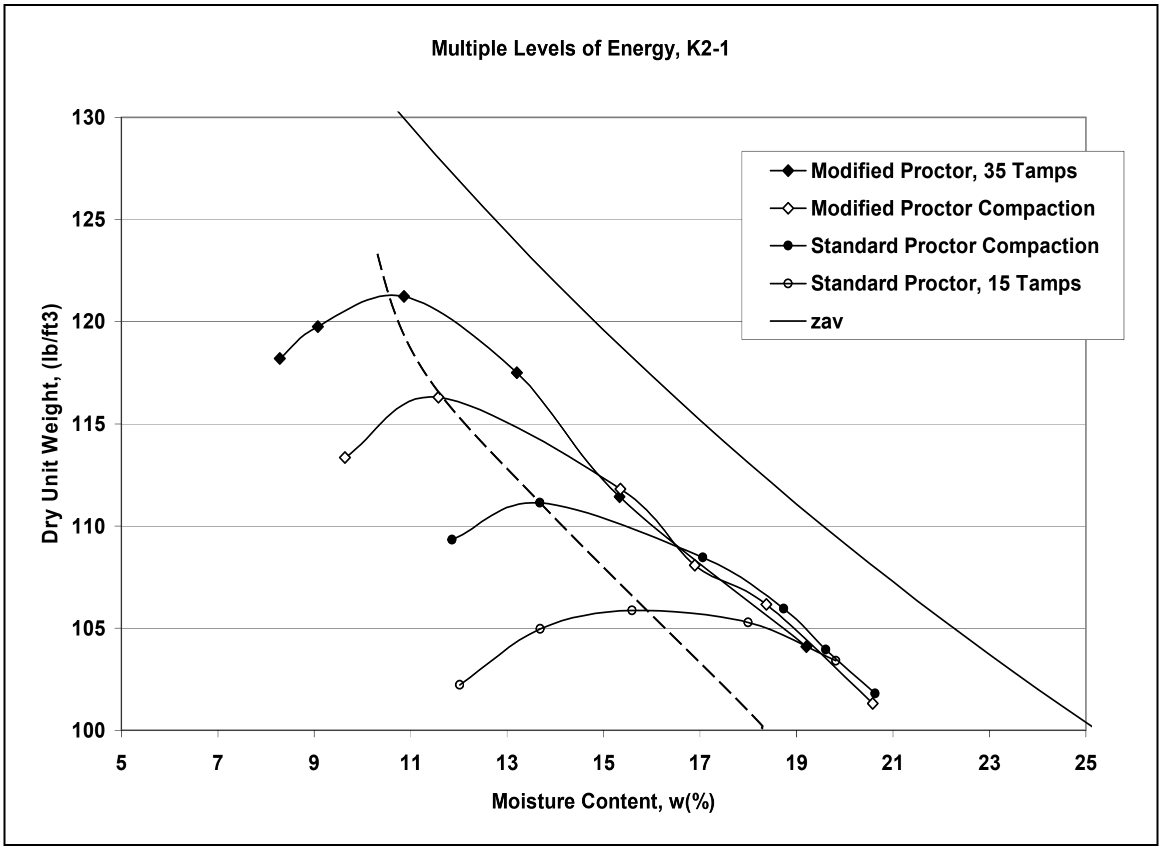

Figure 7C

Compaction curves K2-1

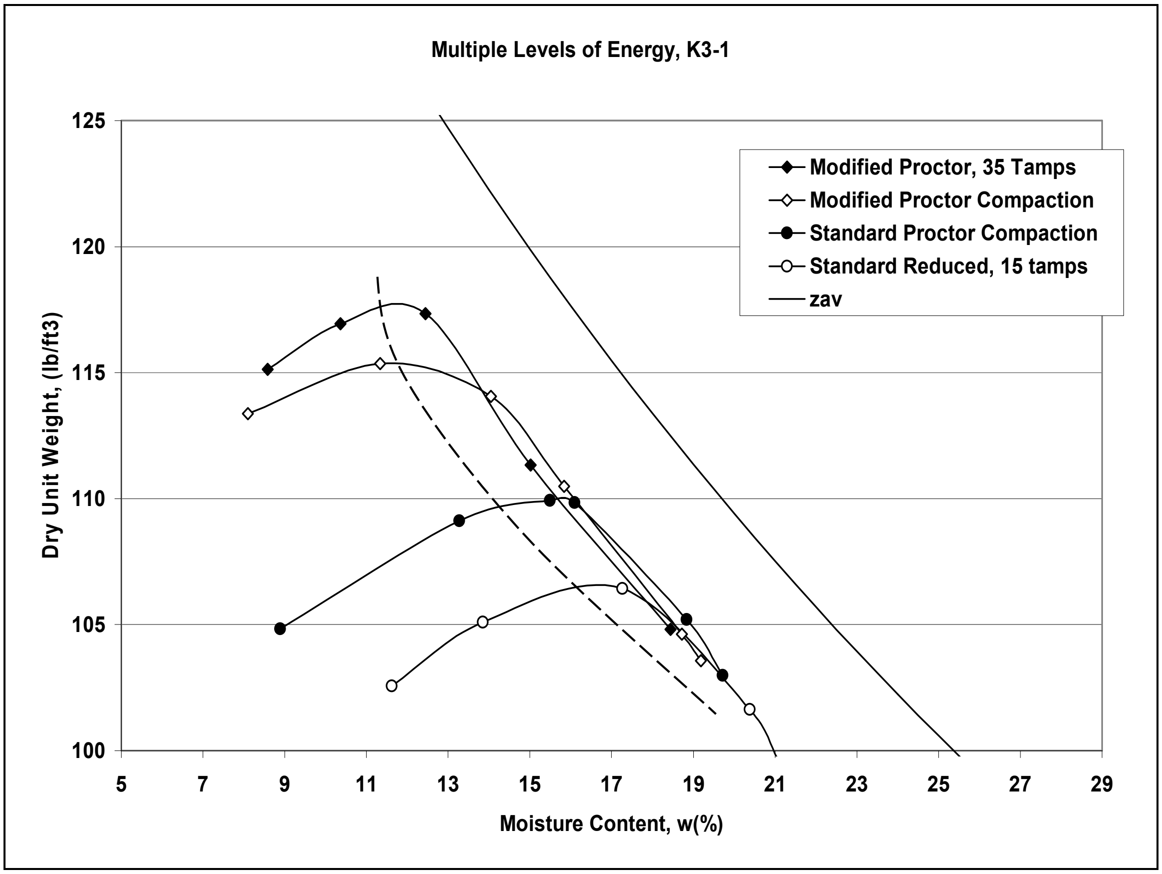

Figure 8C

Compaction curves K3-1

APPENDIX D

Soil	Moisture	γ_{moist}	γ dry	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
ACADIA	9.55	116.56	106.4	47.87
	11.76	124.35	111.27	44.68
	15.42	127.61	110.56	24.61
	16.52	126.48	108.55	17.05
ACADIA + Lime4% DIRECT	10.07	110.42	100.32	52.28
	13.18	118.32	104.54	45.8
	14.21	120.37	105.39	42
	17.03	122.12	104.35	16.57
ACADIA + Lime4% RAPID CURING (RC)	8.18	109.8	101.5	74.55
	11.68	116.61	104.41	75.77
	13.26	118.76	104.86	62
	15.8	121.07	104.55	48.11
ACADIA + Lime4% VACUUM SATURATION (VS)	10.62	109.25	98.762	25.57
	13.84	116.2	102.07	27.82
	15.04	119.23	103.64	30.47
	17.7	121.58	103.3	28
ACADIA + Lime4% HUMIDITY ROOM (HR)	10.62	110.94	100.29	58.33
	13.84	117.74	103.43	56.56
	15.04	120.41	104.67	49.61
	17.7	121.64	103.35	28.65
ACADIA + Lime4%				
After 2 years curing HR	14	122	107.02	72
Acadia+ 4% Cement	17.12	127.37	108.75	16.31
Acadia + 4% Cement HR	17.1	127	108.45	78.77
Acadia +4% Cement VS	17.1	127.67	109.03	41.82
Acadia + 4% Cement				
After 2 years curing HR	16.3	125	107.5	350

Figure 1.1 D

Chemical stabilization, Phase 1, Acadia

Soil	Moisture	γ_{moist}	γ dry	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
ACADIA	9.55	116.56	106.4	47.87
	11.76	124.35	111.27	44.68
	15.42	127.61	110.56	24.61
	16.52	126.48	108.55	17.05
ACADIA +	9.64	107.86	98.377	42.7
Lime2%+FA 8%	12.92	114.51	101.41	47.87
DIRECT	14.41	119.12	104.12	45.92
	17.92	122.23	103.66	12.42
ACADIA +	8.57	107.28	98.812	64.42
Lime2%+FA 8%	11.86	112.7	100.75	63.2
RAPID CURING (RC)	13.5	117.38	103.42	62.78
	16.75	121.49	104.06	40.17
ACADIA +	10.41	106.47	96.431	24.74
Lime2%+FA 8%	13.68	111.88	98.417	22.34
VACUUM	15.02	117.14	101.84	29
SATURATION (VS)	18.88	120.82	101.63	22.22
ACADIA +	10.41	107.48	97.346	67.3
Lime2%+FA 8%	13.68	114.11	100.38	64.32
HUMIDITY ROOM	15.02	118.32	102.87	62
	18.88	121.75	102.41	30.3
ACADIA + Lime 2%+ FA 8%				
After 2 years curing HR	12	126.8	113.21	86
Acadia+ 4% Cement	17.12	127.37	108.75	16.31
Acadia + 4% Cement HR	17.1	127	108.45	78.77
Acadia +4% Cement VS	17.1	127.67	109.03	41.82
Acadia+ 4% Cement				
After 2 years curing HR	16.3			350

Figure 1.2 D

Chemical Stabilization, Phase 1, Acadia

Soil	Moisture			Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
Chase White	10.0228	114.21	103.81	32.69
	13.0452	123.66	109.39	40.19
	15.5856	125.01	108.15	23.23
	17.0646	124.08	105.99	16.48
Chase White + Lime4% DIRECT	13.83	116.95	102.74	39.8
	17.82	119.96	101.82	13.87
	19.21	119.61	100.34	9.21
	22.32	120.68	98.659	9.53
Chase White + Lime4% RAPID CURING (RC)	13.41	116.02	102.3	88.62
	17.5	118.79	101.1	51
	19	117.78	98.975	39
	21.3	120.59	99.415	17.22
Chase White + Lime4% VACUUM SATURATION (VS)	14.6	115.45	100.74	34.31
	18.7	119.06	100.3	27.71
	19.53	118.93	99.498	21.82
	21.73	120.07	98.636	NA
Chase White + Lime4% HUMIDITY ROOM (HR)	14.6	116.83	101.95	53.3
	18.7	120.42	101.45	30
	19.53	121.06	101.28	19.73
	21.73	121.61	99.901	18.6
Chase White + Lime4% After 2 years curing HR	17.6	121	101.9	96
Chase White+ 4% Cement	20.4	120.87	100.39	8.85
Chase White + 4% Cement HR	20.22	121.26	100.87	37.87
Chase White +4% Cement VS	20.22	122.13	101.59	23.16
Chase White + 4% Cement After 2 years curing HR	15.7	123.3	106.6	210

Figure 2.1 D

Chemical Stabilization, Phase 1, Chase White

Soil	Moisture			Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
Chase White	10.0228	114.21	103.81	32.69
	13.0452	123.66	109.39	40.19
	15.5856	125.01	108.15	23.23
	17.0646	124.08	105.99	16.48
Chase White + Lime 2% + FA 8% DIRECT	13.26	111.62	98.552	64.6
	17.35	121.5	103.54	59
	18.16	120.68	102.13	27.82
	21.75	118.35	97.207	7.5
Chase White + Lime 2% + FA 8% RAPID CURING (RC)	11.57	111.28	99.74	70.61
	15.41	119.85	103.85	80.67
	17.77	120.11	101.99	53.26
	21.5	116.46	95.852	14.42
Chase White + Lime 2% + FA 8% VACUUM SATURATION (VS)	13.4	109.55	96.605	26.82
	17.68	119.14	101.24	36.38
	19.2	120.09	100.75	12.35
	21.75	119.13	97.848	6.34
Chase White + Lime 2% + FA 8% (LFA) HUMIDITY ROOM (HR)	13.4	111.23	98.086	65.14
	17.68	120.24	102.18	55.21
	19.2	121.36	101.81	51.44
	21.75	118.54	97.363	16.7
Chase White + LFA After 2 years curing HR	16.8	123	105.31	60
Chase White+ 4% Cement	20.4	120.87	100.39	8.85
Chase White + 4% Cement HR	20.22	121.26	100.87	37.87
Chase White +4% Cement VS	22.7	122.13	99.535	23.16
Chase White + 4% Cement After 2 years curing HR	15.7	123.3	106.6	210

Figure 2.2 D

Chemical Stabilization, Phase 1, Chase White

Soil	Moisture	γ_{moist}	γ dry	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
Deridder White	8.21	117.06	108.18	34.45
	10.24	120.35	109.17	27.21
	12.56	126.07	112	25.74
	15.1	124.93	108.54	17.13
Deridder White + Lime4% DIRECT	9.40	117.95	107.82	42.12
	12.80	124.54	110.41	35.71
	14.17	126.69	110.97	34.5
	16.40	123.05	105.71	9.95
Deridder White + Lime4% RAPID CURING (RC)	7.78	116.94	108.5	60.86
	11.08	123.36	111.06	51.87
	12.5	125.38	111.45	51.48
	15.2	123.2	106.94	22.58
Deridder White + Lime4% VACUUM SATURATION (VS)	9.66	115.36	105.2	27.55
	13	122	107.96	31.79
	14.2	126.5	110.77	33.51
	17.2	121.31	103.51	12.6
Deridder White + Lime4% HUMIDITY ROOM (HR)	9.66	117.3	106.97	43.92
	13	125.15	110.75	39.38
	14.2	126.66	110.91	42.2
	17.2	122.71	104.7	18.8
Deridder White + Lime4% After 2 years curing HR	17	130	111.1	27
Deridder White+ 4% Cement	15.63	126.83	109.69	12.12
Deridder White + 4% Cement HR	14.88	126.07	109.74	48.53
Deridder White +4% Cement VS	14.88	127.32	110.83	33.27
Deridder White + 4% Cement After 2 years curing HR	13	129	114.16	96

Figure 3.1 D

Chemical Stabilization, Phase 1, Deridder White

Soil	Moisture	γ_{moist}	γ dry	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
Deridder White	8.21	117.06	108.179	34.45
	10.24	120.35	109.171	27.21
	12.56	126.07	112.002	25.74
	15.1	124.93	108.54	17.13
Deridder White + Lime 2% + FA 8% DIRECT	9	113.11	103.77	38.16
	12.06	119.42	106.57	38.3
	13.35	121.26	106.98	35.3
	16.84	124.98	106.97	12.55
Deridder White + Lime 2% + FA 8% RAPID CURING (RC)	7.65	112	104.041	52.16
	11	118.17	106.459	52.66
	12.31	121.77	108.423	54.23
	15.15	120	104.212	36.07
Deridder White + Lime 2% + FA 8% VACUUM SATURATION (VS)	9	110.58	101.45	24.61
	12.26	116.73	103.982	27.7
	14	120.48	105.684	32.53
	17.32	120.79	102.958	12.63
Deridder White + Lime 2% + FA 8% (LFA) HUMIDITY ROOM (HR)	9	111.9	102.661	52
	12.26	119.29	106.262	57.51
	14	121.7	106.754	54.24
	17.32	124.62	106.222	25.42
Deridder White + LFA After 2 years curing HR	17.5	130	110.638	43.5
Deridder White+ 4% Cement	15.63	126.83	109.686	12.12
Deridder White + 4% Cement HR	14.88	126.07	109.741	48.53
Deridder White +4% Cement VS	14.88	127.32	110.829	33.27
Deridder White + 4% Cement After 2 years curing HR	13	129	114.16	96

Figure 3.2 D

Chemical Stabilization, Phase 1, Deridder White

Soil	Moisture	γ**moist**	γ **dry**	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
K1-1	9.77	120.25	109.55	25.48
	12.2	124.35	110.83	19.28
	14.03	126.72	111.13	18.72
	14.9	126.48	110.08	15.59
K1-1 + Lime4% DIRECT	9.5	112.89	103.10	29.83
	12.55	119.4	106.09	27.49
	15.06	124.13	107.88	23.95
	17.22	123.22	105.12	11.11
K1-1 + Lime4% RAPID CURING (RC)	8.28	112.5	103.90	122.08
	11.88	121.33	108.45	116.5
	13.95	123.94	108.77	113.65
	17.42	119.95	102.15	45.02
K1-1 + Lime4% VACUUM SATURATION (VS)	8.28	113	104.36	110.85
	11.88	118.5	105.92	120.13
	13.95	123.42	108.31	64.09
	17.42	122.6	104.41	20.5
K1-1 + Lime4% HUMIDITY ROOM (HR)	9.15	113.59	104.07	54.84
	12.49	120.04	106.71	45.27
	15.08	124.7	108.36	44.13
	16.92	122.99	105.19	24.65
K1-1 + Lime4% After 2 years curing HR	14.6	124.1	108.29	96
K1-1+ 4% Cement	18	125.05	105.97	9
K1-1 + 4% Cement HR	17.7	125.18	106.36	66.11
K1-1 +4% Cement VS	17.7	125.79	106.87	42.38
K1-1 + 4% Cement After 2 years curing HR	14.3	124.8	109.20	192

Figure 4.1 D

Chemical Stabilization, Phase 1, K1-1

Soil	Moisture	γ_{moist}	γ dry	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
K1-1	9.77	120.25	109.55	25.48
	12.2	124.35	110.83	19.28
	14.03	126.72	111.13	18.72
	14.9	126.48	110.08	15.59
K1-1 + Lime 2% + FA 8% DIRECT	9.04	112	102.71	40.9
	11.3	116.45	104.63	39.66
	13	119.35	105.62	32.55
	15.22	122.67	106.47	31
K1-1 + Lime 2% + FA 8% RAPID CURING (RC)	8.36	110.2	101.7	70.19
	10.41	115.02	104.18	76.04
	12.53	117.93	104.8	66.91
	14.55	122.2	106.68	66.7
K1-1 + Lime 2% + FA 8% VACUUM SATURATION (VS)	9.61	110.1	100.45	24.88
	11.62	114.86	102.9	32.07
	14.12	119.37	104.6	42.15
	15.63	122.51	105.95	41.03
K1-1 + Lime 2% + FA 8% (LFA) HUMIDITY ROOM (HR)	9.61	111.85	102.04	64.32
	11.62	115.14	103.15	63.60
	14.12	120.61	105.69	59.83
	15.63	121.42	105.01	47.56
K1-1 + LFA After 2 years curing HR	14	125.3	109.91	61.5
K1-1+ 4% Cement	18	125.05	105.97	9
K1-1 + 4% Cement HR	17.7	125.18	106.36	66.11
K1-1 +4% Cement VS	17.7	125.79	106.87	42.38
K1-1 + 4% Cement After 2 years curing HR	14.3	124.8	109.19	192

Figure 4.2 D

Chemical Stabilization, Phase 1, K1-1

Soil	Moisture	γ_{moist}	γ dry	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
K2-1	9.81	120.45	109.69	39.8
	12.22	127.35	113.48	39.74
	14.43	131.11	114.58	29.56
	16.53	127.36	109.29	15.71
K2-1 + Lime4% DIRECT	9.63	112.14	102.29	56.15
	11.3	116.2	104.4	48.96
	14.31	122.65	107.3	43.06
	16.36	124.69	107.16	35.73
K2-1 + Lime4% RAPID CURING (RC)	8.53	111.6	102.83	93.36
	10.5	114.84	103.93	97.63
	13.2	121.8	107.6	92.5
	15.56	123.15	106.57	88.01
K2-1 + Lime4% VACUUM SATURATION (VS)	10	112.83	102.57	50.79
	11.82	115.28	103.09	51.34
	15.21	122.36	106.21	63.37
	16.64	125.26	107.39	56.55
K2-1 + Lime4% HUMIDITY ROOM (HR)	10	111.5	101.36	61.55
	11.82	114.7	102.58	57.92
	15.21	121.81	105.73	51.51
	16.64	124.43	106.68	50.13
K2-1 + Lime4% After 2 years curing HR	13.7	125.3	110.2	110
K2-1+ 4% Cement	16.87	127.09	108.74	24.08
K2-1 + 4% Cement HR	17.38	127.33	108.48	122.17
K2-1 +4% Cement VS	17.38	128.25	109.26	73
K2-1 + 4% Cement After 2 years curing HR	16.6	129	110.6	240

Figure 5.1 D

Chemical Stabilization, Phase 1, K2-1

Soil	Moisture	γ_{moist}	γ dry	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
K2-1	9.81	120.45	109.69	39.8
	12.22	127.35	113.48	39.74
	14.43	131.11	114.58	29.56
	16.53	127.36	109.29	15.71
K2-1 + Lime 2% + FA 8% DIRECT	9.11	112.12	102.76	44.36
	11.38	116.8	104.87	52.47
	13.38	121.52	107.18	50.65
	15.64	126.07	109.02	38.9
K2-1 + Lime 2% + FA 8% RAPID CURING (RC)	8.51	111.78	103.01	81.1
	11	115.75	104.28	97.96
	13.08	120.54	106.6	86.6
	15.26	122.1	105.93	83.3
K2-1 + Lime 2% + FA 8% VACUUM SATURATION (VS)	9.15	111.14	101.82	33.49
	11.27	115.59	103.88	41.05
	13.97	120.95	106.12	41.32
	15.88	125.03	107.9	52.09
K2-1 + Lime 2% + FA 8% (LFA) HUMIDITY ROOM (HR)	9.15	111.11	101.8	56
	11.27	117.05	105.19	85.6
	13.97	121.78	106.85	73.73
	15.88	125.35	108.17	69.32
K2-1 + LFA After 2 years curing HR	14	123.5	108.3	100
K2-1+ 4% Cement	16.87	127.09	108.74	24.08
K2-1 + 4% Cement HR	17.38	127.33	108.48	122.17
K2-1 +4% Cement VS	17.38	128.25	109.26	73
K2-1 + 4% Cement After 2 years curing HR	16.6	129	110.6	240

Figure 5.2 D

Chemical Stabilization, Phase 1, K2-1

Soil	Moisture	γ_{moist}	γ dry	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
K3-1	9.65	118.38	107.96	41.84
	13.2	128.87	113.84	34.14
	14.6	127.54	111.29	19.92
	16.07	126.48	108.97	13.22
K3-1 + Lime4% DIRECT	9.54	111.87	102.13	41.71
	12.25	118.08	105.19	43.88
	14.68	124.2	108.3	39.1
	16.87	123.61	105.77	21.2
K3-1 + Lime4% RAPID CURING (RC)	9	112.12	102.86	67.6
	11.7	117.39	105.09	67
	14.26	123.27	107.89	62.05
	16.52	122.66	105.27	40.27
K3-1 + Lime4% VACUUM SATURATION (VS)	9.42	112.28	102.61	23.78
	12.58	116.89	103.83	29.2
	14.47	121.44	106.09	31.89
	17.03	122.48	104.66	18.67
K3-1 + Lime4% HUMIDITY ROOM (HR)	9.42	111.95	102.31	54.71
	12.58	118.76	105.49	52.06
	14.47	122.7	107.19	45.73
	17.03	122.7	104.84	23.64
K3-1 + Lime4% After 2 years curing HR	14	125.9	110.4	89
K3-1+ 4% Cement	16.85	126.86	108.57	22.5
K3-1 + 4% Cement HR	16.75	127.29	109.03	100
K3-1 +4% Cement VS	16.75	126.39	108.26	54.8
K3-1 + 4% Cement After 2 years curing HR	14.5	126.2	110.2	173

Figure 6.1 D

Chemical Stabilization, Phase 1, K3-1

Soil	Moisture	γ_{moist}	γ dry	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
K3-1	9.65	118.38	107.96	41.84
	13.2	128.87	113.84	34.14
	14.6	127.54	111.29	19.92
	16.07	126.48	108.97	13.22
K3-1 + Lime 2% + FA 8% DIRECT	9.27	110.32	100.96	49.92
	12.01	115.19	102.84	51.43
	14.1	118.37	103.74	45.52
	16.87	123.55	105.72	40.21
K3-1 + Lime 2% + FA 8% RAPID CURING (RC)	8.24	108.57	100.3	69
	11	113.24	102.02	75.23
	13.09	116.52	103.03	69
	16	122.84	105.9	63.07
K3-1 + Lime 2% + FA 8% VACUUM SATURATION (VS)	9.8	108.65	98.953	23.5
	12.65	114.23	101.4	36.77
	14.5	117.53	102.65	32.78
	17.5	123.5	105.11	NA
K3-1 + Lime 2% + FA 8% (LFA) HUMIDITY ROOM (HR)	9.8	109.05	99.317	69.07
	12.65	114.05	101.24	70.6
	14.5	117.5	102.62	63
	17.5	123.63	105.22	55.73
K3-1 + LFA After 2 years curing HR				
	14.8	125.9	109.7	128
K3-1+ 4% Cement	16.85	126.86	108.57	22.5
K3-1 + 4% Cement HR	16.75	127.29	109.03	100
K3-1 +4% Cement VS	16.75	126.39	108.26	54.8
K3-1 + 4% Cement After 2 years curing HR				
	14.5	126.2	110.2	173

Figure 6.2 D

Chemical Stabilization, Phase 1, K3-1

APPENDIX E

Soil	Moisture	γ_{moist}	γ dry	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
CHASE BROWN	24.86	117.95	94.466	11.78
SOIL + 10% PC: Direct	20.57	124.43	103.2	30.36
Cured (in HR for 2 weeks)	17.94	125.78	106.65	179.58
VS	17.94	125.84	106.7	72.63
SOIL+Lime 6%:Direct	22.66	121.27	98.867	32.3
Rapid Curing (RC)	21.1	118.44	97.803	254
VS	21.1	118.11	97.531	55.41
SOIL + FA10%: Direct	20.58	122.73	101.78	17.14
Cured (in HR for 2 weeks)	20.29	124.76	103.72	40.9
VS	20.29	125.22	104.1	26.6
SOIL +L3% FA10%: Direct	20.79	122.85	101.71	37.04
RC	19.3	120.25	100.8	203.73
VS	19.3	119.91	100.51	27

RC = Accel curing:3 days in oven @50^0 C

VS= Vacuum Saturation: 30 min deair and 1 hour complete inundate

Figure 1 E

Chemical Stabilization, Phase 2, Chase Brown

Soil	Moisture	γ_{moist}	γ dry	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
CHASE WHITE	19.67	122.5	102.36	15.34
SOIL + PC: Direct	17.17	124.3	106.09	72.62
Cured (in HR for 2 weeks)	17.59	124.44	105.83	130
VS	17.59	123.91	105.37	75.49
SOIL+Lime 6%:Direct	17.5	119.1	101.36	38.3
Rapid Curing (RC)	13.72	118.61	104.3	310
VS	13	106.5	94.248	275.27
SOIL + FA10%: Direct	17.13	124.34	106.16	13.38
Cured (in HR for 2 weeks)	16.58	122.8	105.34	52.71
VS	16.58	123.88	106.26	24
SOIL +L3% FA10%: Direct	16.34	123.69	106.32	71.07
RC	11.34	119.61	107.43	235
VS	11.88	116.8	104.4	81.7

RC = Accel curing:3 days in oven @50^0 C

VS= Vacuum Saturation: 30 min deair and 1 hour complete inundate

Figure 2 E

Chemical Stabilization, Phase 2, Chase White

Soil	Moisture	γ_{moist}	γ dry	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
DERIDDER Brown	19.91	121.1	100.99	13.11
SOIL + PC 10 %: Direct	17.26	127.85	109.03	34.75
Cured (in HR for 2 weeks)	17.4	130.17	110.88	293.25
VS	17.4	129.18	110.03	239.8
SOIL+Lime 6%:Direct	18.06	125.9	106.64	33.14
Rapid Curing (RC)	12.71	120.47	106.88	518
VS	12.71	121.27	107.59	126.5
SOIL + FA10%: Direct	16.97	125.88	107.62	24.05
Cured (in HR for 2 weeks)	16.37	128.4	110.34	64.3
VS	16.37	129.86	111.59	33.42
SOIL +L3% FA10%: Direct	16.31	127.75	109.84	78.75
RC	12.07	122.94	109.7	301.6
VS	12.07	123.33	110.05	36

RC = Accel curing:3 days in oven @50^0 C

VS= Vacuum Saturation: 30 min deair and 1 hour complete inundate

Figure 3 E

Chemical Stabilization, Phase 2, Deridder Brown

Soil	Moisture	γ_{moist}	γ_{dry}	Strength	
	%	lb/ft^3	lb/ft^3	lb/in^2	kPa
DERIDDER WHITE	19.42	128.43	107.54	14.86	102.456
SOIL + PC (8% by weight): Direct	16.36	127.95	109.96	16.43	113.281
Cured (in HR for 2 weeks)	16.28	125.43	107.87	83.35	574.678
VS	16.28	126.38	108.69	34.87	240.42
SOIL+Lime 6%:Direct	17.07	125.56	107.25	15.6	107.558
Rapid Curing (RC)	14.1	122.21	107.11	58.85	405.756
VS	14.1	122.26	107.15	24.8	170.99
SOIL + FA10%: Direct	16.24	126.68	108.98	11.47	79.0829
Cured (in HR for 2 weeks)	17.1	111.97	95.619	16.45	113.419
VS	NA, disintegrated				0
SOIL +L3% FA10%: Direct	15.32	126.89	110.03	31	213.737
RC	12.43	124.8	111	62.24	429.13
VS	12.43	124.14	110.42	30	206.843

RC = Accel curing:3 days in oven @50^0 C

VS= Vacuum Saturation: 30 min deair and 1 hour complete inundate

Figure 4 E

Chemical Stabilization, Phase 2, Deridder White

APPENDIX F

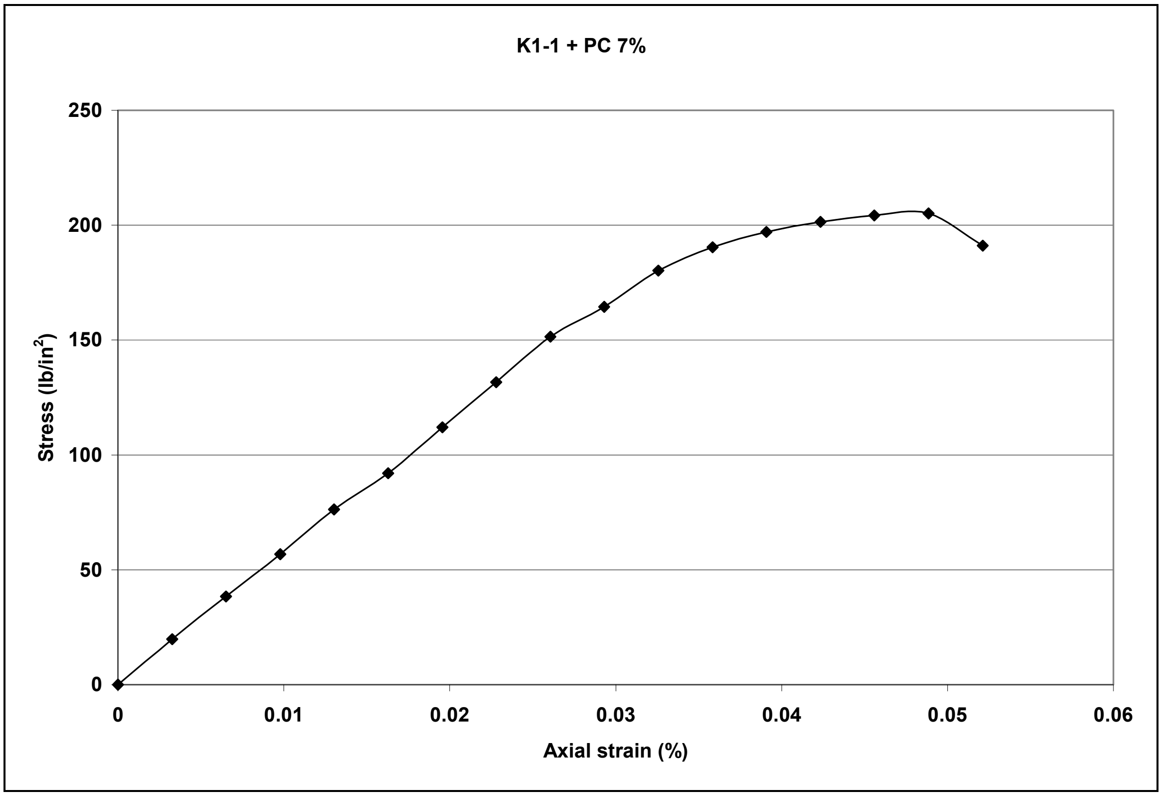

Figure 1 F

Unconfined Compression strength for K1-1+PC 7%

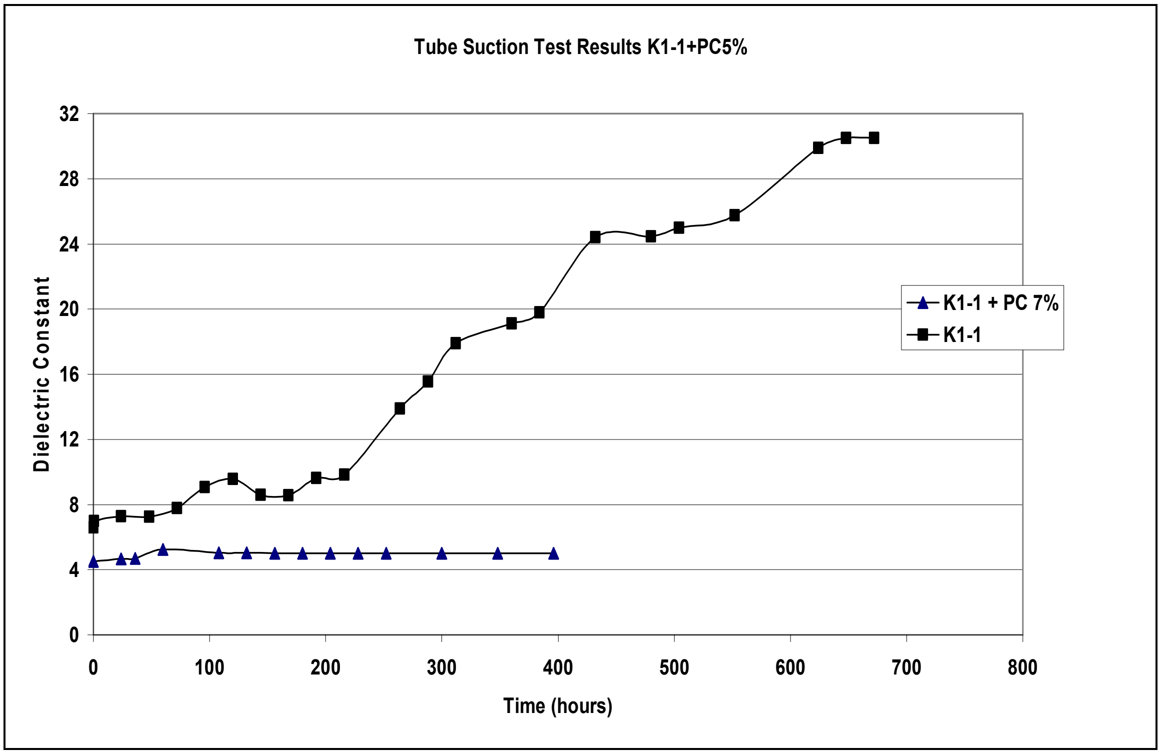

Figure 2F

Tube Suction Results for K1-1 before and after stabilization

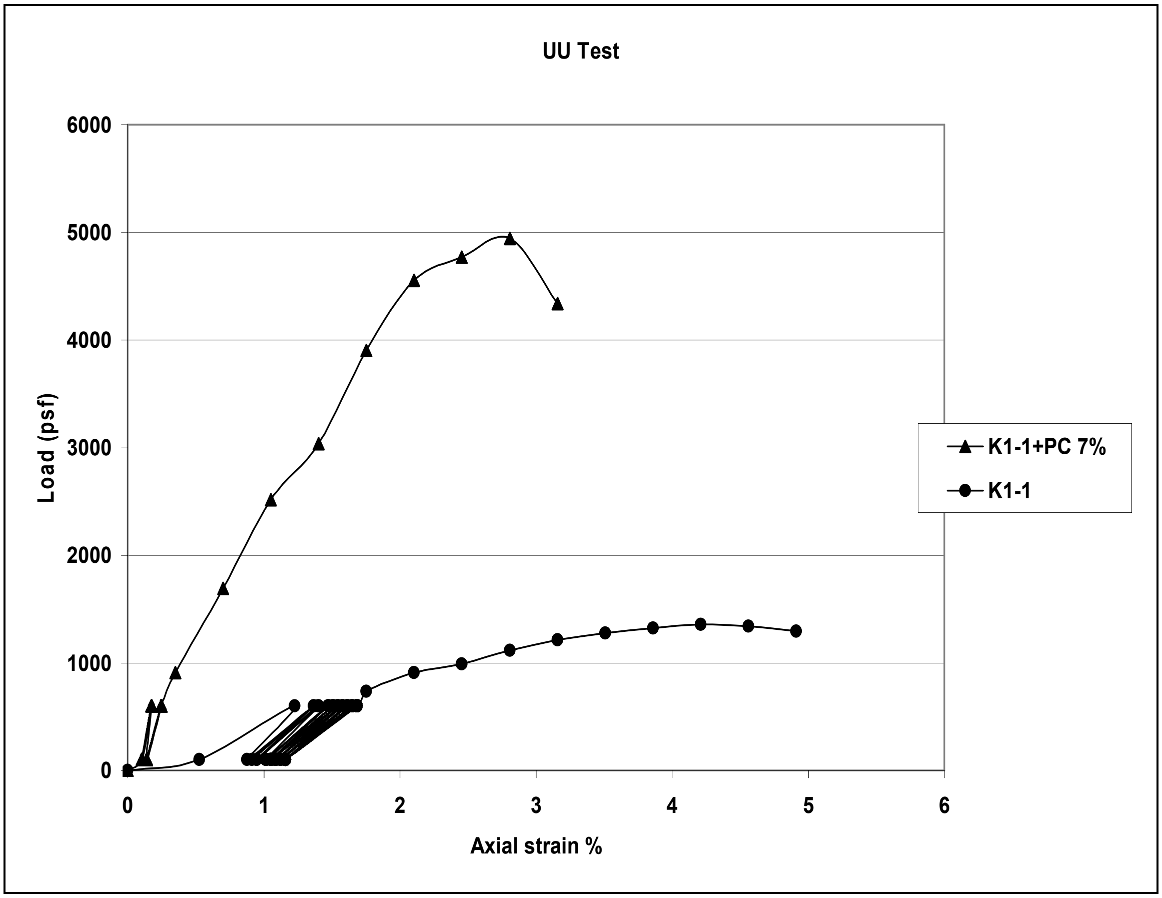

Figure 3F

Cyclic Triaxial Test K1-1

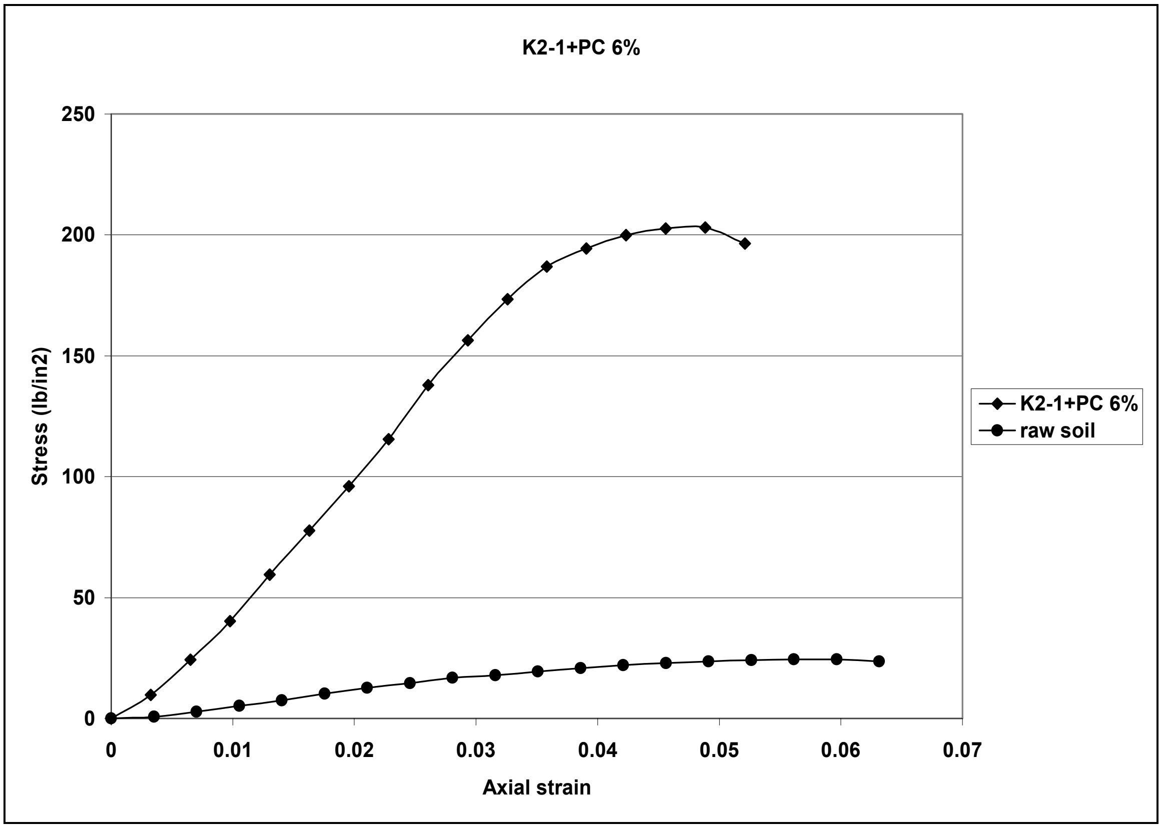

Figure 4 F

Unconfined Compression strength for K2-1+PC 6%

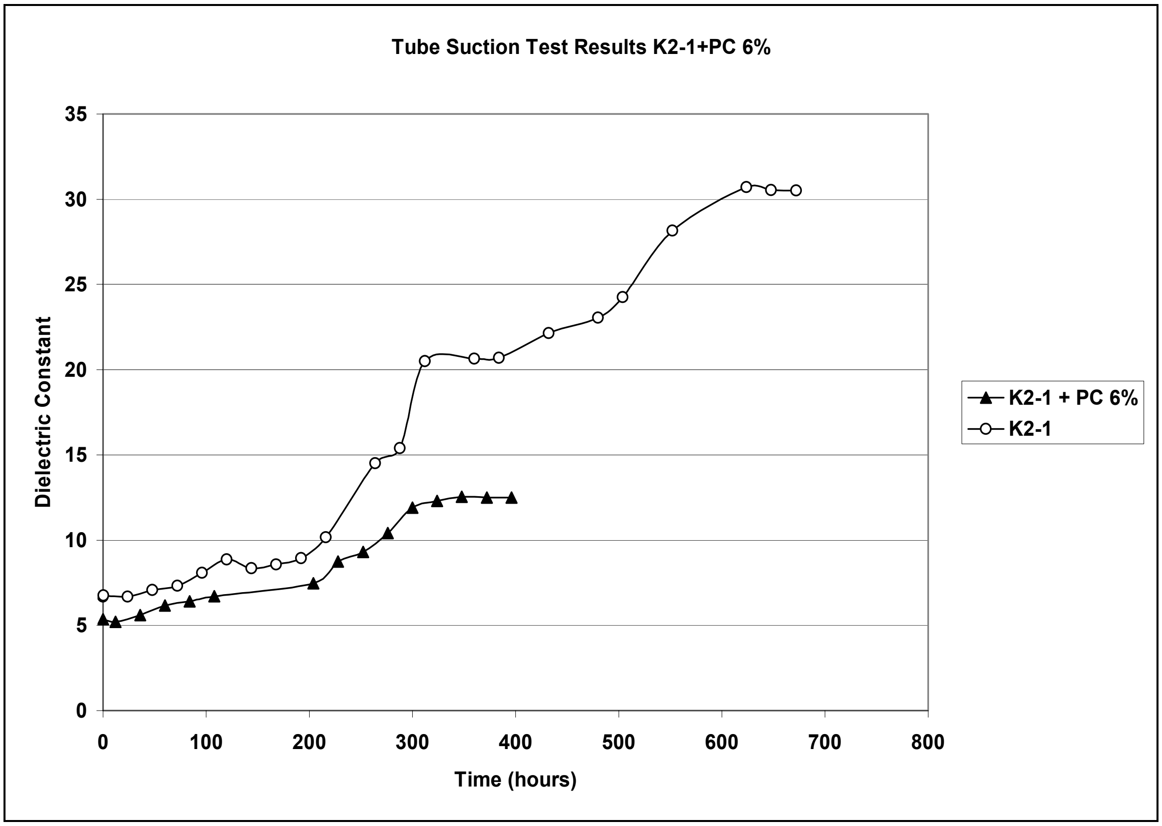

Figure 4F

Tube Suction Results for K2-1 before and after stabilization

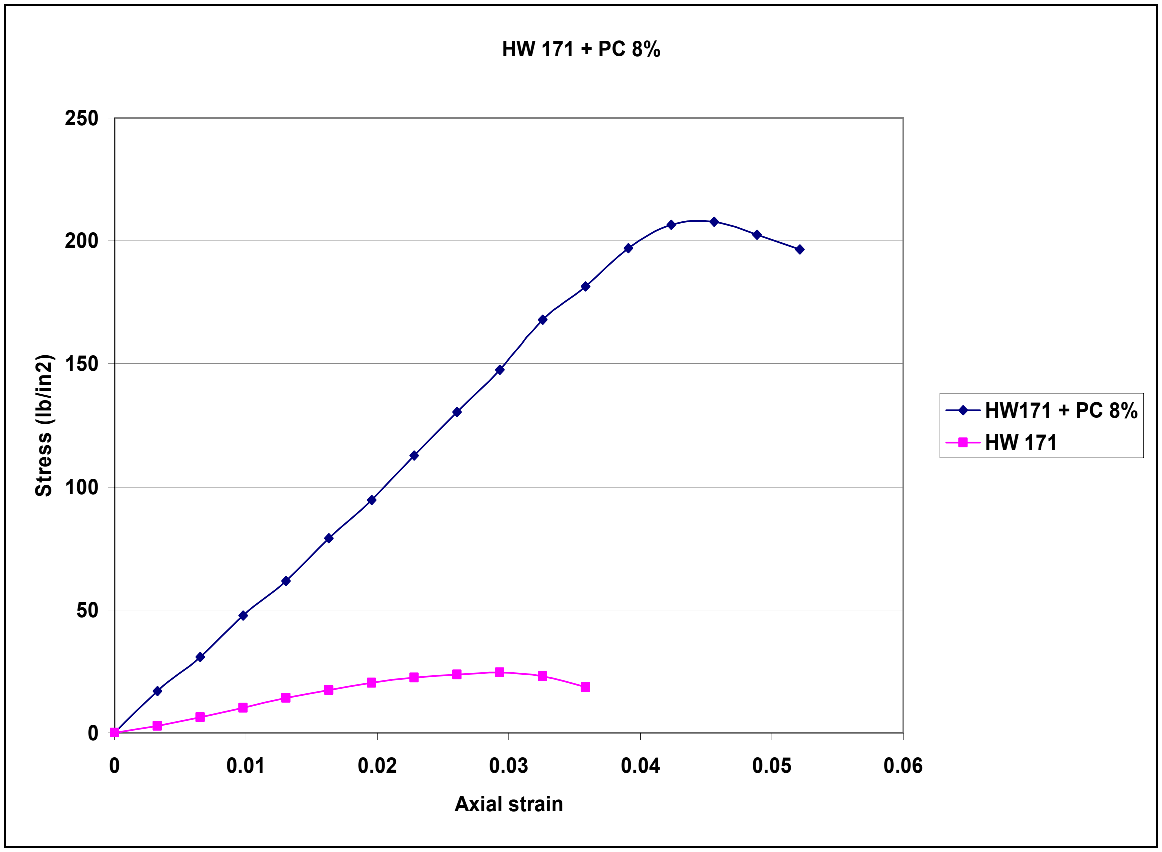

Figure 6 F

Unconfined Compression strength for HW 171

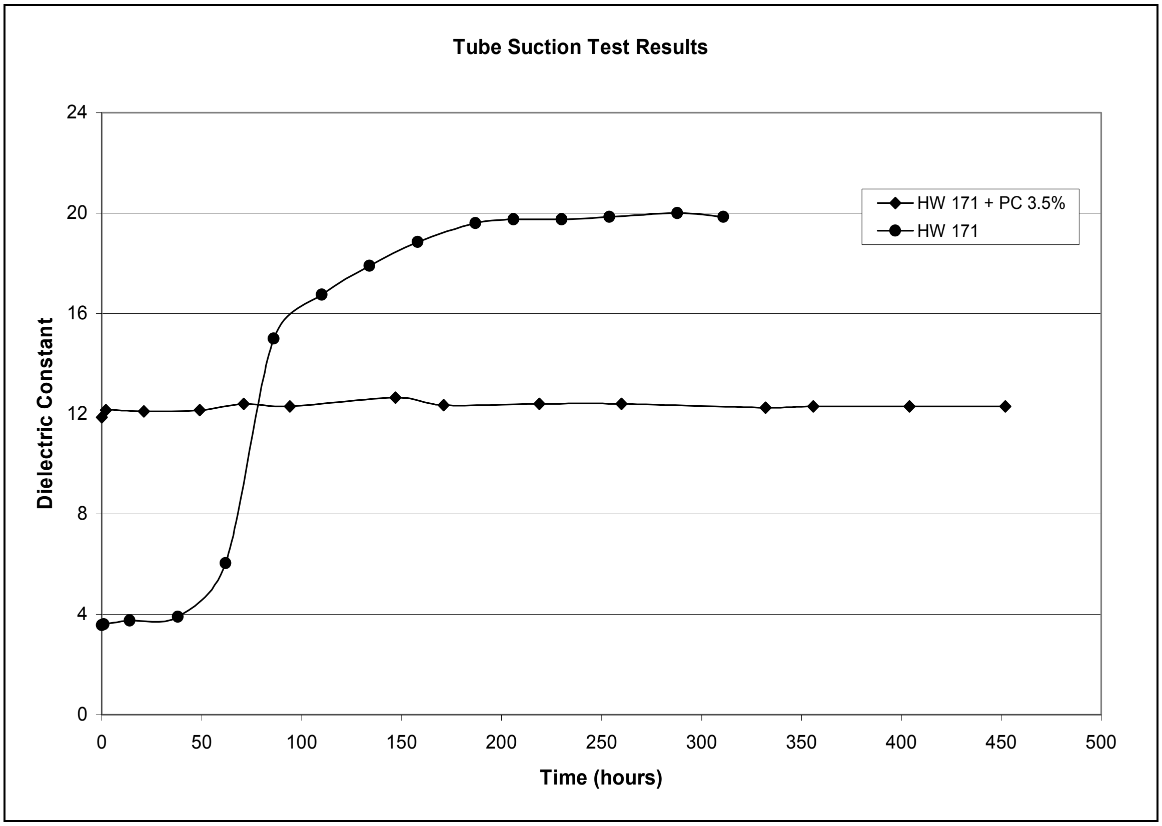

Figure 7F

Tube Suction Results for HW 171 before and after stabilization

APPENDIX G

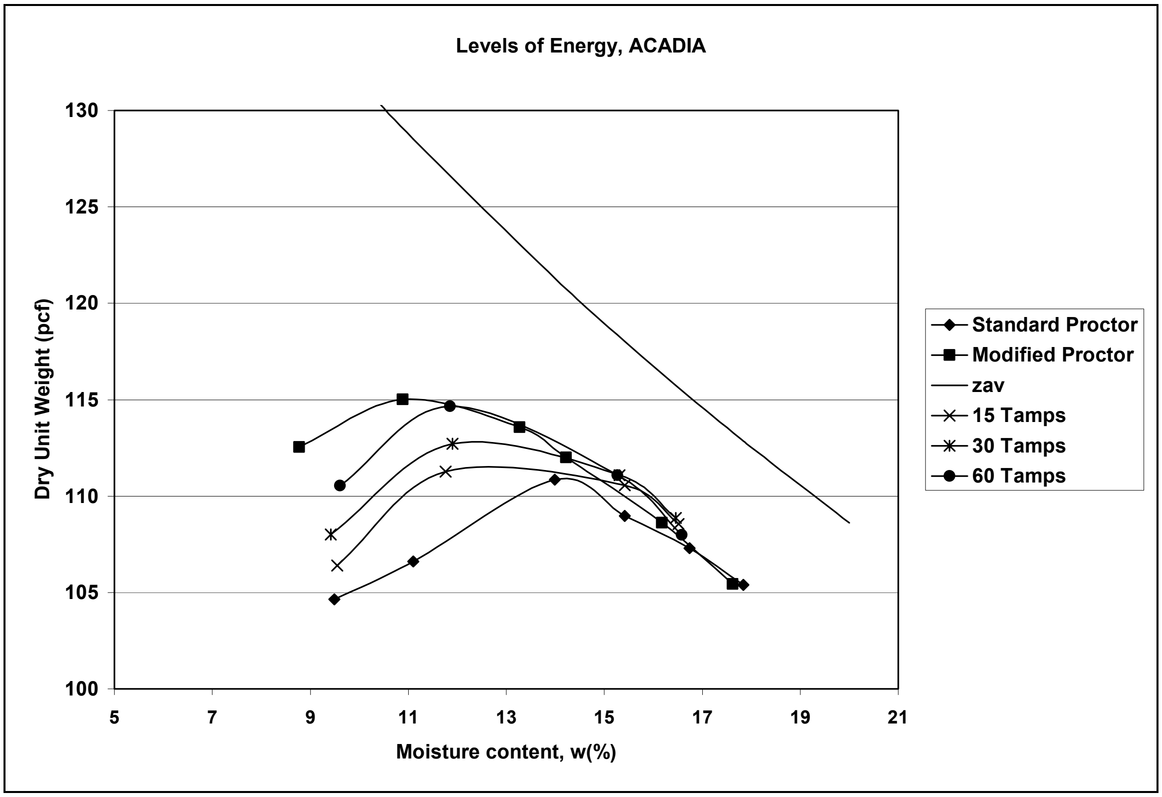

Figure 1 G

Levels of energy, Acadia

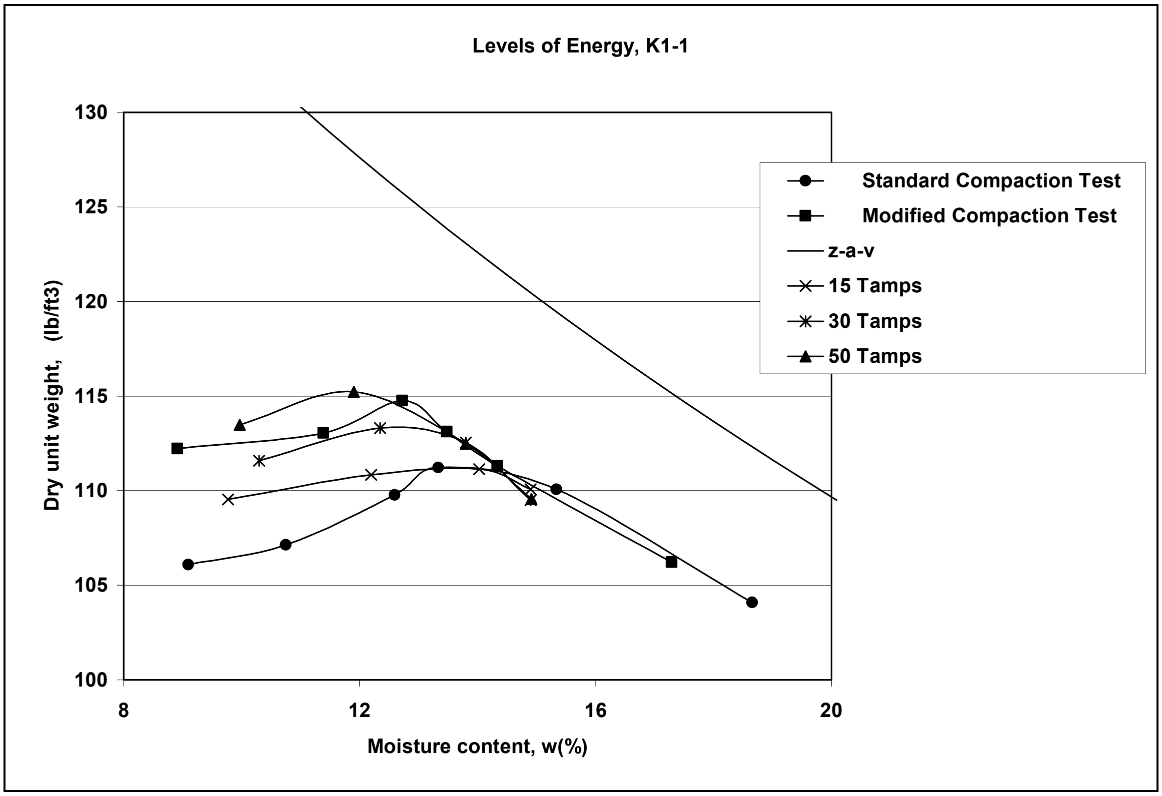

Figure 2 G

Levels of energy K1-1

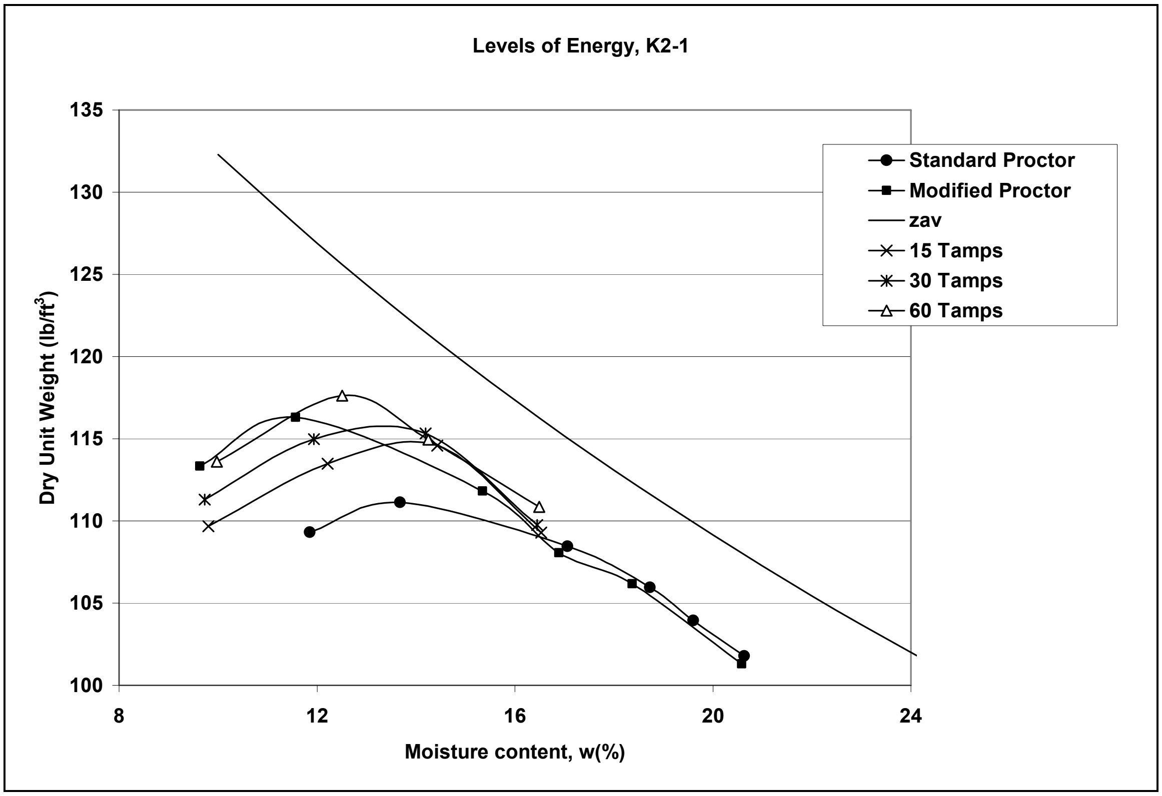

Figure 3 G

Levels of energy K2-1

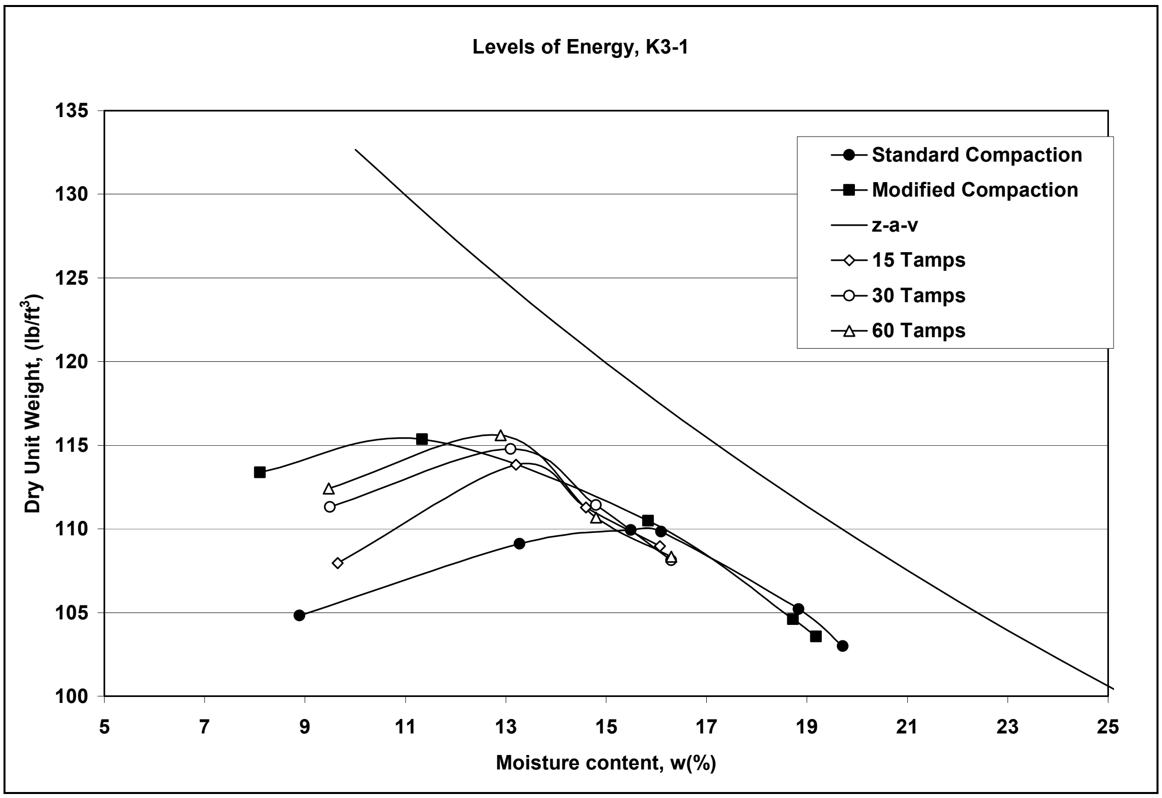

Figure 4 G

Levels of energy, K3-1

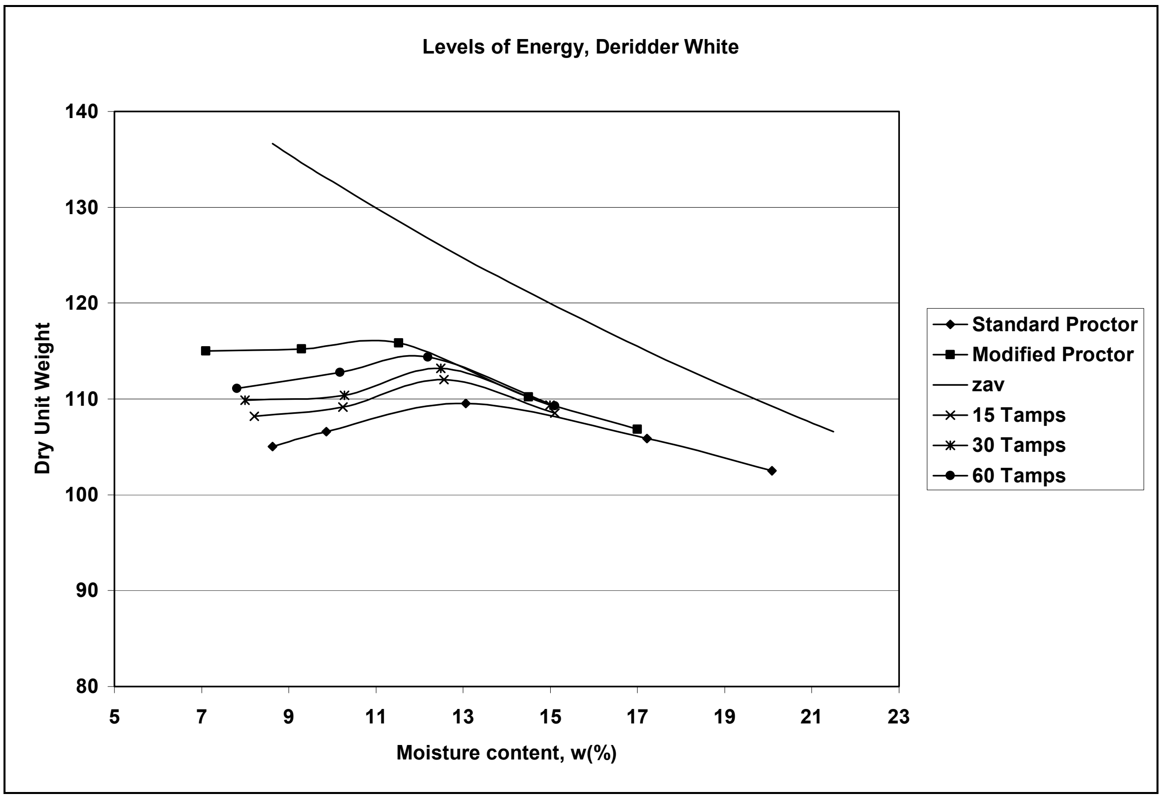

Figure 5 G

Levels of energy Deridder White

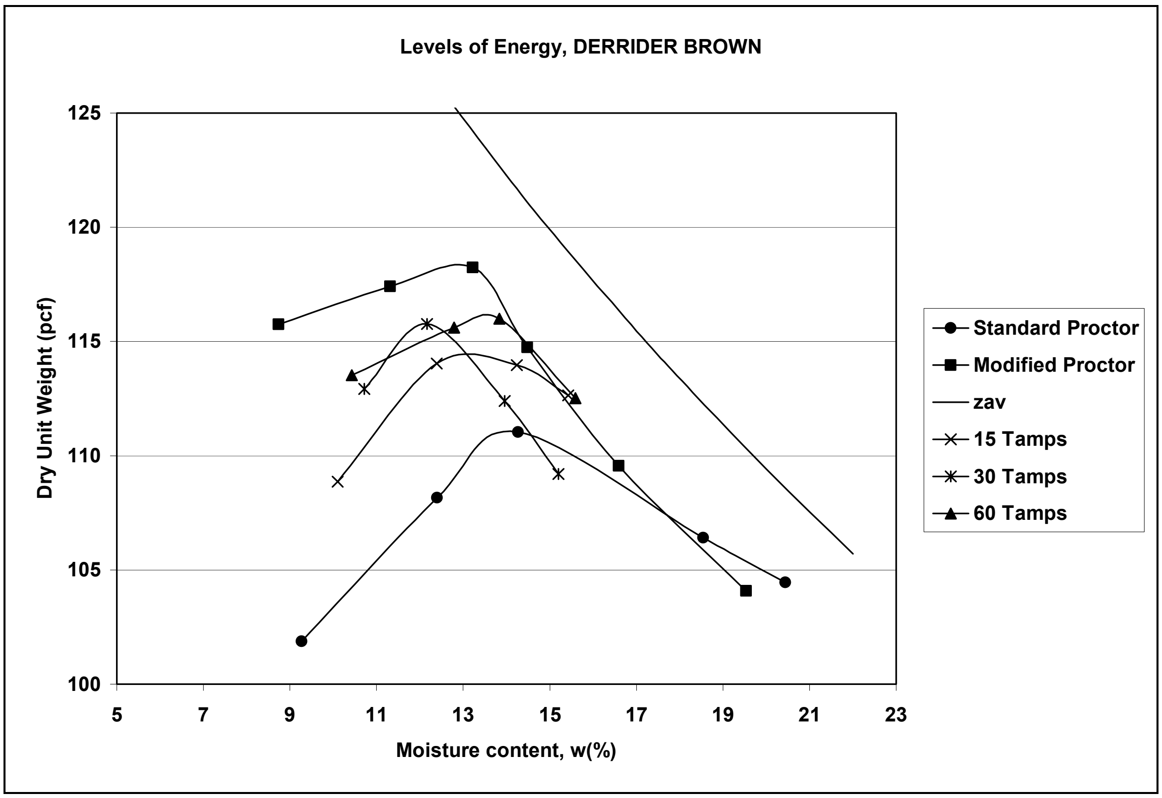

Figure 6 G

Levels of energy Deridder Brown

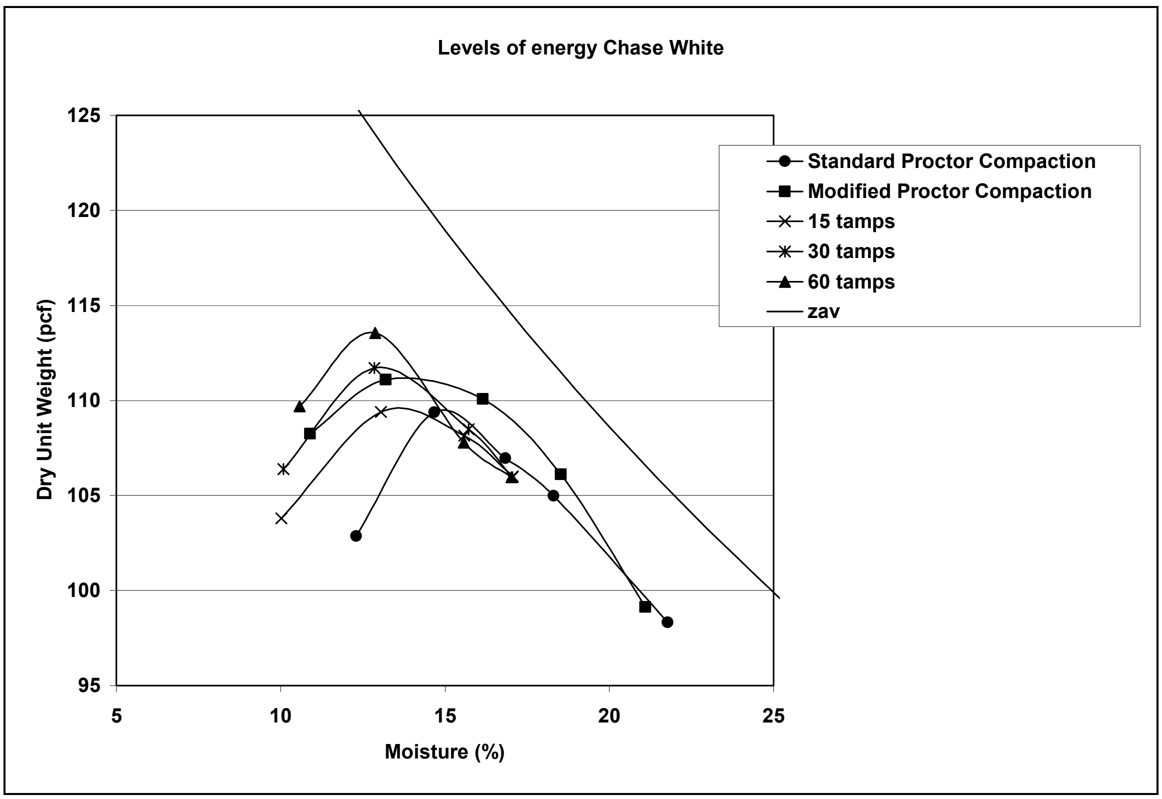

Figure 7 G
Levels of energy Chase White

VITA

Mr. Barbu Bogdan was born in Ploiesti, Romania and received his B.Sc from University of Ploiesti in 1994, followed by his M.Sc in 1995 from the same university.

From 1994 to 1998, Mr. Bogdan Barbu was employed as Engineering Intern, Staff Engineer, and Project Engineer with “GEOTEC” Institute, Bucharest, Romania.

In the fall of1999, Mr. Bogdan Barbu began his doctoral studies in civil engineering at the University of New Orleans.