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16. Abstract

Many areas of Louisiana consist of soils with high silt contents, low strengths, and minimal bearing capacity. Construction traffic in these soils can cause detrimental pumping action in areas with a high water table. These wet subgrades under Louisiana pavements cause both construction and performance problems. Common solutions to the problem include excavation and replacement, lime treatment, or cement stabilization. Special provisions are often included in the contract for chemical additives in lieu of undercutting.

The research emphasis of this study was placed on efforts to refine the pumping problem and on the development of guidelines for identifying the problem silt-soils. Secondary importance was given to the identification of alternate methods for stabilization.

The study consisted of two phases. Phase 1 documented the field experiences of the DOTD districts. Phase 2 consisted of a testing program to investigate the nature of the problem, the character of the silt materials, and their performance with modifying and stabilizing agents. Eight soil samples from four of the DOTD districts were used in the laboratory program. The soils were typical examples of those commonly encountered with a high-silt content. Several were acquired from current projects in which pumping problems were occurring.

The basic characteristic-parameters of the natural samples 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 significant during testing. Anomalies were also found to exist between the DOTD's earthwork specifications and the physical properties of the high silt-content soils.

The potential for the modification and 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 in order that they 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. Limited tests for evaluating long-period stability of the stabilized silt-subgrade subjected to accelerated curing followed by vacuum-saturation conditions were also conducted.

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Identification and Stabilization Methods for Problematic Silt Soils

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ABSTRACT

Many areas of Louisiana consist of soils with high silt contents, low strengths, and minimal bearing capacity. Construction traffic in these soils can cause detrimental pumping action in areas with a high water table. These wet subgrades under Louisiana pavements cause both construction and performance problems. These problematic soils have contributed to the establishment of the Louisiana Department of Transportation and Development (DOTD) standard specification's definition of usable soils, consisting of a maximum of 65 percent silt content for embankments and a maximum of 60 percent for chemically stabilized bases. Soils, which do not meet these requirements, must be removed and replaced. However, this requirement has become very expensive especially when usable soils must be hauled any significant distance. Common solutions to the problem include excavation and replacement and lime treatment or cement stabilization. Special provisions are often included in the contract for chemical additives in lieu of undercutting.

The research emphasis of this study was placed on efforts to refine the pumping problem and on the development of guidelines for identifying the problem silt-soils. Secondary importance was given to the identification of alternate methods for stabilization.

The study consisted of two phases. Phase 1 documented the field experiences of the DOTD districts. Phase 2 consisted of a testing program to investigate the nature of the problem, the character of the silt materials, and their performance with modifying and stabilizing agents. Eight soil samples from four of the DOTD districts were used in the laboratory program. The soils were typical examples of those commonly encountered with a high-silt content. Several were acquired from current projects in which pumping problems were occurring.

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The potential for the modification and 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 in order that they 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. Limited tests for evaluating long-period stability of the stabilized silt-subgrade subjected to accelerated curing followed by vacuum-saturation conditions were also conducted.

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Significant assistance and information was received from the engineering staff of the involved DOTD districts. Their insight and experience was very helpful in defining the problem and providing focus to the study. Many of the factors contributing to the problem and possible solutions were identified through personal communication and through the responses provided in survey questions. Their work experience and observations associated with specifications, construction, and design policy, and their agreement or lack of agreement with the material properties was invaluable.

The microanalysis and the mineral identification of the samples was performed by Mr. Al Falster, Laboratory Manager, UNO Department of Geology and Geophysics. The analysis and supervision of the cyclic-triaxial tests were provided by Dr. Gordon Boutwell of Soil Testing Engineers, Inc., Baton Rouge.

IMPLEMENTATION STATEMENT

To identifying pumping soils, fine-grained soils (including fine sands) containing silt percentages of 50 percent or more and whose plastic character is such that the Plastic Index (PI) is less than 10 should be considered as having a high pumping potential. The more plastic, high-silt soils (PI > 10) may pump under specific conditions but are less susceptible to pumping.

Current compaction specifications for soils, which allow a moisture range of –2 to +4 percent of optimum moisture for field compaction, should be changed to –2 to +2 percent of optimum moisture for the high-silt soils or those with a high potential for pumping.

Since, these soils are currently used with modification methods (primarily lime as a drying agent), it is recommended that a more specific end-product method be developed. A testing protocol for evaluating a mixture design and acceptance standards is needed to guide the selection for modification and stabilization of subgrade soils. They should include considerations for 1) placement and compaction, 2) construction activities after subgrade compaction, and 3) the design performance in service as a pavement subgrade.

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INTRODUCTION

Soils with high-silt contents are common to many areas in Louisiana. These high-silt soils frequently have low strengths and minimal bearing capacity. When located in areas with a high water table, soil compaction efforts and construction traffic can produce detrimental pumping action. Thus, the high-silt soils when wet and located under pavement subgrades or within pavement embankments can cause both construction and performance problems. When these soils are encountered, the solutions to these problems have generally included excavation and replacement, lime treatment, or cement stabilization.

The Louisiana Department of Transportation and Development standard specifications limits usable soils to a maximum of 65 percent silt content for use in embankments and a maximum of 60 percent silts for chemically stabilized bases (Louisiana Standard Specifications For Roads and Bridges, 1992 Edition). 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 has become very expensive, especially when usable soils must be hauled any significant distance. 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, their long-term performance is uncertain. Treatment with lime has been used commonly in an effort to temporarily dry the soil enough to construct the pavement. However, concern for long-term performance and integrity of the base and pavement structure has been expressed because premature failures may occur with 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 reported mixed results.

OBJECTIVES

The objectives of this research are to (1) identify the soil properties and characteristics that contribute to a pumping condition, (2) evaluate the effectiveness of selected chemical stabilization techniques, and (3) provide an evaluation and recommendation for alternative solutions.

SCOPE

The investigation emphasized further developing the description of the problem and method for identifying the pumping conditions for soils with high silt content through

1. a review of the experiences encountered by the LADOTD districts with silt soils (soil character, site conditions, and the construction experiences), and review of relevant literature,
2. a laboratory characterization program of tests with soils typical of the DOTD districts and identified as pumping during construction, not usable, and/or other marginally acceptable soils meeting the DOTD specifications section 203.06 for soil usage, and
3. a laboratory evaluation of the potential of stabilization methods limited to the more common techniques employing lime, fly ash, and Portland cement.

A variety of problems and a wide range of issues were identified through the initial district interviews. The Project Review Committee (PRC) recommended that the scope of the project be prioritized to focus on activities specific to the major objectives. Thus, in an effort to further focus the study for meeting the project objectives the scope was refined as follows:

1. The new DOTD specifications are being revised to require construction plans to identify unstable subgrade conditions and include items for correction. Thus, the primary objective of this research is to recommend a process and/or guide to identify soil characteristics that define an unstable subgrade condition. The PRC committee decided that the best way to accomplish this task was to obtain samples from existing construction projects that are experiencing pumping problems.
2. The second priority is to address the issue of usable soils for embankment construction. A survey of engineers and contractors will be made in an attempt to identify the proper techniques and control that should be used to achieve a satisfactory embankment.
3. The third objective identified was an effort to establish the potential for chemically modifying or stabilizing the problematic silts soils. The scope of this effort was limited with respect to the number of modifiers/stabilizers and the number of mix combinations. Three of the more common additives were selected for the study: lime, Portland cement, and Class C fly ash.

METHODOLOGY

Literature: State of Practice

In addition to considering the material properties that are unique to the high silt soils, a review of existing specifications, engineering and construction methodology, descriptions of related phenomena, and the investigations of others was made to assist in the identification and description of a pumping condition.

State of Practice – Subgrade and Embankment Earthwork

Specifications for earthwork are either given as end product specifications and/or methods specifications. Section 203.7 of the Louisiana Standard Specifications For Roads and Bridges (1992 Edition) requires compaction by approved methods to a minimum of 95.0 percent of maximum dry density. Maximum dry density is determined in accordance with DOTD TR 415 or TR 418 and percent in-place in accordance with DOTD TR 401. The moisture content required at the time of compaction must fall within a range of –2 percent and +4 percent of the optimum moisture content as established with DOTD TR 418. A specified density can be achieved with different combinations of moisture contents and compaction efforts.

Six LADOTD districts were included in the DOTD survey: Lafayette District 03, Lake Charles District 07, Alexandria District 08, Chase District 58, Baton Rouge District 61, and Hammond District 62. All six of the districts were contacted, and copies of the proposed research study were provided to the district construction and laboratory engineers. While the occurrence of high silt soils and problems associated with them appear to be common for much of the state, the description of the extent of problems and the solutions attempted varied in detail.

In discussions with the construction and laboratory engineers of the districts concerned, it was determined that the 95 percent relative compaction is based on field identified densities and moistures (using the 3-point method or Family of Curves, DOTD TR 415). However, the details, interpretation, and specifics may vary with different approaches. Some of those interviewed felt that the specifications were too broad for interpretations (Cryer). For example, -2 to +4 percent moisture range may be sufficient for clays but not sands or silts. Further descriptions of the issues that were raised in district office visits are presented in the discussion of results.

The Character of Silts and Unstable Soils

Silts are nonplastic fines. They are inherently unstable in the presence of water and have a tendency to become quick when saturated, i.e., they assume the character of a viscous fluid and can flow (Bureau of Reclamation, 1987). Also, they are fairly impervious, difficult to compact, and highly susceptible to frost heave. A silty soil undergoes a change in volume with a change in shape (dilatancy), while clays retain their volume with a change in shape (plasticity). A typical silt in a loose, wet state can be identified by its dilative character and quick reaction to vibration.

Silts can differ among themselves in grain size and shape. This is usually reflected in their compressibility. Generally, the higher the liquid limit of a silt, the more compressible it is. The liquid limit of a typical bulky-grained, inorganic silt is approximately 30 percent, whereas highly micaceous or diatomaceous silts (elastic silts), consisting of mainly flaky grains, may have liquid limits as high as 100 percent (Bureau of Reclamation, 1987).

Silty soils have the highest potential for frost damage. They are sufficiently fine-grained to promote high capillary forces but they have an adequate coarse-grain character to permit a high-rate of capillary conductivity. The basic conditions required for ice layers to form in soil include freezing temperatures in the soil, sources of water close enough to supply capillary water, and frost- susceptible soil type and grain (pore) size distribution. Systems for identifying frost susceptibility give silts, coarse clays (clay content 15-25 percent), and silty tills the highest potential for frost action damage (Hansbro 1975, Casagrande 1931).

Collapsible Silts

Metastable or 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 (Clemence and Finbarr, 1981). Rapid compression with excess pore water and loss of grain-to-grain content leads to liquefaction and or a complete loss of strength. The soils involved vary tremendously although the majority of these are silt size. Frequently, there is evidence of some clay content. The most extensive deposits, identified as collapsible soils, are wind-deposited silts and fine-sands. However, alluvial flood plains, fans, mud flows, colluvial deposits, and residual soils may produce unstable soils as well.

Lawton et al (1992) discussed the problems associated with collapsible settlements in compacted fills (damages to pavements and subgrades placed on highway embankments). It was suggested that the collapse of compacted soils occurs primarily as a result of the softening and distortion of micropeds. Compacted soils containing between 10 percent to 40 percent clay particles exhibited the highest collapse potential. The pre-wetting conditions of the soil, not the as-compacted conditions, determine its collapse potential. Potential post compaction changes in density, moisture, and stress must be evaluated to determine a reasonable estimation of collapse potential. There is a critical degree of saturation above which negligible collapse occurs. This degree of saturation varies with the overburden pressure and approximately follows the line of optimums found in Proctor compaction testing.

In southwest Louisiana, surface deposits of gray collapsible silt have been encountered. In dry weather, these silts support farm tractors, construction equipment, houses, and concrete slabs. However, when it rains, the silt loses its strength, and, when under pressure, it decreases in volume and flow like soft mud (Arman and Thornton, 1972). These silts are indistinguishable from the normal silts of the region. Areas of potentially unstable silt surround the known collapsing silt deposits. These areas generally include, but are not limited to, the soil areas of the Coastal Prairies, Loessal Hills, and Mississippi Terraces.

While the in-place unit weight of the undisturbed collapsible silt is 80 pcf, the maximum dry unit weight obtained with standard compaction is 104 pcf. Neither the grain-size, the grain-shape, the grain-structure, the fossil content, nor the Atterberg limits of the collapsible silts indicate the difference in behavior between the stable and unstable silts. The supernatant liquid of the silt is black after the solids have settled out of the solution of Calgon used in the hydrometer test. Arman and Thornton (1972) conducted further tests with mixtures of lime and cement. The results demonstrate that the collapsible silts can be stabilized with either lime or cement, provided that the construction equipment can incorporate them into the soil.

The Pumping Phenomenon

In the field, the specified density is achieved by progressively increasing the compaction effort. This may include using either heavier equipment or more passes. However, lower strength also results with higher compaction energies when the soil is wet of optimum. The conditions where this occurs are demonstrated qualitatively in Figure 1.

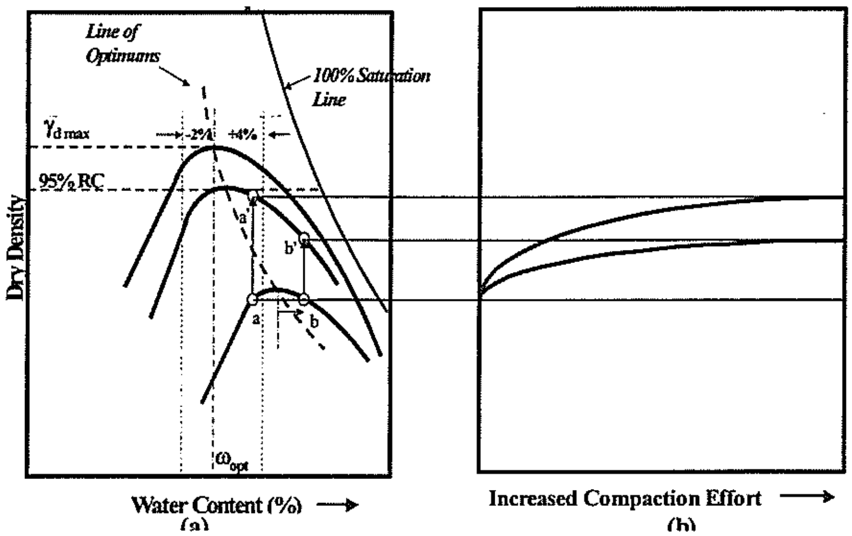


Figure 1

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 similarly but at different energy levels are shown. Points *a* and *b* on the curve represent two conditions of the soil compacted at the lower compaction and to the same density at different moisture contents. With increased compaction, the density of the drier moisture conditions Figure 1 (a) increases more rapidly than that of the wetter moisture content of *(b)*. The material of the lesser moisture content is stiffer, and the compaction energy helps produce a denser soil. However, the wetter material, *b*, is soft and the shear stresses imposed on the soil during compaction are greater than the shear strength. Under these conditions, pumping occurs with some soils. The compaction energy is dissipated largely in shearing the compacted material without adding much density. The soil behavior at this point has been identified as overcompaction (Holtz & Kovacs, Spangler and Handy). 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. If the soil is too wet and the applied compaction energy is too great, pumping or weaving occurs as the wheel shoves the weaker soil ahead of its motion. A sheepsfoot roller will not be able to walk out.

From the test and observations, the basic conditions that contribute to a pumping condition are:

1. The presence of a soil with characteristics susceptible to pumping,
2. Excess moisture conditions (above optimum) and/or access to water,

3. Excessive compaction effort.

Proof-Rolling or Test-Rolling

A technique known as proof-rolling or test-rolling has been employed to test the degree of compaction attained compared to the specified compaction. It generally involves the use of heavy to moderately heavy wheel load pneumatic-tire compactors to test the effectiveness of rolling. It is used to determine areas receiving insufficient coverage, weak areas, or excessively wet areas. It provides the engineer with a means to test the entire roadway instead of a few select spots and requires less interpolation of test data. It can, however, give a false sense of security for moisture content on the dry side of the proper moisture range. Compaction deficiencies can only be determined for areas with excessive moisture. While additional compaction may be achieved with a soil of the acceptable range of wetness, shoving and shearing in the proof-rolling will weaken the soil and make it unstable.

Some soils, like the silts, that are highly sensitive to moisture variations, may be sufficiently stable for traffic loads when properly compacted. With the higher stresses caused by proof-rolling, these soils may also shove and lose strength. If this occurs, removal and recompaction to meet its original bearing capacity is necessary.

Liquefaction Susceptibility

Not all soils are susceptible to liquefaction. For many years, liquefaction was believed to be limited to sands. More recently, the bounds on gradation criteria for liquefaction susceptibility have changed to include silts. Furthermore, while the pumping that occurs during soil compaction or under construction wheel loads may not fully fit the traditional concept of liquefaction, both types of pumping contribute to a loss of shear strength with the development of pore pressures produced in soils of similar soil character.

Liquefaction occurs only in saturated soils; therefore, the depth to ground water influences liquefaction susceptibility. The susceptibility decreases with increasing groundwater depth. The effects of liquefaction are most commonly observed at sites where groundwater is within a few meters of the ground surface. At sites where groundwater levels fluctuate significantly, liquefaction hazards may also vary.

Since liquefaction requires the development of excess pore pressure, liquefaction susceptibility is influenced by the compositional characteristics that influence volume change behavior. Compositional characteristics associated with high volume change potential tend to be associated also with high liquefaction susceptibility. These characteristics include particle size, shape, and gradation. Liquefaction of non-plastic silts has been observed (Terzaghi et al., 1996) to determine whether plasticity characteristics rather than grain size alone influence the liquefaction susceptibility of fine-grained soils. Coarse silts with bulky particle shape, which are nonplastic and cohesionless, are fully susceptible to liquefaction (Ishihara, 1985, 1993), whereas finer silts with flaky or plate-like particles generally exhibit sufficient cohesion to inhibit liquefaction.

The criteria of the Chinese of fine-grained soils that may be considered susceptible to significant strength loss and liquefaction have been given as the four following measurements (Wang, 1979):

1. A fraction finer than 0.005 mm is less than or equal to 15%
2. Liquid limit is less than or equal to 35%
3. Natural water content is greater than or equal to 0.9 times the liquid limit
4. Liquidity index is less than or equal to 0.75

To account for differences between the Chinese and U.S. practice, the U.S. Army Corps of Engineers modified the measured index properties by decreasing the fines content by 5%, increasing the liquid limit by 1%, and increasing the natural water content by 2% before applying the Chinese criteria to a clayey silt (Finn et al. 1994).

Liquefaction susceptibility is influenced by gradation. Well-graded soils are generally less susceptible to liquefaction than poorly graded soils. Field evidence indicates that most liquefaction failures have involved uniformly graded soils. Particle shape can also influence liquefaction susceptibility. For example, soils with rounded particle shapes densify more easily than soils with angular grains. Consequently, they are usually more susceptible to liquefaction than angular-grained soils. Particle rounding frequently occurs in fluvial and alluvial environments where loosely deposited saturated soils are frequently found. Liquefaction susceptibility is often high in those areas.

Other studies published also suggest 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 increases with increasing PI. In the study conducted by Guo and Prakash (1999), one of two situations occurred when a small percentage of highly plastic material is added to a nonplastic silt under cyclic loads: 1) the pore pressure increases due to a reduction of the hydraulic conductivity, and/or 2) the plasticity and cohesive character of the silt-clay mixture may increase the resistance to liquefaction.

Sandoval (1989) and Prakash and Sandoval (1992) conducted liquefaction tests on a silt (96% passing No. 200 sieve and PI = 1.7 percent) with various amounts of commercial kaolinite added to increase the PIs to 2.6 and 3.4. In these low plasticity ranges, the test results implied that the increase in PI lowered the cyclic stress ratio required to initiate liquefaction. In addition, tests conducted by Puri (1984, 1990) on reconstituted silt and silt-clay mixtures with PI of 10 to 20 percent showed an increase in liquefaction resistance. Gao and Prakash (1999) provided the hypothesis that

- a reduction in the liquefaction resistance is produced in the low plasticity range due to the lower hydraulic conductivity and the higher pore pressures, and
- the cohesive character of the silt-clay mixture in the higher plasticity ranges produces an increase in the resistance to liquefaction.

Day (1999) proposed that the main criterion governing the shear strength of silty sands is whether the fines are plastic. The shear strength of a silty-sand with non-plastic fines is governed by the frictional and interlocking resistance between particles. However, if the fines are plastic, the shear strength is governed by its plastic character (LL and PI). An extended classification of the soils as either non-plastic (cohesionless) or plastic (cohesive) was proposed. A modification of the Unified Classification System (or the Inorganic Soil Classification System Based on Plasticity) was recommended to include classification for high, intermediate, or low plasticity. A new classification of MN for non-plastic silts was added as a coarse grain soil. Silts and clays were broken down as ML, MI, and MH, and CL, CI, and CH, respectively. Low plasticity (ML and CL) corresponds to a PI ≤ 10, intermediate plasticity (MI and CI) corresponds to $10 < PI \leq 30$, and high plasticity (MH and CH) corresponds to PI > 30.

Method of Investigation

The method of investigation was broken down into two phases. Phase 1 involved a fact finding effort to better describe the problem and focus the research objectives. It attempted to identify and describe the problems and field experiences of the DOTD districts, and to develop a testing program for characterizing the silts with respect to their tendency to pump during compaction or when subjected to construction traffic after compaction. Phase 1 was divided into 3 tasks.

Task 1. Information from DOTD engineers was solicited to establish the design approach and construction methods for pavements over soft, wet silts (locations where the silt limitations exceed those of the DOTD standard specifications) employed by different DOTD district offices. This included interviews, site visits, and phone communication with the laboratory and construction engineers of six DOTD districts in which high-silt soils are commonly encountered. The districts were Lafayette (03), Lake Charles (07), Alexandria (08), Chase (58), Baton Rouge (61) and Hammond (62). Field visits to observe the subgrade compaction at several construction sites were also conducted. Preliminary soil samples of the problem silts observed were acquired and classification tests were conducted when possible.

A survey was also developed with the Louisiana Transportation Research Center to address the experiences of DOTD engineers and contractors in the construction of embankments using the high-silt soils. This survey was sent to the participating DOTD district offices and to the American General Contractors Association (AGC) for solicitation of their members involved in these projects.

Task 2. Using the information gathered in task 1, a laboratory program was developed to characterize the soils, further refine the description of the pumping problem, and, if possible, identify the problem soils in terms of their the pumping potential and evaluate the potential for stabilization efforts.

Task 3. The results from the review of the engineering practices in task 1 and the sampling/testing program proposed in task 2 were submitted to the LTRC Project Manager and Project Review Committee (PRC) as an Interim Report for approval and/or modification of the work plan.

Phase 2 involved the selection and acquisition of samples and a testing program to investigate the nature of the problem and character of the silt materials. The major emphasis was placed on efforts to refine a description of the pumping problem and to develop guidelines for identifying the problem soils. Of secondary importance was the identification of alternate methods for stabilization. Both elements of phase 2 included tests for investigating the potential of stabilization with respect to construction needs and, if possible, post-construction performance. A limited number of specific additives were proposed with consideration for their ability to dry the subgrade silts sufficiently for compaction and for their ability to provide a working table for the construction of the base and pavement. These additives included hydrated lime, portland cement, and fly ash. Tests for evaluating a longer performance period for the stabilized subgrade when subjected to accelerated weathering/saturation conditions were also proposed.

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, Lake Charles District 07, the Alexandria District 08, and Chase District 58. They involved eight samples of soils from different project sites. The soils were representative of those commonly encountered with a high-silt content. Most were acquired from current projects in which pumping problems were occurring. At least two of the eight samples provided were noted as not pumping and were used as a comparison with the pumping silts. All of the samples were of a high silt content.

Testing Program

The testing program focused on characterization of soils and identifying attributes contributing to the pumping nature of the silty soils. Tests selected for identification of those characteristics indicate the soil's relative susceptibility to pumping. A second objective was to consider methods for modifying and stabilizing the natural soils 1) to prevent the pumping during compaction and 2) to improve the long-term performance of the subgrade.

Classifications Tests

Gradation tests (DOTD TR 407) 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 particles was determined according to ASTM D854 (AASHTO T100).

Compaction Tests

Laboratory compaction curves, optimum moisture, and maximum dry density of the soils were established using the standard Proctor compaction method (DOTD TR 418 Method (ASTM D698 and AASHTO T99, 12,375 ft-lbf/ft^3 or 590 kN-m/m^3). A family of curves and a comparison of the individual soil compaction character with other impact compaction efforts were also conducted. 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
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 natural soil at various moisture contents and the modified soil with various chemical agents were prepared for strength tests. These specimens were molded by means of a spring-loaded plunger and the Harvard Miniature Compaction Apparatus (described in ASTM D4609). The apparatus consists of a mold 1.3125 inches in diameter and 2.816 inches long with a volume of 1/454 cubic feet. The weight of the entire soil specimen produced is equal to the unit weight of the soil in pounds per feet cubed. The specimens were molded at different compaction efforts by varying the number of tamps per layer with the spring tamping plunger.

Harvard compaction employs a kneading action in molding the specimens.

Undrained Strength Tests

Unconfined compressive strength tests (ASTM D2166 or AASHTO 208) were conducted on the compacted, natural soil at various moisture contents. A soil's strength generally decreases at moisture levels that exceed the optimum value for compaction. Therefore, extent of the strength loss and the corresponding strain that occur in the different samples can provide insight into the relative affinity of the different samples to pump or not pump. This is considered a static load test.

In this series of tests, the Harvard compaction curves for the natural soils were produced using different levels of compaction efforts. The unconfined compression strength corresponding to different moistures and densities was established for the different soil samples. The tests permitted an analysis for strength-moisture relationships. This approach was used to consider the feasibility for predicting the silt subgrade's strength at moisture levels beyond those used for compaction. The analysis attempts to mimic seasonal variations or the impact of moisture changes for long-term performance.

Moreover, the unconfined strengths of the chemically stabilized specimen were measured. Strength tests for the chemically stabilized specimens were conducted on the as compacted specimen and the cured specimens. Lime stabilized soil cures at a slower rate under ambient conditions than those stabilized with Portland cement or Class C fly ash. However, lime stabilization can be accelerated under higher curing temperatures. Thus, in an attempt to compare the full potential for both stabilizing agents, two different curing techniques were used. Ambient curing for a period of two weeks in a humidity room and an accelerated curing with a one-day or three-day curing period at 50^0 C temperature were arbitrarily selected. The accelerated curing periods were used with lime and lime-fly ash specimens. The two-week curing period was used with Portland cement and Class C fly ash specimens.

The long-term stability or durability of the stabilization efforts was evaluated with the vacuum-saturation test ASTM C 593. After curing, test specimens were de-aired in vacuum for 30 minutes followed by total inundation in a saturation chamber. They were allowed to soak in the water-filled chamber for one hour. The specimen was then tested using the unconfined compression test.

Cyclic Triaxial Compression Tests.

Cyclic triaxial tests, similar to ASTM D3999, were also conducted on selected specimens. The specimens were compacted wet with the optimum moisture content. Subsequently, a confining pressure corresponding to field conditions was applied. Test specimens were subjected to a shearing stress equivalent to the anticipated wheel loads as a cyclic deviator stress. The specimens then underwent cyclic compression and extension loading. In this test, specimens are normally loaded until liquefaction/failure strains occur or the pore pressure equals the confining stress. The number of load cycles required to produce this state were noted. For this study, the test was conducted as a drained test to simulate field conditions with a water table nearby. A source of water was provided at the base of the specimen.

Mineralogy

A mineralogy study of the natural soils was conducted to determine the frequency of clay minerals, quartz and feldspar minerals, and metal oxides that were present. Also, the soil mixtures with the chemical additives were investigated to identify any cementitious or other products that may have formed.

The mineralogy investigation was conducted by the University of New Orleans Geology laboratory. Two methods were used in analyzing 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 in collecting the energy dispersive spectra. A software page called "Iridium," (IXRF Systems, Inc.) was also 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 and potential reaction products.

DISCUSSION OF RESULTS

Survey of Practice

Six DOTD districts were included in the initial DOTD survey: Lafayette District 03, Lake Charles District 07, Alexandria District 08, Chase District 58, Baton Rouge District 61, and Hammond District 62. All six of the districts were contacted, and copies of the proposed research study were provided to the district construction engineers and district laboratory engineers. Visits were made to four of the districts, Alexandria, Chase, Lafayette and Lake Charles while phone discussions were conducted with the district laboratory engineers for Baton Rouge and Hammond.

Through conversations with individuals and groups, it became apparent that the specific information and organization of individual cases was either not available or not formally documented. However, the problems associated with high silt soils appear to be similar for much of the state. Individual experiences and feedback provided by DOTD personnel from the various district offices follows.

Lafayette District 03.

Individual discussions were held with the district laboratory engineer, construction engineer, maintenance engineer, design engineer, project engineer and a contractor. The problem identified involved the liquefaction of the silt soils with the presence of water and construction traffic. This is a major problem because of the wide distribution of the high silt soils and the flat, poorly-drained nature of the prairie in which the district is located. The 60/65 percent silt definition of usable soils in the DOTD specifications is particularly problematic to the Lafayette district. Soils with high silt contents are distributed widely over this district, and select materials are not readily available. While the district also has areas of collapsible silts, they were not considered as the focus of concern in this study.

It was noted that prior to the current DOTD specifications these same high-silt soils were routinely used as backfill for pipes and as select materials within roadway embankments. It was stated that these silts could pump with a 95 percent compacted density and with a silt content as low as 40 to 50 percent. Even if satisfactory compaction is achieved, the support provided by the compacted subgrade is compromised with the accumulation of water and increase in moisture content.

A three to five foot fill constructed dry of optimum on highway 167 was identified as having had significant problems in the past when an asphalt shoulder (4 to $4^1/_2$ in.) was placed directly on the compacted surface. The asphalt shoulder broke up due to a lack of support. Borings within the embankment showed free-standing water even though the embankment was as much as five feet above the natural subgrade. Other problem projects identified were U.S. 90 (between Crowley and Esterwood), Highway 167 (Abbeville to Maurice), and the I-10 embankment repair jobs.

Without select materials, lime was cited as being a common material used for drying out and constructing a working table to support the base. The contractor stated that the in place soils could be used if compacted properly, (i.e., if they were not wet, and that lime or Portland cement could be used to achieve the end product.

A visit to the intersection of U.S. 90 crossing of the Southern Pacific Rail Road outside of Crowley was made to review a pumping subgrade problem. The solution employed was reffered as "cut and lime" which is an effort to dry and increase the support. The need to improve the subgrade was due to the fact that the embankment was less than 3 feet and the inability to compact an 8 inch stone (610) base over unstable silt subgrade (described as being spongy). A sample of the silt subgrade was taken for testing as depicted in Tables 1 and 2.

Lake Charles District 07

A group meeting was held with the district laboratory engineer, construction engineer, project engineer and the assistant laboratory engineer. Again, the compaction problems and occurrence of high silt soils were cited as not being unique but were common to much of the district. A USDA map of the state based on the suitability for road support was used to demonstrate the poor quality of the natural soils. The current delimiters used to identify usable soils were stated as being too limited (gradation, PI, etc.). It was suggested that Recent and Pleistocene silts/soils are poor and not as stable compared to older sediment that had been weathered. Also, chemical or mineralogical constituents present may be necessary to properly identify the engineering or stabilization solution. While cases could be cited where Portland cement worked well, there were other situations where high percentages of cement did not set up. Lime was also cited as requiring compatible soil (clay) constituents.

It was pointed out that 90+ percent of the district's work involved pavement reconstruction versus that of new construction. In the reconstruction activities, the replacement pavement requires that a working platform be constructed first in the existing embankment, which is typically wet and not stable, Figure 2.

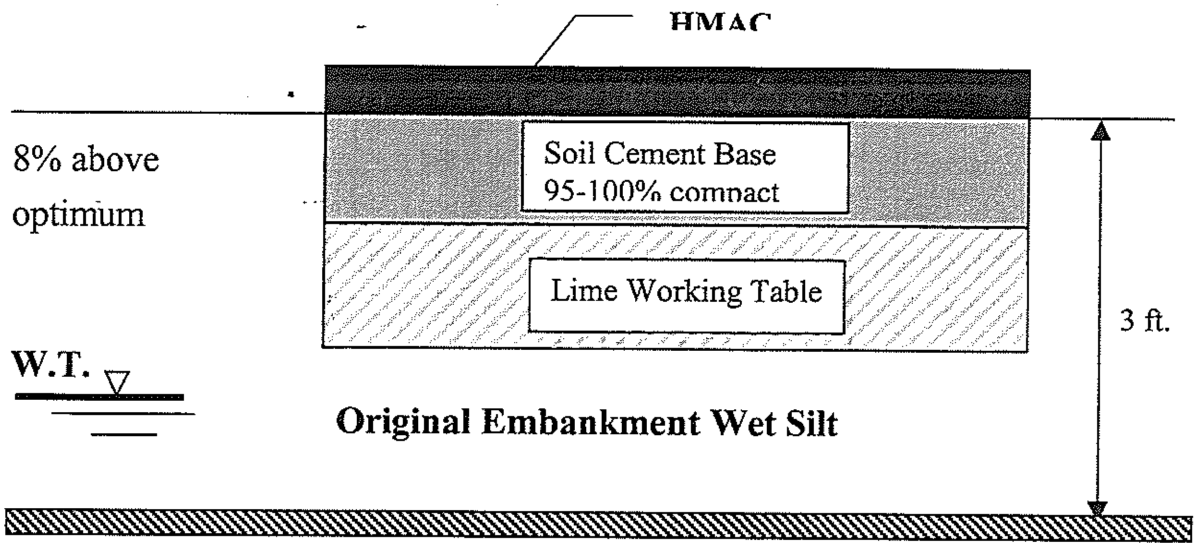


Figure 2
Pavement reconstruction in existing silt embankment

Investigations within the district established that almost all of the previously constructed embankments in the area had moisture contents that exceed 5 to 8 percent of optimum moisture. Generally silts cannot be compacted above optimum. DOTD specifications allow compaction moisture to be +4 percent of optimum moisture content, which is not adequate for silts. Former sites that were identified with unstable silts and that caused significant problems included Moss Bluff Road and Ronnie Dupont Road.

The Lafayette study stressed the need for research, including the chemistry and mineralogy of the soil, the geology of the site, and a more precise method for identifying collapsible soils.

Alexandria District 08

A group meeting was held with the district administrator, construction engineer, laboratory engineer, project engineers, and the LTRC project manager. The discussion focused on high-silt problems 1) encountered with the in-place subgrade and 2) used in an embankment and 3) the DOTD specifications for "usable soils." The district relies heavily on replacement materials and mechanical stabilization methods when pumping silts are encountered in the subgrade. Select materials are readily available in this district and are preferred by contractors. Furthermore, deep cuts with liming are generally not considered effective. In most cases, silts in high embankments were not a problem if the top three feet are select materials. If a problem occurs in the compaction of silts in an embankment, it is due to inadequate current construction techniques. Previously, compaction methods were more efficient. For example, an effective method was compacting with sheep's-foot roller until it "walks out." There was much difficult in achieving density with high-silt soils, however.

A field inspection of the Keyser Ave/La 1 project in Natchitoches provided an example of a pumping silt problem. The original design called for a 12-inch stone base beneath the pavement. The contractor requested and was granted a construction change for an 8-inch stone base plus 2 inches of asphalt. Ultimately, the silt subgrade was observed to be pumping through the stone base after traffic was allowed on the completed section. In order to let the traffic flow, construction was conducted one lane at a time with traffic running on the completed lane of stone base. When a filter fabric was used, it was noted that in some places, the filter fabric was pushed up through the stone base. The final construction design consisted of the removal of 2 feet of the subgrade and replacement with 2 feet of wrapped sand beneath 8 inches of stone base. A sample of the silt was secured for tests as exhibited in Tables 1 and 2.

Chase District 58

The district's laboratory engineer and project engineer for the Wisner Project provided information on the location of pumping silts within the district and construction experiences with these soils. According to the laboratory and project engineers the construction of embankment using high silt borrow pit material can be accomplished with smaller lifts even if they "crawled" slightly. They emphasized that in place saturated silts can be improved with time and effort. Moreover, DOTD focuses on high silts in terms of silt percentages present without considering the presence of other constituents. A suggestion was made that perhaps a minimum plasticity index (PI) should be required to provide a cohesive character to prevent pumping.

A sample of a pumping subgrade was taken along La 15 construction at Wisner at the intersection of La 562, Tables 1 and 2. An approach embankment for the Cane River Bridge at Franklin Parish was also reviewed. At this site, a marginally "usable" silt with a silt percentage of ± 60 percent was being used from a borrow pit. Tables 1 and 2 show the results of tests with these silts.

DOTD Headquarters – Baton Rouge

The research objectives of the project and the problems associated with constructing the pavement system in pumping silts were reviewed with the DOTD's geotechnical-pavement design engineer. In discussing the current construction cases involving pumping silts, he suggested that much was due to the use of vibratory equipment. For example, in the case of the Natchitoches Keyser Avenue project, the contractor's design changes and the allowance of traffic on the stone base without completing the entire system were responsible for much of the problems. It was emphasized that the pumping problem, in general, developed with the evolution of construction equipment (weight and vibratory equipment). The absence of the pumping problem in constructing the original highways of the 1960's can be attributed to older construction equipment used (smaller dozers, dump trucks, and rollers, and the lack of heavy-duty vibratory equipment) in compacting the silt soils. In essence, silts should not be compacted with vibratory equipment.

To account for unknown areas with silt contents above 60 percent (i.e., unusable soils), the standard pavement design allows for the inclusion of type-D lime treatment, 9 percent by volume for a 12-inch thickness, by the project engineer when required.

Character of Pumping Subgrades.

The four samples obtained from the construction sites during the DOTD district visits were secured in DOTD sample bags and transported to the UNO geotechnical laboratory. The natural moisture content was determined with a small portion of the samples. Others were submitted to the UNO geochemistry laboratory for a mineral identification, and classification tests were conducted with the remaining sample.

Table 2 shows the silt content of all samples taken at the project sites as above 75 percent with PI ranging from 3 to 10. All classify as an A-4, a nonplastic, or moderately plastic silty soil. The liquid limit-plasticity index of three of the samples (Keyser Ave, Wisner LA 15, and the Wisner LA15 bridge embankment) would plot almost on the "A" line of the Plasticity Chart.

Table 2.
District survey of pumping soils – Properties and Classifications

	%Silt	%Sand	%Clay	PI	LL	AASHTO	USCS
Keyser Ave, Natchitoches	80	9	11	7.4	28.4	A-4	CL-ML
LA 15-LA 562, Wisner	84	7	9	3.2	24.8	A-4	ML
LA 15 Embank, Wisner	77	5	18	10	33.9	A-4	CL/ML
U.S. 90 – SPRR, Crowley	76	3	21	4.3	29.8	A-4	ML

Because the limited amount of material available, compaction tests were not conducted. However, based on the Atterberg results, optimum moisture content (standard compaction test) was estimated using the Johnson and Sallberg chart (1962, p. 125). The chart has been found very accurate in other tests with similar soils. In recognizing the possibility for variations and inconsistencies, the field moisture content for all of the materials at the time of sampling was wetter than the estimated optimum moisture content, as shown in Table 1.

Table 3 summarizes the abundance of various mineral phases identified on two randomly selected areas for each of the four samples. Some minor differences are exhibited by the samples. The Chase (Wisner LA 15 bridge embankment) contains the highest proportion of biotite or chlorite (altered biotite) and the lowest quartz contribution. Other variations among the samples are minor. Dolomite, for example, was only found in the Natchitoches samples, and some accessory minerals such as ilmenite and zircon were more notable in the Chase-bridge EMB samples.

Practice in Other States

Information on the practice of other states was also solicited. Arkansas' State Transportation Department and Mississippi's DOT responded. The Arkansas DOT does not have specific guidelines to address high-silt or wet subgrades. They report that they do not have significant areas of high-silt soil deposits.

Table 3.
Pumping Soils - Mineral Phases

	Natchitoches-Keyser	Chase Bridge EMB – LA 15	Chase-Wisner	Crowley – US 90 SPRR
Quartz	119	103	158	149
K-feldspar	39	20	38	11
Na-plagioclase	16	21	8	10
muscovite	0	3	3	0
biotite/chlorite	4	35	4	23
other clay minerals	8	9	3	7
ilmenite	0	6	0	0
hematite	0	6	0	0
zircon	0	2	0	0
dolomite	6	0	0	0
titanium oxide	0	3	0	0
Total # of grains	192	208	214	200
%				
Quartz	62	50	74	75
K-feldspar	20	10	18	6
Na-plagioclase	8	10	4	5
muscovite	0	1	1	0
biotite/chlorite	2	17	2	12
other clay minerals	4	4	1	4
ilmenite	0	3	0	0
hematite	0	3	0	0
zircon	0	1	0	0
dolomite	3	0	0	0
titanium oxide	0	1	0	0

However, corrective measurements for those isolated areas with wet subgrades that cause performance and construction difficulties include chemical stabilization or excavation and replacement. Individual problems areas are corrected as needed.

The Mississippi DOTD reported that they do not have any special provisions for dealing with the loess or high-silt soils common to that state. Cement or lime-fly ash stabilization have been used for dealing with this type of material. Also, MDOT has adopted a policy of chemically treating all subgrade soils for their pavement work.

Embankment Construction with Soils Containing High Silt Contents

Construction Experience Survey:

Currently, the DOTD standard specifications require that usable soils not exceed 65 percent silt content to be used for embankments. Soils in Louisiana commonly exhibit high silt contents. Finding materials meeting the specifications on many projects can be difficult and very expensive. Many engineers and contractors believe that soils with high silt content can be used with the proper constructing methods. However, these soils are known for their compaction problems, especially concerning their long-term performance and continued pumping potential.

To further address the objectives of reviewing the existing DOTD specifications for usable soils, a survey on past and present experiences with construction practice with high silts was conducted (Appendix A). Both DOTD engineers and contractors were solicited for their participation. The survey was sent to all district offices and to contractors through the Association of General Contractors (AGC). Twenty-five responses to the survey were received. They encompassed all of DOTD district offices and three contractors with offices located in Alexandria, Baton Rouge, and Minden, Louisiana.

The survey consisted of five major sections or questions that addressed the individual's opinion of the DOTD's specifications and experiences on projects involving silty soils. An abbreviated synopsis of the question and their multiple parts that could be quantified are 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% Si? - 72% Had Experience
3.2) Problems Encountered soils >65% Si ? - 56% Had Experienced Problems
3.5) Are you aware of any long-term problems for soils >65% Si? – 52% Say No
4) Construction Problems with soils <65% Si ? - 60% Had Experienced Problems
4.5) Are you aware of any long-term problems with soils <65% Si - 36% Say No
5) Should soil with silt > 65% Si 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 survey's respondents were, in general, divided on the question of the adequacy of the DOTD's definition of usable soils although a small majority (56 percent) found them to be acceptable. The silt-content was considered a good indicator for identifying a usable soil by most (76 percent) as shown in question 2. However, a number decided against using the percent silt alone as an indicator or wrote that the current silt limits were not properly established and that other factors such as plasticity needed to be considered.

Question 3 established that construction projects involving soils with silt contents greater than 65 percent is a common experience in Louisiana (72 percent of those surveyed). Of those responding affirmatively, fifty-six percent (56 percent) had also experienced problems using the high-silt soils. The problems cited involved 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. The compaction equipment cited on these jobs included a variety of methods such as BOMAG stabilizers, rubber tire rollers, sheep's-foot rollers, padded foot roller, waffle wheel, vibratory rollers, water trucks, dump trucks, dozers, motor graders, 9 wheel roller, tractor, etc. The quality-control used for these jobs were primarily moisture content and density with the emphasis on density. Most interviewees were unaware of any long-term problems associated with the embankment/pavement. One interviewee noted that previous construction experiences have often dictated design changes to minimize the effects. Pavement structures have increased in thickness. It is not uncommon to have a total pavement structure thickness approaching 2 feet. Also, lime treatment, especially when subsequently stabilized with cement, has provided benefits.

In question 4, engineers and contractors were asked whether they had experienced any moisture, pumping problems, and/or density problems with construction projects that used soils with silt contents less than 65 percent. Sixty percent (60 percent) indicated that they had experienced these problems with lower silt soils (as low as 53 percent silt cited). Again, most cited (with the exception of one) that they were not aware of any long-term problems associated with the embankment/pavement structure.

When asked whether soils with greater than 65 percent silt should be allowed (question number 5), a slight majority (56 percent) answered no. When asked what modifications to the specifications should be made to assure proper construction, there were a number of suggestions. Extensive detail was provided on the need to dictate construction methods. Fifty percent indicated that there should not be specifications on the type of construction equipment allowed for an embankment constructed of high silt content soil. However, most of those that responded to the question also stated that vibrating compacting equipment should not be used with the high silt soils. Others emphasized the need for moisture control, soil conditioners or pre-treating with lime, filter fabrics, increasing the allowable silt percentage with a PI greater than 20 ($20 \leq PI < 35$), usable soils based on pavement usage (ADT numbers), other soil properties, etc. The DOTD specification allowance for four percent moisture content above optimum moisture without limitations of soil types was cited as an example of where specifications applied broadly may create problems rather than resolving them.

Sixty-eight percent expressed that moisture infiltration into a properly constructed embankment of a soil with high-silt content causes long-term performance problems. Suggestions for additional specifications or design considerations that assist in long-term performance included drainage systems and chemical stabilization. A set of bona fide acceptance criteria that included a higher level of performance (modified proctor requirements) was also cited.

In the additional comments section, the responding engineers emphasized the problem and costs associated with eliminating the use of high silt soils in some parts of the state. It was also noted that in current practices using "95 percent standard proctor, the corner stone of embankment and base construction will not give us the adequate performance," and that the "cheapest materials constructed with the least amount of attention (embankment) have created a very expensive problem." Also, " if a contractor is dealing with a 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 (DOTD) specifications tend to give him support for poor moisture control while constructing embankment lifts even though that is not the intent. The DOTD needs to develop QC specifications with a specified number for acceptance that will force the contractor to appropriately select, process and compact the soil with compatible equipment."

Laboratory Testing Program

Samples

Samples from current projects were provided from four DOTD districts. Approximately six sampling bags 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) also by virtue of color shades. The four sets of samples from the four districts included soils that had been observed as 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 De Ridder site. There were a total of eight different samples involved in the extended testing program of phase 2.

Characterization Tests

The results of the classification tests are summarized in Table 3. The silt content ranges from 56 percent to 78 percent. Six of the samples, representatives from all four 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. Figures 3 and 4 plot the soils on the plasticity chart and on the ASTM D 3282 chart for liquid limit and index ranges for silt-clay materials.

The eight samples are clustered in two groups on the plasticity charts. The Chase Brown and DeRidder Brown samples exhibit PI's greater than 10 with liquid limits of 37 and 38, respectively. Both of the samples classify as AASHTO A-6 soils and as CL soils in the USCS system. The other samples classify as A-4 soils (ML) and range from non-plastic to low plasticities for PI's less than 5. All of the liquid limits of the low PI soils are approximately 25 percent, with the exception of DeRidder White (20 percent liquid limit).

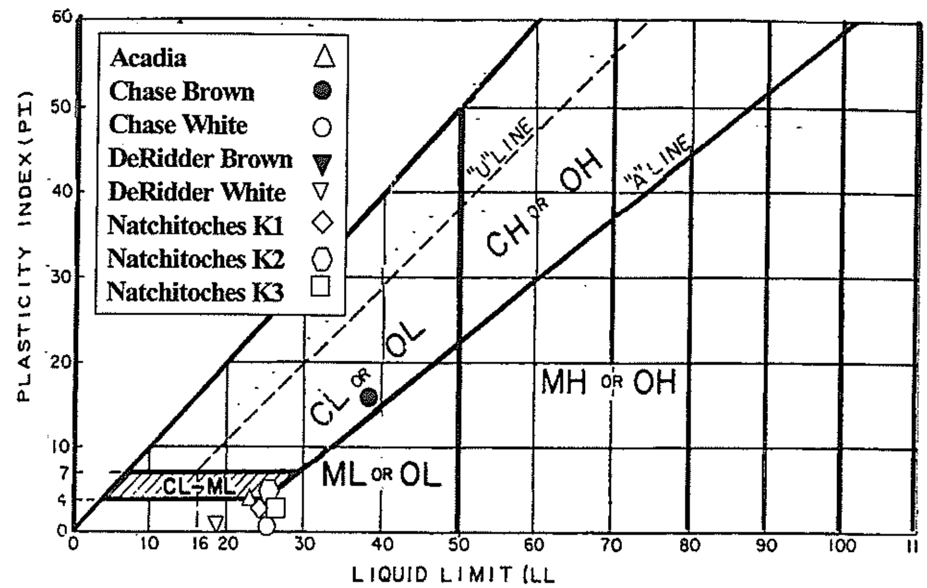


Figure 3
Plasticity Chart w/ Unified Classification of Soil Samples

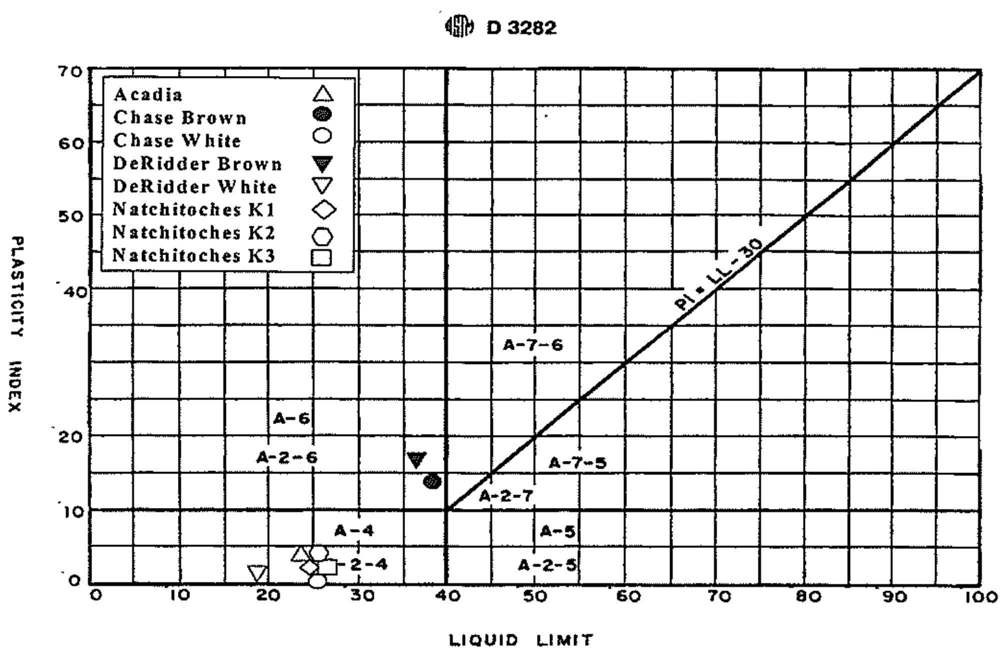


Figure 4
AASHTO Classifications of Soil Samples

The DeRidder Brown classifies as a silty clay loam according to the textural classification of DOTD TR 423-89. All other samples fall in the silty loam area of the textural diagram, as shown in Figure 5.

DOTD TR 423-89

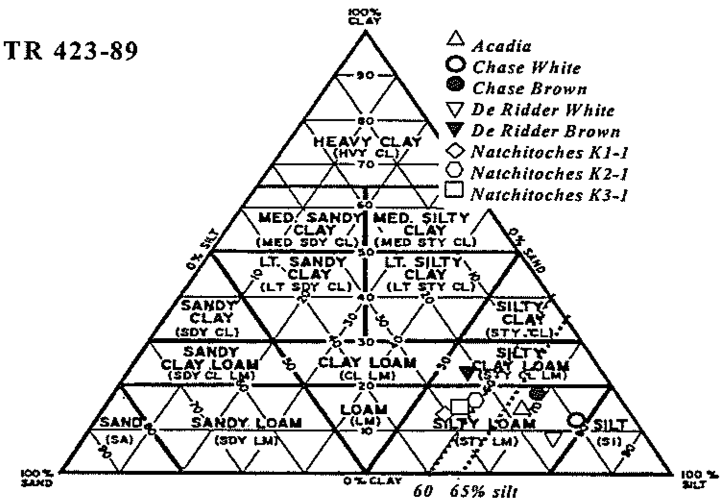


Figure 5
Textural Classification, DOTD TR 423

The Acadia, Chase Brown and White, and the DeRidder white samples have silt contents that are greater than the allowable amount noted in the DOTD specifications for usable soils (i.e., greater than 60 percent). The Natchitoches samples (K1-1, K2-1, and K3-1) have silt contents that are approximately 60 percent and are, therefore, marginal with respect to being considered a usable soil. The silt content of the Natchitoches samples range from 60 percent to 78 percent silt content. The DeRidder Brown sample meets the usable soil criteria with a silt percentage of 56 percent (<60 percent) and PI of 17 (<20). None of the soils showed the collapsible silt characteristic described by Arman and Thornton (1972) in the hydrometer tests (i.e., black liquid in the hydrometer test).

Mineralogy of Natural Soil

For the microanalysis of the soils, dried and disaggregated specimens were carbon coated and investigated in an AMRAY 1820 scanning electron microscope. X-ray diffraction scans were also run on a Scintag XDS 2000 X-ray diffractometer. The mineral distribution for each of the sample sets is summarized in Table 4.

Table 4.

Mineral distribution of the silt samples

Mineral Distribution	ACADIA	K1-1	K2-1	K3-1	Chase Brown	Chase White	DeRidder Brown	DeRidder White
Quartz	68	67	72	75	59	60	73	72
K – feldspar	6	5	8	5	16	11	11	11
Na - plagioclase	9	7	4	5	11	14	5	10
muscovite/illite	5	4	5	4		2	1	<1
biotite/chlorite	6	7	4	5	4	1	6	8
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
Ilmenite	0	<1	0	<1				
hematite or Fe oxide	<1	1	1	1	2		1	0
Zircon	1	<1	<1	0		1	<1	<1
titanium dioxide	<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 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 in their percentages. The quantities of clays present in these samples ranged from approximately 13 to 17 percent. All are noted as including muscovite/illite, biotite/chlorite, and montmorillonite clays in approximately equal quantities (4 to 6 percent). Small quantities of kaolinite and other clay minerals were also noted in some samples.

The DeRidder and Chase samples consisted of less quartz, but with more of the K-feldspar in both and with more Na-plagioclase in the Chase samples. The clay minerals reported for the Chase samples were noted as small percentages (3 and 4 percent) of muscovite/illite and biotite/chlorite, but with a larger percentage (7 and 9 percent) of other clay minerals. The DeRidder sample had the largest clay mineral representation, including a significant portion consisting of the more active montmorillonite clay. The DeRidder soils also had the largest representation of oxides. Clumping was noted as a problem with the DeRidder soils by the UNO geochemistry laboratory.

The more stable clay minerals have lower specific surfaces and are less active. The activity index for all samples is noted in Table 5. A correlation exists between the type of clay mineral present and the activity of the soil as follows (after Holtz and Kovacs, 1981):

Mineral	Activity
Na-Montmorillonite	4 – 7
Ca-Montmorillonite	1.5
Illite	0.5 – 1.3
Kaolinite	0.3 – 0.5
Mica (muscovite)	0.2
Calcite	0.2
Quartz	0

A classification system for relative activity is as follows:

ACTIVITY	CLASSIFICATION
Less than 0.75	inactive clays
0.75 to 1.25	normal clays
Greater than 1.25	active clays

Table 5.
Soil Properties and Classifications

Parameter	Acadia	Chase Brown 408+00	Chase White 295+00	DeRidder Brown	DeRidder White	Natchitoches K1	Natchitoches K2	Natchitoches K3
Sand %	18	11	10	21	19	29	24	24
Silt %	68	68	78	56	74	59	61	60
Clay %	14	19	12	23	7	12	15	16
LL %	23	38	25	37	19	24	25	25
PI %	3	14	NP	17	1	2	3	2
Activity A	0.26	0.71	0.03	0.75	0.19	0.125	0.18	0.125
Toughness Index, TI	0.29	0.99	0.03	1	0.79	0.39	0.4	0.31
Group Index, GI	1	15	0	15	18	0	0	0
AASHTO Classification	A-4 Silty Soil	A-6 Clayey Soil	A-4 Silty Soil	A-6 Clayey Soil	A-4 Silty Soil	A-4 Silty Soil	A-4 Silty Soil	A-4 Silty Soil
Unified Classification	ML Silt	CL Lean Clay	ML Silt	CL Lean Clay	ML Silt	ML Silt	ML Silt	ML Silt

The soil samples with the largest percentages of clays were the Chase White (12 percent) and Brown (19 percent) samples and the DeRidder Brown sample (23 percent). The Chase Brown and DeRidder Brown samples exhibited the greatest plasticity. The largest silt and fine sand concentrations with low to non-plastic characteristics were those of the Acadia and Natchitoches soils. Thus, the mineral concentrations, types, and groups of clays present in the samples is consistent with the expected plastic or non-plastic character.

Compaction

The factors that are most influential in the compaction of a soil are 1) the soil type, 2) the moisture content, and 3) the compaction effort. For a specific soil and compaction effort, a relationship generally exists between the unit weight achieved and the moisture content of the soil during compaction. The moisture to unit weight relationship is represented as a compaction curve. The standard Proctor (AASHTO T99) and the modified (AASHTO T180) compaction curves for the eight samples are shown in Figures 6 and 7, respectively.

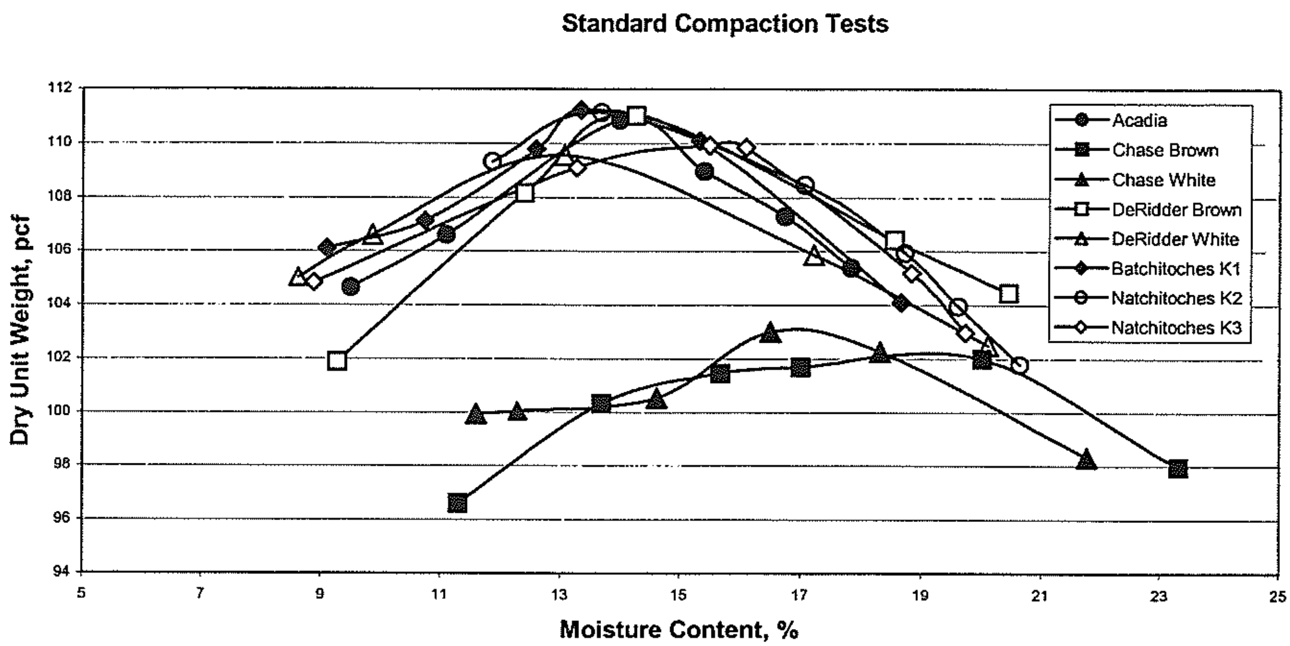


Figure 6
Standard Compaction Curves

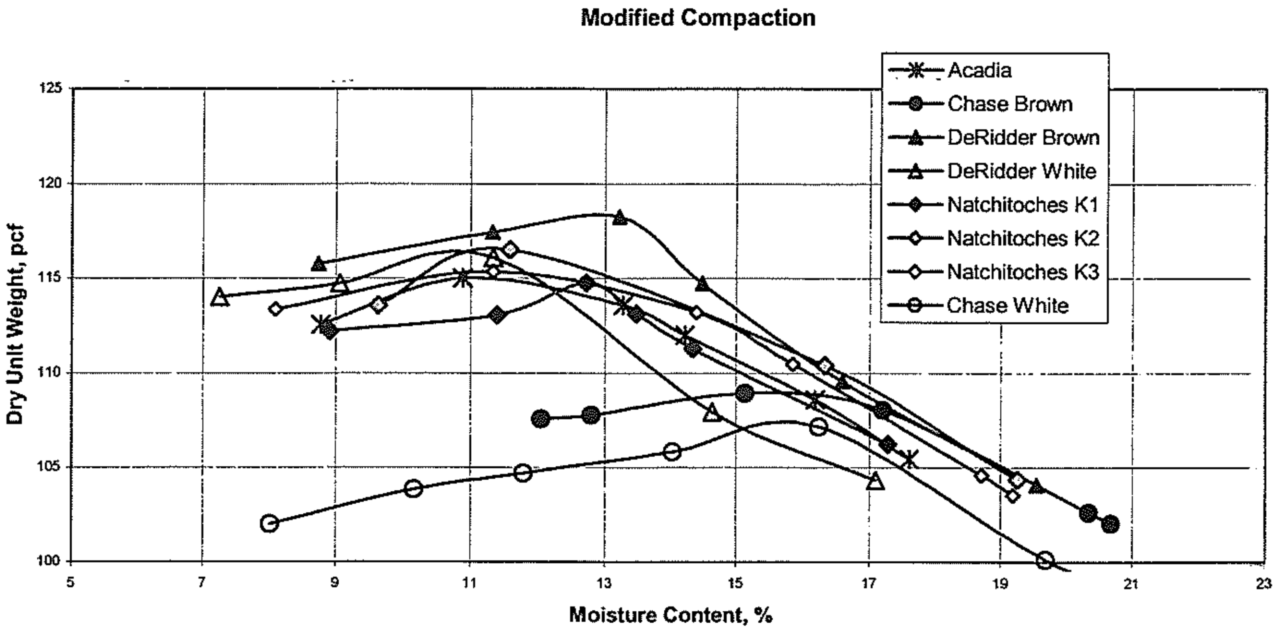


Figure 7
Modified Compaction Curves

The compaction curves are plotted as two separate groups with the dry densities and optimum moisture contents differing by 10 pcf and 4 percent, respectively. The lowest unit weights (102 and 103 pcf) and the highest optimum moisture contents (17 and 19 percent) were produced by the Chase samples with the standard Proctor compaction effort (AASHTO T99). Acadia, DeRidder Brown, and Natchitoches K1 and K2 produced the highest dry unit weights (approximately 111 pcf) at an optimum moisture content of 13 to 14 percent. The DeRidder White and Natchitoches K3 samples had maximum dry unit weights of 110 pcf and optimum moisture contents of 13 and 16 percent, respectively

Plastic soils usually have a greater response (unit weight and optimum moisture) to an increase in the compaction effort. A comparison of the modified compaction curves with the standard proctor curves, shown in Figure 8, seemed to produce a greater response by the DeRidder Brown and Chase Brown samples with an increased compaction effort.

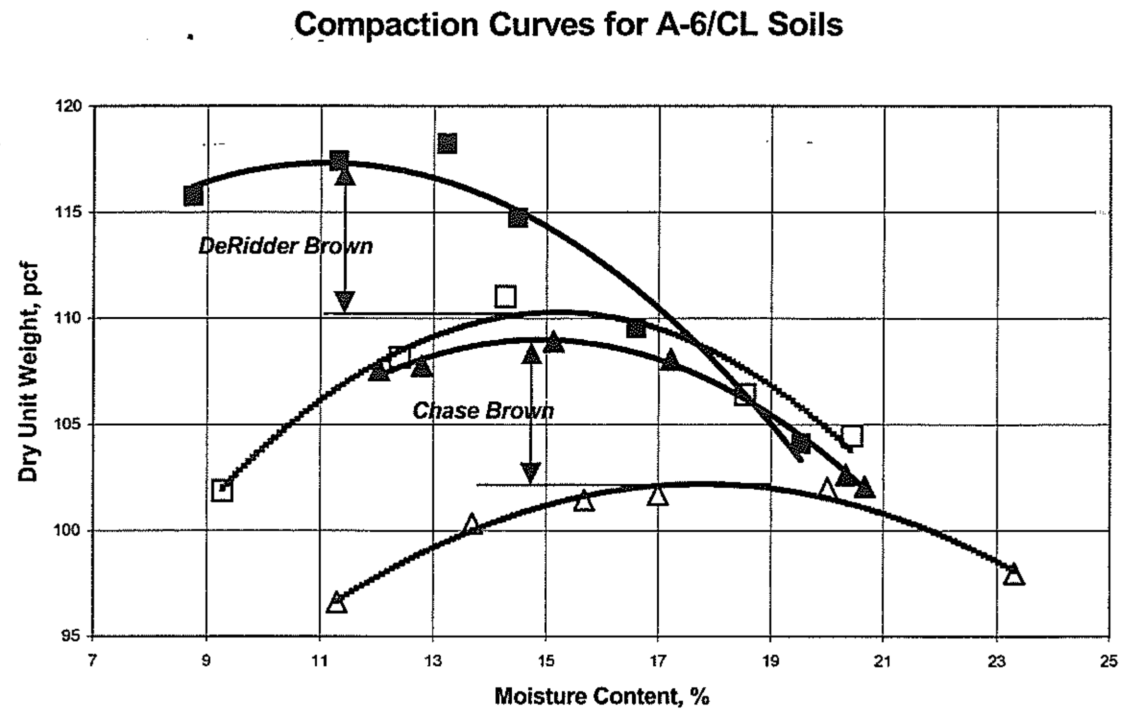


Figure 8
Influence of the compaction effort for soils with higher plasticity

Figures 6 and 7 exhibit the relative movement of the compaction curves with respect to other soil types. The DeRidder/Chase-Brown samples were the most plastic soils. They were classified as A-6 (CL) soils.

Comparison of Compaction Effort

In addition to the more standard compaction tests (AASHTO T99 and T180), the samples were compacted using the impact hammer at a level higher than the modified compaction and lower than the standard Proctor compaction method. The reduced compaction effort was conducted similarly to the standard test (i.e., AASHTO T99) but with a reduced number of blows per layer (15 blows/layer). The modified plus compaction consisted of the same techniques and hammer as in the modified test (AASHTO T180) but with an additional blow per layer of soil (35 blows/layer). Typical curves for the Chase soils are show in Figure 9.

The compaction curves for other samples are provided in the appendix B. The effect that changes in compaction effort have on a soil is demonstrated in a plot of maximum unit weight versus compaction effort given in Figure 10.

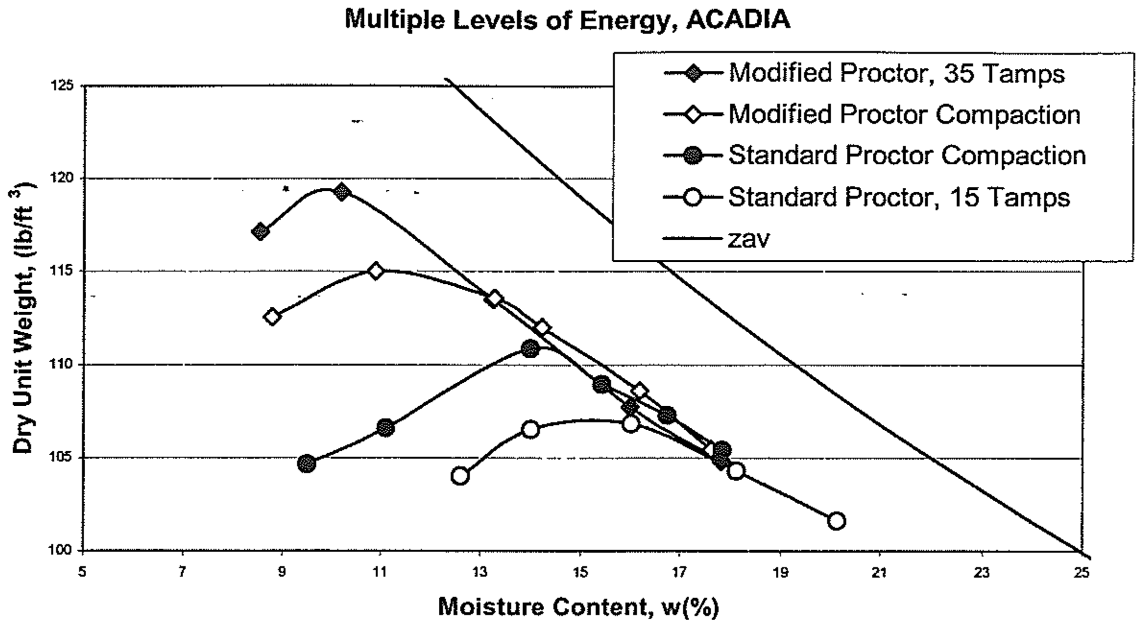


Figure 9

Acadia's family of curves for different compaction efforts

These differences can be of practical significance to the engineer that prepares specification requirements for compaction. They may also assist the interpretation of the unit densities resulting from tests when specifications are stated in terms of percent relative compaction. For example, the compaction effort necessary to achieve 90 percent or 95 percent of maximum unit weight can vary greatly with the soil type as measured by the slope of the curve.

The shapes of the different compaction curves of some individual samples were irregular. Their convergence on the wet side was especially erratic because of the crossing of the lower and higher energy levels of compaction. This irregularity seems to be more pronounced in the A-4 soils (Acadia, Chase White, DeRidder White, and the Natchitoches samples). This erratic behavior may signify a greater pumping potential for these soils and a greater sensitivity to moisture increases above optimum.

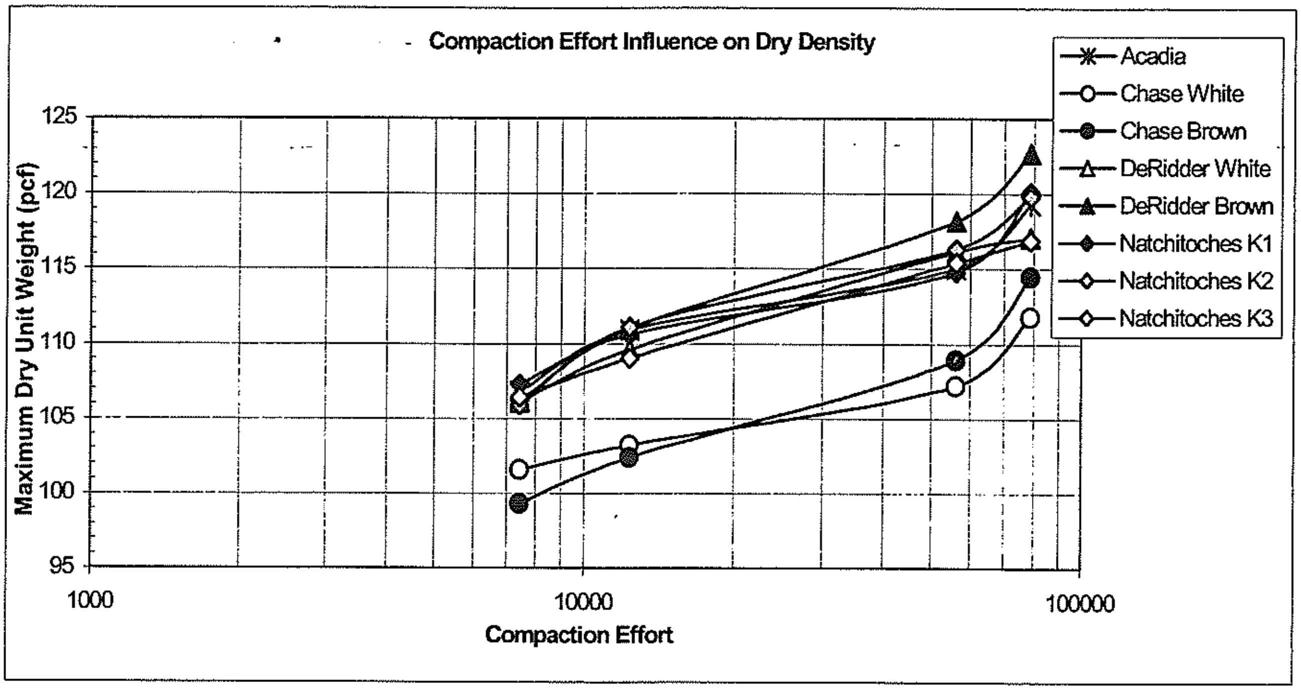


Figure 10
Compaction effort influence on maximum dry density achieved.

On the wet side of the optimum moisture, a less densely compacted soil may be produced although a higher compaction effort may be necessary. In the field, this would be observed as pumping of the soil during compaction.

Figure 11 shows that similarities resulted in the optimum moisture content predicted by the Atterberg limits with the Johnson and Sallberg Chart (1962) and the standard compaction tests (AASHTO T99).

The degree of saturation for the maximum dry unit weights and optimum moisture contents for the different compaction energy levels are presented in Table 6.

The saturation figures within a sample group vary to some extent for the different compaction efforts. Some of this variation may be attributed to testing errors, but the differences are not great. However, the magnitude of the saturation produced indicates a lower saturation level of maximum dry density for the A-4 (ML) soils as opposed to the A-6 (CL) soils. The saturation at maximum dry density for the A-4 soils averaged less than 75 percent.

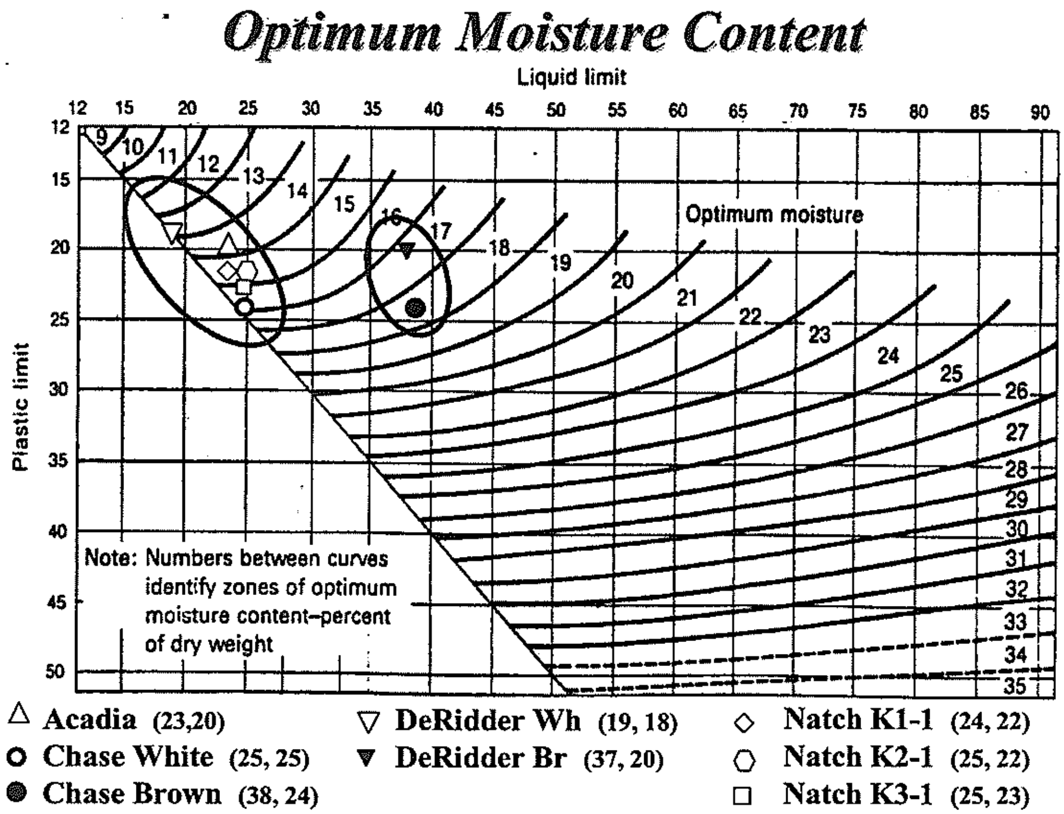


Figure 11

Optimum moisture content estimates by Johnson and Sallberg (1962)

The saturation of the two A-6 soils, Chase Brown and DeRidder Brown, at maximum dry density, averaged 81 and 78 percent, respectively. The maximum density for the more cohesive soils usually occurs at ± 85 percent saturation. For these high silt (low to non-plastic) soils, it is likely that the maximum density achieved will occur at lower levels of saturation and that further compaction effort will result in increased pore pressures that produce the observed pumping. There are, of course, significant differences in the compaction methods and efforts employed in the field compared with those in the laboratory.

TABLE 6.

Compaction effort and character

ACADIA	OMC ω_{dmax}	Dry Unit Wt. γ (lb/ft^3)	Saturation S
Modified Proctor, 35 tamps	10	119.3	0.673
Modified Proctor	10.88	115	0.647
Standard Proctor	14.2	111	0.757
Standard Proctor, Reduced	15.5	107	0.743
CHASE BROWN			
Modified Proctor, 35 tamps	14.2	115	0.838
Modified Proctor	16.2	109	0.813
Standard Proctor	18.5	102.3	0.781
Standard Proctor, Reduced	20.95	99.26	0.820
CHASE WHITE			
Modified Proctor, 35 tamps	11.32	111.76	0.616
Modified Proctor	16	107	0.767
Standard Proctor	17	103.2	0.739
Standard Proctor, Reduced	18.5	101.8	0.776
DERIDDER BROWN			
Modified Proctor, 35 tamps	10.5	122.65	0.759
Modified Proctor	13.22	118.24	0.840
Standard Proctor	14.26	111	0.744
Standard Proctor, Reduced	17	106.5	0.789
DERIDDER WHITE			
Modified Proctor, 35 tamps	10.5	117.15	0.658
Modified Proctor	11	116.5	0.677
Standard Proctor	13	109.5	0.661
Standard Proctor, Reduced	15.8	106	0.733

TABLE 6. (continued)
Compaction effort and character

Natchitoches K1-1			
Modified Proctor, 35 tamps	10	120.33	0.675
Modified Proctor	12.72	114.76	0.734
Standard Proctor	13.75	111.5	0.726
Standard Proctor, Reduced	15.75	107.35	0.747
Natchitoches K2-1			
Modified Proctor, 35 tamps	10.86	121.24	0.760
Modified Proctor	11.57	116.3	0.702
Standard Proctor	13.67	111.13	0.720
Standard Proctor, Reduced	16	105.8	0.734
Natchitoches K3-1			

Pumping with Compaction.

While conducting the compaction tests, it was observed 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. The pumping was more pronounced in the A-4 samples than in the more plastic A-6 samples. However, although the dilatant character and pumping appears to be tempered with an increase in plastic character, there was speculation that all of the high-silt samples in this study could pump or become unstable at moisture levels that exceed the optimum.

Figure 12 shows the family of curves at different levels of compaction energy and their line of optimums produced for the Natchitoches K3 soil. The maximum dry unit weight for the modified compaction curve (compaction curve 2) is 116 pcf. If the unit weight is specified as 95 percent of the maximum dry unit weight based on the modified compaction effort, the required unit weight would be approximately 110 pcf. This is slightly less than the maximum dry unit weight produced by the standard proctor compaction (curve 3).

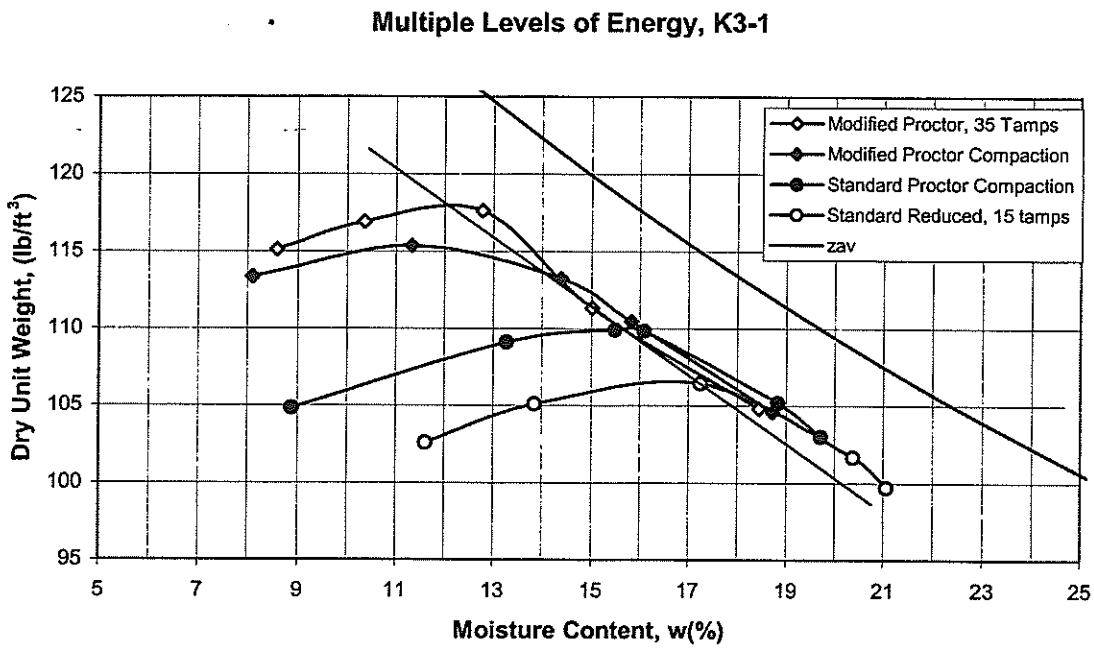


Figure 12

Natchitoches family of compaction curves for different compaction efforts.

Consider situations where the moisture content in the field is represented by either points *a* or *b* and correspond to the initial pass in the field compaction. Both are wet of the optimum of the modified compaction. However, *a* is on the dry-side and *b* is on the wet side of the optimum moisture for the standard compaction effort. Both fall on the dry side of the optimum moisture content for the reduced standard compaction curve (curve 4). With additional compaction, the moisture-density curve approaches that of the standard compaction curve (curve 3) with points *a* and *b* becoming *a'* and *b'*. While both have achieved the specified unit weight, there is a high possibility that pumping will occur at *b'*.

Allowable Moisture Range for Compaction.

The DOTD specifications require a 95 percent relative compaction determined in accordance with DOTD TR 415 or TR 418 (Louisiana Standard Specifications for Roads and Bridges, 1992 Edition). The moisture content at the time of compaction must be within the range of –2.0 percent and +4.0 percent of the optimum established in accordance with DOTD TR 418 or the lifts will be reprocessed and recompacted until these requirements are met. Considering the instability of the silty soils with excessive moisture, the effect of the greater moisture towards on the wet side was reviewed in terms of the soils tested in this study. The pumping zone for these soils occurred at small increases in the moisture content above optimum. A saturation of 75 percent is common to the saturation that occurs at maximum dry density. Using the 75 percent saturation line to approximate the line of optimums, the greatest moisture content wet of optimum that could achieve the 95 percent relative compaction was computed as shown in Figure 13. The moisture content computed represents the optimum moisture content at a minimum compaction effort required to achieve the specified 95 percent maximum dry density.

0

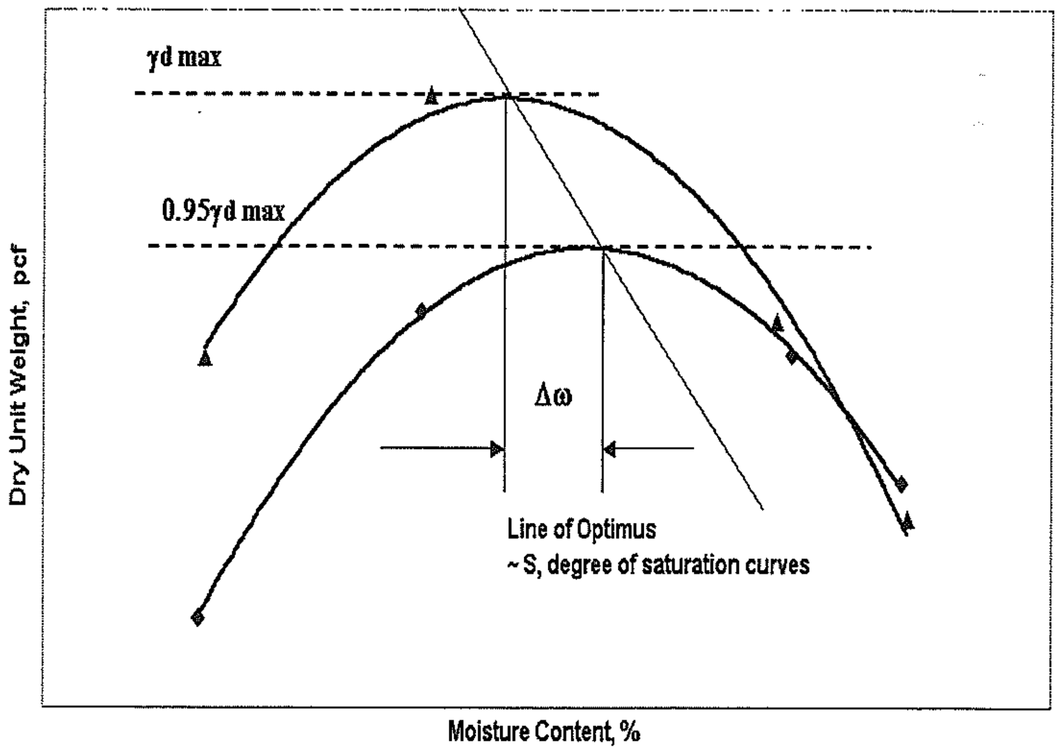


Figure 13

Approximation of Δω for degree of saturation and minimum compaction effort for 0.95γdmax

The range of moisture content was also computed for the measured degree of saturation values from the compaction curves (third degree fit) and is presented in Table 7.

The range of the values computed for the Δω differences between the optimum moisture contents and the moisture content at $0.95\gamma_{dmax}$ for the eight samples varied from a high of 4.3 percent to a low of 0.5 percent. Most were estimated to be slightly more than 2 percent, however. The average for all estimates was 2.3 percent. Considering the nature of the high-silt soils to pump at moisture contents above optimum, the current allowance of a –2 percent to +4 percent of optimum is not compatible with construction objectives.

Table 7.
Moisture Content Required for 95 Percent of Maximum Dry Density

Modified Compaction	γ_{dmax} pcf	ω_{opt}, %	S %	ω, %	$\Delta\omega$, %	S %	ω, %	$\Delta\omega$, %
Acadia	115	11.0	75	14.3	3.3	68	13.0	2.0
Chase White	106	14.7		18.4	3.7	70	16.8	2.1
Chase Brown	109	15.2		16.7	1.5	79	17.6	2.4
DeRidder White	116.5	9.5		13.8	4.3	61	11.2	1.7
DeRidder Brown	118	11.0		13.2	2.2	74	13.1	2.1
Natchitoches K1	114	11.5		14.7	3.2	72	14.1	2.6
Natchitoches K2	116	12.0		14.0	2.0	76	14.2	2.2
Natchitoches K3	115.8	11.0		14.0	3.0	69	12.9	1.9
Standard Compaction								
Acadia	110	14.0	75	15.9	1.5	74	16.1	2.1
Chase White	103	18.0		19.3	0.5	80	20.7	2.7
Chase Brown	102.5	18.0		19.5	1.5	79	20.6	2.6
DeRidder White	109.5	13.0		16.5	3.5	67	14.9	2.0
DeRidder Brown	110	14.4		16.3	1.9	77	16.7	2.3
Natchitoches K1	110.5	14.5		16.1	1.6	79	16.9	2.4
Natchitoches K2	111.5	14.0		15.7	1.7	78	16.3	2.3
Natchitoches K3	110	15.0		16.3	1.3	80	17.4	2.4

Strength

Using the Harvard compaction device, test specimens of all of the samples were compacted at different compaction efforts or energy levels (i.e., tamps per layer). The unconfined compressive strength of each sample set was determined for different unit weight and moisture contents of the different compaction efforts.

The variation between the strengths achieved by compacting on the dry side or wet side of the optimum moisture content is shown in Figure 14. On the wet side of optimum, there is a tendency for the specimens with the higher compaction effort to have lower strengths than those with a lower compaction effort.

These soils exhibit significant strength when compacted at the optimum moisture content or dry side of the optimum moisture content. The penetration resistance measured on some of the modified Proctor compacted samples exceeded the capacity of the pocket penetrometer (i.e., greater than 4.5 tsf). However, when subjected to an increase in moisture content, they became very soft.

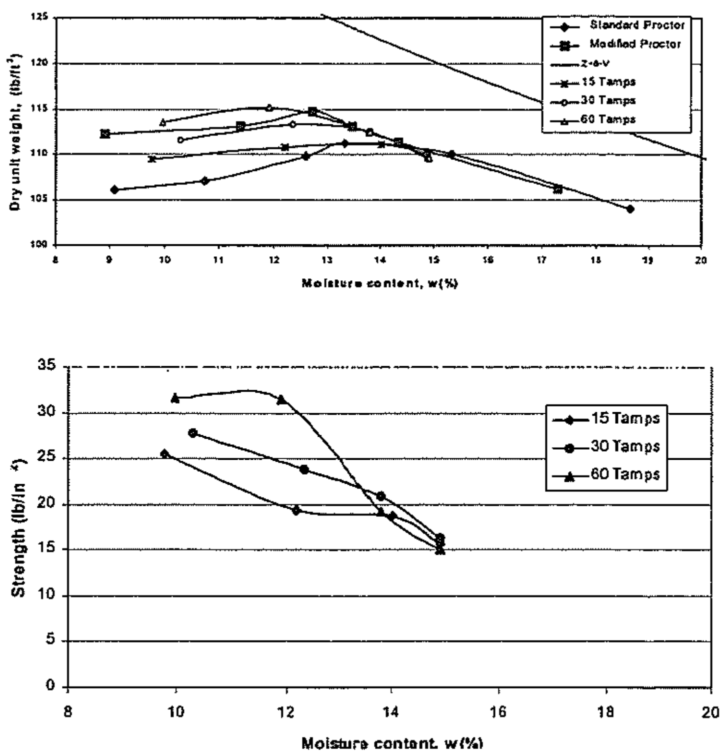


Figure 14
Natchitoches K1 compaction effort / strength variation

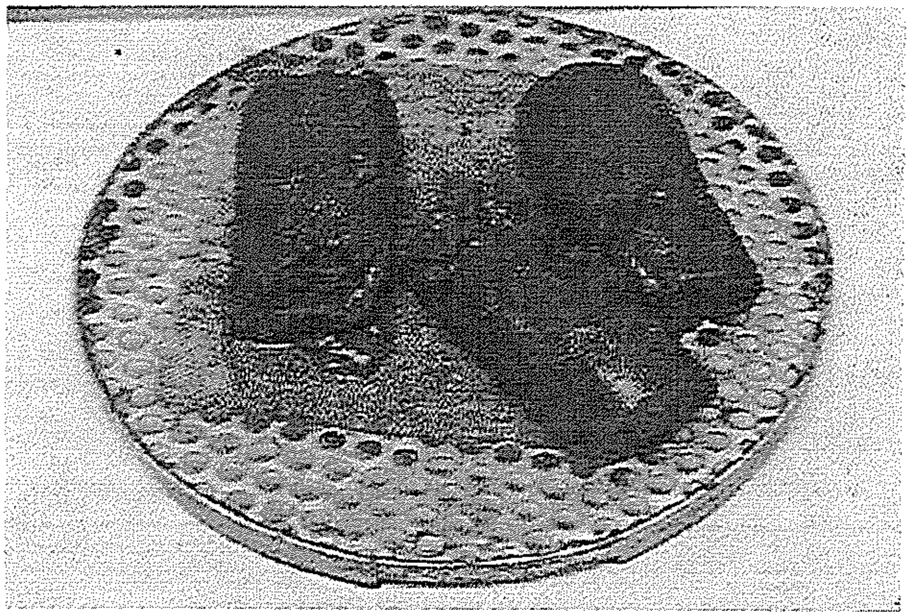


Figure 15
Natchitoches soil after being subjected to vacuum saturation

Figure 15 shows what occurred when three of the Harvard compacted samples of the Natchitoches soils (K1, K2, and K3), remolded at optimum moisture content, were inundated and subjected to vacuum saturation for one hour. The saturated specimens lost all strength. Other specimens were placed in a vacuum chamber with their base surface in contact with water. The chamber was subjected to a vacuum for one hour. At the end of the one hour period, the base of the remolded specimen was bulging with addition moisture. Thus, even though these high silt soils can be compacted and stabilized under conditions that do not produce a pumping situation, it is not a permanent condition. Increased moisture and traffic loads can produce unstable support after achieving the specified unit weight.

A comparison of the stress / strain relationship typical of these high-silt soils, as exhibited by DeRidder White and Brown samples, portrays a more in depth description of how soft the higher silt soils become at moisture contents on the wet side of optimum. Figures 16 and 17 present the stress / strain relationship of the DeRidder White and Brown samples. The DeRidder White A-4 soil demonstrates a much greater loss in stiffness due to the additional moisture than that of the DeRidder Brown, A-6 soil. However, the A-6 soils may be prone to pumping and experience a loss of strength with excess moisture. Their added cohesive character makes them less susceptible to pumping.

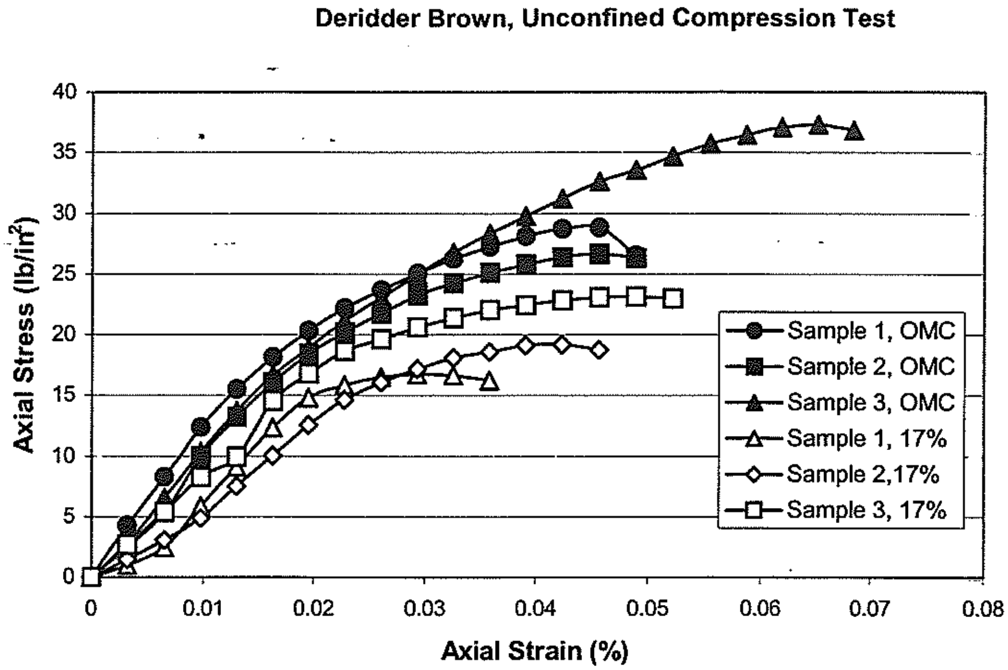


Figure 16

DeRidder Brown stress / strain curves for specimens at optimum and wet of optimum

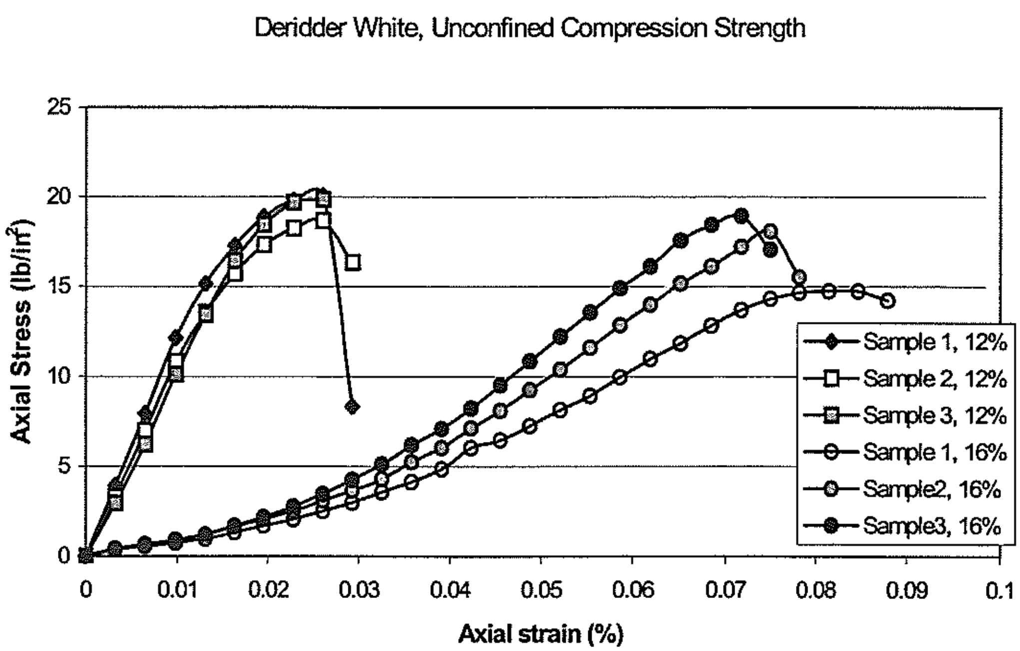


Figure 17

DeRidder White stress / strain curves for specimens at optimum and wet of optimum

Strength / Moisture Density Relationship

A regression analysis of the strength relationship in terms of the moisture content and dry density was conducted for each of the soils. The dependency of the strength on the variation of moisture content and density is expressed in the following form.

$$\text{Log } s_u = A + B\gamma_d + C\omega$$

Where,
s_u = undrained strength
γ_d = dry unit weight
ω = moisture content
A, B, C = regression coefficients

The coefficients for the equation that defines the regression line and the best fit for the data points was determined for all of the soils as shown in Appendix C. Indicators for estimating the degree to which this regression relationship represents the corresponding variables were generally very good. For an exact fit of the data, the coefficient of determination (the square of the correlation coefficient, r^2) equals unity, (i.e., all data points will fall on the regression line). The r^2 value for the eight soils varied from a high of 0.99 to 0.89. In general, the low to non-plastic silts seem to have produced equations with the best fit for the data as demonstrated by the r^2 value.

The regression analyses are summarized in the analysis of variance (ANOVA) table provide in the Appendix. The significance and confidence for the model and variables is verified in the statistical analyses for the F-test, t-statistic, etc. An example of the lines of equal strength for the Acadia soil is shown in Figure 18. Graphs for the other soils are also presented in Appendix C.

The strength/moisture/density relationship can be used as a tool for construction planning or in pavement design. It provides an accurate prediction of the strength of the soil under different scenarios. For a compaction density, the strength produced with seasonal changes in moisture levels can be estimated.

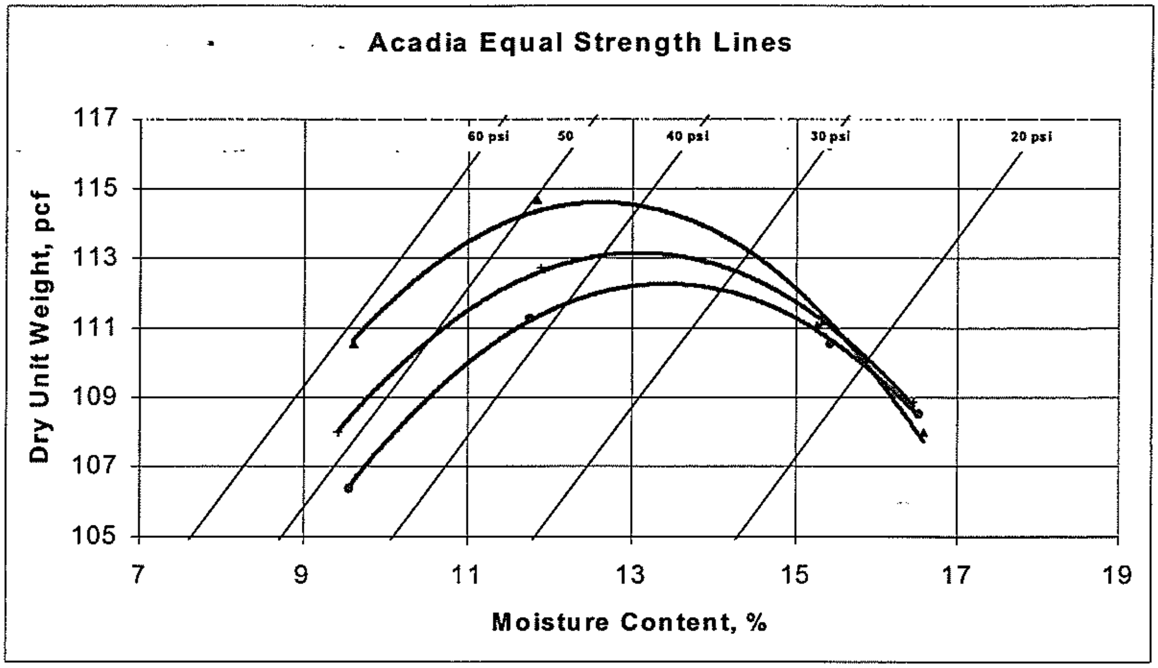


Figure 18
Acadia Strength / Moisture / Density Relationships

Cyclic Triaxial Compression Tests

In the cyclic triaxial tests, the stress loads went from overburden to LDH maximum wheel-load stresses calculated using the elastic layer theory. The cyclic loads, $\sigma_1 - \sigma_3$, varied from 100 to 600 psf (1 to 5 psi). The chambers confining pressure, σ_3, for all tests measured 2 psi. The number of cycles (N), the axial strain (ε), and the change in axial strain due to cyclic loading ($\Delta\varepsilon$) were noted during the tests. The test specimens were free to absorb water during load cycles. The on-set of pumping was manifested by greatly increased strains at the same shear stress for soils susceptible to pumping or liquefaction.

The different responses to cyclic loading of the plastic (CL/A-6) and the non-plastic (ML/A-4) soils can be observed in Figures 19 and 20 for the DeRidder Brown (PI=17) and White (PI=1) specimens. The strain continues to increase as the load cycles continue in the non-plastic DeRidder White as shown in Figure 19. The strain response to the load cycles for the Deridder Brown test specimen, however, approaches a constant value. The strain rebounds with the cyclic reduction in load.

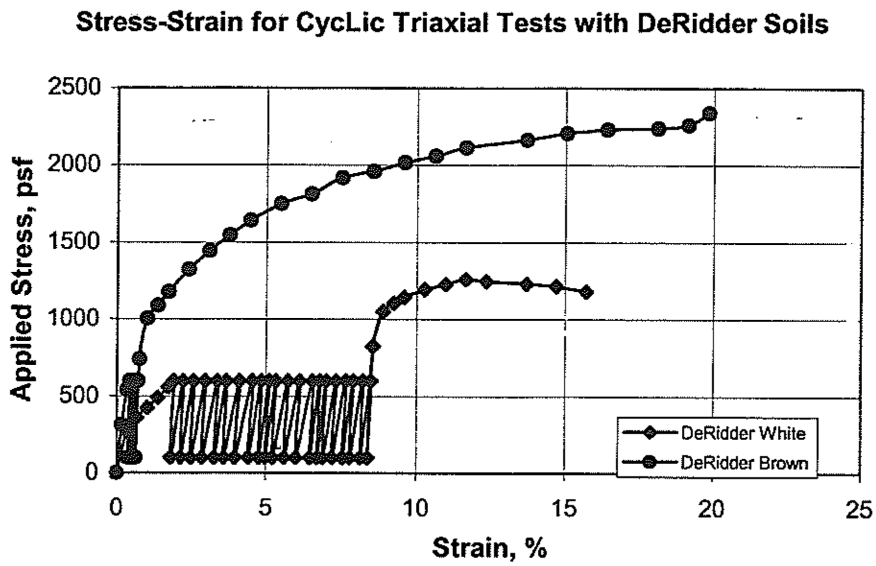


Figure 19
Stress-strain cyclic triaxial tests comparison for ML and CL Soils

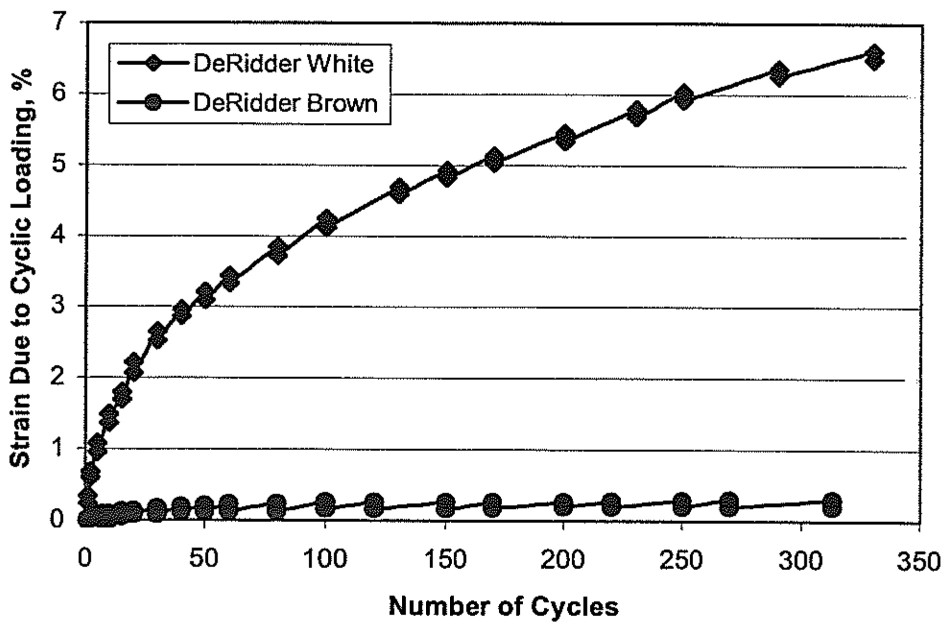


Figure 20
Cumulative strain produced by cyclic loads for DeRidder Brown (CL) and White (ML) soils

Figure 20 demonstrates the reserve strength beyond the cyclic loading for the two soil types, plastic and non-plastic. Very little additional strength beyond the maximum cyclic load is developed with the DeRidder White specimen (ML/A-4). The increasing pore pressure corresponding to an increasing strain diminishes the strength of this DeRidder specimen. Contrastingly, the cyclic loading did not impede the stress-strain relationship for the DeRidder Brown specimen. The CL soil continues to provide additional support (or resistance) with loading increased beyond the 600 psf maximum cyclic load.

Chemical Stabilization

Current DOTD specifications permit the use of lime for stabilization of soils that do not meet the DOTD's usable criteria in situations or locations where select materials may be costly or unavailable. Chemical stabilization with lime is a technique used to construct the working table, prevent pumping, and to achieve the relative compaction requirements for the subgrade. Its primary purpose is to dry the wet silts and allow them to be compacted. Additional budget allowances are provided on projects with problems involving high silt soils for this purpose.

Section 305.04 (1992 Edition) gives minimum amounts of cement or lime and cement for treatment of a subgrade layer as:

PI	Lime or Cement (by vol.)
0 – 10	8% cement
11– 20	10% cement
21 – 35	10% lime and 8% cement

Lime treatment conforms to Section 304 for Type C treatment (conditioning for cement treatment), as specified as above. For cement treatment the quantity of cement shall be as specified above.

Lime Treatment specifications in Section 304, 1992, states that, "unless specified, the percent lime shall be determined in accordance with DOTD TR 416" for Type B, C, D, and E Lime Treatment. The DOTD TR 416-93 method identifies the minimum percentage of hydrated or quicklime required is that which meets specifications for Liquid Limit and Plasticity Index. DOTD TR 433-81 gives the minimum lime content for a subbase as a maximum liquid limit of 40 and plasticity index of 35 following lime treatment. A specification does not appear that provides a target or minimum support value expected of the subgrade or provides for the possibility of using a subbase. None of the existing specifications seem to apply to the silty soils, which may be compatible with the DOTD's concept of a usable soil (Section 203.06, 1992 Edition) that excludes soils whose silt content is more than 60 percent. However, without performance criteria, a comparison of stabilization efforts or mixture requirements is difficult to establish.

The major concerns expressed by DOTD engineers in this investigation have involved 1) the ability to properly compact and achieve the specified unit weight for the subgrade or embankment without pumping and 2) to avoid the conditions of instability or pumping as a result of construction traffic loads during or before the placement of the base and pavement. In effect, the primary goal is to provide a working table capable of supporting the base-pavement construction. Any support or lack of support provided by the subgrade after construction is not included in the design considerations for the pavement, (i.e., the base/pavement structure is designed to carry the load). The only performance requirement by DOTD is that "the contractor shall construct a subgrade layer that will provide adequate support for construction of the base/pavement structure," Section 305.01, 1992.

An evaluation of stabilization efforts in this study was limited to chemical stabilization using three of the most common additives, hydrated lime, Portland cement, and Class C fly ash. The hydrated lime (ASTM C207, Type N) used in the tests was produced by the United States Gypsum's Lime Division's New Orleans Plant. The typical chemical and physical analyses for their product is provided in Appendix E. The class C fly ash (ASTM C618) was obtained from Bayou Ash of Baton Rouge, Louisiana. The chemical and physical analysis provided by Bayou Ash is shown in Appendix E. A Type I, Portland cement (ASTM C150) was used.

Test Series 1. General Stabilization Character

The effects of lime, lime plus fly ash, and Portland cement on the compaction character and strength of the eight samples were investigated. The percentages of additive by dry weight selected for the compaction and strength tests with molded specimens from the eight samples included 1) four-percent lime-soil mixtures, 2) two-percent lime plus eight-percent fly ash-soil mixtures, and 3) the strength produced by specimens molded from the silty soils (A-4/ML) and four-percent Portland cement-soils. These tests provided a comparison of the performance of the natural soil with the modifying/stabilizing additives molded under similar moisture conditions.

The Harvard compaction apparatus and method was used to mold the lime and fly ash test specimens. The compaction effort approximated the results obtained in the standard compaction test method (AASHTO T99). In producing the compaction curves for the first series of tests, four specimens were molded for each sample type at different moisture contents with the 4-percent lime 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 "as molded". The three remaining 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 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 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 moisture content approximately equal to the optimum moisture content plus 6 percent. One specimen was tested immediately. 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. Tests results are provided in Appendix E.

Test Series 2. Modification/Stabilization of Wet Silts

A second series of tests were conducted with the Chase White (A-4/ML) and Brown (A-6/CL) and the DeRidder White (A-4/ML) and Brown (A-6/CL). Using the soil in a state significantly wet of the optimum moisture content, different percentages (dry weight basis) of Portland cement (8 and 10 percent), lime (6 percent), Class C fly ash (10 percent), and 3 percent lime plus 10 percent fly ash were used for comparison. These mix combinations were arbitrarily selected for comparison of the differences between sample specimens molded with the different additives and the wet natural soil. The percentage of additives specimens produced for the corresponding sample included:

	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%

This second series of tests attempts to simulate a situation where a wet, high-silt soil is encountered. The initial moisture content of the soils exceeded their optimum moisture contents by several percentage points, but it was not too wet for compacting the natural or raw soil with the Harvard spring plunger. A specimen was fabricated with the natural soil at the moisture content established. Four test specimens were compacted for each set consisting of a soil sample and selected mixture of additives. Thus, the percentages of Portland cement, lime, fly ash, and lime-fly ash were added to the four wet soils (Chase/DeRidder White and Brown), and three test specimens from each mixture were molded with the Harvard compaction equipment, approximating the standard compaction effort.

The natural soil specimen and one of the specimens molded with the different admixtures were tested as molded. This was done as a measure of the potential for modifying and/or drying the soil. The other two specimens 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 100 percent 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 in unconfined compression.

Compaction Character with Additives

In the first series of tests, the addition of the chemical additives produced compaction curves for the mixtures that typically had lower unit weights and slightly higher optimum moisture contents, Figure 21 and 22, and Appendix E.

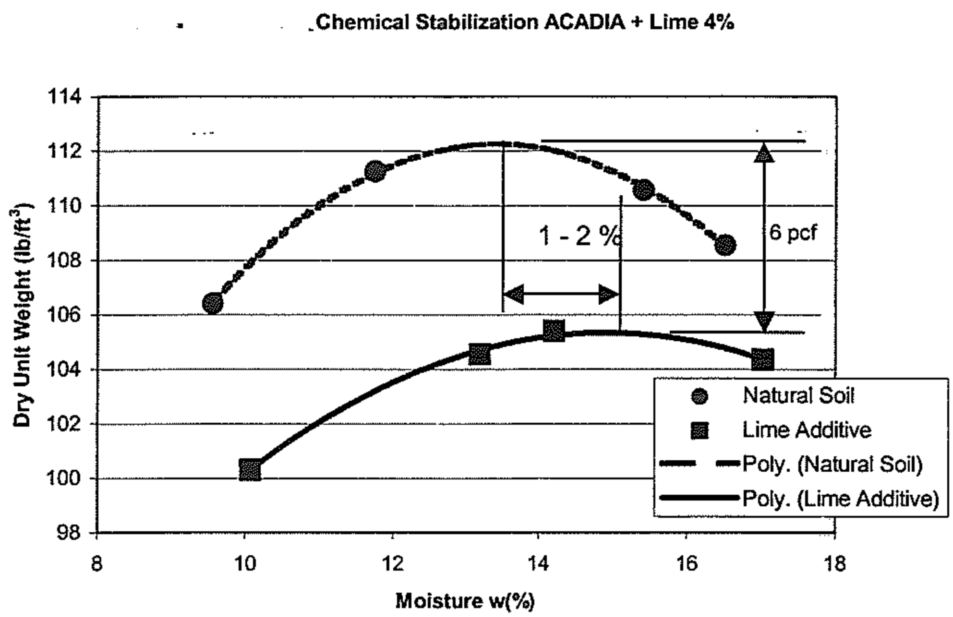


Figure 21

Compaction curves for the Acadia soil alone and with 4 percent lime

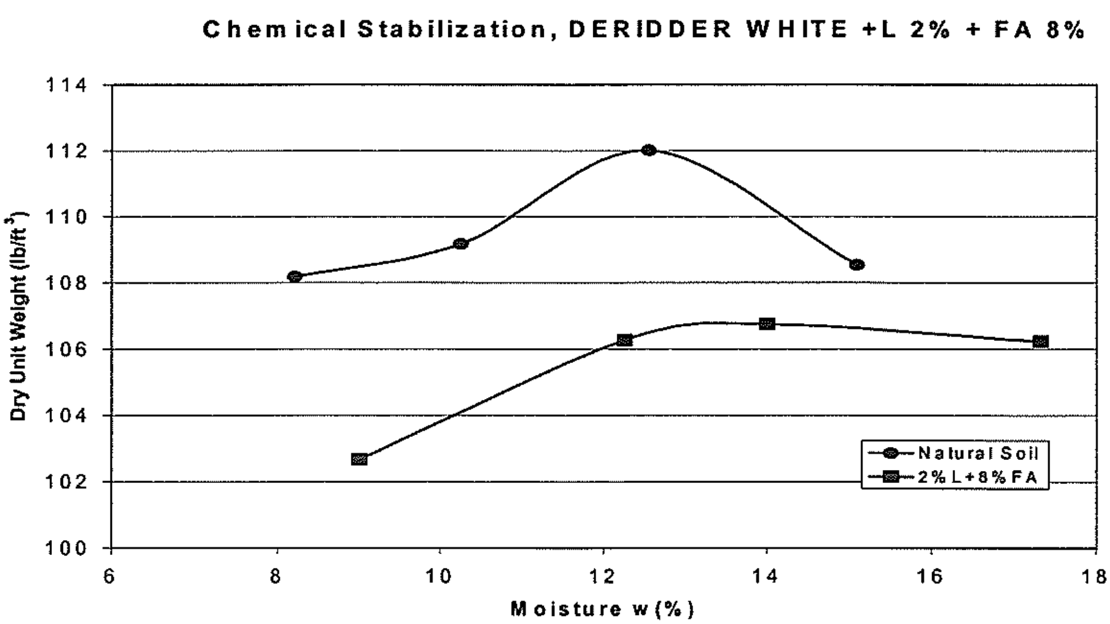


Figure 22

DeRidder White Compaction Curves with 2% Lime+8% Fly Ash

The maximum dry unit weights for the percentages of the lime and the lime plus fly ash specimens ranged from 3 to 12 pcf less than the natural soil. The range of moisture contents used in the compaction efforts approximated those used to determine the natural soil's compaction curve. This moisture range was not sufficient in producing a maximum unit weight for the Natchitoches K2 and K3 sample mixtures as shown in Appendix E. The optimum moisture content required to produce maximum dry density for the mixtures of Acadia, Chase, and DeRidder soils varied from 1 to 3 percentage points greater than the optimum moisture content of the natural soils.

Stabilized/Modified Strengths.

In general, the addition of the lime, lime-fly ash, and Portland cement improved the strength of all samples in the first series of tests. Shown in Figure 23 is an example of the strengths developed by the soil mixtures as molded and cured (2 weeks in the humidity room, HR, and/or a 24-hour accelerated, rapid cure, RC) and the impact of changing environmental conditions (i.e., with vacuum saturation, VS).

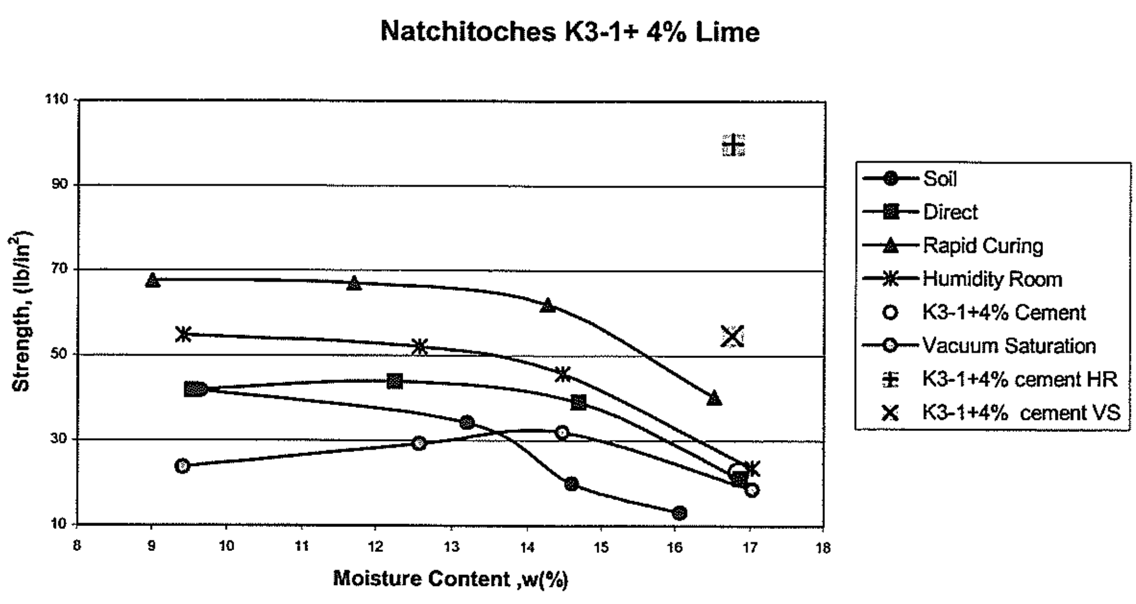


Figure 23

Stabilized/Modified Strength for Natchitoches K3 Soil with 4% Lime

The variation for the measured strengths of the different additives and mix proportions used is approximated as:

<u>Lime:</u>	RC > HR > VS/as molded > natural soil
<u>Lime-Fly Ash:</u>	RC / HR > as molded > natural soil / VS
<u>Portland Cement:</u>	HR > VS > as molded / natural soil

All of the chemical additives used in this test series appear to improve the natural soil's strength. In some cases, this improvement is significant, especially if the soil is allowed to cure. However, when subjected to vacuum saturation, much of the strength gain for the lime or lime-fly ash mixtures was lost. This may be partially because the quick curing periods (24 hours) were insufficient time for cementitious products to be developed. Also, if the percentages of lime/fly ash used are insufficient for their distribution within the silt, the opportunity for pozzolanic activity may be impeded. This was not the case for the Portland cement specimens. The strength increase of the Portland cement specimens over the natural soil's at the higher moisture contents was significant and much strength was retained after vacuum saturation. The vacuum saturation strength was approximately twice that of the compacted natural soils.

Microanalysis of Stabilization Products

A microanalysis of the specimens from the first test series was conducted to identify whether any cementitious products were being produced. A digital AMRAX 1820 scanning electron microscope was used to digitize the specimen images and to determine the dispersive spectra present. A SCINTAG XDS x-ray diffractometer was used with the pulverized specimens in determining the common minerals expected as well as potential reaction products. The formation of secondary cementitious phases (Ca-Al-silcates, Ca-silicates, etc.) was detected as summarized in Table 8. The bulk composition (quartz through titanium oxide) is normalized to 100 percent and does not include the secondary cementitious materials. The cementitious components are based on a percentage of these products found in the whole sample.

The secondary cementitious phases of these mixtures did not exhibit as high a degree of crystallinity and may have escaped detection by XRD. Poorly crystalline materials are difficult to detect. It is possible that with additional aging and with contact of moisture the formation of the crystals in these material would be improved. Only crusty coatings over the pre-existing quartz, feldspars, and other components of the initial material were found in the energy dispersal spectral analysis. The EDS analysis did indicate more prevalent Ca-bearing silicates, however, even though well-formed crystals were not found.

The one-day (24-hour) rapid curing time was concluded to be insufficient for the full development of the cementitious crystals. Thus, in the second series of tests, the rapid curing time was increased to three days at 50° C.

Table 8

Cementitious Phases Found in Soil Mixtures with Lime and Lime-Fly Ash

	Acadia			**DeRidder White**			
	Lime HR	**LFA HR**	**LFA RC**	**Lime RC**	**Lime HR**	**LFA RC**	**LFA HR**
quartz	57	57	54	62	56	59	62
K-feldspar	12	10	13	15	13	9	12
Na-plagioclase	15	12	11	10	15	14	14
muscovite	2	3	3	3	3	1	
biotite/chlorite	7	6	5	4	3	5	5
Other clay minerals	5	5	5	5	6	6	6
ilmenite		1				1	
hematite	2	4	4		2	1	
zircon		1	2			2	1
dolomite		1	3	1	2	2	
titanium oxide							
Cementitious materials	The following proportions of cementitious phases were identified based on counts of particles for 4% lime-soil and 2% lime 8% fly ash-soil mixtures.						
calcite/portlandite	6	4	2	6	3	4	5
Ca-silicate	1	1	1	1	2	3	1
Ca-Al-silicate	3	3	1	1	2	2	2
Ca-Al-S-silicate	2	1	1				1

Table 8 (continued)

	Chase Brown			Chase White			
	Lime RC	LFA HR	LFA RC	Lime RC	Lime HR	LFA RC	LFA HR
quartz	55	55	57	58	55	60	58
K-feldspar	20	15	16	13	12	9	14
Na-plagioclase	8	14	11	17	18	12	16
muscovite	1	1	4	2	4	3	1
biotite/chlorite	4	3	3	2	1	5	5
Other clay minerals	8	5	6	4	5	6	5
ilmenite				2			
hematite	3	6	2		1	2	
zircon				1	3	1	
dolomite						2	1
titanium oxide	1		1	1	1		
Cementitious materials	The following proportions of cementitious phases were identified based on counts of particles for 4% lime-soil and 2% lime 8% fly ash-soil mixtures.						
calcite/portlandite	8	2	3	3	3	1	3
Ca-silicate		2	2	1	2	2	1
Ca-Al-silicate		3	1	1	1	2	2
Ca-Al-S-silicate		1	1		1	1	

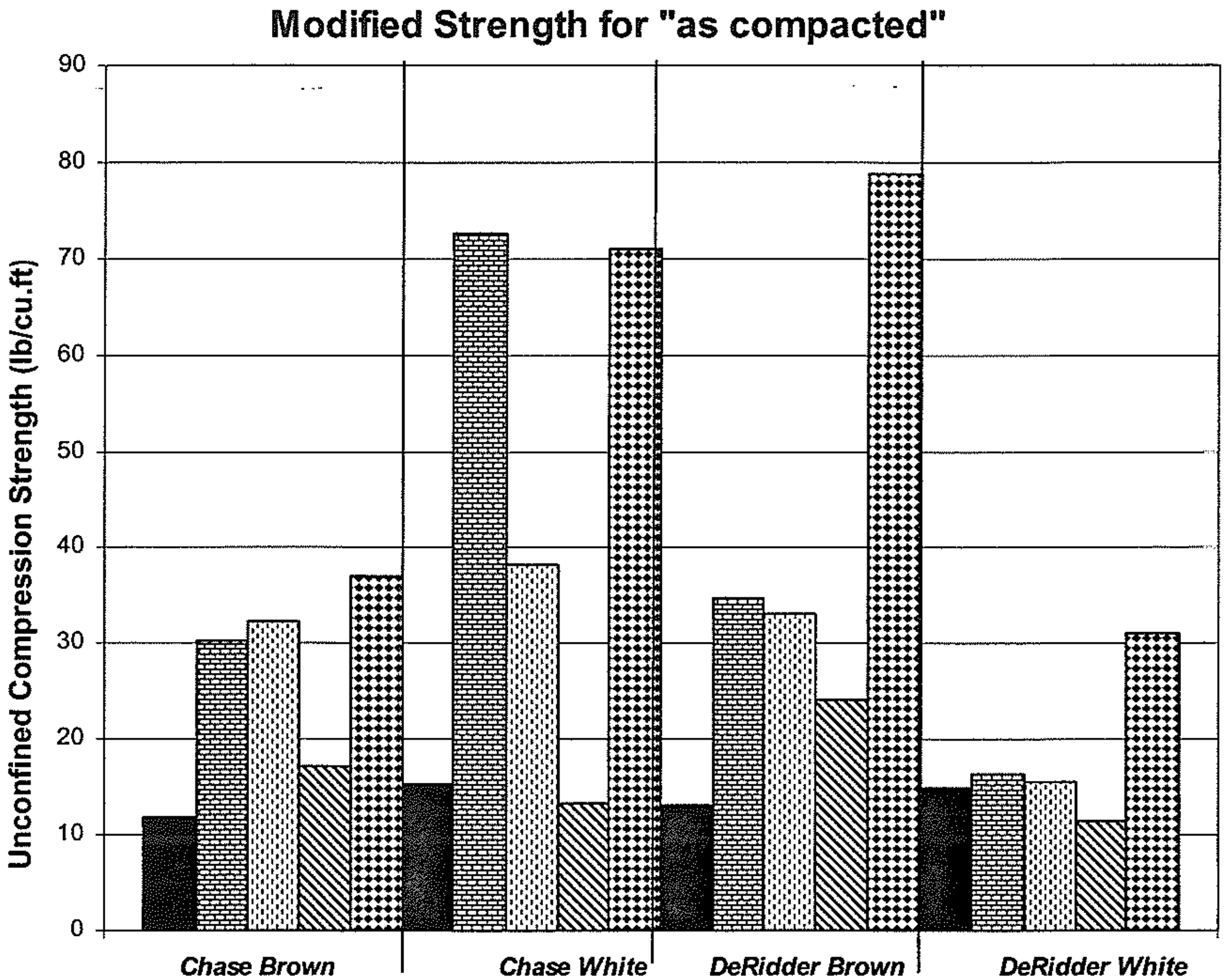
Modifying/Stabilizing Performance with a Wet Soil

The second series of tests simulated efforts to modify and stabilize an excessively wet soil that might be encountered in the field. The molded specimen strengths were measured under the test conditions for 1) as compacted, 2) cured plus vacuum saturation, and 3) the fully cured strengths for the selected mixes. Under the first test conditions, "as compacted", the strength achieved with the additive mixture represents the modification potential. The mixture providing the greatest improvement or modification to the natural soil's strength should indicate a greater resistance to pumping during field compaction. The second condition, curing followed by vacuum saturation, was predicted to produce a reduced strength of the mixture as might occur with seasonal inundation of the in-place soil. This strength would be important in questions involving the subgrade's long-term performance in supporting the pavement structure. The cured strength from the third set of tests provided an estimate of the maximum strength achievable.

The mix proportions used were arbitrarily selected for comparison. The test results do not necessarily identify the acceptability of a specific additive or the mix percentages used without a target support value or strength requirement. However, the resulting strength measurements for the different test conditions do provide information for a relative comparison. In general, for the mix proportions used, the additives and combination of additives seemed to improve the soil's performance in terms of construction and long term seasonal conditions.

Modification of Wet Silts

Figure 24 provides a comparison of the "as compacted" strengths achieved by the mixtures for the four samples. The lime-fly ash mixtures seem to consistently provide the best performance (higher strengths). Fly ash, used as a lone additive, performed the poorest when used with the A-4 soils, Chase White and DeRidder White. The fly ash strengths produced were less than those of the natural soils. The performance of the lime and Portland cement was similar for the four soils except that the Portland cement strength for the Chase White soil rivaled the produced strength of the lime-fly ash. With the exception of DeRidder White, the lime, lime-fly ash, and Portland cement significantly improved the strengths of the natural soils. However, only the lime-fly ash showed an improvement for the DeRidder White. In other words, the lime-fly ash-soil's strength was twice that of the natural soil's strength. The DeRidder White specimens had the poorest performance in strength gain for the samples tested. It was also the most unstable of the soils tested.



PC L FA LFA PC L FA LFA PC L FA LFA PC L FA

Figure 24

Comparison of "as compacted" modified soil

The ability of the additives to act as a drying agent for the DeRidder White is shown in Figure 25.

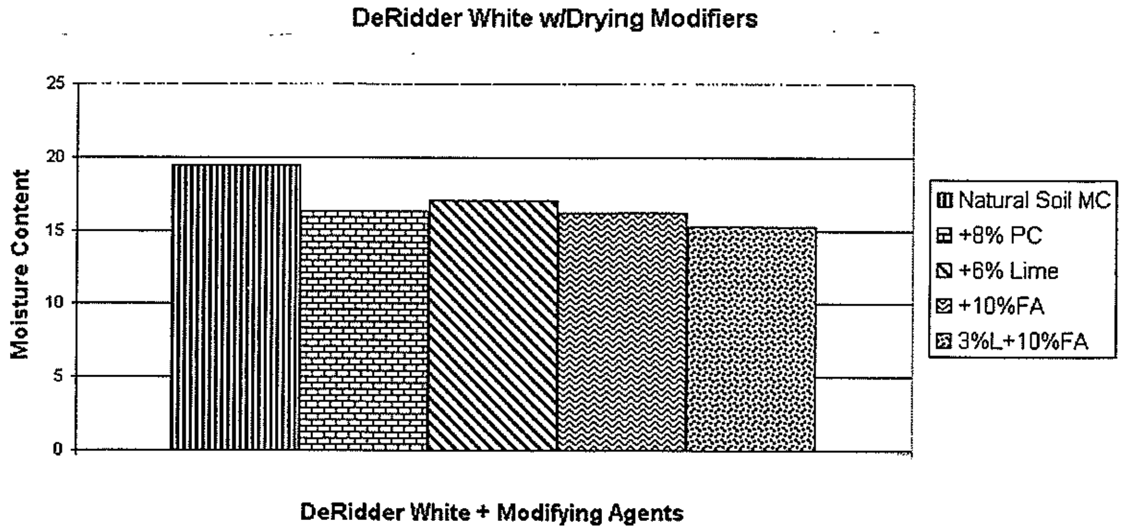


Figure 25
Additives drying ability with DeRidder White Soil

The decrease of the natural soil's moisture with the introduction of the additives for all the samples was typically less than three to five percentage points for the mixtures used. The effect of lime and fly ash as lone additives on the plastic character for three of the A-4 soils (Chase White, DeRidder White, and Natchitoches K2) was measured as shown in Table 8. This table also shows the reduction in moisture content after the additives have been mixed with the wet soils. The liquid limit, plastic limit, plasticity index, and estimate of the required optimum moisture content for the various mixtures is shown in the table. The optimum moisture content (OMC) was estimated using the Johnson and Sallberg chart (1962). Assuming the OMC to be correct, a comparison between the OMC and the resulting moisture content of the mixture can be made to see if the two values converge, therefore reducing their tendency to pump (drying out due to the resulting OMC). Figures 26 and 27 plot the variation of the parameters (PL, LL, OMC, ω_{mix}) for the percentages of lime and fly ash used, respectively, with the Chase White soil.

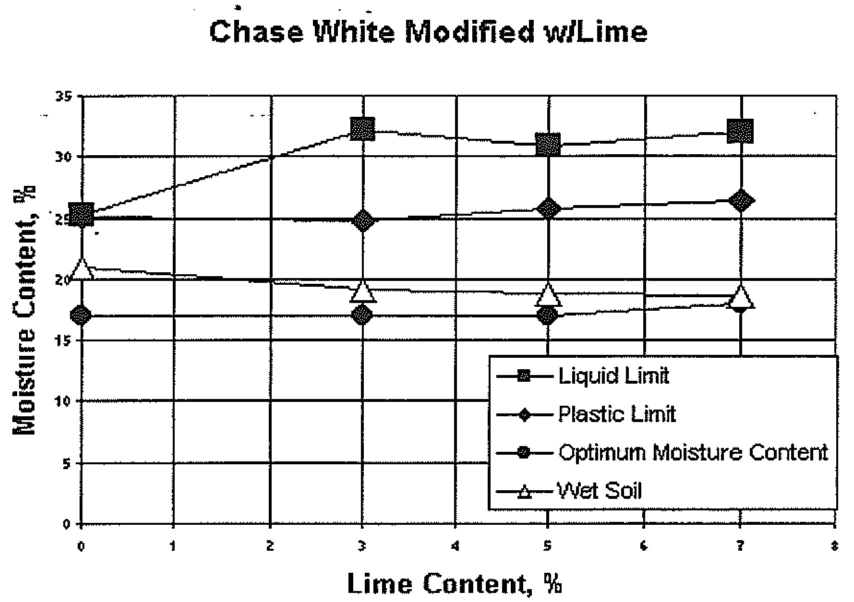


Figure 26

Variation of plasticity, OMC, and moisture with lime content

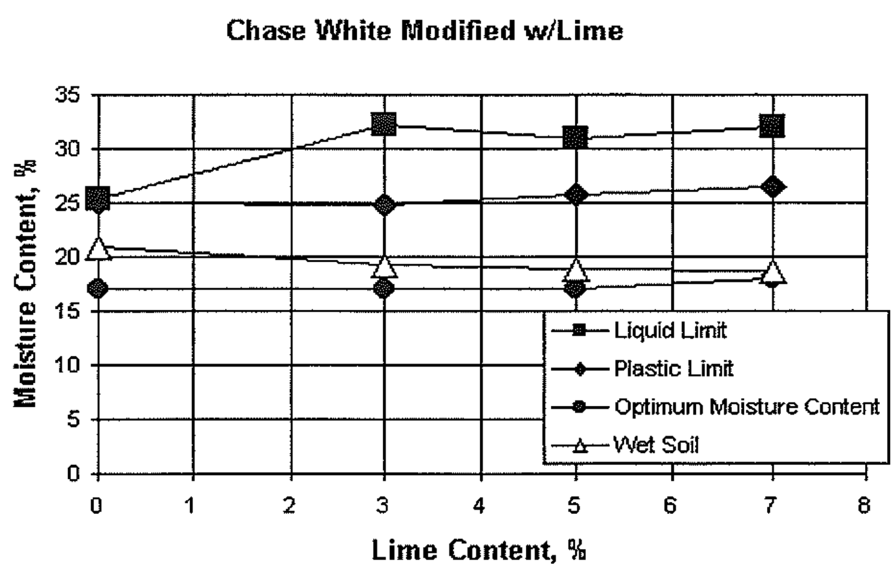


Figure 27.

Variation of plasticity, OMC, and moisture with fly ash content

An inspection of Table 9 shows that the addition of lime and fly ash generally produce an increase in the PI values (their plastic character). The DeRidder White PI values, however, were only slightly altered.

Table 9.
The Effects of Lime and Fly Ash on the Atterberg Limits of the Silt Soils

	LL	PL	PI	OMC	Moisture of soil	Moisture of mixture
Chase White	25.3	25.02	0.28	17	-	21
Chase White + L 3%	32.16	24.76	7.4	17	21	19.18
Chase White + L 5%	30.88	25.78	5.1	17	21	18.83
Chase White + L 7%	32	26.47	5.53	18	21	18.62
Chase White + FA 8%	30.63	20.74	9.89	15	20.3	18.05
Chase White + FA 12%	29.33	21.83	7.5	15	20.3	17.71
K 2-1	24.72	19.78	4.94	14.2	-	20.9
K 2-1 + L 3%	32.5	26.62	5.88	18	20.9	19.59
K 2-1 + L 5%	33.24	25.62	7.62	18	20.9	18.77
K 2-1 + L 7%	33.37	26.09	7.28	18	20.9	18.48
K 2-1 + FA 8%	30.5	21.62	8.88	15	20.9	18.31
K 2-1 + FA 12%	30.48	21	9.48	15	20.9	17.62
Deridder White	19.35	18.15	1.2	13.06	-	17.4
Deridder White + Lime 3%	21.38	18.72	2.66	13	17.4	16.58
Deridder White + Lime 5%	22.35	18.74	3.61	13	17.4	15.32
Deridder White + Lime 7%	22.14	19.75	2.39	13	17.4	15.22
Deridder White + FA 8%	18.6	15.75	2.85	11	17.4	15.21
Deridder White + FA 12%	18.78	15.66	3.12	11	17.4	15.04

A plot of the PI versus the percent additive content for Chase White is shown in Figure 28.

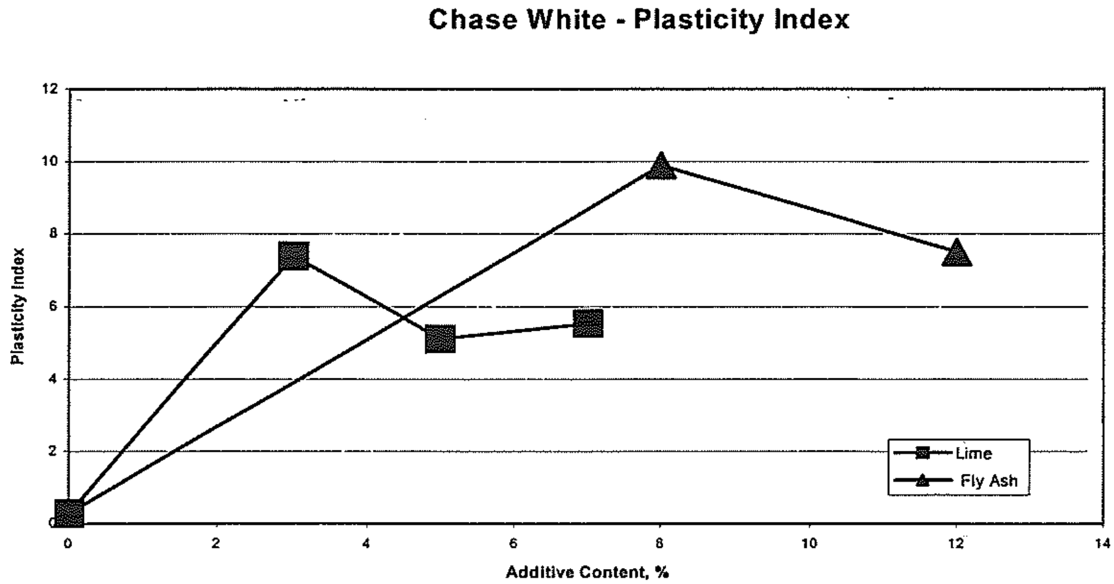


Figure 28
Plasticity Index vs Percent Additive

Long-Term Stabilzation of Wet Silts

Curing produced significant gains in strength for the lime, lime-fly ash, and Portland cement as shown in Figure 29. The curing was accelerated for the lime and lime-fly ash mixtures. The rapid cure (RC) was conducted at 50° C for 3 days.

The RC curing period was extended in the second series of tests due to the microanalysis findings of the first series of tests. The one-day RC cure had only produced premature cementitious products. The class C fly ash used alone and the Portland cement were cured in a humidity room for 2 weeks. It is possible that the different curing techniques used do not provide specimens that are comparable for review of the produced strengths.

The accelerated cure of the 6 percent lime specimens produced the greatest gains in strength (200 to 500 psi). However, the cured specimens of the lime-fly ash (3 and10 percent, respectively) and Portland cement (10 percent) performed almost as well with the four soils. The fly ash used alone provided some minor gains, but these strengths were much weaker than those produced with other additives (lime, lime-fly ash, and Portland cement).

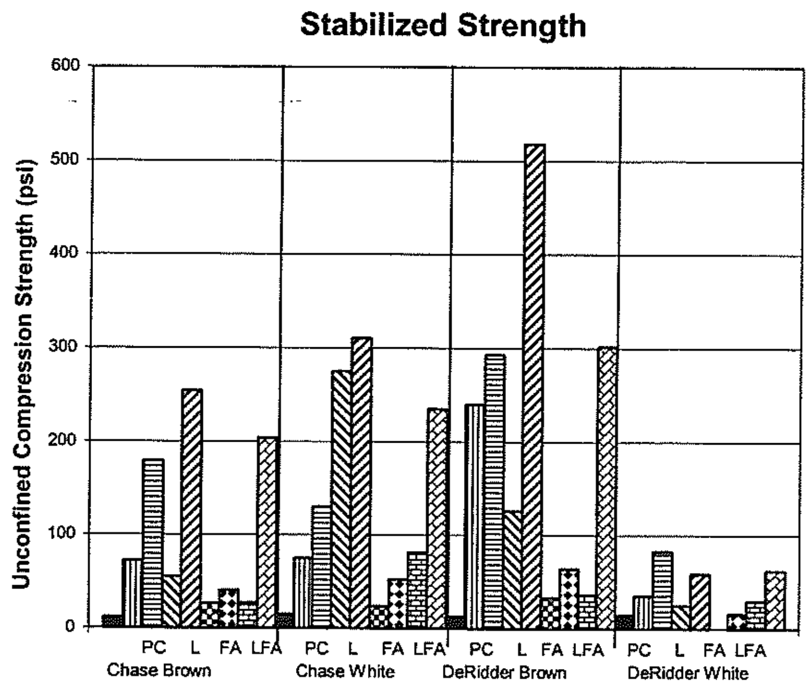


Figure 29

Soil Sample Strengths for natural soil, soil mixtures cured , soil mixtures cured and subjected to vacuum saturation.

The strengths produced in the DeRidder White soil with the lime, lime-fly ash, and Portland cement were similar, but represent the smallest gain in strength for the four samples. The mixture percentages used for this sample were insufficient in producing a significant difference in strength. For these additives and the mix proportions used, the gain in strength for all samples would be sequenced as Lime $\geq$ Lime-Fly Ash and/or Portland Cement >> Fly Ash alone.

Testing the cured specimens for strength after subjecting them to vacuum-saturation attempted to simulate seasonal changes or inundation for long-term performance considerations. Figure 30 provides a comparison between the full strength potential provided by the cured/stabilized soil mixture and the strength that remains after vacuum saturation.

As can be observed, most of the curved strength of the samples is reduced by 50 percent or more after vacuum saturation. Also, the vacuum saturation strength in all but three of the tests was less than 100 psi.

The vacuum saturation strengths for three of the soils (Chase Brown/White and DeRidder Brown) also remained at a level slightly greater than the natural soils' "as compacted" strength. However, the natural soils in most cases, especially the A-4 soils, lose all strength when subjected to vacuum saturation. The DeRidder White soil with fly ash also disintegrated when subjected to vacuum saturation. A higher percentage of additives is probably required to provide the formation of a cemented structure between the granular constituents and through out the specimen.

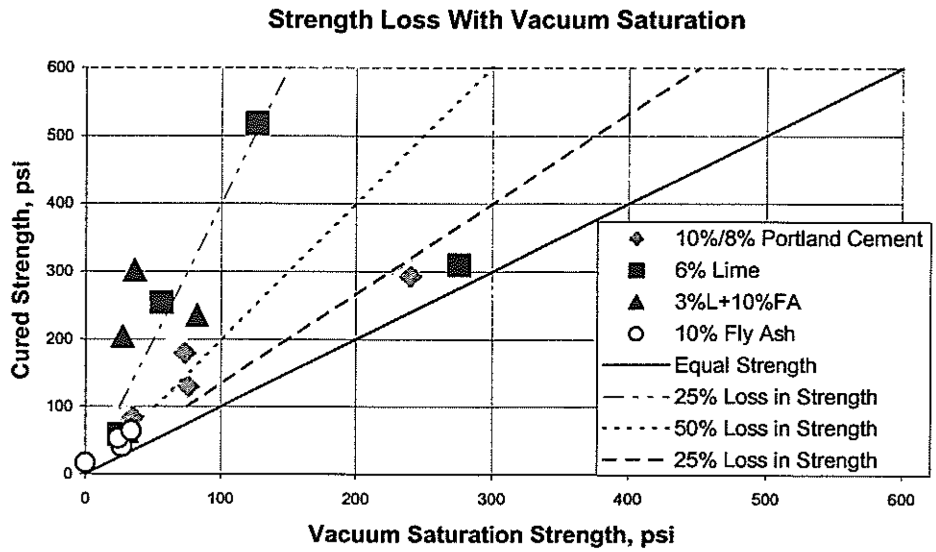


Figure 30
Reduction in Strength with Vacuum Saturation

CONCLUSIONS

This study documents problems associated with the compaction of and the support provided for subgrades/embankments by soils containing high percentages of silt-size particles. Site visits, personal interviews, and a DOTD-industry survey were used to document and further describe the problem. A laboratory testing program using eight high-silt soils typical of those commonly associated with the pumping problem was conducted to establish their physical, compaction, and stability character and their performance with modifying or stabilizing agents.

Occurrence and Identification

The district surveys have demonstrated that soils with high silt percentages (+50 percent) are 1) widely distributed across the state, 2) commonly encountered in the construction of pavements, and 3) are a difficult, if not a problematic, soil for use in the construction of pavement subgrades and embankments. The soils with a high-silt content (+50 percent) and a low plasticity to non-plastic character commonly pump under construction equipment wheel loads. Fine-grained soils (including fine sands) with silt percentages greater than or equal to 50 percent and whose plasticity index, PI, are less than 10 are highly susceptible to pumping.

Description of Pumping

When an excessively wet, silty-soil is subjected to a compaction effort in which the energy applied is too great, pumping or weaving will result as the equipment wheels shove the wet, weaker soil ahead of itself. The conditions that contribute to a pumping situation are:

1. a soil with a high silt content that is non-plastic or of low plasticity,
2. the presence of excess soil moisture content or access to a source of moisture, and
3. an excessive compaction effort or construction traffic that produces strain wet of optimum.

Under the strain imposed by compacting equipment or construction traffic, the dilatant character of silts produces volume changes with corresponding pore pressure increases. The nonplastic silts are fine enough for relatively high capillary pressures to develop and yet not so fine that the flow of water (permeability) is restricted. Their strength is reduced and they pump when the soils are wet of optimum and have under-dynamic loads. With the low-plastic silty-soils (PI < 10), pumping is further enhanced due to the lower hydraulic conductivity and the higher pore pressures. The cohesive character of the silt-clay soils with higher plasticity ranges (PI > 10) produces an increase in cohesion and a resistance to pumping.

Specifications for Compaction and Usable Soils

The district surveys and the construction questionnaire identified problems with the current DOTD specifications that were specific to high silt soils. These include the specifications that address usable soils and compaction.

The definition of a usable soil as currently given in the 1992 edition of the Louisiana Standard Specifications for Roads and Bridges is 1) inadequate or too limited, 2) may be unenforceable, and, 3) as a general rule, does not provide a good engineering solution.

Soils with high silt contents can be compacted without pumping to meet specifications for unit weight and to provide a firm load bearing subgrade. The current allowable range of moisture conditions (-2 to +4 percent of optimum moisture content) afforded contractors is inappropriate for these soils. With close attention to quality control during construction and assuming that post-compaction activities do not alter (increase) the moisture conditions, the support provided by the subgrade/embankment should be adequate to complete the pavement. However, they do become unstable with increased moisture and/or wheel loads that can produce a pumping or low-load bearing situation. Continuous support and long-term performance is uncertain.

Modification/Stabilization

The modification/stabilization issues addressed involved 1) the performance of modification agents that would permit the compaction of the high-silt soils without pumping, 2) the increased stability potential provided through stabilization, and 3) an investigation of long-term performance or stability that could be provided. The three agents considered included lime, class c fly ash, and Portland cement used either alone or in combination. The soil samples were provided by the DOTD districts. The number of tests and mixture variations were limited to the samples and volumes of soil provided.

Modification

For the mixtures used, the lime-fly ash specimens seemed to perform the best and greatly improved (modified) the strengths of the soils in the laboratory "as compacted" tests. However, the lime and Portland cement modification of the soil also produced a significantly improved, compacted strength. Fly ash, alone, did not perform well in these tests.

Long-Term Stabilization

The long-term stabilization of these soils was a secondary consideration in this investigation. Thus, the testing program was conducted in an effort to establish the technical feasibility of stabilization. High silt soils can be stabilized with chemical additives. Lime, lime-fly ash, and Portland cement can potentially modify the soils pumping character during construction, and they have the potential to provide long-term stabilization if proper mixtures and placement are accomplished in the field. Thus, the volume or percentage of additive and the field activities necessary to achieve this must be determined. Before these factors can be established, definitive objectives must be stated with respect to the requirements for the subgrade's performance during construction and post-construction.

The accelerated cure of the lime specimens produced the greatest gains in strength in the laboratory tests with these soils; however, the cured specimens of the lime-fly ash and Portland cement performed almost as well. The fly ash used alone provided some minor gains, but the strengths achieved were much less than the other additives.

RECOMMENDATIONS

The following recommendations are proposed.

Identification

Fine-grained soils (including fine sands) containing silt percentages of 50 percent or more and whose plastic character is such that the Plastic Index (PI) is less than 10 should be considered to have a high pumping potential. The more plastic, high-silt soils (PI > 10) may pump under specific conditions, but are less susceptible.

The study should be extended to further consider the impact of the plastic/cohesive character in retarding or in the development of a pumping condition.

Usable Soils

The concept of usable soils should be reconsidered or eliminated. The wide occurrence of the high-silt soils and the lack of acceptable replacement materials in some districts make this specification extremely difficult to achieve. These soils are currently being used with modification methods, primarily with lime as a drying agent. It is recommended that a more specific, end-product method be developed for the use of these soils.

Compaction Specifications

The allowable moisture range of –2 to +4 percent of optimum moisture for field compaction should be changed to –2 to +2 percent of optimum moisture for the high-silt soils. Further consideration for compaction density based on the modified compaction test (AASHTO T180) may be warranted. This may address the use of today's heavier equipment and produce a denser, more stable subgrade. The higher compaction effort will also require a reduced moisture content to achieve the unit weight requirements.

Modification/Stabilization Alternatives

A testing protocol for evaluating the mixture design, and acceptance standards is needed to guide the selection for modification/stabilization of subgrade soils. They should include considerations for 1) placement and compaction, 2) construction activities after subgrade compaction, and 3) the design performance in service as a pavement subgrade.

REFERENCES

Louisiana Standard Specifications For Roads and Bridges, 1992 Edition
Cryer, M. M., Engineering Geologist, LDOTD District 07, (personal communications and survey response)

Holtz R., and Kovacs, W. An introduction to geotechnical engineering, Prentice-Hall, Inc., 1981.

Spangler, M., and Handy, R. Soil engineering, 4th Ed., Harper & Row, Publishers, 1982.

Sherard, J.; Woodward, R.; Gizienski, S,; and Clevenger, W. Earth and earth-rock dams, John Wiley & Sons, Inc., 1963.

Terzaghi, K, Peck, R.B., and Mesri, G., Soil Mechanics in Engineering Practice, 3rd Edition, John Wiley and Sons, New York, 1996.

Ishihara, K., "Stability of natural deposits during earthquakes", Proceedings, 11th International Conference on Soil Mechanics and Foundation Engineering, Vol.1, pp.321-376, 1985.

Ishihara, K., "Liquefaction and flow failure during earthquakes", Geotechnique, Vol., 43, No.3, pp.351-415, 1993.

Wang, W.A., "Some findings in soil liquefaction", Water Conservancy and Hydroelectric Power Scientific Research Institute, Beijing, China, 1979.

Finn, W.D.L., Ledbetter, R.H., and Wu. G., " Liquefaction in silty soils: design and analysis", Ground failures under seismic conditions, Geotechnical STP, No.44, ASCE, pp.51-76, 1994

Guo, T., and Prakash, S. "Liquefaction of silts and silt-clay mixtures." Journal of Geotechnical and Geoenvironmental Engineering, ASCE Vol. 125, No. 8, August 1999.

Sandoval, J. "Liquefaction and settlement characteristics of silt soils." Ph.D. thesis, University of Missouri-Rolla, MO, 1989.

Prakash, S., and Sandoval, J. "Liquefaction of low plasticity silts." Journal of Soil Dynamics and Earthquake Engineering, Vol 7(7), 1992.

Day, R., "Discussion of effects of fines and confining stress on undrained shear strength of silty sands," Journal of Geotechnical and Geoenvironmental Engineering, Vol 125, No. 8, ASCE, 1999.

Design of small dams: A water resources technical publication. 3rd Ed., Bureau of Reclamation, United States Department of the Interior, 1987.

Clarence, S., and Finbarr, A. "Design considerations for collapsible soils." Journal of Geotechnical Engineering, Vol. 107, No. 3, ASCE, 1981

Arman, A., and Thornton, S., "Collapsible soils in Louisiana," Engineering Research Bulletin No. 111, Louisiana State University, Division of Engineering Research for the Louisiana State Department of Highways, 1972

Casagrande, A., "Discussion of frost heave," Proceedings, Highway Research Board, Washington, D.C., 1931

Appendix A
Construction Experience Survey

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 "I don't know" and wrote)*
  High silt content soils cannot be used on slopes yet "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.

- *an'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, "Hydraulic 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 ->*
- *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 "pump" 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 "pumping" 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 Densityyes -------> 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, 1983Many 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 onlyvibratory 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*

- *Standqrd*

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 "None - 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 "in the pit (soil usage: PI, % organic, % silt)," "on 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 "slight 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 "End 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 – 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 "no." 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 soilNo - 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 "magic 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 "up 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 proper 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 "super" saturation of subgrade soils.*
- *no answer and wrote "I 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.

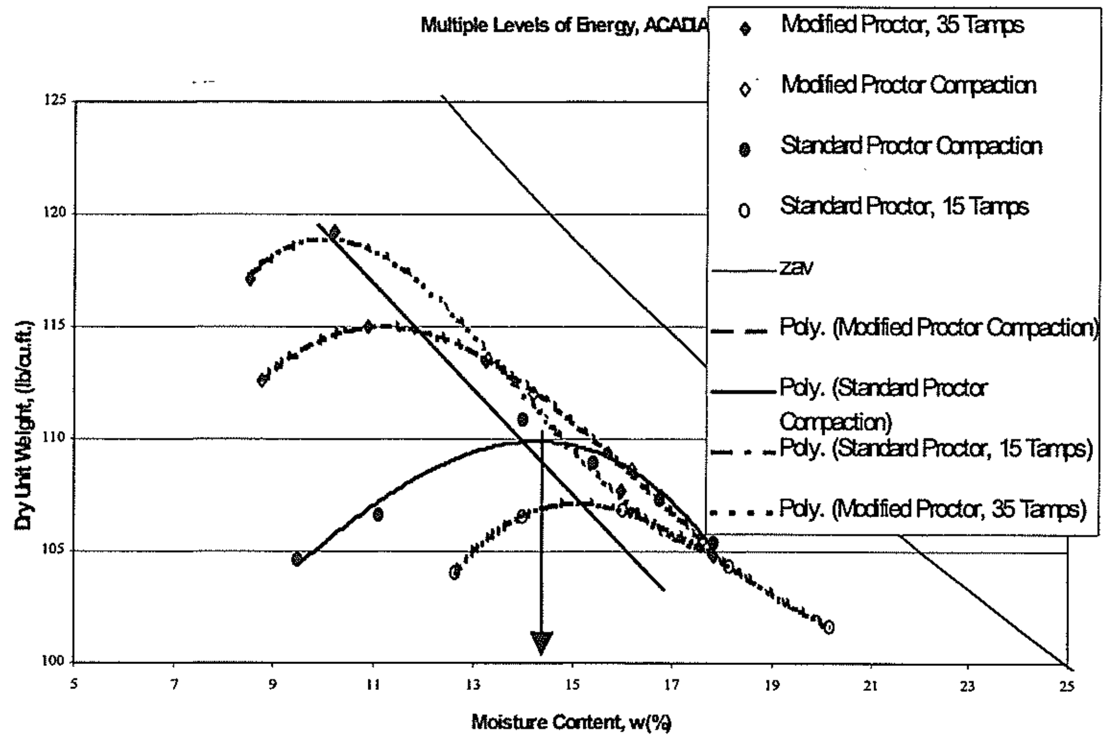
Name:
Title:
Company:
Address:

phone:
fax:
email:

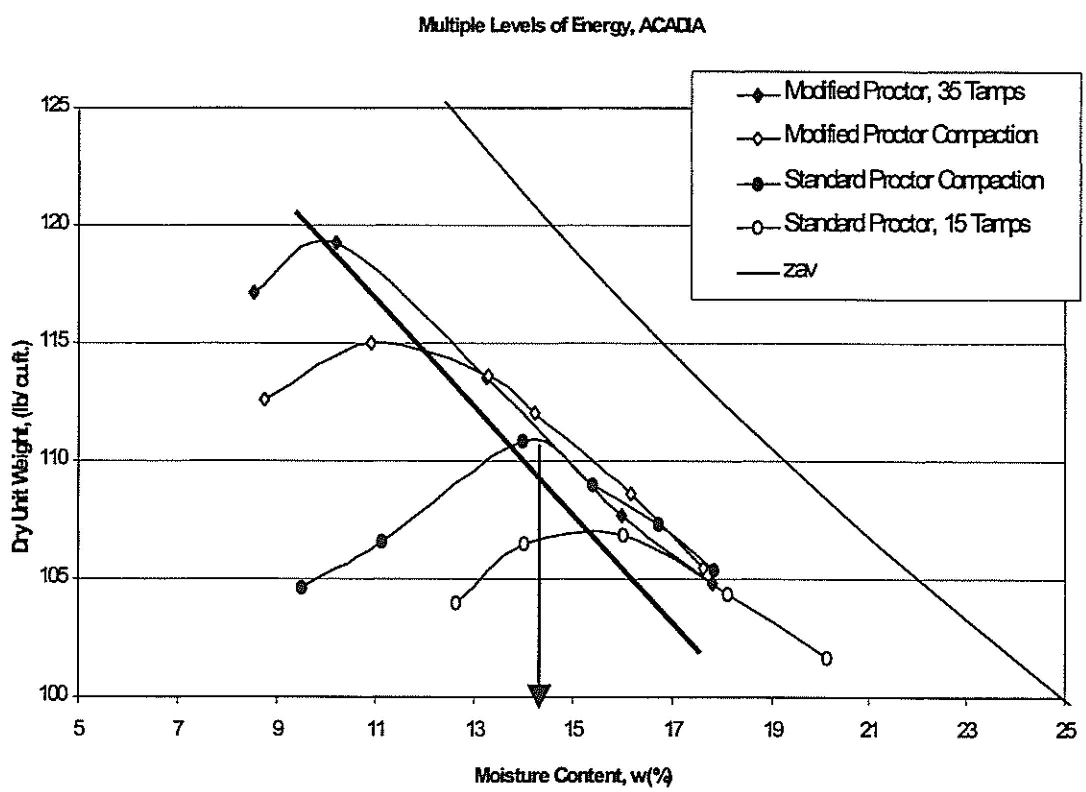
Please return the survey to: Dr. Kenneth McManis
University of New Orleans

If you have any questions you may contact Dr. McManis at (504) 280-6271 or Mr. Mark Morvant, LTRC at (225)767-9124

Appendix B
Compaction Curves of Natural Soil

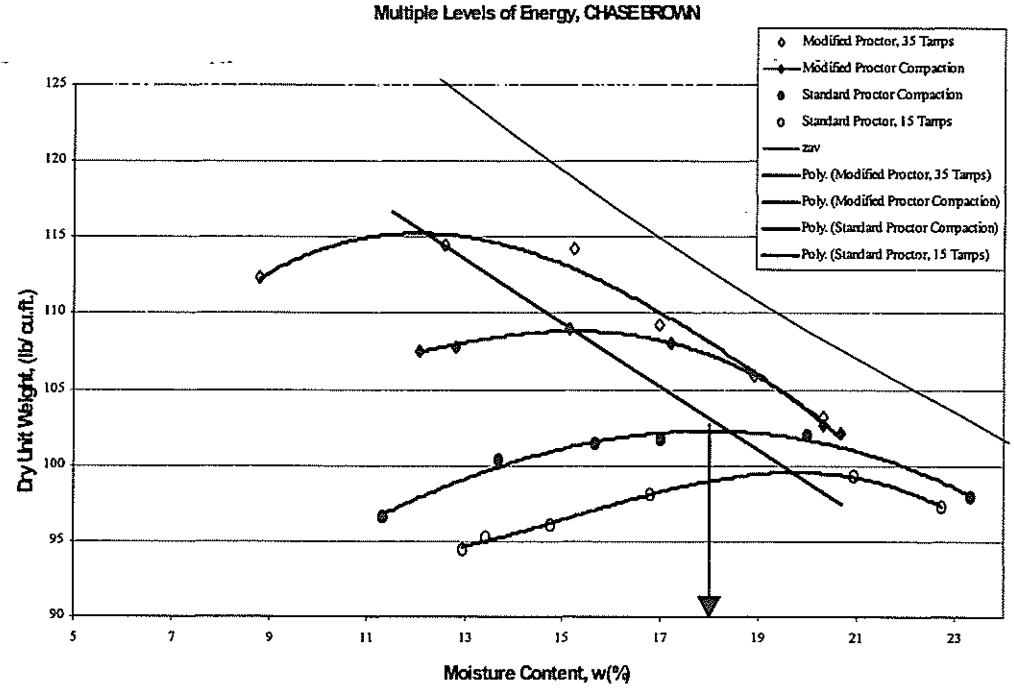


Multiple Levels of Energy, ACADIA
Modified Proctor, 35 Tamps
Modified Proctor Compaction
Standard Proctor Compaction
Standard Proctor, 15 Tamps
zav
Poly. (Modified Proctor Compaction)
Poly. (Standard Proctor Compaction)
Poly. (Standard Proctor, 15 Tamps)
Poly. (Modified Proctor, 35 Tamps)
Dry Unit Weight, (lb/cu.ft.)
Moisture Content, w(%)

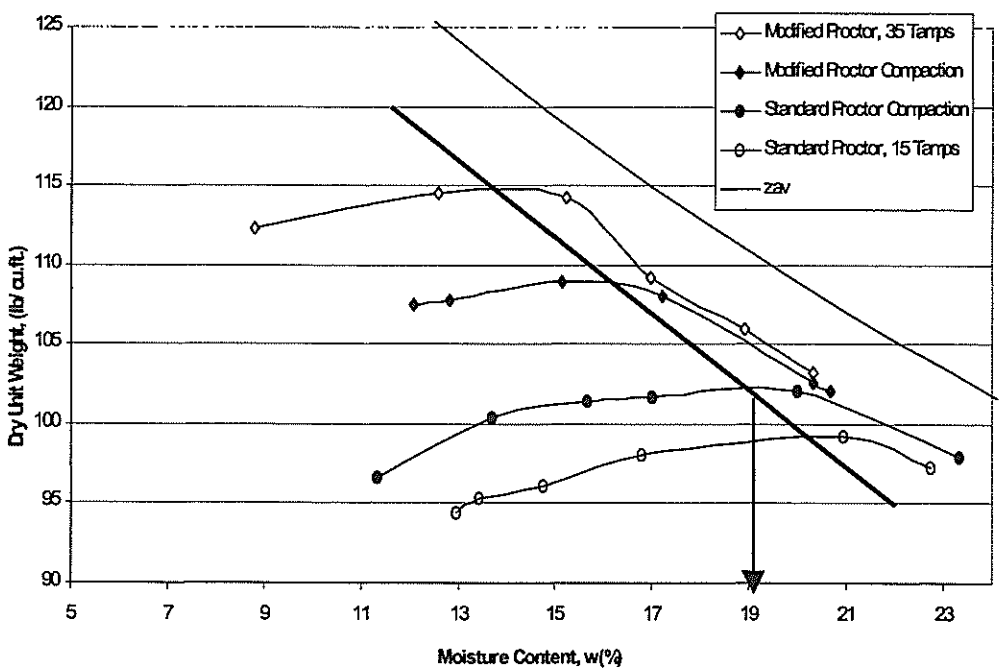


Multiple Levels of Energy, ACADIA
Modified Proctor, 35 Tamps
Modified Proctor Compaction
Standard Proctor Compaction
Standard Proctor, 15 Tamps
zav
Dry Unit Weight, (lb/cu.ft.)
Moisture Content, w(%)

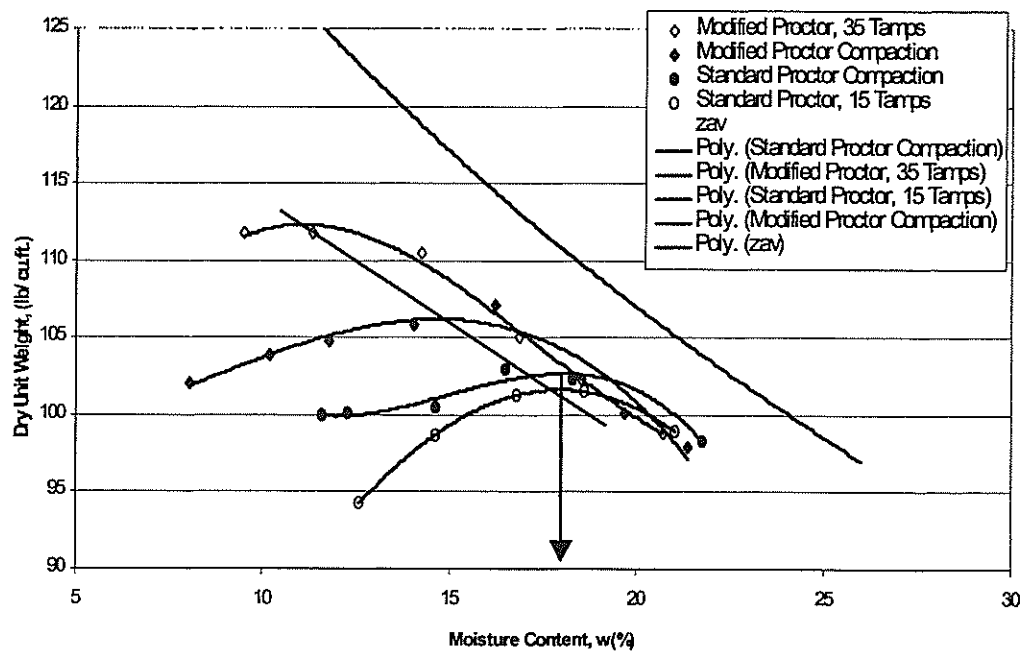
Multiple Levels of Energy, CHASEBROWN



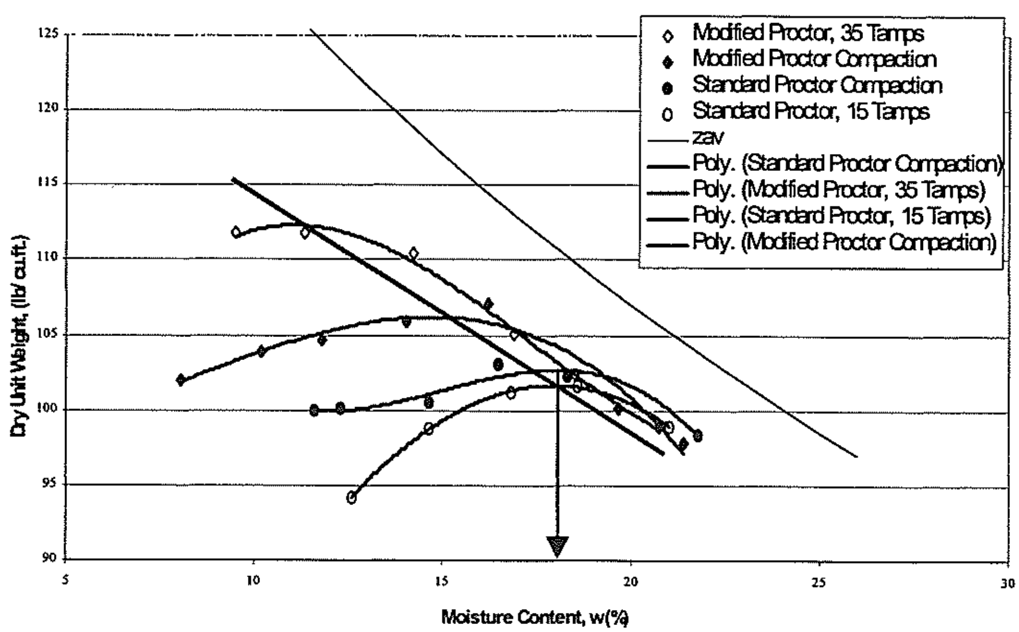
Multiple Levels of Energy, CHASEBROWN

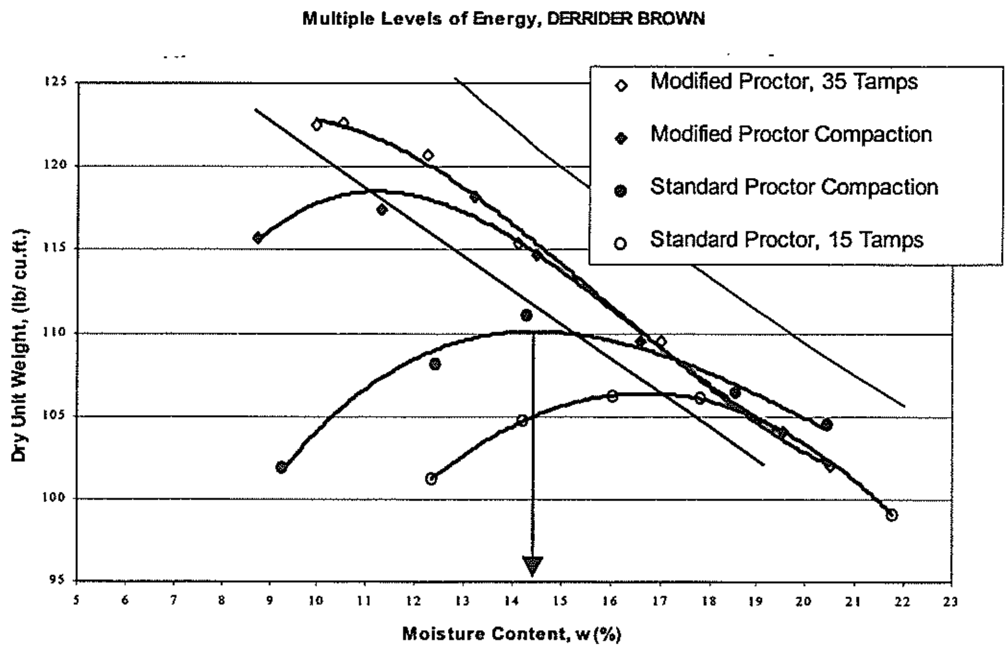


Multiple Levels of Energy, CHASEWHITE

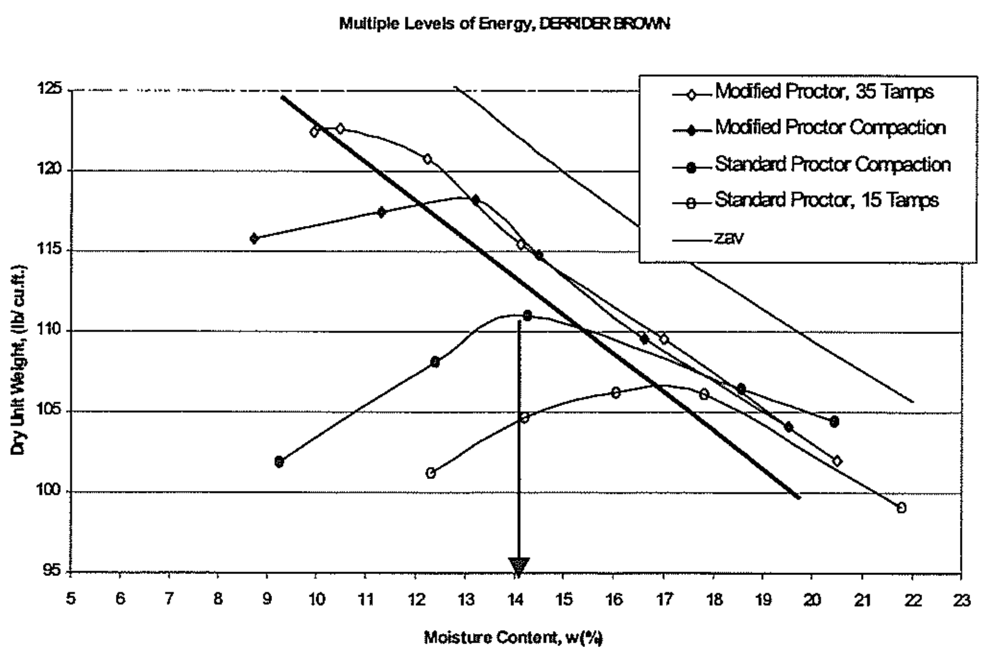


Multiple Levels of Energy, CHASEWHITE

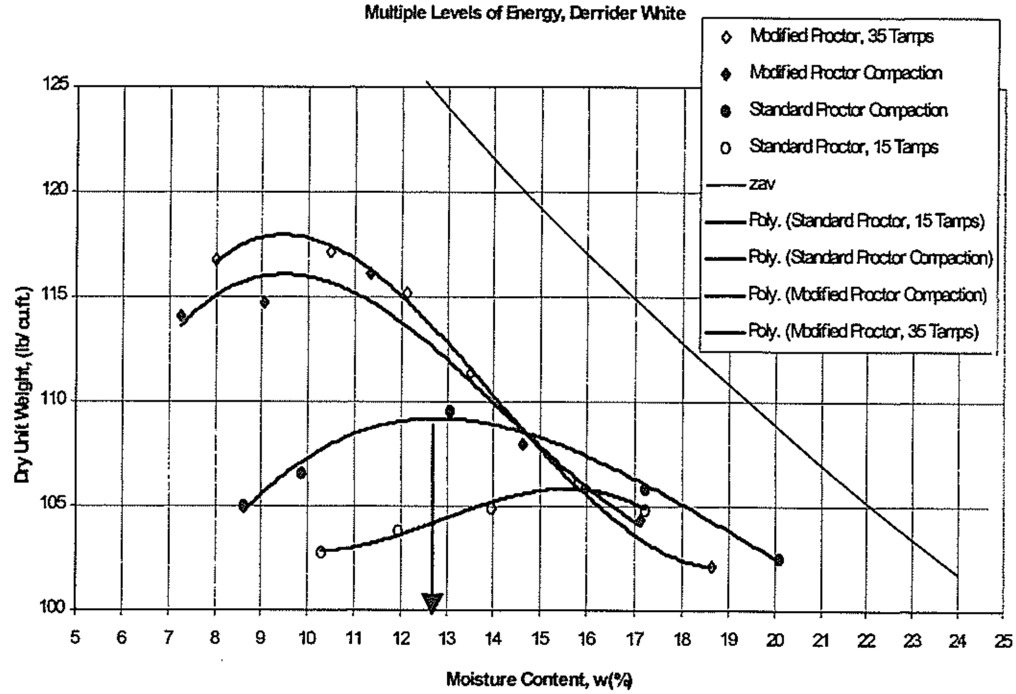


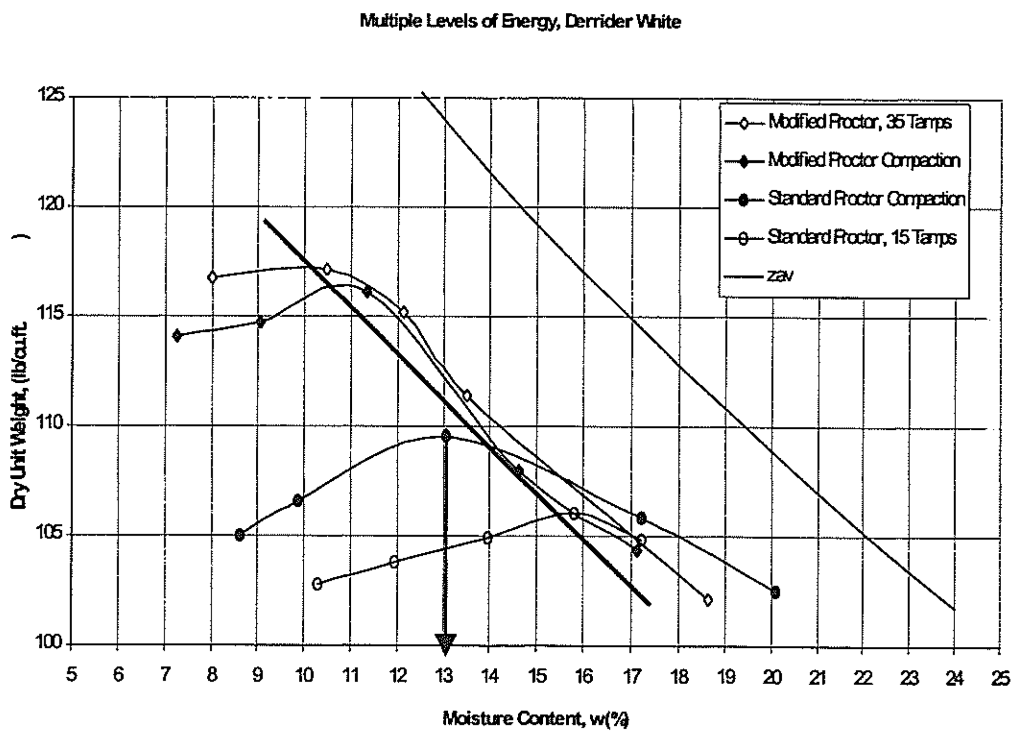


Multiple Levels of Energy, DERRIDER BROWN
Modified Proctor, 35 Tamps
Modified Proctor Compaction
Standard Proctor Compaction
Standard Proctor, 15 Tamps
Dry Unit Weight, (lb/ cu.ft.)
Moisture Content, w (%)

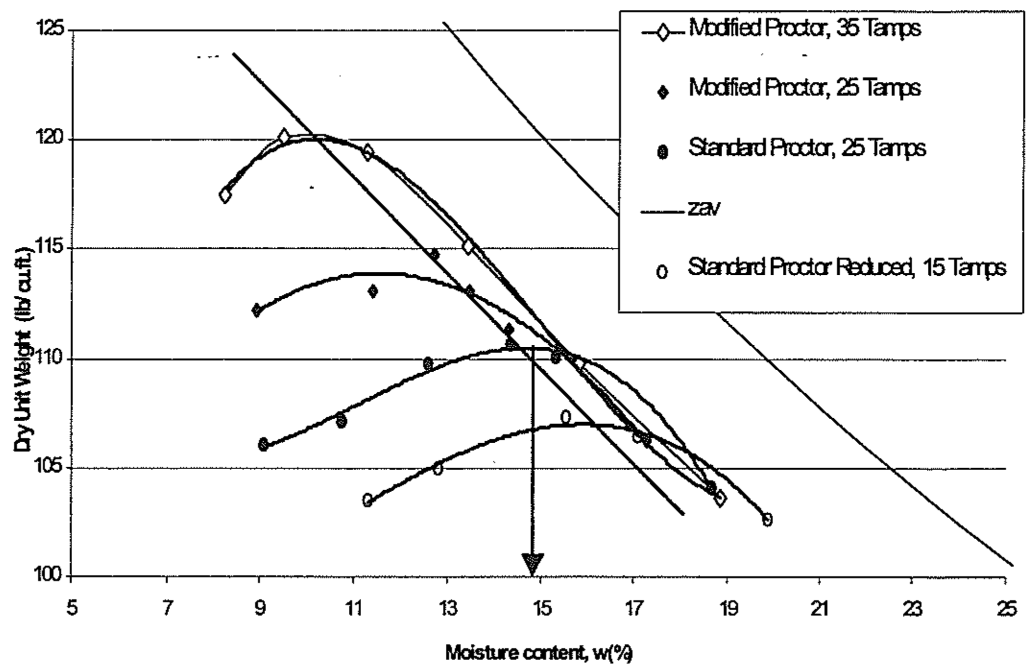


Multiple Levels of Energy, DERRIDER BROWN
Modified Proctor, 35 Tamps
Modified Proctor Compaction
Standard Proctor Compaction
Standard Proctor, 15 Tamps
zav
Dry Unit Weight, (lb/ cu.ft.)
Moisture Content, w(%)

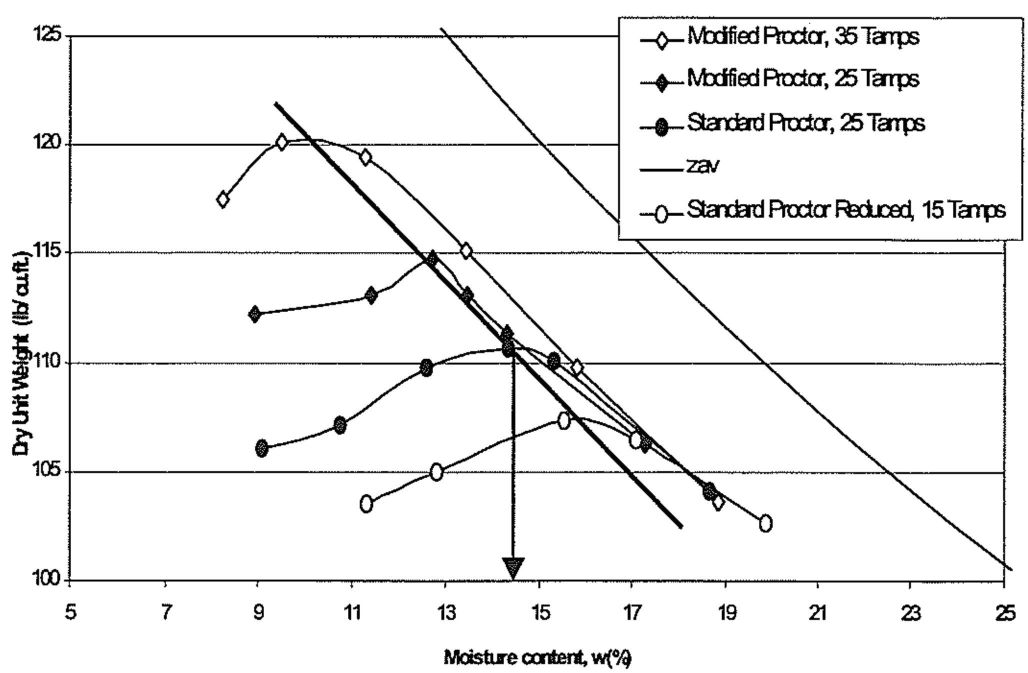

Multiple Levels of Energy, Derrider White
Modified Proctor, 35 Tamps
Modified Proctor Compaction
Standard Proctor Compaction
Standard Proctor, 15 Tamps
zav
Poly. (Standard Proctor, 15 Tamps)
Poly. (Standard Proctor Compaction)
Poly. (Modified Proctor Compaction)
Poly. (Modified Proctor, 35 Tamps)
Dry Unit Weight, (lb/ cu.ft.)
Moisture Content, w(%)


Multiple Levels of Energy, Derrider White
Modified Proctor, 35 Tamps
Modified Proctor Compaction
Standard Proctor Compaction
Standard Proctor, 15 Tamps
zav
Dry Unit Weight, (lb/cu.ft.
Moisture Content, w(%)

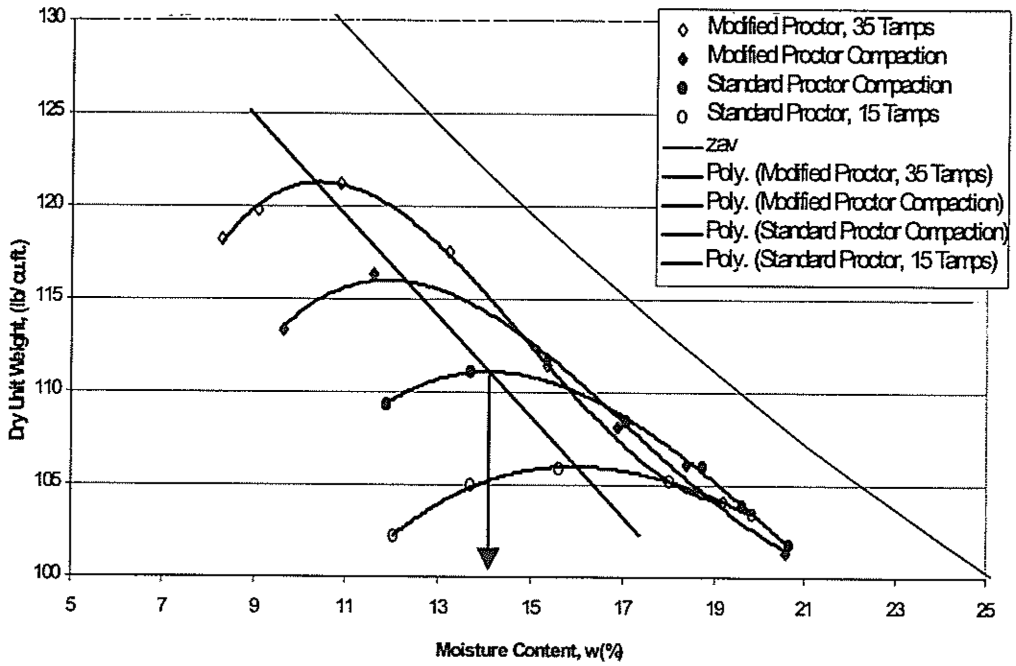
Multiple Levels of Energy, K1-1



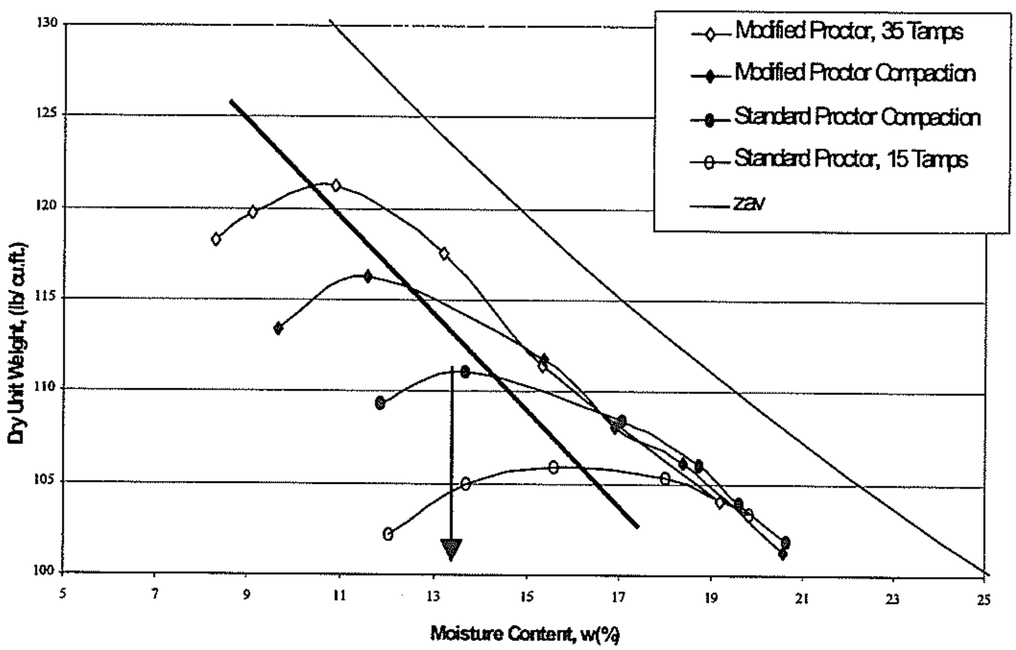
Multiple Levels of Energy, K1-1

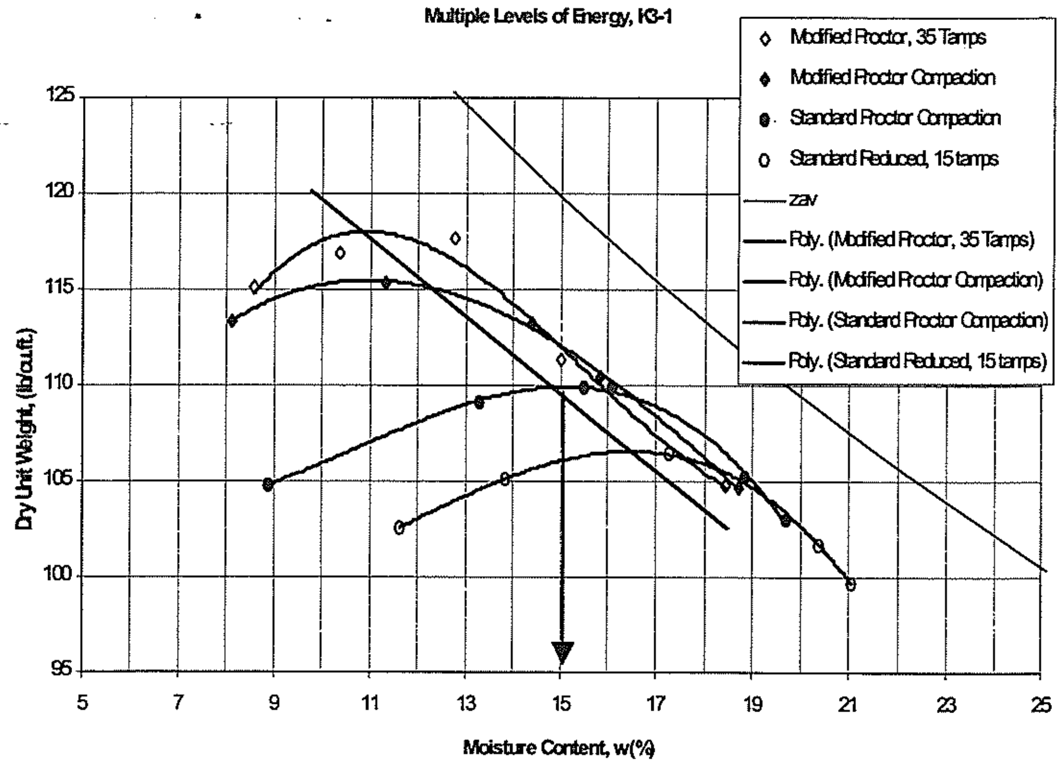


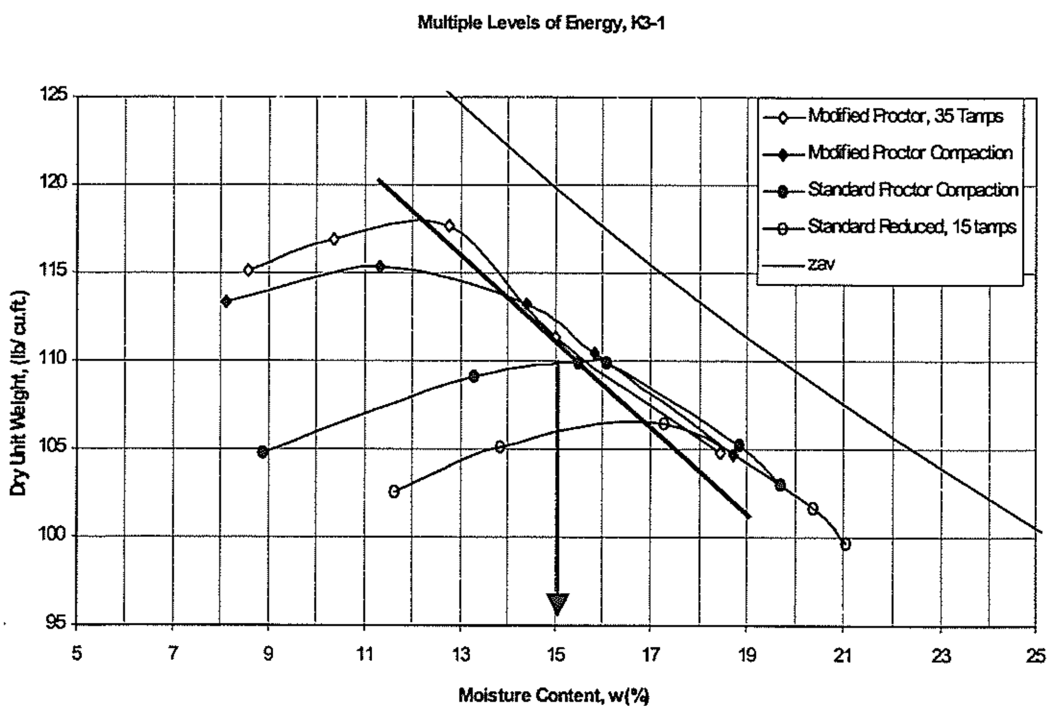
Multiple Levels of Energy, K2-1



Multiple Levels of Energy, K2-1




Multiple Levels of Energy, K3-1
Modified Proctor, 35 Tamps
Modified Proctor Compaction
Standard Proctor Compaction
Standard Reduced, 15 tamps
zav
Poly. (Modified Proctor, 35 Tamps)
Poly. (Modified Proctor Compaction)
Poly. (Standard Proctor Compaction)
Poly. (Standard Reduced, 15 tamps)
Dry Unit Weight, (lb/cu.ft.)
Moisture Content, w(%)


Multiple Levels of Energy, K3-1
Modified Proctor, 35 Tamps
Modified Proctor Compaction
Standard Proctor Compaction
Standard Reduced, 15 tamps
zav
Dry Unit Weight, (lb/ cu.ft.)
Moisture Content, w(%)

Appendix C
Strength-Moisture-Density Natural Soils

ACADIA

Number of Tamps	Moisture content	Moist Unit Weight	Dry Unit Weight	Maximum Stress
-	%	lb/ft^3	lb/ft^3	lb/in^2
15	9.55	116.56	106.4	47.87
15	11.76	124.35	111.27	44.68
15	15.42	127.61	110.56	24.61
15	16.52	126.48	108.55	17.05
30	9.42	118.17	108	50.78
30	11.9	126.13	112.72	47.68
30	15.31	128.08	111.07	25.57
30	16.46	126.79	108.87	16.32
60	9.61	121.17	110.55	55.13
60	11.85	128.25	114.66	47.69
60	15.27	128.05	111.09	18.67
60	16.58	125.9	107.99	16.02

Regression Statistics	
Multiple R	0.9845
R Square	0.9692
Adjusted R Squ	0.9623
Standard Error	0.0427
Observations	12

Log s = -0.5287 - 7.17632w + 0.022656d

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	0.5155	0.2577	141.4909	1.58E-07
Residual	9	0.0164	0.0018		
Total	11	0.5319			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95%*	*Upper 95%*
Intercept	-0.0529	0.6237	-0.0848	0.934293	-1.4637	1.358	-1.4637	1.358
X Variable 1	-7.1763	0.4436	-16.1782	5.84E-08	-8.1798	-6.1729	-8.1798	-6.1729
X Variable 2	0.0227	0.0056	4.0335	0.002957	0.01	0.0354	0.01	0.0354

CHASE BROWN

Number of Tamps	Moisture content	Moist Unit Weight	Dry Unit Weight	Maximum Stress
-	%	lb/ft^3	lb/ft^3	lb/in^2
15	13.1	118.72	104.97	60.21
15	16.6	124.82	107.05	48.94
15	18.07	125.8	106.55	33.74
15	19.35	126.48	105.97	18
30	13.54	121.67	107.16	68.75
30	16.45	127.89	109.82	53.31
30	17.43	125.97	107.27	43.16
30	19.2	125.8	105.54	16.8
60	13.58	122.31	107.69	67.11
60	16.27	129.82	111.65	47.89
60	17.82	128.99	109.48	35.91
60	18.95	125.9	105.84	17.04

Regression Statistics	
Multiple R	0.9409
R Square	0.8854
Adjusted R Squ	0.8599
Standard Error	0.0852
Observations	12

Log s = -0.71599 - 8.77362w + 0.035021d

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	0.5044	0.2522	34.7557	5.85E-05
Residual	9	0.0653	0.0073		
Total	11	0.5697			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95%*	*Upper 95%*
Intercept	-0.716	1.4311	-0.5003	0.6289	-3.9534	2.5214	-3.9534	2.5214
X Variable 1	-8.7736	1.1546	-7.5989	0	-11.3855	-6.1618	-11.3855	-6.1618
X Variable 2	0.035	0.013	2.6879	0.0249	0.0055	0.0645	0.0055	0.0645

CHASE WHITE

Number of Tamps	Moisture content	Moist Unit Weight	Dry Unit Weight	Maximum Stress
-	%	lb/ft^3	lb/ft^3	lb/in^2
15	10.02	114.21	103.81	32.69
15	13.05	123.66	109.39	40.19
15	15.59	125.01	108.15	23.23
15	17.06	124.08	106	16.48
30	10.08	117.1	106.38	44.97
30	12.85	126.05	111.7	46.7
30	15.72	125.53	108.48	21.78
30	17.05	124.03	105.96	17.25
60	10.58	121.29	109.69	47.89
60	12.88	128.17	113.55	52.41
60	15.57	124.57	107.79	20.33
60	17.03	124	105.96	17.99

Regression Statistics	
Multiple R	0.9867
R Square	0.9735
Adjusted R Squ	0.9676
Standard Error	0.0351
Observations	12

Log s = -1.54068 - 5.60527w +0.035037d

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	0.407	0.2035	165.1824	8.06E-08
Residual	9	0.0111	0.0012		
Total	11	0.418			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95%*	*Upper 95%*
Intercept	-1.5407	0.4354	-3.5386	0.006328	-2.5256	-0.5558	-2.5256	-0.5558
X Variable 1	-5.6053	0.3916	-14.3132	1.69E-07	-6.4912	-4.7194	-6.4912	-4.7194
X Variable 2	0.035	0.0039	8.9359	9.05E-06	0.0262	0.0439	0.0262	0.0439

DERIDDER BROWN

Number of Tamps	Moisture content	Moist Unit Weight	Dry Unit Weight	Maximum Stress
-	%	lb/ft^3	lb/ft^3	lb/in^2
15	10.1	119.85	108.86	72.52
15	12.39	128.16	114.03	59.16
15	14.24	130.2	113.97	46.61
15	15.42	130.01	112.64	28.31
30	10.72	125.03	112.92	87.65
30	12.16	129.85	115.77	55.98
30	13.96	128.08	112.39	52.89
30	15.2	125.8	109.2	28.9
60	10.43	125.36	113.52	97.74
60	12.79	130.4	115.61	67.89
60	13.84	132.05	116	64.98
60	15.59	130.05	112.51	28.14

Regression Statistics	
Multiple R	0.9471
R Square	0.897
Adjusted R Squ	0.8741
Standard Error	0.0661
Observations	12

Log s = -0.142785 - 8.58607w +0.23917d

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	0.3421	0.171	39.1908	3.61E-05
Residual	9	0.0393	0.0044		
Total	11	0.3814			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95%*	*Upper 95%*
Intercept	0.14279	0.99122	0.14405	0.88864	-2.0995	2.3851	-2.0995	2.3851
X Variable 1	-8.58607	1.02128	-8.40718	0.00001	-10.8964	-6.2758	-10.8964	-6.2758
X Variable 2	0.02392	0.00868	2.75583	0.02226	0.0043	0.0435	0.0043	0.0435

DERIDDER WHITE

Number of Tamps	Moisture content	Moist Unit Weight	Dry Unit Weight	Maximum Stress
-	%	lb/ft^3	lb/ft^3	lb/in^2
15	8.21	117.06	108.18	34.45
15	10.24	120.35	109.17	27.21
15	12.56	126.07	112	25.74
15	15.1	124.93	108.54	17.13
30	8	118.66	109.87	39.94
30	10.28	121.74	110.39	29.16
30	12.49	127.34	113.2	31.2
30	14.99	125.76	109.37	15.92
60	7.81	119.78	111.1	49.06
60	10.18	124.25	112.77	39.11
60	12.19	128.31	114.37	37.29
60	15.1	125.77	109.27	13.17

Regression Statistics	
Multiple R	0.9539
R Square	0.91
Adjusted R Squ	0.89
Standard Error	0.0565
Observations	12

Log s = -2.68278 - 4.98368w + 0.042345d

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	0.2907	0.1453	45.5067	1.97E-05
Residual	9	0.0287	0.0032		
Total	11	0.3194			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95%*	*Upper 95%*
Intercept	-2.6828	0.9505	-2.8226	0.019964	-4.8329	-0.5326	-4.8329	-0.5326
X Variable 1	-4.9837	0.6239	-7.988	2.24E-05	-6.395	-3.5723	-6.395	-3.5723
X Variable 2	0.0423	0.0085	4.9578	0.000783	0.023	0.0617	0.023	0.0617

Natchitoches K1-1

Number of Tamps	Moisture content	Moist Unit Weight	Dry Unit Weight	Maximum Stress
-	%	lb/ft^3	lb/ft^3	lb/in^2
15	9.77	120.25	109.55	25.48
15	12.2	124.35	110.83	19.28
15	14.03	126.72	111.13	18.72
15	14.9	126.48	110.08	15.59
30	10.3	123.07	111.58	27.8
30	12.35	127.3	113.31	23.76
30	13.8	128.08	112.55	20.88
30	14.9	125.8	109.49	16.24
50	9.97	124.8	113.49	31.59
50	11.9	128.94	115.23	31.41
50	13.8	127.99	112.47	19.15
50	14.91	125.9	109.56	14.979

Regression Statistics	
Multiple R	0.9804
R Square	0.9612
Adjusted R Squ	0.9526
Standard Error	0.0247
Observations	12

$\text{Log } s = -1.33254 - 4.20098w + 0{,}02865d$

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	0.136	0.068	111.451	4.47E-07
Residual	9	0.0055	0.0006		
Total	11	0.1414			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95%*	*Upper 95%*
Intercept	-1.3325	0.5	-2.6651	0.025829	-2.4636	-0.2015	-2.4636	-0.2015
X Variable 1	-4.201	0.4072	-10.3168	2.76E-06	-5.1221	-3.2798	-5.1221	-3.2798
X Variable 2	0.0287	0.0043	6.6613	9.25E-05	0.0189	0.0384	0.0189	0.0384

Natchitoches K2-1

Number of Tamps	Moisture content	Moist Unit Weight	Dry Unit Weight	Maximum Stress
-	%	lb/ft^3	lb/ft^3	lb/in^2
15	9.81	120.45	109.69	39.8
15	12.22	127.35	113.48	39.74
15	14.43	131.11	114.58	29.56
15	16.53	127.36	109.29	15.71
30	9.73	122.12	111.29	50.56
30	11.94	128.69	114.96	46.66
30	14.19	131.67	115.31	29.71
30	16.45	127.8	109.75	15.52
60	9.98	124.94	113.6	55.94
60	12.51	132.33	117.62	46.59
60	14.25	131.32	114.94	32.09
60	16.49	129.13	110.85	15.47

Natchitoches K2 Strength-Moisture-Density

Log s = -0.91247 - 6.70911w +0.029208d

Regression Statistics	
Multiple R	0.9942
R Square	0.9885
Adjusted R Squ	0.9859
Standard Error	0.0244
Observations	12

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	0.4601	0.2301	386.416	1.88E-09
Residual	9	0.0054	0.0006		
Total	11	0.4655			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95%*	*Upper 95%*
Intercept	-0.9125	0.3232	-2.8231	0.019947	-1.6436	-0.1813	-1.6436	-0.1813
X Variable 1	-6.7091	0.2886	-23.2453	2.40E-09	-7.362	-6.0562	-7.362	-6.0562
X Variable 2	0.0292	0.0028	10.517	2.35E-06	0.0229	0.0355	0.0229	0.0355

Natchitoches K3-1

Number of Tamps	Moisture content	Moist Unit Weight	Dry Unit Weight	Maximum Stress
-	%	lb/ft^3	lb/ft^3	lb/in^2
7	11	120.03	108.14	32.16
7	13.14	126.45	111.76	28.74
7	15.2	127.44	110.63	19.62
7	17.11	124.64	106.43	10.36
15	9.65	118.38	107.96	41.84
15	13.2	128.87	113.84	34.14
15	14.6	127.54	111.29	19.92
15	16.07	126.48	108.97	13.22
30	9.5	121.89	111.32	54.39
30	13.1	129.82	114.78	35.53
30	14.8	127.93	111.44	17.28
30	16.3	125.75	108.13	10.82
60	9.47	123.07	112.42	55.4
60	12.9	130.52	115.61	37.52
60	14.8	127.06	110.68	19.24
60	16.3	126	108.34	10.41

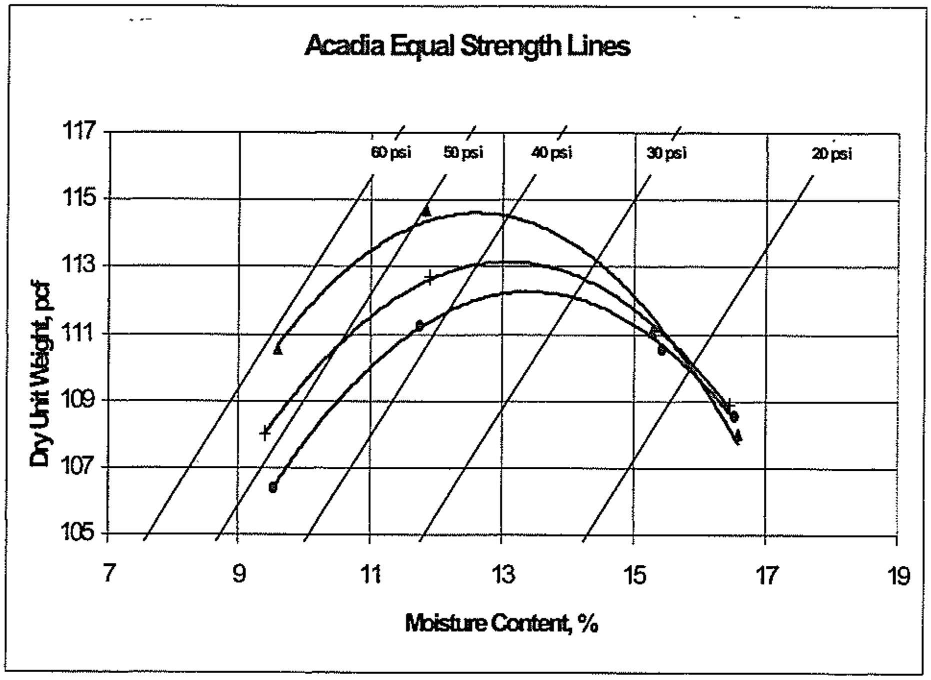
Regression Statistics	
Multiple R	0.9904
R Square	0.981
Adjusted R Squ	0.9781
Standard Error	0.0368
Observations	16

Log s = -0.88652 - 8.23344w + 0.030529d

ANOVA

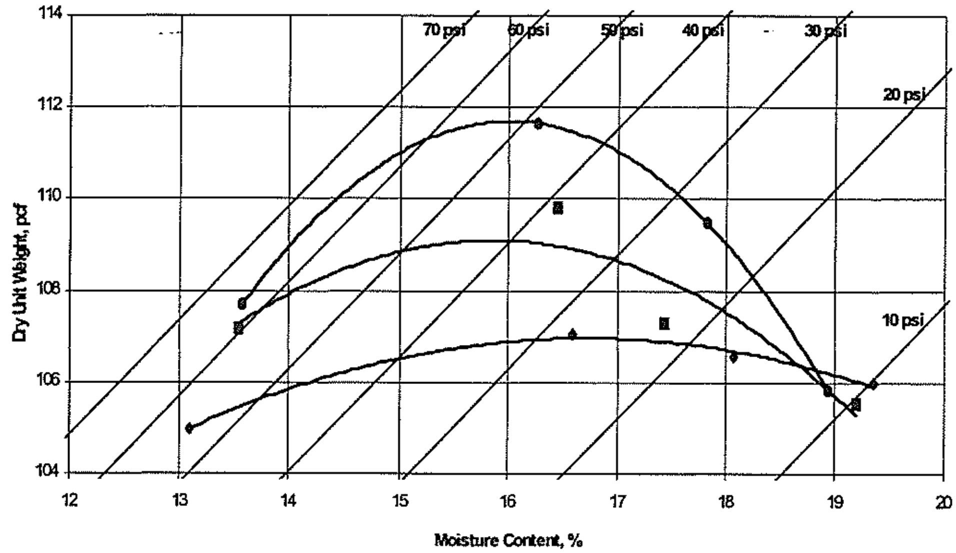
	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	0.9064	0.4532	335.2038	6.53E-12
Residual	13	0.0176	0.0014		
Total	15	0.9239			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95%*	*Upper 95%*
Intercept	-0.8865	0.4397	-2.0161	0.064935	-1.8365	0.0634	-1.8365	0.0634
X Variable 1	-8.2334	0.3926	-20.9713	2.09E-11	-9.0816	-7.3853	-9.0816	-7.3853
X Variable 2	0.0305	0.0038	8.0417	2.11E-06	0.0223	0.0387	0.0223	0.0387

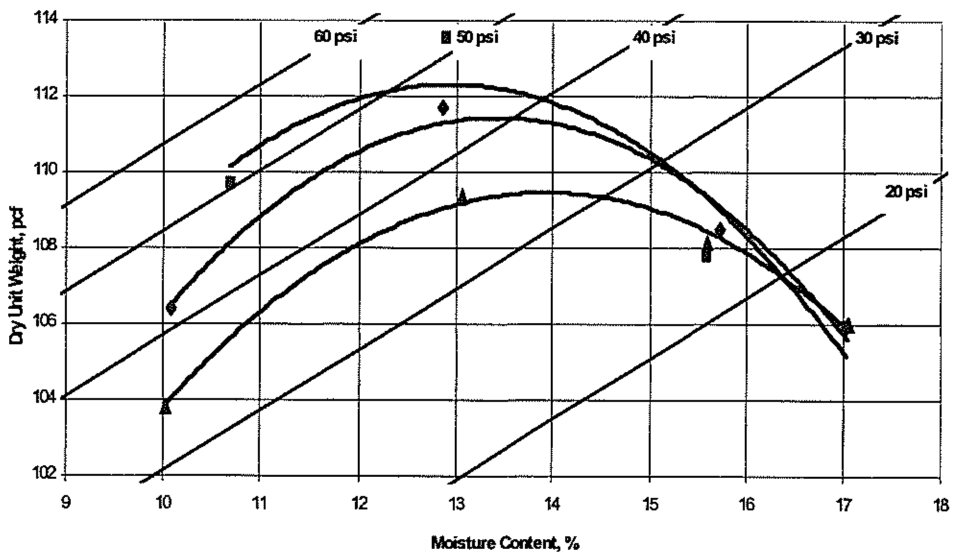


Acadia Equal Strength Lines
60 psi
50 psi
40 psi
30 psi
20 psi
Dry Unit Weight, pcf
117
115
113
111
109
107
105
7
9
11
13
15
17
19
Moisture Content, %

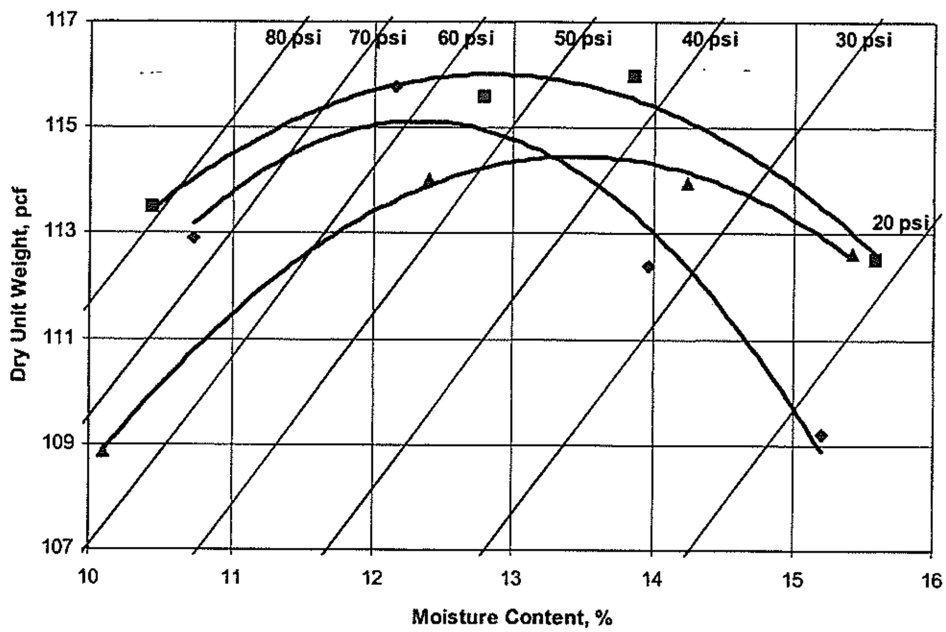
Chase Brown Equal Strength Lines



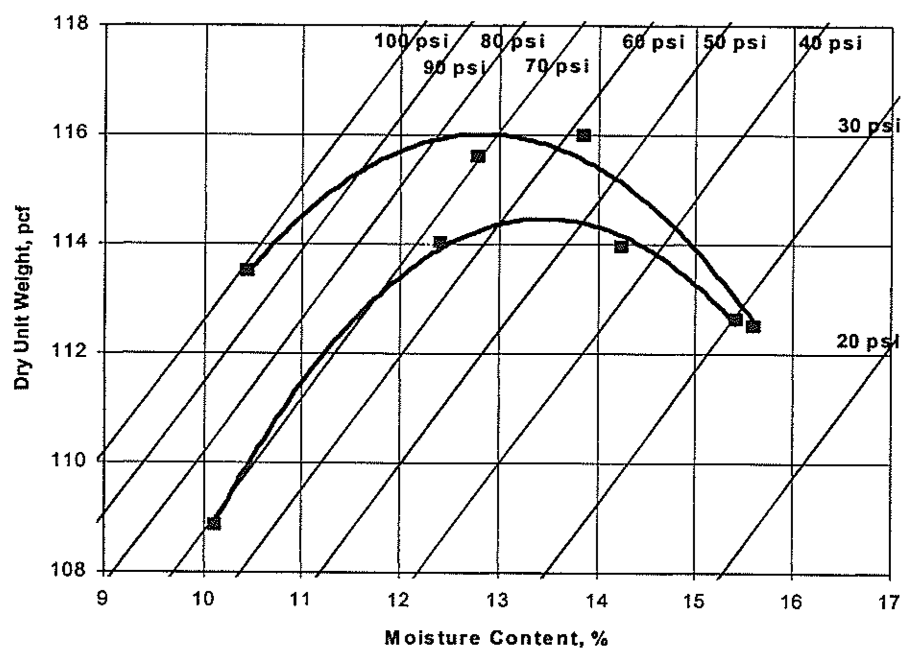
Chase White Equal Strength Lines



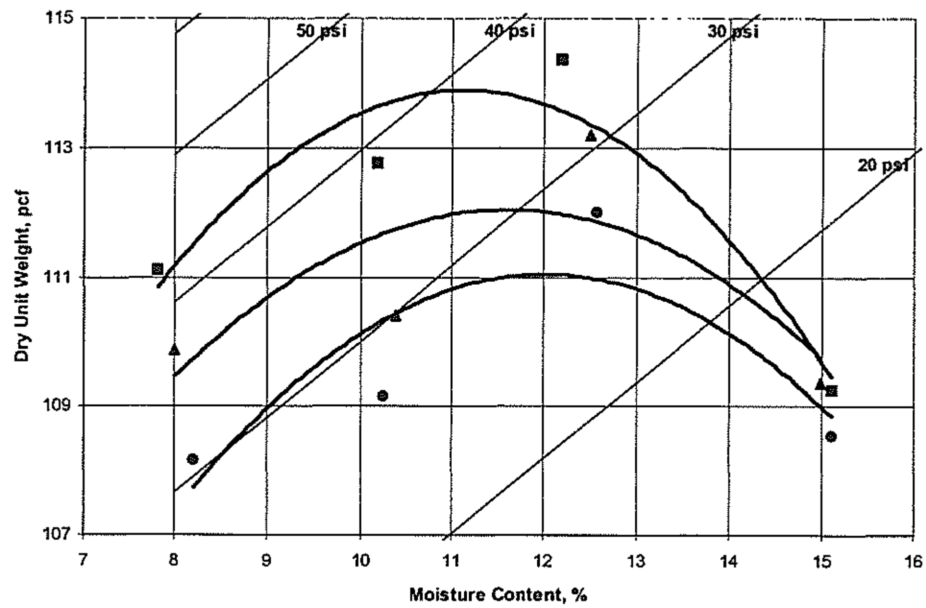
DeRidder Brown Equal Strength Lines



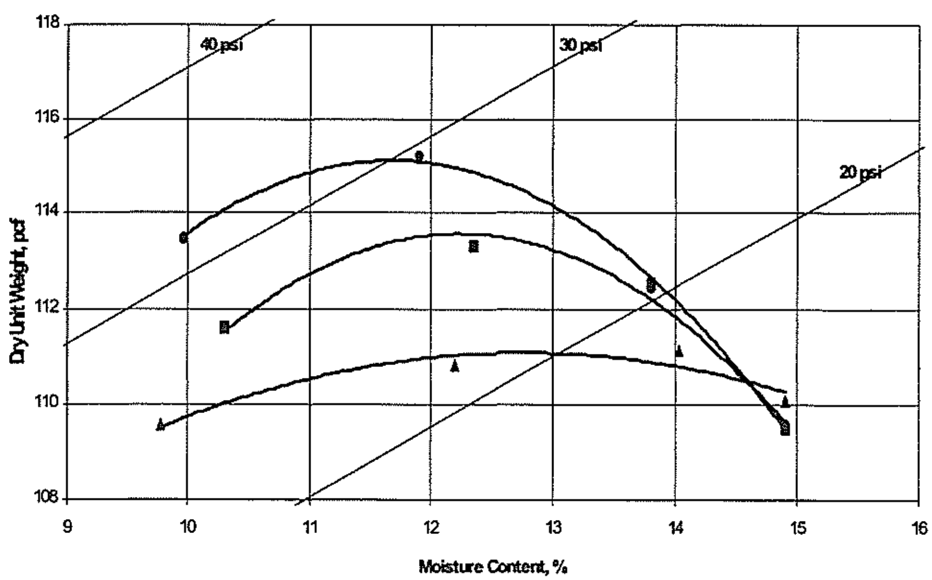
DeRidder Brown Equal Strength Lines (Mod. Data)



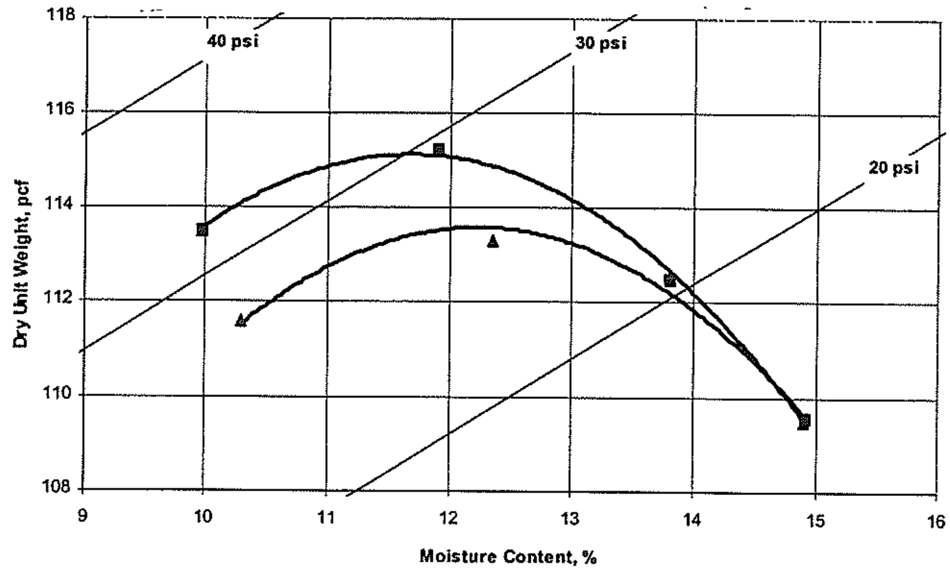
DeRidder White Equal Strength Lines



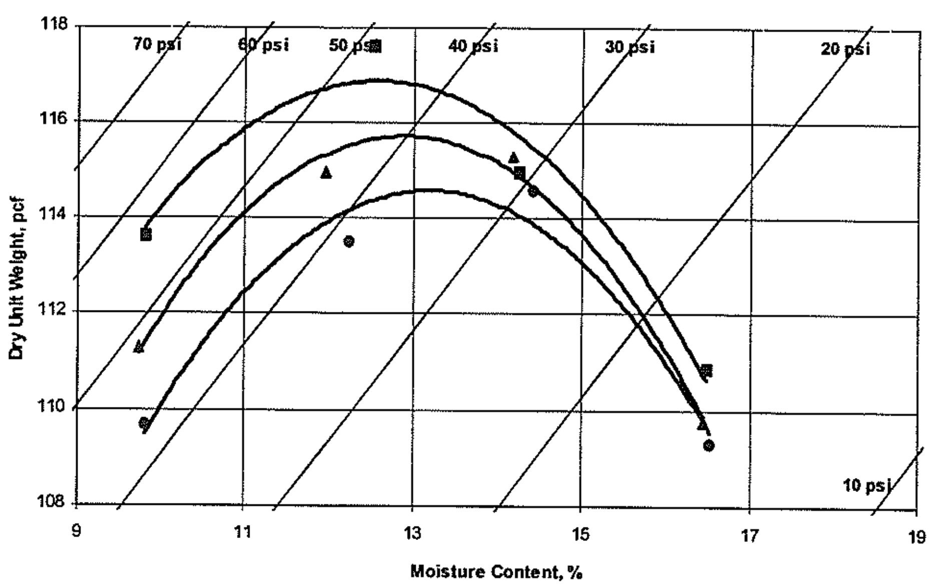
Natchitoches K1 Equal Strength Lines



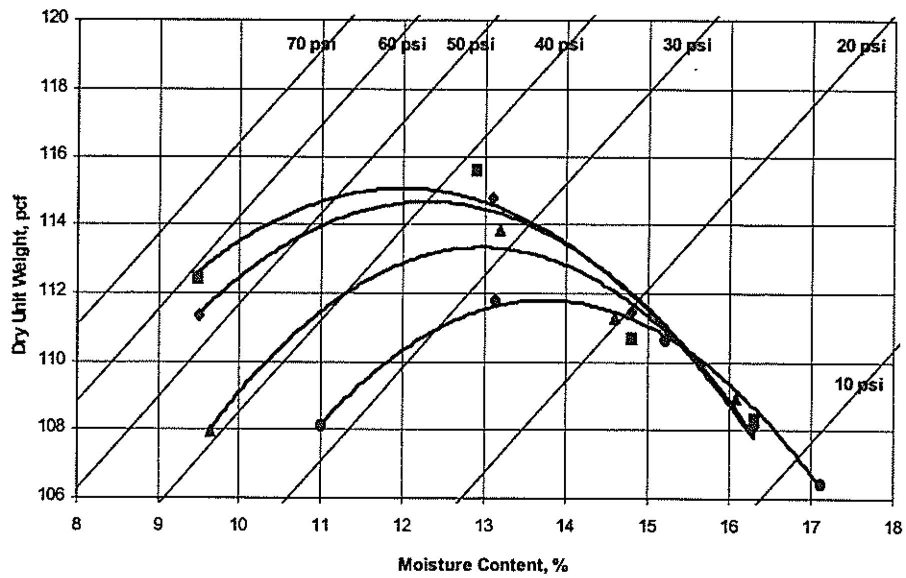
Natchitoches K1 Equal Strength Lines (Mod. Data)



Natchitoches K2 Equal Strength Lines

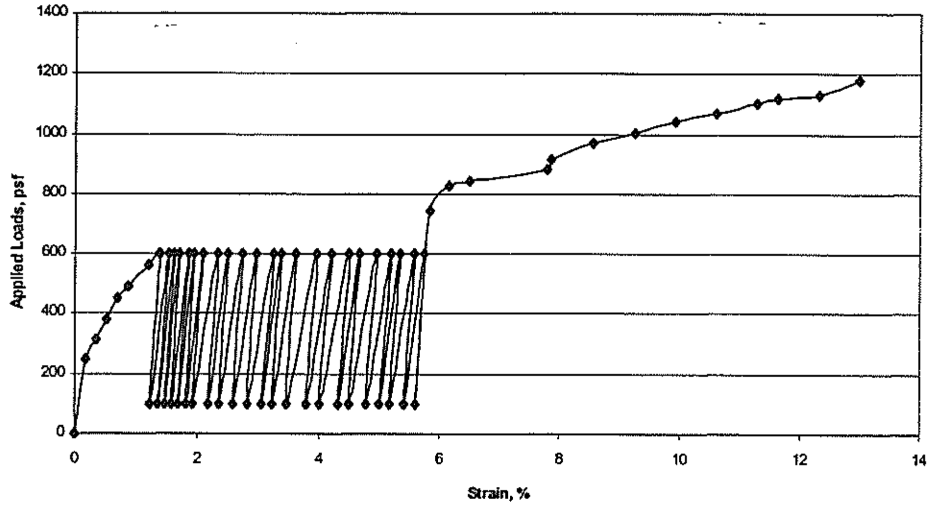


Natchitoches K3 Equal Strength Lines

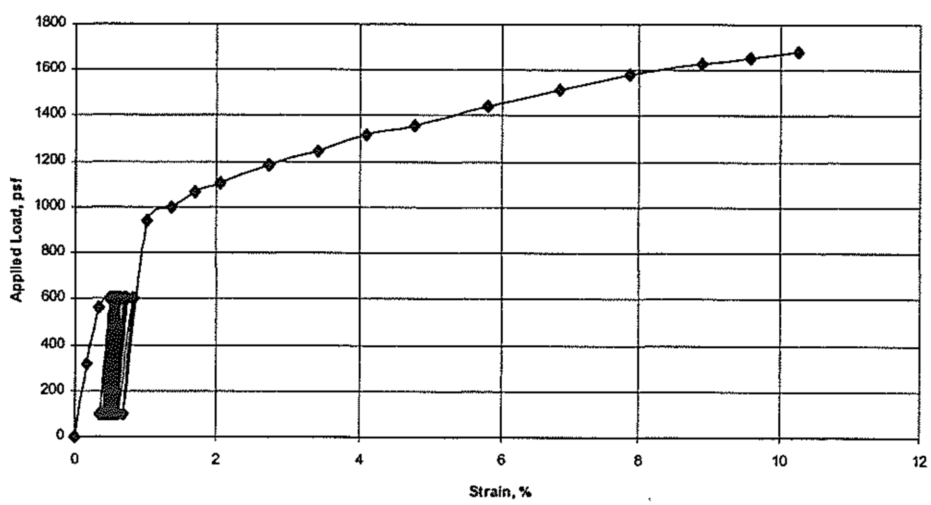


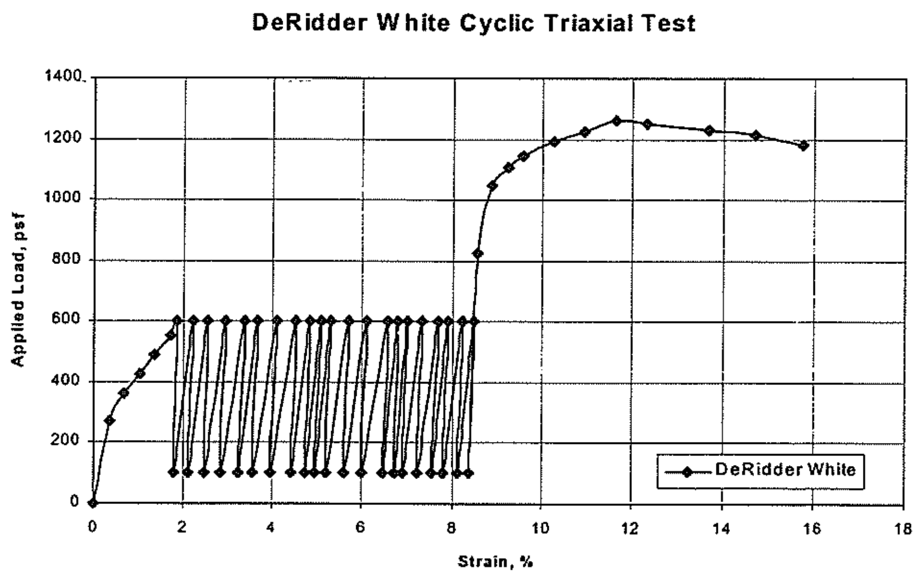
Appendix D
Cyclic Triaxial Tests Natural Soils

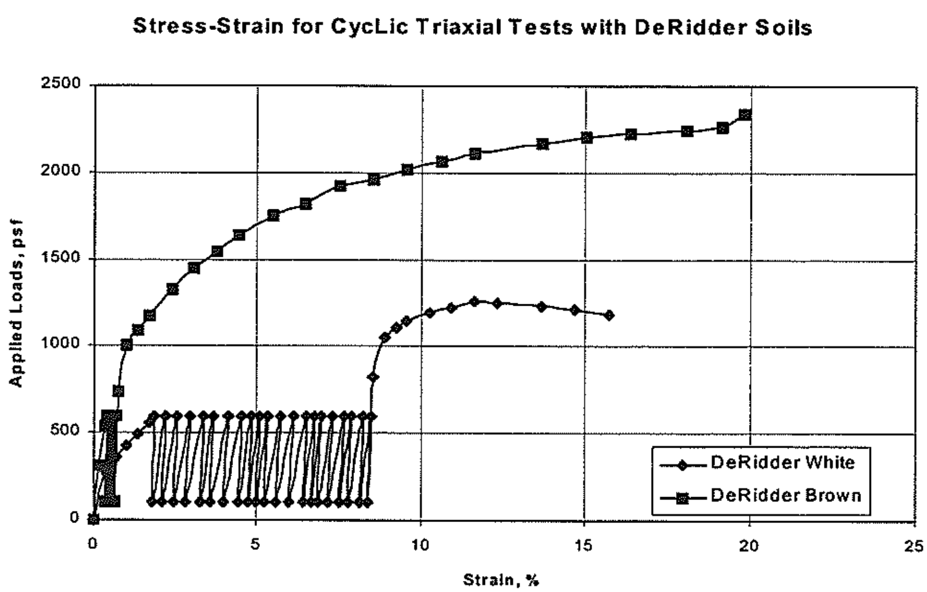
Chase White Cyclic Triaxial Stress Strain

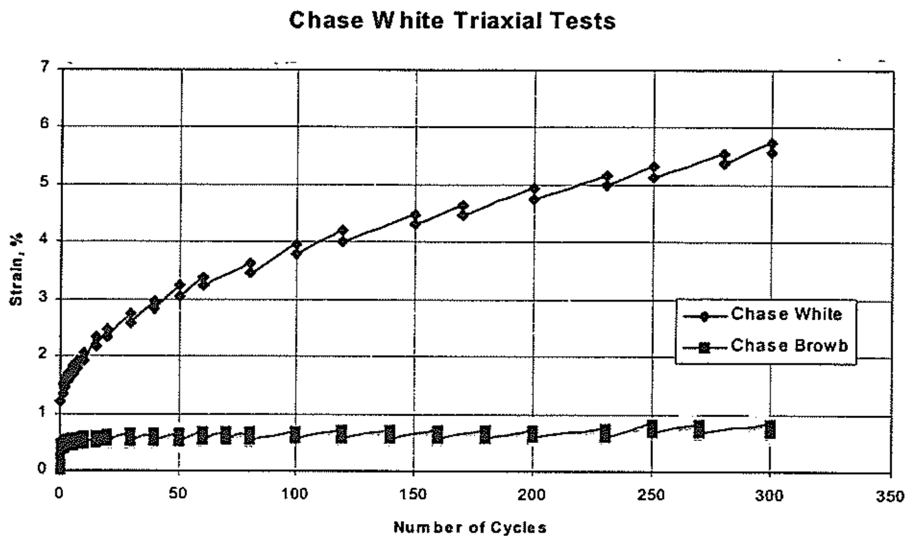


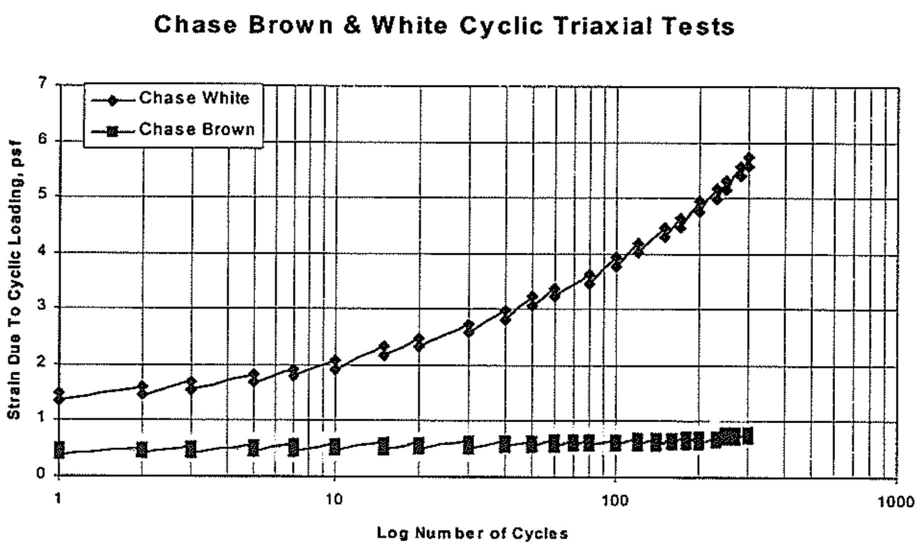
Chase Brown



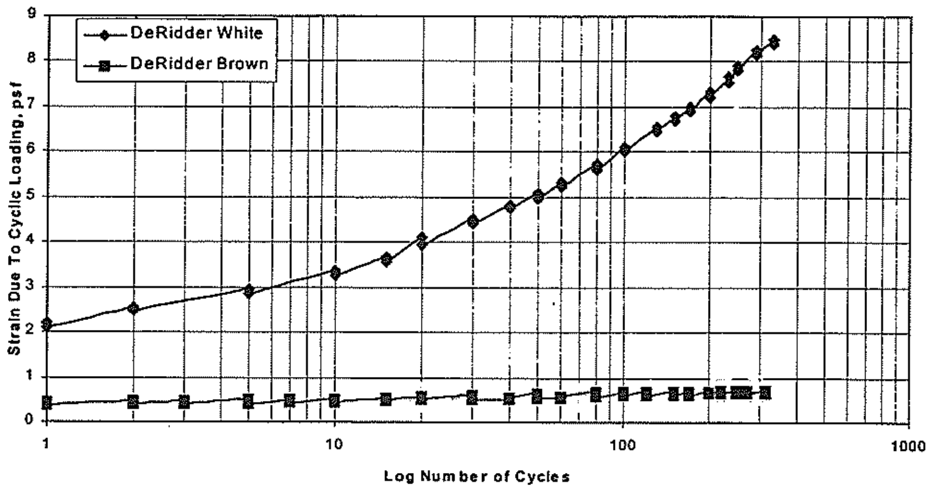

DeRidder White Cyclic Triaxial Test
1400
1200
1000
800
600
400
200
0
Applied Load, psf
0
2
4
6
8
10
12
14
16
18
Strain, %
DeRidder White


Stress-Strain for CycLic Triaxial Tests with DeRidder Soils
2500
2000
1500
1000
500
0
Applied Loads, psf
0
5
10
15
20
25
Strain, %
DeRidder White
DeRidder Brown

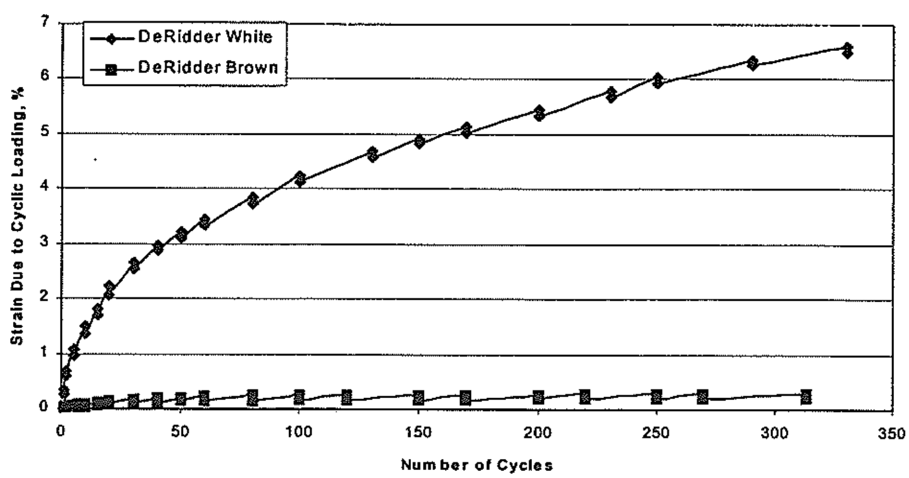

Chase White Triaxial Tests
Strain, %
Number of Cycles
Chase White
Chase Browb
0
1
2
3
4
5
6
7
0
50
100
150
200
250
300
350

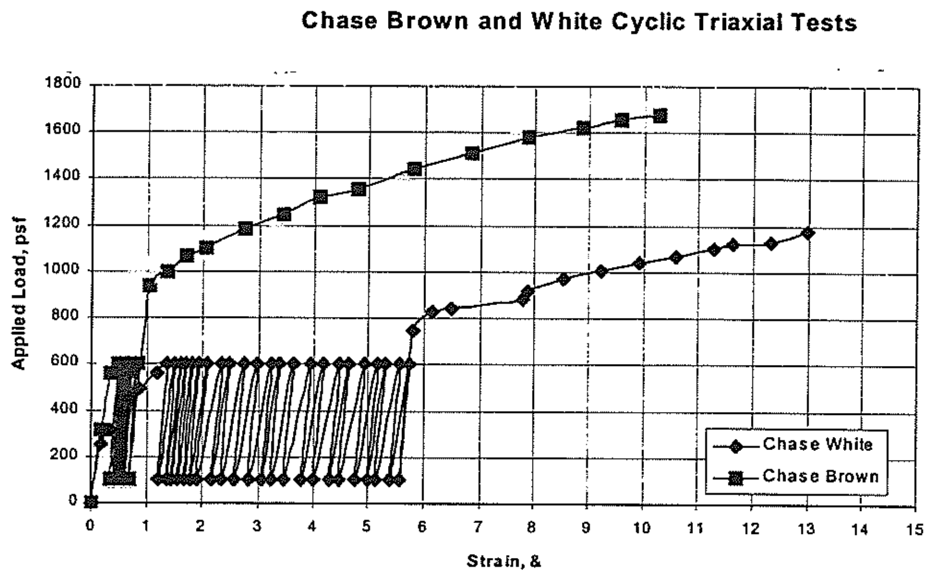

Chase Brown & White Cyclic Triaxial Tests
Strain Due To Cyclic Loading, psf
Log Number of Cycles
Chase White
Chase Brown
0
1
2
3
4
5
6
7
1
10
100
1000

DeRidder White/Brown Cyclic Triaxial Tests

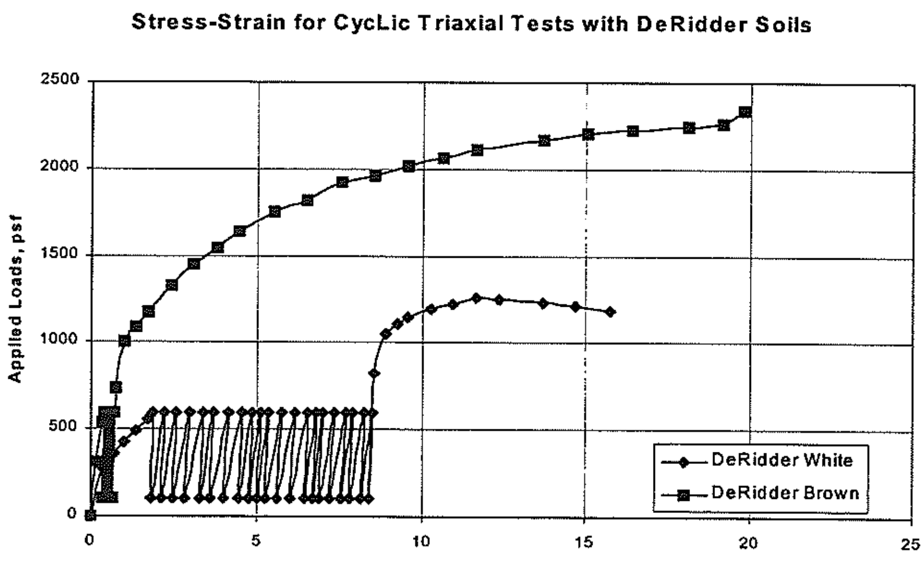


Cyclic Triaxial Tests for DeRidder Soil Samples

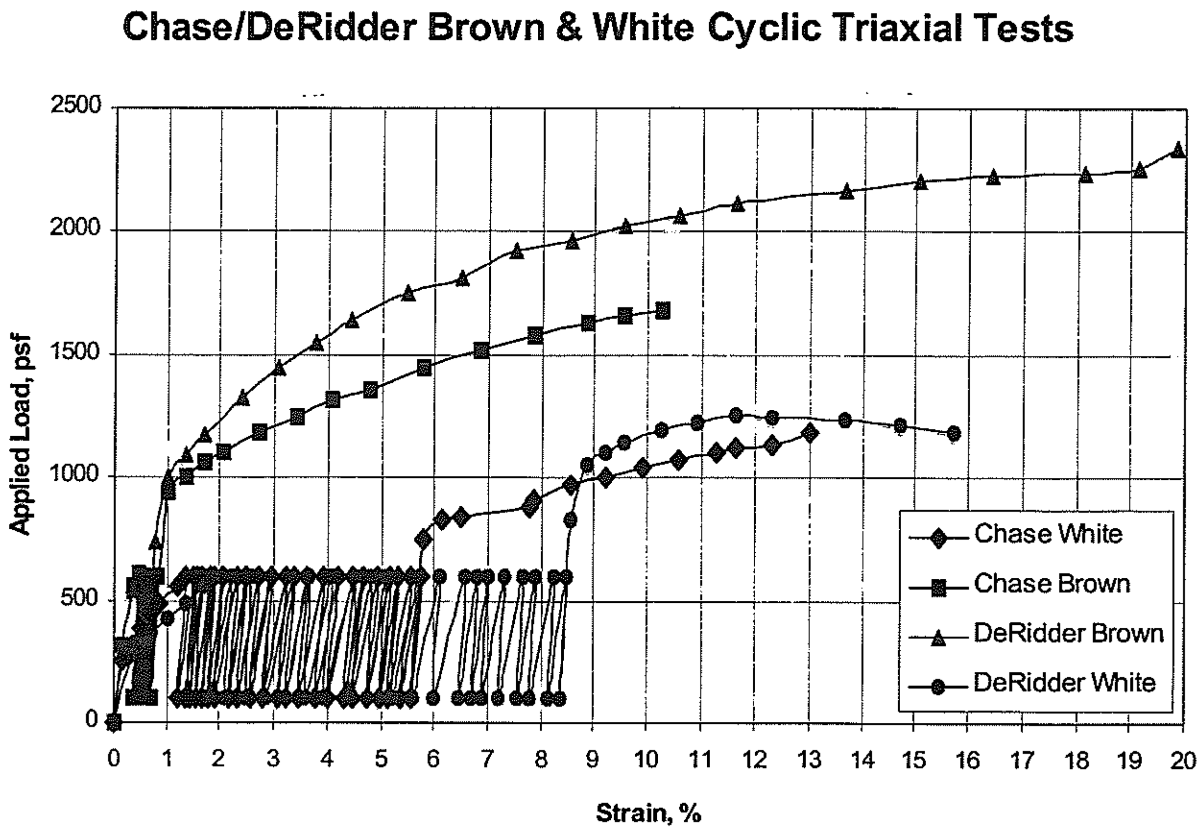




Chase Brown and White Cyclic Triaxial Tests
Applied Load, psf
1800
1600
1400
1200
1000
800
600
400
200
0
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
Strain, &
Chase White
Chase Brown



Stress-Strain for CycLic Triaxial Tests with DeRidder Soils
Applied Loads, psf
2500
2000
1500
1000
500
0
0
5
10
15
20
25
Strain, %
DeRidder White
DeRidder Brown



Chase/DeRidder Brown & White Cyclic Triaxial Tests
Applied Load, psf
Strain, %
Chase White
Chase Brown
DeRidder Brown
DeRidder White
0
500
1000
1500
2000
2500
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20

Appendix E
Stabilization Test Series 1

STABILIZATION TEST SERIES 1

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	111.3	44.68
	15.42	127.61	110.6	24.61
	16.52	126.48	108.5	17.05
ACADIA + Lime4% DIRECT	10.07	110.42	100.3	52.28
	13.18	118.32	104.5	45.8
	14.21	120.37	105.4	42
	17.03	122.12	104.3	16.57
RAPID CURING	8.18	109.8	101.5	74.55
	11.68	116.61	104.4	75.77
	13.26	118.76	104.9	62
	15.8	121.07	104.6	48.11
VACUUM SATURATION	10.62	109.25	98.76	25.57
	13.84	116.2	102.1	27.82
	15.04	119.23	103.6	30.47
	17.7	121.58	103.3	28
HUMIDITY ROOM	10.62	110.94	100.3	58.33
	13.84	117.74	103.4	56.56
	15.04	120.41	104.7	49.61
	17.7	121.64	103.3	28.65
Acadia+ 4% Cement	17.12	127.37	108.8	16.31
Acadia + 4% Cement HR	17.1	127	108.5	78.77
Acadia +4% Cement VS	17.1	127.67	109	41.82

STABILIZATION TESTSERIES 1

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	111.3	44.68
	15.42	127.61	110.6	24.61
	16.52	126.48	108.5	17.05
ACADIA + Lime2%+FA 8% DIRECT	9.64	107.86	98.38	42.7
	12.92	114.51	101.4	47.87
	14.41	119.12	104.1	45.92
	17.92	122.23	103.7	12.42
RAPID CURING	8.57	107.28	98.81	64.42
	11.86	112.7	100.8	63.2
	13.5	117.38	103.4	62.78
	16.75	121.49	104.1	40.17
VACUUM SATURATION	10.41	106.47	96.43	24.74
	13.68	111.88	98.42	22.34
	15.02	117.14	101.8	29
	18.88	120.82	101.6	22.22
HUMIDITY ROOM	10.41	107.48	97.35	67.3
	13.68	114.11	100.4	64.32
	15.02	118.32	102.9	62
	18.88	121.75	102.4	30.3
Acadia+ 4% Cement	17.12	127.37	108.8	16.31
Acadia + 4% Cement HR	17.1	127	108.5	78.77
Acadia +4% Cement VS	17.1	127.67	109	41.82

PHASE 1

Soil	Moisture	γ_{moist}	γ_{dry}	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
Chase Brown	13.1	118.72	104.97	60.21
	16.6	124.82	107.05	48.94
	18.07	125.80	106.55	33.74
	19.35	126.48	105.97	18
w/4% LIME DIRECT	14.55	111.58	97.41	53.2
	17.88	117.41	99.6	48.92
	21.77	119.75	98.34	27.94
	23.32	120.02	97.32	26.82
RAPID CURING	13	110.35	97.65	859.95
	15.65	116.76	101	938.38
	20.34	118.3	98.3	611.50
	22.12	118.45	96.99	563.4
VACUUM SATURATION	14.52	109.68	95.77	66.34
	17.8	115.9	98.39	62.64
	20.8	119.85	99.21	57
	22	119.58	98.02	48.88
HUMIDITY ROOM	15	111.22	96.71	65.65
	18.2	117.53	99.43	73.06
	21.04	120.19	99.3	41.38
	22	120.01	98.37	41

Soil	Moisture	γ_{moist}	γ_{dry}	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
CHASE BROWN	13.1	118.72	105	60.21
	16.6	124.82	107	48.94
	18.07	125.8	106.5	33.74
	19.35	126.48	106	18
CHASE BROWN + Lime2%+Fly Ash 8% DIRECT	13.38	111.21	98.09	71.43
	15.72	116.17	100.4	75.09
	18.26	120.26	101.7	52.65
	20.14	121.65	101.3	37.09
RAPID CURING	12.58	110.35	98.02	116.86
	15.06	114.1	99.17	102.15
	17.41	119.24	101.6	95.68
	19.42	120.94	101.3	69.61
	20.5	120.79	100.2	65
VACUUM SATURATION	13.3	110.41	97.45	41.05
	17.43	113.92	97.01	36.92
	19.36	119.17	99.84	41.18
	21.5	120.84	99.46	27.83
HUMIDITY ROOM	13.3	111.35	98.28	76.05
	17.43	115.98	98.77	85.55
	19.36	118.98	99.68	68.14
	21.5	121.09	99.66	52.35

Soil	Moisture	γ_{moist}	γ_{dry}	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
CHASE WHITE	10.023	114.21	103.8	32.69
	13.045	123.66	109.4	40.19
	15.586	125.01	108.2	23.23
	17.065	124.08	106	16.48
CHASE WHITE + Lime 4% DIRECT	13.83	116.95	102.7	39.8
	17.82	119.96	101.8	13.87
	19.21	119.61	100.3	9.21
	22.32	120.68	98.66	9.53
RAPID CURING	13.41	116.02	102.3	88.62
	17.5	118.79	101.1	51
	19	117.78	98.97	39
	21.3	120.59	99.41	17.22
VACUUM SATURATION	14.6	115.45	100.7	34.31
	18.7	119.06	100.3	27.71
	19.53	118.93	99.5	21.82
	21.73	120.07	98.64	NA
HUMIDITY ROOM	14.6	116.83	101.9	53.3
	18.7	120.42	101.4	30
	19.53	121.06	101.3	19.73
	21.73	121.61	99.9	18.6
Ch. White +4% Cement	20.4	120.87	100.4	8.85
Ch. White +4% Cement HR	20.22	121.26	100.9	37.87
Ch. White +4% Cement VS	20.22	122.13	101.6	23.16

Soil	Moisture	γ_{moist}	γ_{dry}	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
CHASE WHITE	10.023	114.21	103.8	32.69
	13.045	123.66	109.4	40.19
	15.586	125.01	108.2	23.23
	17.065	124.08	106	16.48
CHASE WHITE + L 2% +FA 8% DIRECT	13.26	111.62	98.55	64.6
	17.35	121.5	103.5	59
	18.16	120.68	102.1	27.82
	21.75	118.35	97.21	7.5
RAPID CURING	11.57	111.28	99.74	70.61
	15.41	119.85	103.8	80.67
	17.77	120.11	102	53.26
	21.5	116.46	95.85	14.42
VACUUM SATURATION	13.4	109.55	96.6	26.82
	17.68	119.14	101.2	36.38
	19.2	120.09	100.7	12.35
	21.75	119.13	97.85	6.34
HUMIDITY ROOM	13.4	111.23	98.09	65.14
	17.68	120.24	102.2	55.21
	19.2	121.36	101.8	51.44
	21.75	118.54	97.36	16.7
Ch. White +4% Cement	20.4	120.87	100.4	8.85
Ch. White +4% Cement HR	20.22	121.26	100.9	37.87
Ch. White +4% Cement VS	22.7	122.13	99.54	23.16

Soil	Moisture	γ_{moist}	γ_{dry}	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
DERIDDER BROWN	10.1	119.85	108.9	72.52
	12.39	128.16	114	59.16
	14.24	130.20	114	46.61
	15.42	130.01	112.6	28.31
DERIDDER BROWN+ Lime 4% DIRECT	12.56	112.15	99.64	62.53
	14.98	116.62	101.4	59.78
	17.52	118.85	101.1	42.36
	19	118	99.16	39.45
RAPID CURING	12	111.95	99.96	67.8
	14.12	115.95	101.6	65.32
	16.92	118	100.9	52.3
	18.75	117.69	99.11	43.6
VACUUM SATURATION	12.42	112	99.63	60.2
	14.85	115.65	100.7	52.35
	17.3	118.8	101.3	39.6
	18.8	117.35	98.78	27.4
HUMIDITY ROOM	12.42	112.06	99.68	75.6
	14.85	116.5	101.4	70.2
	17.3	118.65	101.2	58.2
	18.8	117.85	99.2	49.5

Soil	Moisture	γ_{moist}	γ_{dry}	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
DERIDDER BROWN	10.1	119.85	108.9	72.52
	12.39	128.16	114	59.16
	14.24	130.20	114	46.61
	15.42	130.01	112.6	28.31
DERIDDER BROWN+ Lime 2% + FA 8% DIRECT	12.48	116.8	103.8	68.2
	14.36	118.95	104	55.3
	16.05	119.75	103.2	49
	18.2	119	100.7	32.35
RAPID CURING	12.1	116.2	103.7	74.5
	13.92	118.5	104	68.7
	15.85	119	102.7	60.5
	17.8	118.2	100.3	42.5
VACUUM SATURATION	12.35	115.85	103.1	58.6
	14.3	117.95	103.2	42.5
	15.98	118.8	102.4	22.8
	18	118.25	100.2	18.4
HUMIDITY ROOM	12.35	116.5	103.7	75
	14.3	119.8	104.8	70.5
	15.98	120.3	103.7	62.5
	18	120.8	102.4	51.6

Soil	Moisture	γ_{moist}	γ_{dry}	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
DERIDDER WHITE	8.21	117.06	108.2	34.45
	10.24	120.35	109.2	27.21
	12.56	126.07	112	25.74
	15.1	124.93	108.5	17.13
DERIDDER White +Lime 4% 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
RAPID CURING	7.78	116.94	108.5	60.86
	11.08	123.36	111.1	51.87
	12.5	125.38	111.4	51.48
	15.2	123.2	106.9	22.58
VACUUM SATURATION	9.66	115.36	105.2	27.55
	13	122	108	31.79
	14.2	126.5	110.8	33.51
	17.2	121.31	103.5	12.6
HUMIDITY ROOM	9.66	117.3	107	43.92
	13	125.15	110.8	39.38
	14.2	126.66	110.9	42.2
	17.2	122.71	104.7	18.8
Deridder White+4% Cement	15.63	126.83	109.7	12.12
Deridder White+4% Cement HR	14.88	126.07	109.7	48.53
Deridder White+4% Cement VS	14.88	127.32	110.8	33.27

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 Lime 2% + Fly Ash 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
RAPID CURING	7.65	112	104.04	52.16
	11	118.17	106.46	52.66
	12.31	121.77	108.42	54.23
	15.15	120	104.21	36.07
VACUUM SATURATION	9	110.58	101.45	24.61
	12.26	116.73	103.98	27.7
	14	120.48	105.68	32.53
	17.32	120.79	102.96	12.63
HUMIDITY ROOM	9	111.9	102.66	52
	12.26	119.29	106.26	57.51
	14	121.7	106.75	54.24
	17.32	124.62	106.22	25.42
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

	Moisture content	Moist Unit Weight	Dry Unit Weight	Maximum Stress
	%	lb/ft^3	lb/ft^3	lb/in^2
Natchitoches 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
DIRECT 4% lime	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
OVEN 4% lime (rapid curing)	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
Humidity Room (14 days)	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+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
Vacuum Saturation	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

Soil	Moisture	γ_{moist}	γ_{dry}	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
K1-1 Natchitoches	9.77	120.25	109.5	25.48
	12.2	124.35	110.8	19.28
	14.03	126.72	111.1	18.72
	14.9	126.48	110.1	15.59
K1-1+2%L+8%FA DIRECT	9.04	112	102.7	40.9
	11.3	116.45	104.6	39.66
	13	119.35	105.6	32.55
	15.22	122.67	106.5	31
RAPID CURING	8.36	110.2	101.7	70.19
	10.41	115.02	104.2	76.04
	12.53	117.93	104.8	66.91
	14.55	122.2	106.7	66.7
VACUUM SATURATION	9.61	110.1	100.4	24.88
	11.62	114.86	102.9	32.07
	14.12	119.37	104.6	42.15
	15.63	122.51	106	41.03
HUMIDITY ROOM	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+4% Cement	18	125.05	106	9
K1-1+4% Cement HR	17.7	125.18	106.4	66.11
K1-1+4% Cement VS	17.7	125.79	106.9	42.38

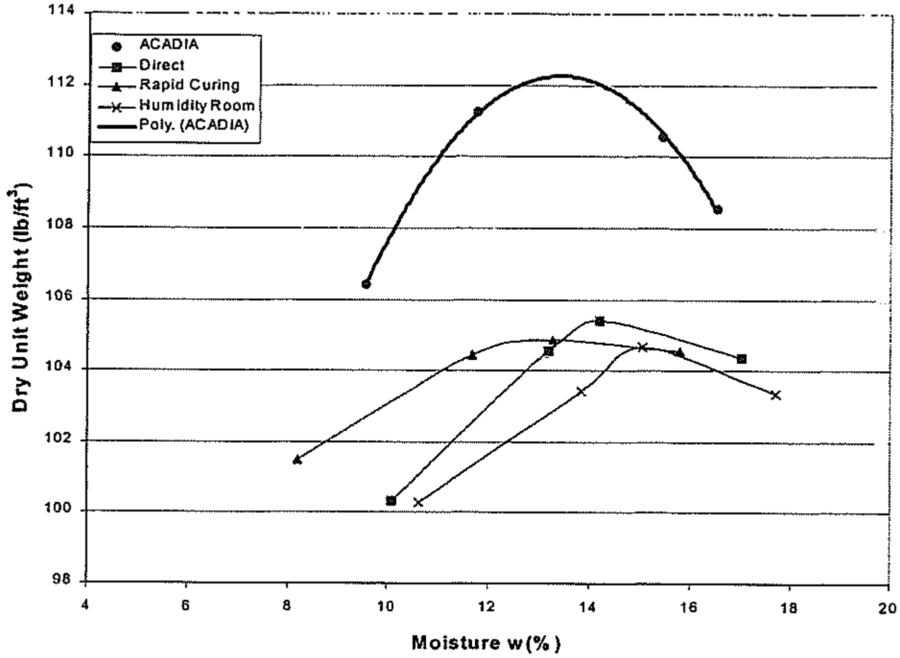
Soil	Moisture	γ_{moist}	γ_{dry}	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
K2-1	9.81	120.5	109.7	39.8
NATCHITOCHES	12.22	127.4	113.5	39.74
	14.43	131.1	114.6	29.56
	16.53	127.4	109.3	15.71
K2-1+L4%	9.63	112.1	102.3	56.15
DIRECT	11.3	116.2	104.4	48.96
	14.31	122.7	107.3	43.06
	16.36	124.7	107.2	35.73
RAPID CURING	8.53	111.6	102.8	93.36
	10.5	114.8	103.9	97.63
	13.7	122.4	107.6	100.74
	15.56	123.2	106.6	88.01
VACUUM SATURATION	10	112.8	102.6	50.79
	11.82	115.3	103.1	51.34
	15.21	122.4	106.2	63.37
	16.64	125.3	107.4	56.55
HUMIDITY ROOM	10	111.5	101.4	61.55
	11.82	114.7	102.6	57.92
	15.21	121.8	105.7	51.51
	16.64	124.4	106.7	50.13
K2-1+4% Cement	16.87	127.1	108.7	24.08
K2-1+4% Cement HR	17.38	127.3	108.5	122.17
K2-1+4% Cement VS	17.38	128.3	109.3	73

Soil	Moisture	γ_{moist}	γ_{dry}	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
K2-1 NATCHITOCHES	9.81	120.5	109.7	39.8
	12.22	127.4	113.5	39.74
	14.43	131.1	114.6	29.56
	16.53	127.4	109.3	15.71
K2-1+L2%+FA8% DIRECT	9.11	112.1	102.8	44.36
	11.38	116.8	104.9	52.47
	13.38	121.5	107.2	50.65
	15.64	126.1	109	38.9
RAPID CURING	8.51	111.8	103	81.1
	11	115.8	104.3	97.96
	13.08	120.5	106.6	86.6
	15.42	125.7	108.9	110.76
VACUUM SATURATION	9.15	111.1	101.8	33.49
	11.27	115.6	103.9	41.05
	13.97	121	106.1	41.32
	15.88	125	107.9	52.09
HUMIDITY ROOM	9.15	111.1	101.8	56
	11.27	117.1	105.2	85.6
	13.97	121.8	106.9	73.73
	15.88	125.4	108.2	69.32
K2-1+4% Cement	16.87	127.1	108.7	24.08
K2-1+4% Cement HR	17.38	127.3	108.5	122.17
K2-1+4% Cement VS	17.38	128.3	109.3	73

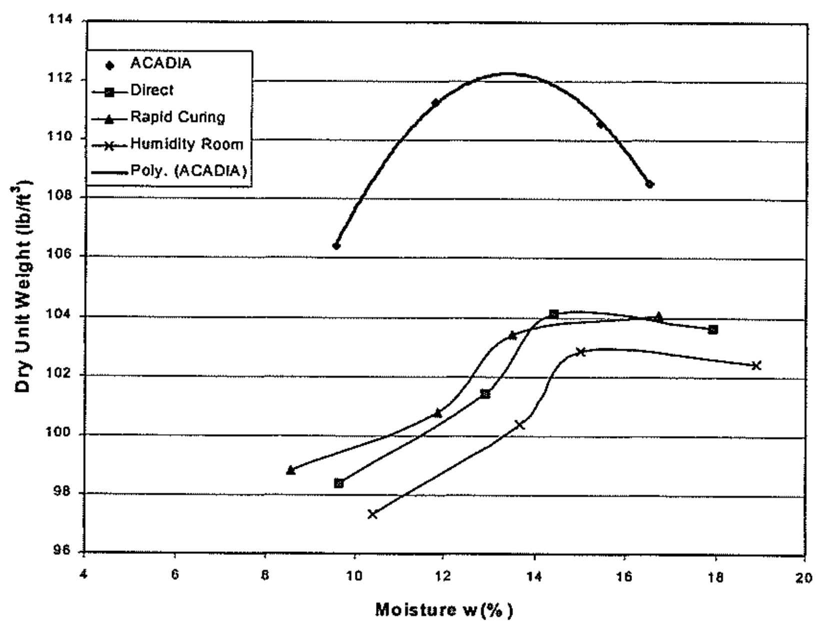
Soil	Moisture	γ_{moist}	γ_{dry}	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
K3-1 NATCHITOCHES	9.65	118.38	108	41.84
	13.2	128.87	113.8	34.14
	14.6	127.54	111.3	19.92
	16.07	126.48	109	13.22
K3-1+L4% DIRECT	9.54	111.87	102.1	41.71
	12.25	118.08	105.2	43.88
	14.68	124.2	108.3	39.1
	16.87	123.61	105.8	21.2
RAPID CURING	9	112.12	102.9	67.6
	11.7	117.39	105.1	67
	14.26	123.27	107.9	62.05
	16.52	122.66	105.3	40.27
VACUUM SATURATION	9.42	112.28	102.6	23.78
	12.58	116.89	103.8	29.2
	14.47	121.44	106.1	31.89
	17.03	122.48	104.7	18.67
HUMIDITY ROOM	9.42	111.95	102.3	54.71
	12.58	118.76	105.5	52.06
	14.47	122.7	107.2	45.73
	17.03	122.7	104.8	23.64
K3-1+4% cement	16.85	126.86	108.6	22.5
K3-1+4% cement HR	16.75	127.29	109	100
K3-1+4% cement VS	16.75	126.39	108.3	54.8

Soil	Moisture	γ_{moist}	γ_{dry}	Strength
	%	lb/ft³	lb/ft³	lb/in²
K3-1 NATCHITOCHES	9.65	118.38	108	41.84
	13.2	128.87	113.8	34.14
	14.6	127.54	111.3	19.92
	16.07	126.48	109	13.22
K3-1+L2%+FA8% DIRECT	9.27	110.32	101	49.92
	12.01	115.19	102.8	51.43
	14.1	118.37	103.7	45.52
	16.87	123.55	105.7	40.21
RAPID CURING	8.24	108.57	100.3	69
	11	113.24	102	75.23
	13.09	116.52	103	69
	16	122.84	105.9	63.07
VACUUM SATURATION	9.8	108.65	98.95	23.5
	12.65	114.23	101.4	36.77
	14.5	117.53	102.6	32.78
	17.5	123.5	105.1	NA
HUMIDITY ROOM	9.8	109.05	99.32	69.07
	12.65	114.05	101.2	70.6
	14.5	117.5	102.6	63
	17.5	123.63	105.2	55.73
K3-1+4% cement	16.85	126.86	108.6	22.5
K3-1+4% cement HR	16.75	127.29	109	100
K3-1+4% cement VS	16.75	126.39	108.3	54.8

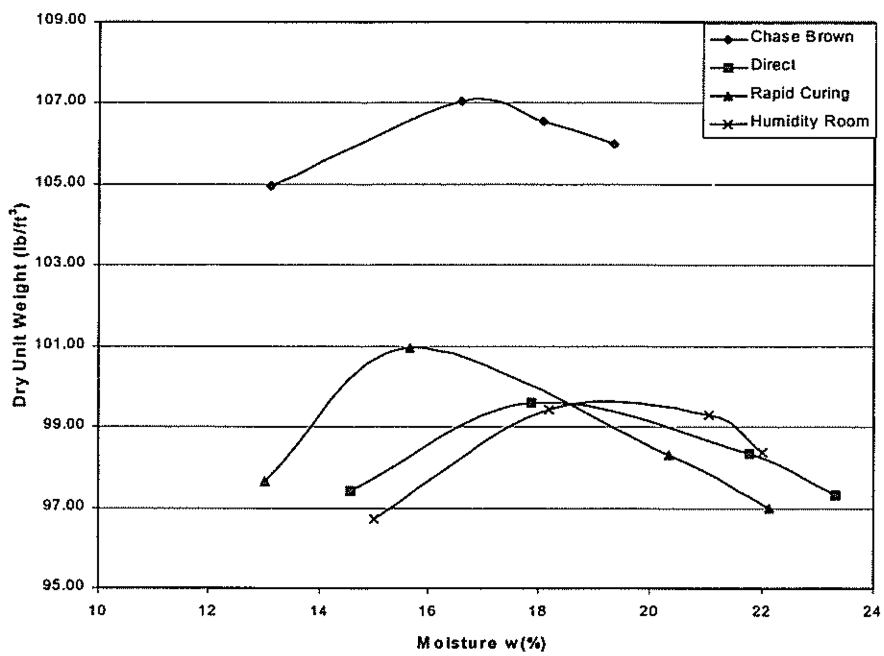
Chemical Stabilization ACADIA + Lime 4%



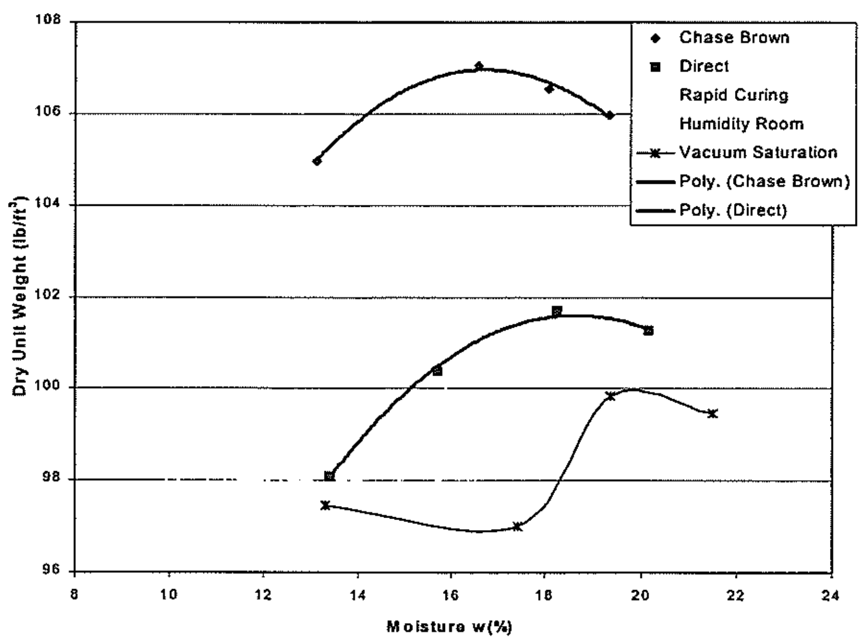
Chemical Stabilization ACADIA + L2% + FA8%

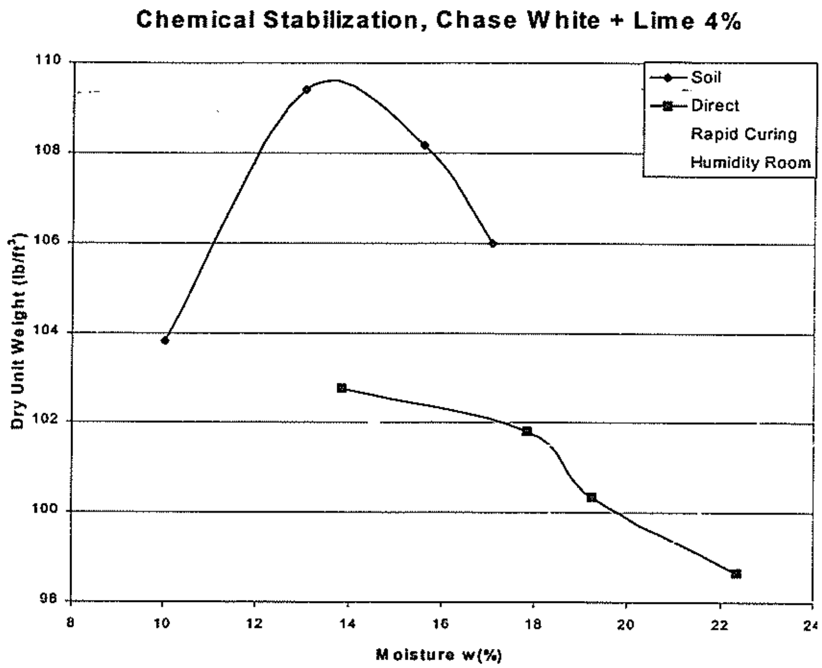


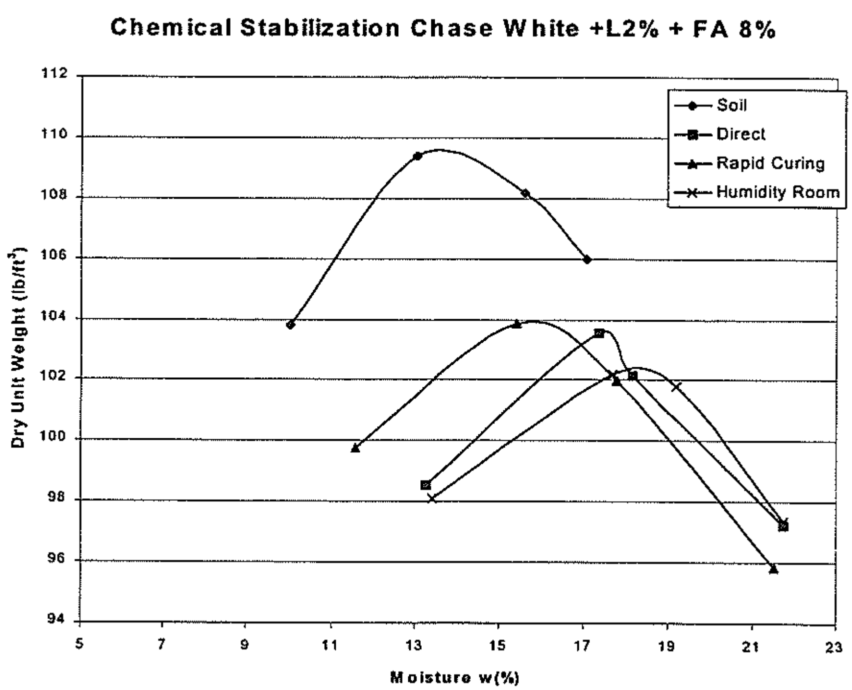
Chemical Stabilization Ch Brown + Lime 4%



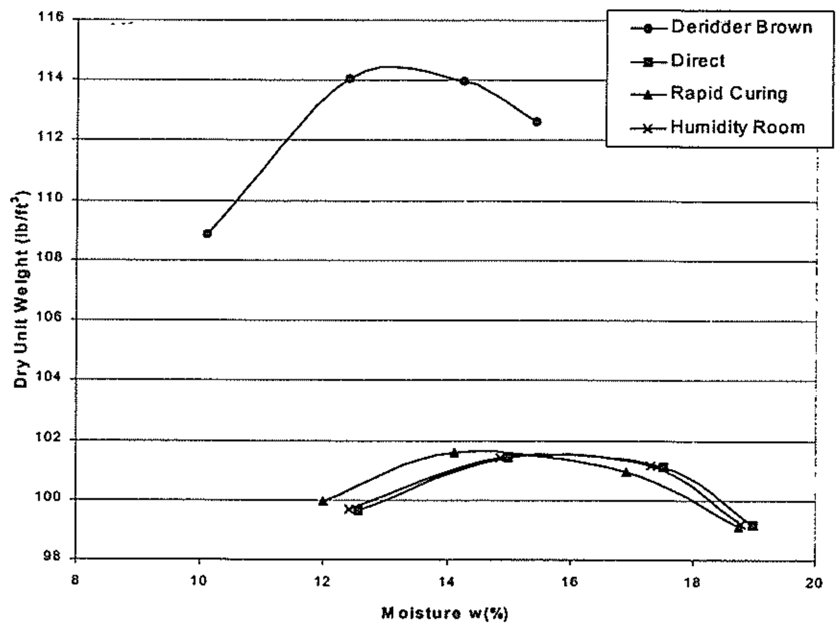
Chemical Stabilization Chase Brown + L2%+FA 8%



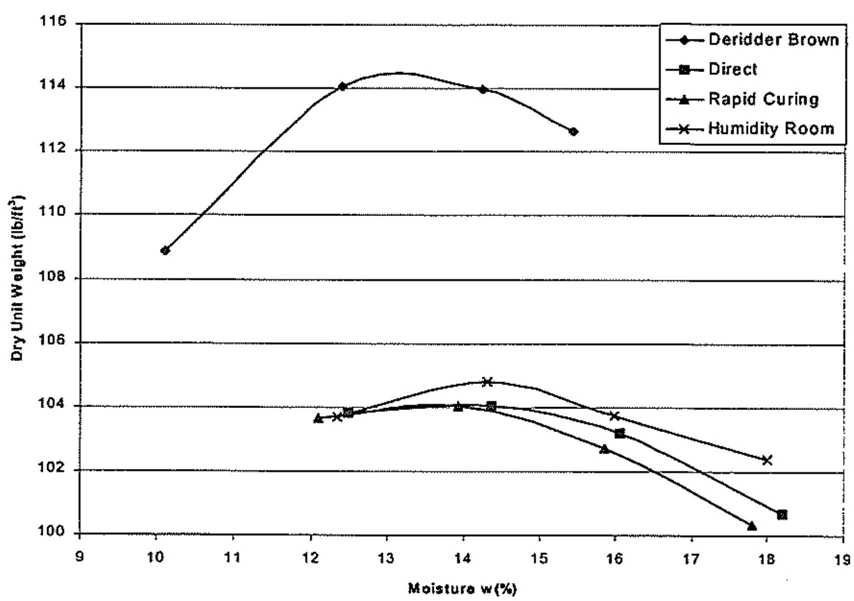

Chemical Stabilization, Chase White + Lime 4%
Soil
Direct
Rapid Curing
Humidity Room
Dry Unit Weight (lb/ft³)
Moisture w(%)
110
108
106
104
102
100
98
8
10
12
14
16
18
20
22
24

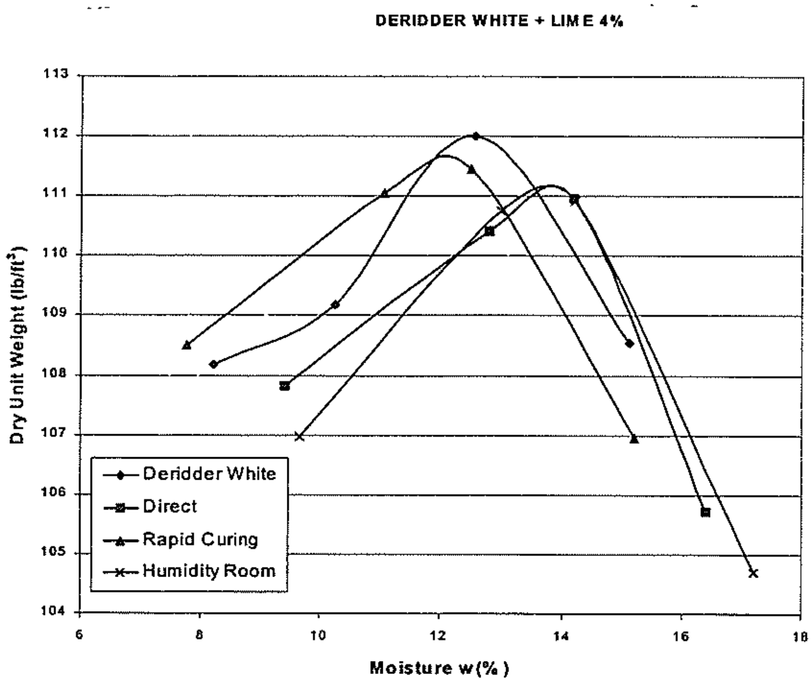

Chemical Stabilization Chase White +L2% + FA 8%
Soil
Direct
Rapid Curing
Humidity Room
Dry Unit Weight (lb/ft³)
Moisture w(%)
112
110
108
106
104
102
100
98
96
94
5
7
9
11
13
15
17
19
21
23

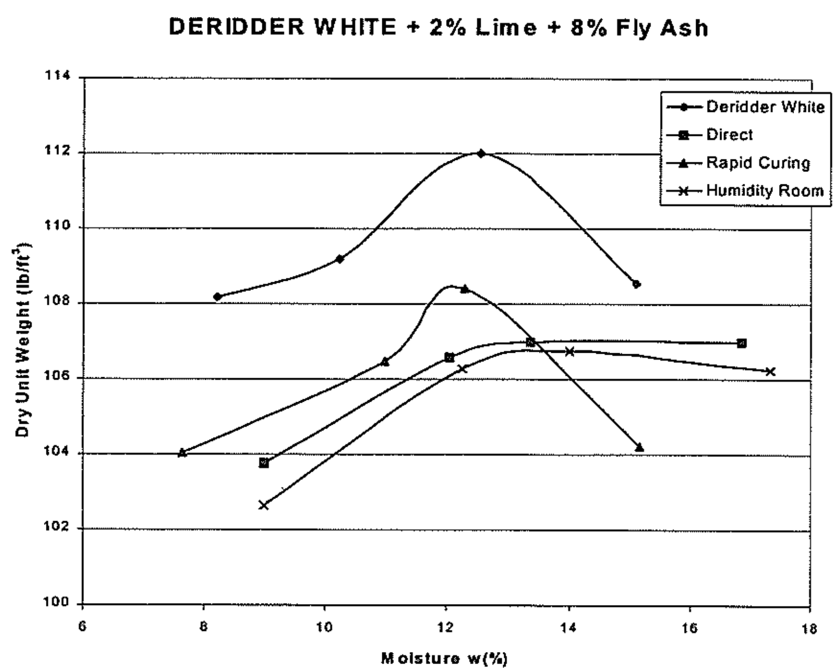
Chemical Stabilization Deridder Brown + Lime 4%

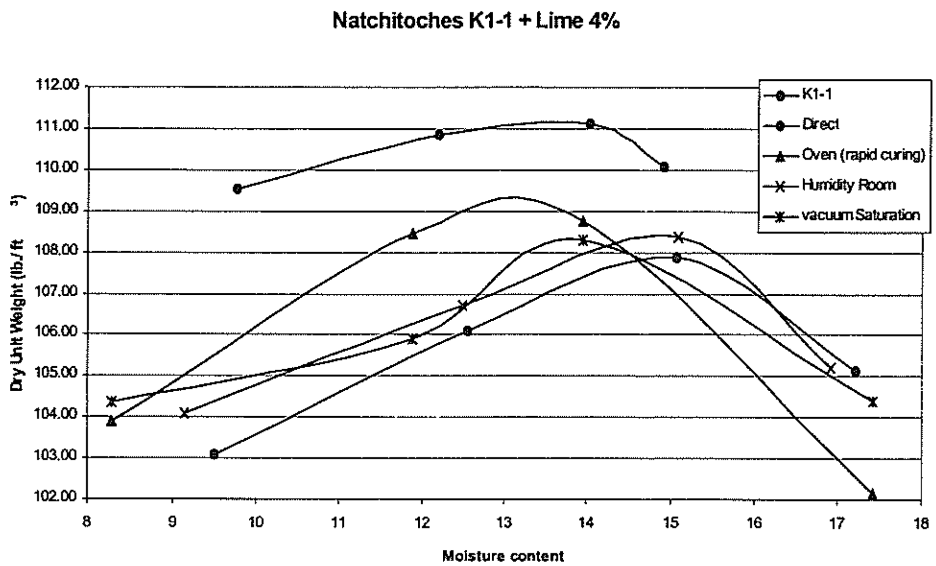


DeRidder Brown + 2% Lime + 8% Fly Ash

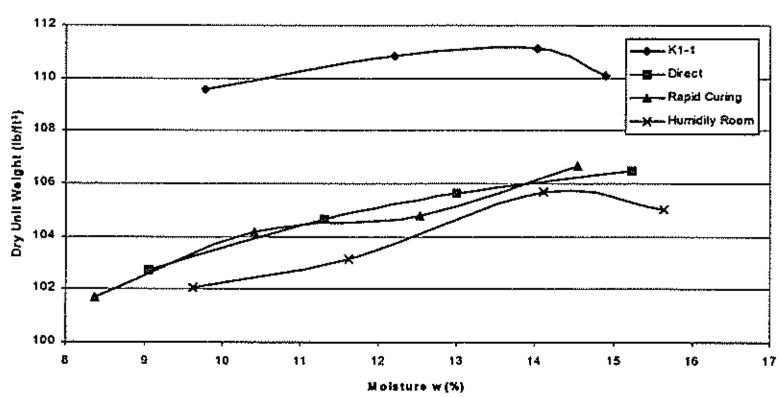


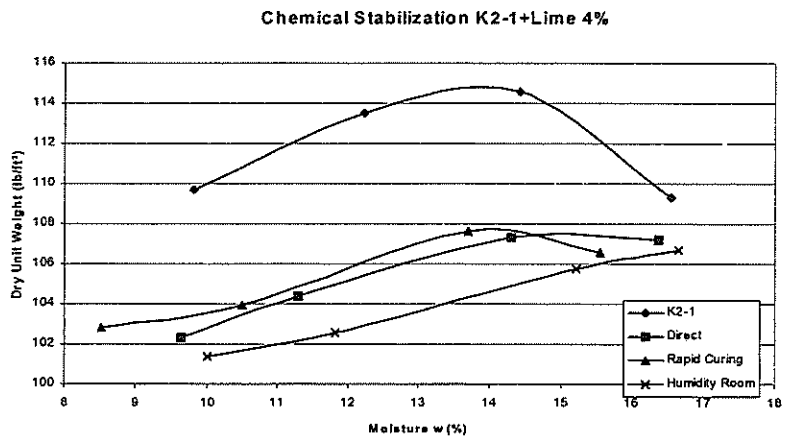

DERIDDER WHITE + LIME 4%
113
112
111
110
109
108
107
106
105
104
Dry Unit Weight (lb/ft^3)
6
8
10
12
14
16
18
Moisture w(%)
Deridder White
Direct
Rapid Curing
Humidity Room

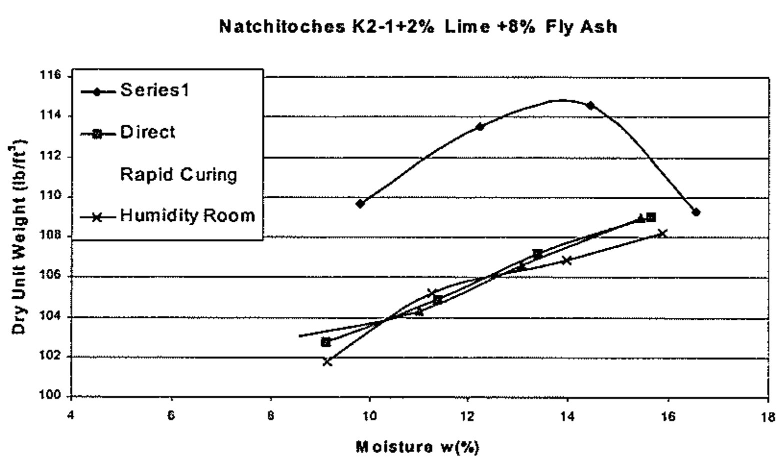

DERIDDER WHITE + 2% Lime + 8% Fly Ash
114
112
110
108
106
104
102
100
Dry Unit Weight (lb/ft^3)
6
8
10
12
14
16
18
Moisture w(%)
Deridder White
Direct
Rapid Curing
Humidity Room



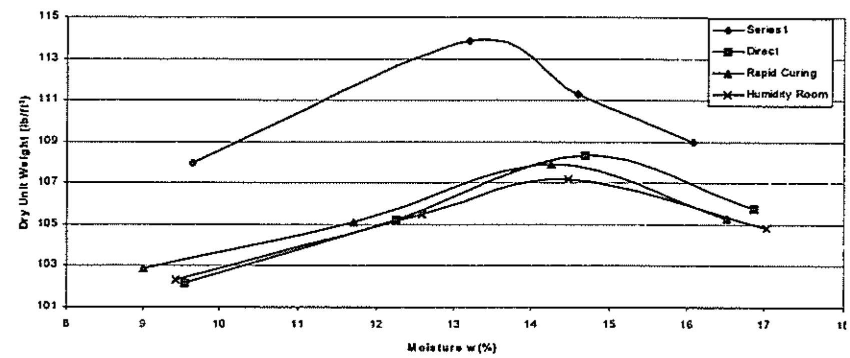
Natchitoches K1 – 2% Lime + 8% Fly Ash



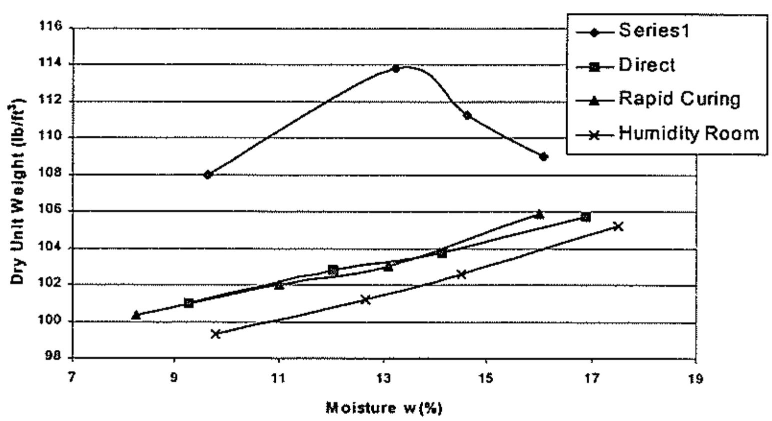

Chemical Stabilization K2-1+Lime 4%
Dry Unit Weight (lb/ft³)
Moisture w(%)
K2-1
Direct
Rapid Curing
Humidity Room


Natchitoches K2-1+2% Lime +8% Fly Ash
Dry Unit Weight (lb/ft³)
Moisture w(%)
Series1
Direct
Rapid Curing
Humidity Room

Natchitoches K3-1+4% Lime



Natchitoches K3 – 2% Lime + 8% Fly Ash



Appendix F
Stabilization Test Series 2

CHASE BROWN

Soil - TEST SERIES 2	Moisture	γ_{moist}	γ_{dry}	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
CHASE BROWN	24.86	117.95	94.47	11.78
SOIL + 10% PC: Direct	20.57	124.43	103.2	30.36
Cured (in HR for 2 weeks)	17.94	125.78	106.6	179.58
VS	17.94	125.84	106.7	72.63
SOIL+Lime 6%:Direct	22.66	121.27	98.87	32.3
RC	21.1	118.44	97.8	254
VS	21.1	118.11	97.53	55.41
SOIL + FA10%: Direct	20.58	122.73	101.8	17.14
Cured (in HR for 2 weeks)	20.29	124.76	103.7	40.9
VS	20.29	125.22	104.1	26.6
SOIL +LFA: Direct	20.79	122.85	101.7	37.04
RC	19.3	120.25	100.8	203.73
VS	19.3	119.91	100.5	27

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

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

CHASE WHITE - Phase 2

Soil	Moisture	γ_{moist}	γ_{dry}	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
CHASE WHITE	19.67	122.5	102.4	15.34
SOIL + PC: Direct	17.17	124.3	106.1	72.62
Cured (in HR for 2 weeks)	17.59	124.44	105.8	130
VS	17.59	123.91	105.4	75.49
SOIL+Lime 6%:Direct	17.5	119.1	101.4	38.3
RC	13.72	118.61	104.3	310
VS	13	106.5	94.25	275.27
SOIL + FA10%: Direct	17.13	124.34	106.2	13.38
Cured (in HR for 2 weeks)	16.58	122.8	105.3	52.71
VS	16.58	123.88	106.3	24
SOIL +LFA: Direct	16.35	123.69	106.3	71.07
RC	11.34	119.61	107.4	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

DERIDDER WHITE
Phase 2

Soil	Moisture	γ_{moist}	γ_{dry}	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
DERIDDER WHITE	19.42	128.43	107.5	14.86
SOIL + PC: Direct	16.36	127.95	110	16.43
Cured (in HR for 2 weeks)	16.28	125.43	107.9	83.35
VS	16.28	126.38	108.7	34.87
SOIL+Lime 6%:Direct	17.07	125.56	107.3	15.6
RC	14.1	122.21	107.1	58.85
VS	14.1	122.26	107.2	24.8
SOIL + FA10%: Direct	16.24	126.68	109	11.47
Cured (in HR for 2 weeks)	17.1	111.97	95.62	16.45
VS	NA, disintegrated			
SOIL +LFA: Direct	15.32	126.89	110	31
RC	12.43	124.8	111	62.24
VS	12.43	124.14	110.4	30

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

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

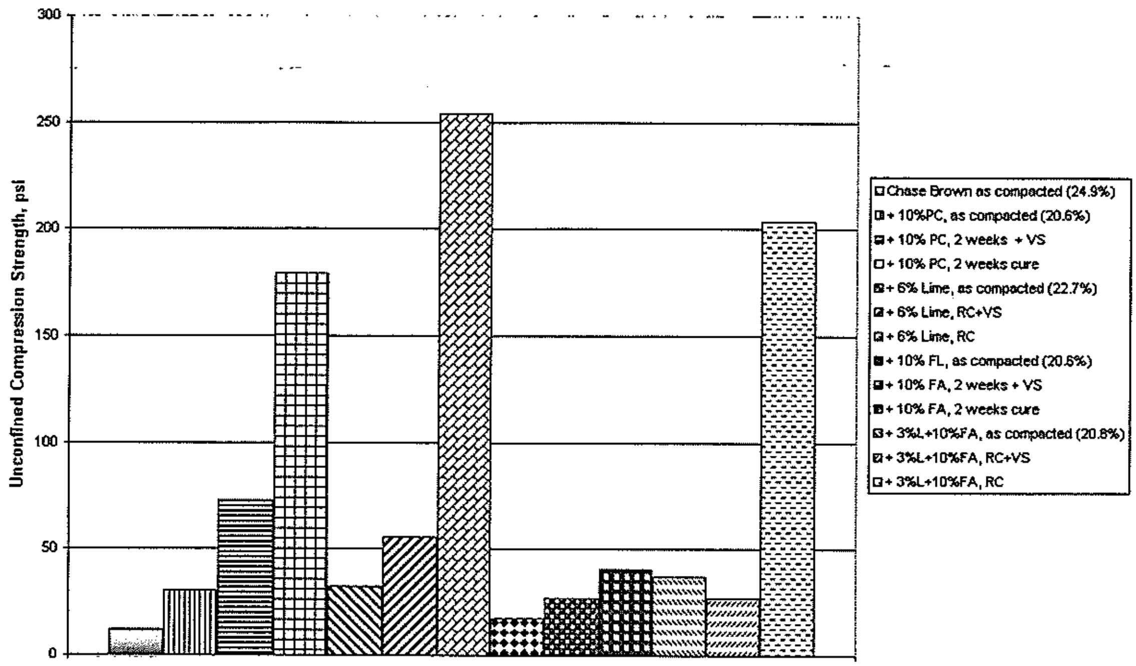
DERIDDER BROWN
Phase 2

Soil	Moisture	γ_{moist}	γ_{dry}	Strength
	%	lb/ft^3	lb/ft^3	lb/in^2
DERIDDER Brown	19.91	121.1	101	13.11
SOIL + PC 10 %: Direct	17.26	127.85	109	34.75
Cured (in HR for 2 weeks)	17.4	130.17	110.9	293.25
VS	17.4	129.18	110	239.8
SOIL+Lime 6%:Direct	18.06	125.9	106.6	33.14
RC	12.71	120.47	106.9	518
VS	12.71	121.27	107.6	126.5
SOIL + FA10%: Direct	16.97	125.88	107.6	24.05
Cured (in HR for 2 weeks)	16.37	128.4	110.3	64.3
VS	16.37	129.86	111.6	33.42
SOIL +LFA: Direct	16.31	127.75	109.8	78.75
RC	12.07	122.94	109.7	301.6
VS	12.07	123.33	110	36

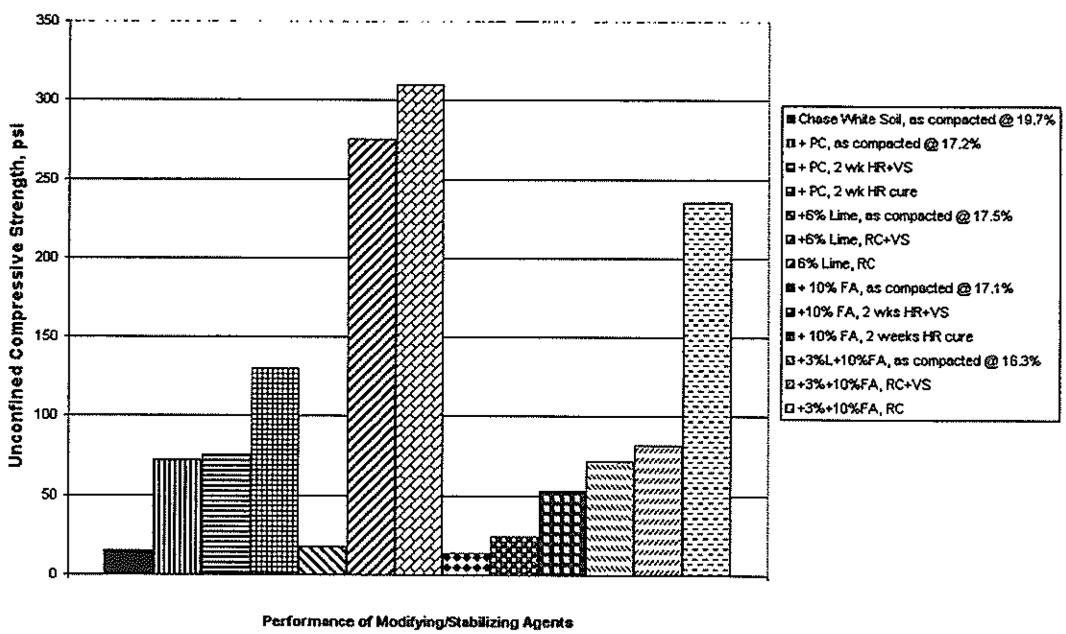
RC = Accel curing:3 days in oven @50^0 C

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

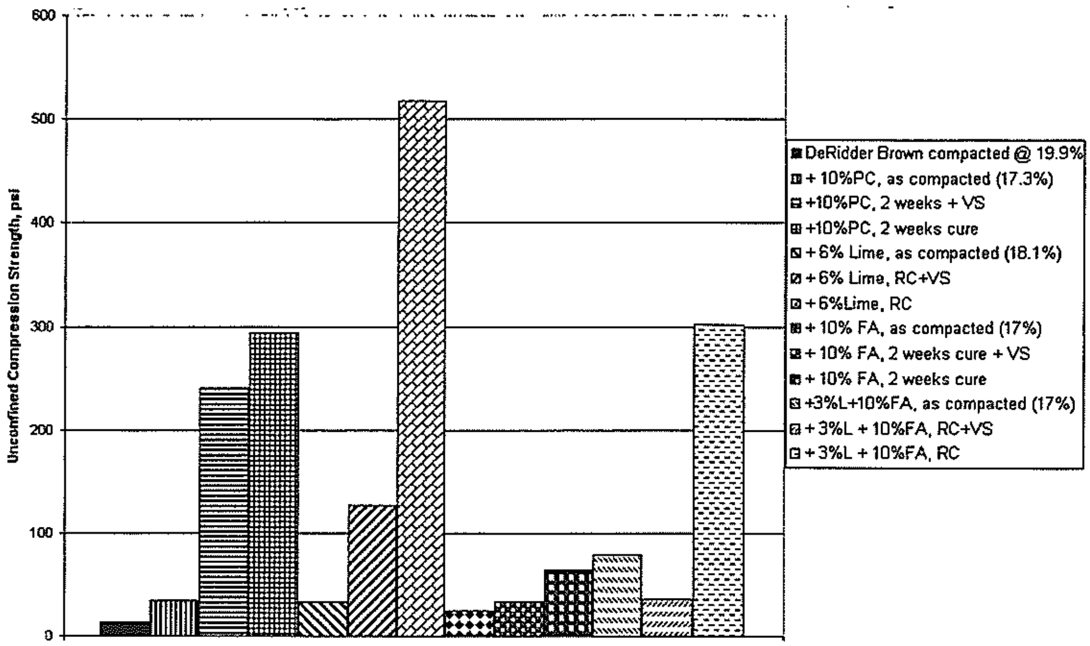
Chase Brown Modification/Stabilization Performance



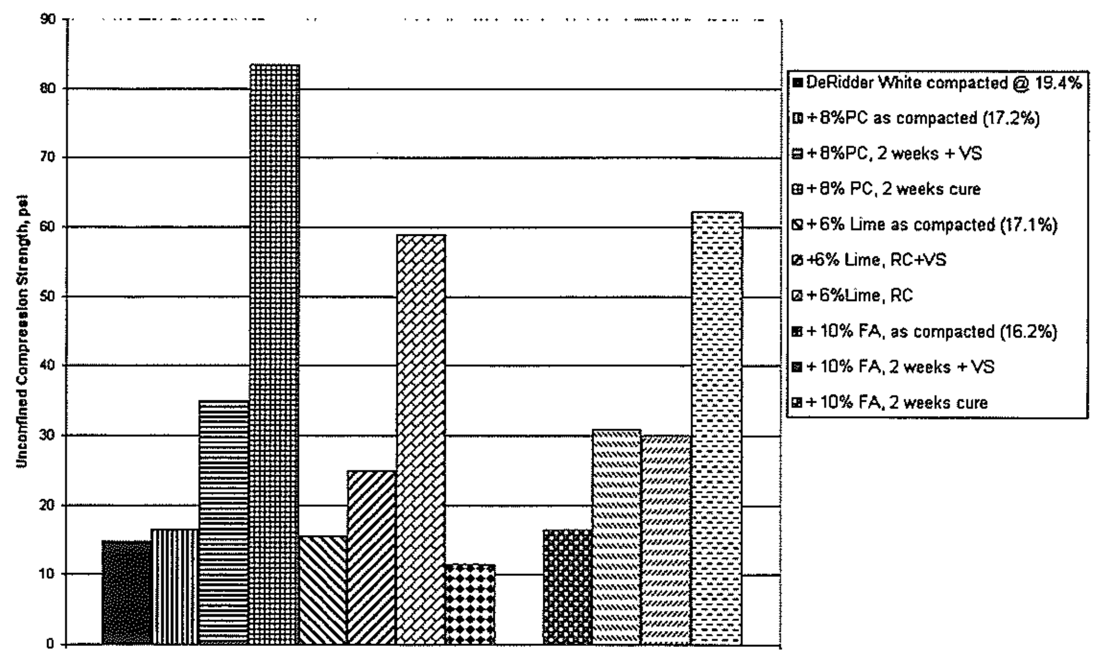
Chase White: Modified/Stabilized w/PC, Lime, Fly Ash, & LFA

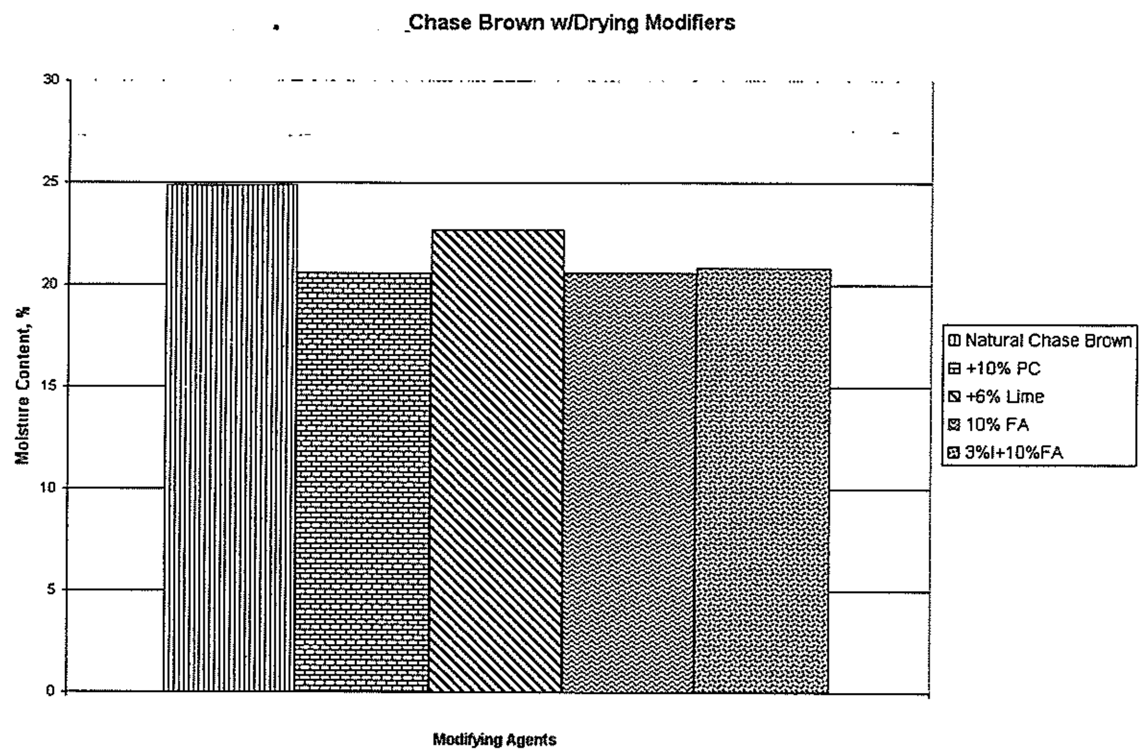


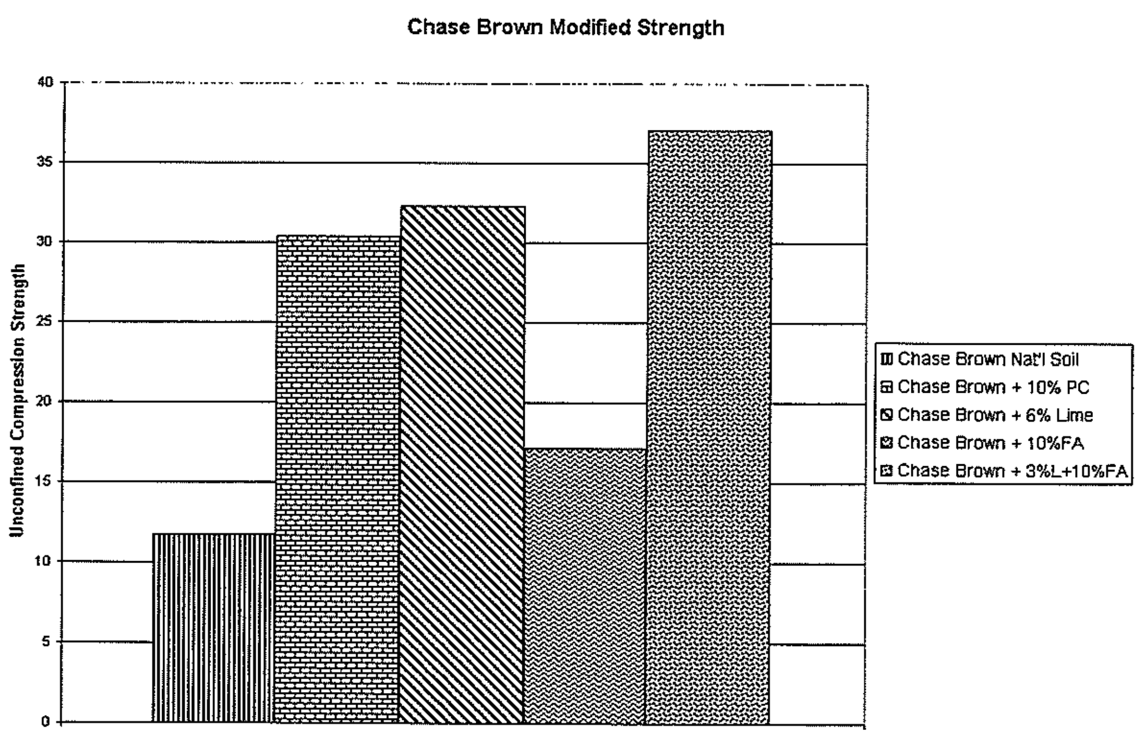
DeRidder Brown: Modification/Stabilization Performance

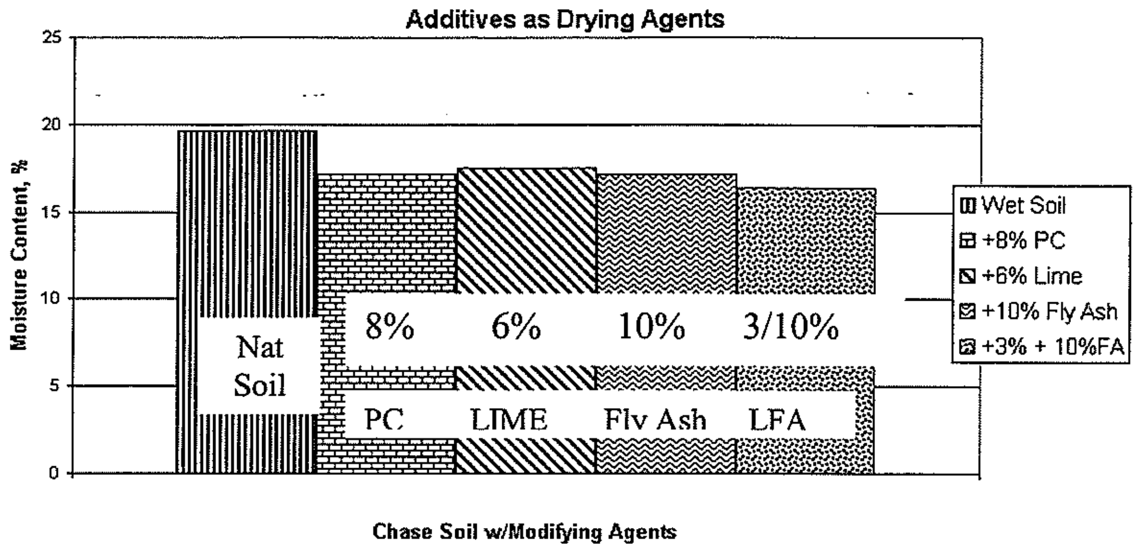


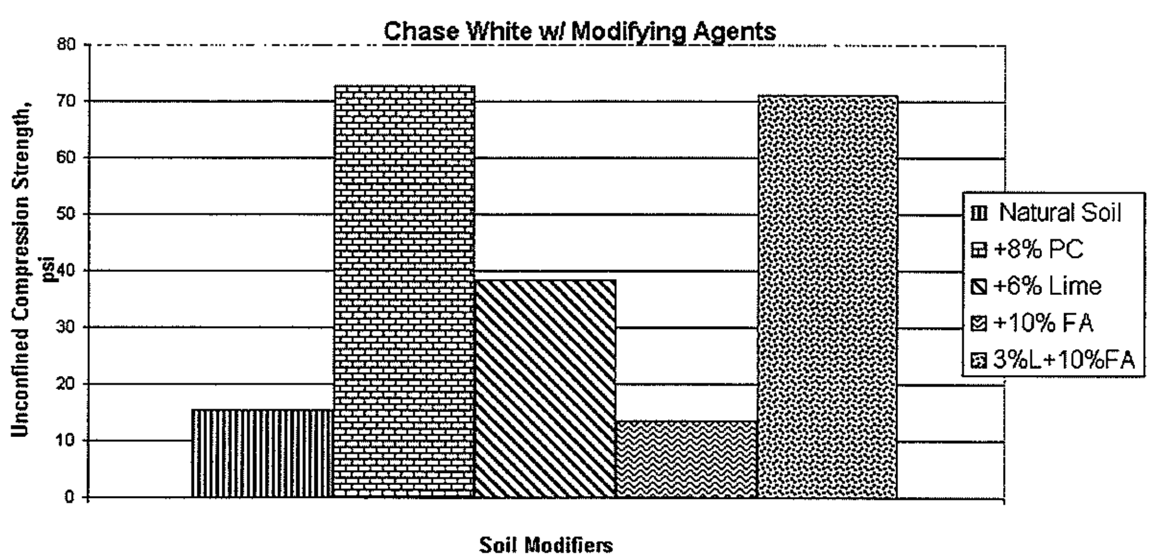
DeRidder White Modification/Stabilization Performance

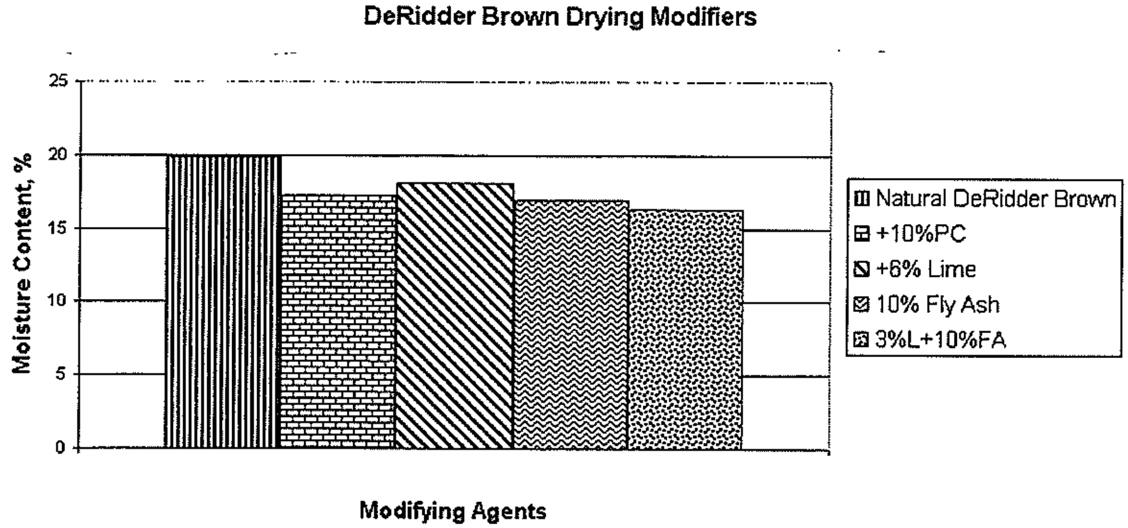


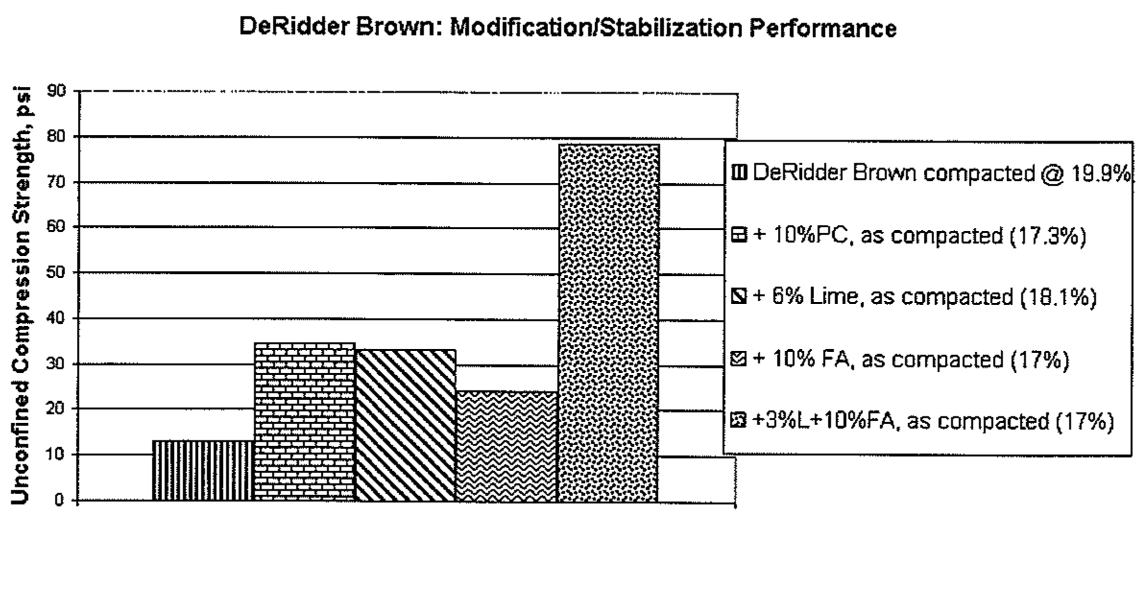

Chase Brown w/Drying Modifiers
30
25
20
15
10
5
0
Moisture Content, %
Modifying Agents
Natural Chase Brown
+10% PC
+6% Lime
10% FA
3%l+10%FA


Chase Brown Modified Strength
40
35
30
25
20
15
10
5
0
Unconfined Compression Strength
Chase Brown Nat'l Soil
Chase Brown + 10% PC
Chase Brown + 6% Lime
Chase Brown + 10%FA
Chase Brown + 3%L+10%FA

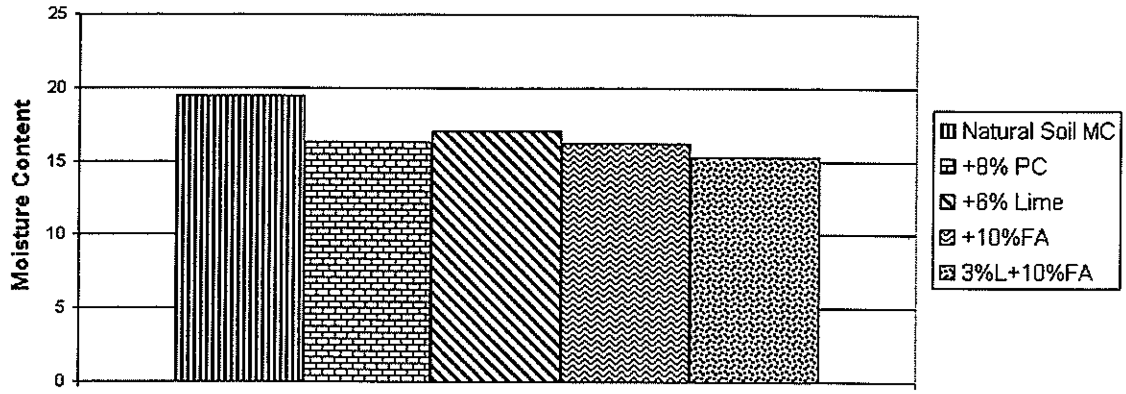

Additives as Drying Agents
Moisture Content, %
25
20
15
10
5
0
Nat Soil
8%
6%
10%
3/10%
PC
LIME
Fly Ash
LFA
Wet Soil
+8% PC
+6% Lime
+10% Fly Ash
+3% + 10%FA
Chase Soil w/Modifying Agents


Chase White w/ Modifying Agents
Unconfined Compression Strength, psi
80
70
60
50
40
30
20
10
0
Natural Soil
+8% PC
+6% Lime
+10% FA
3%L+10%FA
Soil Modifiers


DeRidder Brown Drying Modifiers
Moisture Content, %
0
5
10
15
20
25
Modifying Agents
Natural DeRidder Brown
+10%PC
+6% Lime
10% Fly Ash
3%L+10%FA

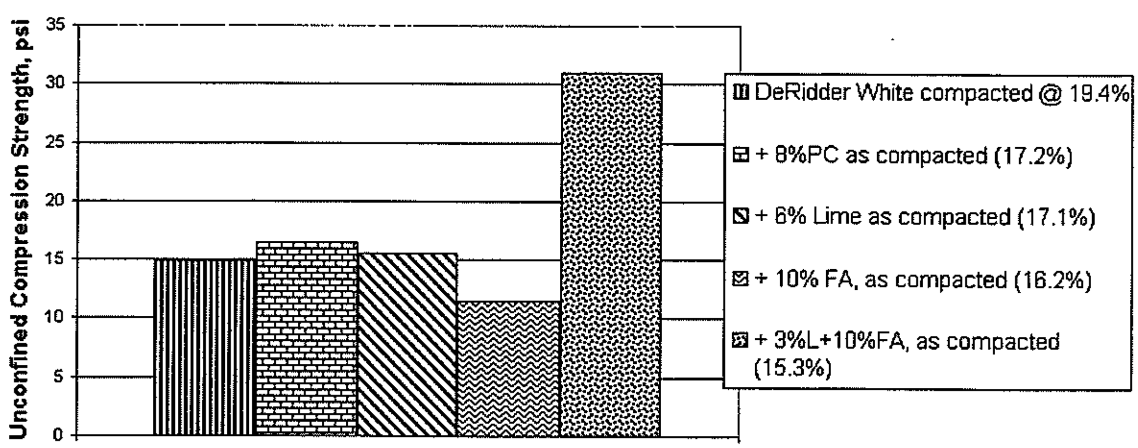

DeRidder Brown: Modification/Stabilization Performance
Unconfined Compression Strength, psi
0
10
20
30
40
50
60
70
80
90
DeRidder Brown compacted @ 19.9%
+ 10%PC, as compacted (17.3%)
+ 6% Lime, as compacted (18.1%)
+ 10% FA, as compacted (17%)
+3%L+10%FA, as compacted (17%)

DeRidder White w/Drying Modifiers



DeRidder White + Modifying Agents

DeRidder White Modification/Stabilization Performance



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