LABORATORY EVALUATION OF FLY ASH TREATED EMBANKMENT AND BASE MATERIALS

FINAL REPORT

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ABSTRACT

This study was undertaken to provide the Louisiana Department of Transportation and Development (LADOTD) with a data base from which recommendations can be made concerning the modification or stabilization of soils using a Class C fly ash as a full or partial replacement for hydraulic cement or hydrated lime. It includes test data from two road base soils and two soils that are used in embankment construction. The test specimens were combined with lime, cement, and/or fly ash from three generating plants in Louisiana.

The base soils consisted of an A-3 sand and an A-2-4 sandy-silt. Unconfined compressive and vacuum saturation strengths were compared to those presently required for stabilization using cement as a single additive. The test results indicate the importance of the characteristics of the gradation of the materials and the effects of the presence of fines in the soil.

Two clays, an A-6(9) and an A-7-6(20), were evaluated by comparing the improvements in the plastic properties and the gain in soil support with the addition of fly ash and/or lime. The amount of free lime present in the Class C fly ashes tested was insufficient to effectively compete with lime alone in modifying the plasticity of the clays. The fly ash did enhance the soil's resistance or R-Value when combined with lime.

IMPLEMENTATION STATEMENT

The use of locally produced fly ash should be considered as an alternate method for soil stabilization in pavement and embankment construction. Due to the wide range in physical characteristics between different fly ashes, procedures for source qualifying should be identified. The use of fly ash should be evaluated on a case-by-case basis.

Methods for proportioning fly ash as a lone stabilizing agent or in combination with other additives should be studied and formalized. An approach based on a gradation analysis can be used in the initial selection of the percentage of fly ash required in a sandy soil. In order to maximize the stabilization effort the gradation of the combined materials should be based on maximum density (page 22). This will provide an upper bound for those improvements produced in the engineering properties of the soil-stabilizer mix. Consideration should also be given to the silt-size portion of the soil as it affects the continuity of the pozzolanic reactions in the fly ash matrix.

The Class C fly ashes studied were not as effective as lime in the modification of the clay soils studied (page 28). However, fly ash may be advantageous in preparing or stabilizing the work area or in achieving an early strength. Mixture design criteria similar to that used for lime (29) in the stabilization of clays should be used.

Field evaluations should be undertaken to determine performance and construction criteria to be used. Criteria selected for the analysis of laboratory testing should be compatible with field operations.

METRIC CONVERSION FACTORS*

To Convert from	TO	Multiply by
	Length	
foot	meter (m)	0.3048
inch	millimeter (mm)	25.4
yard	meter (m)	0.9144
mile (statue)	kilometer (km)	1.609
	Area	
square foot	square meter (m^2)	0.0929
square inch	square centimeter (cm^2)	6.451
square yard	square meter (m^2)	0.8361
	Volume (Capacity)	
cubic foot	cubic meter (m^3)	0.02832
gallon (U.S. liquid)**	cubic meter (m^3)	0.003785
gallon (Can. liquid)**	cubic meter (m^3)	0.004546
ounce (U.S. liquid)	cubic centimeter (cm^3)	29.57
	Mass	
ounce-mass (avdp)	gram (g)	28.35
pound-mass (avdp)	kilogram (kg)	0.4536
ton (metric)	kilogram (kg)	1000
ton (short, 2000 lbs)	kilogram (kg)	907.2
	Mass per Volume	
pound-mass/cubic foot	kilogram/cubic meter (kg/m^3)	16.02
pound-mass/cubic yard	kilogram/cubic meter (kg/m^3)	0.5933
pound-mass/gallon (US)**	kilogram/cubic meter (kd/m^3)	119.8
pound-mass/gallon(Can)**	kilogram/cubic meter (kg/m^3)	99.78
	Temperature	
deg Celsius (C)	kelvin (K)	$t_K=(t_C+273.15)$
deg Fahrenheit (F)	kelvin (K)	$t_K=(t_F+459.67)/1.8$
deg Fahrenheit (F)	deg Celcius (C)	$t_C=(t_F-32)/1.8$

* The reference source for information on SI units and more exact conversion factors is "Metric Practice Guide" ASTM E 380.

** One U.S. gallon equals 0.8327 Canadian gallon.

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INTRODUCTION

Growing shortages of natural aggregates in many areas of Louisiana have created the need for the conservation and/or replacement of these materials in highway construction. For example, shell has been used over the years in base construction as an aggregate and as an alternative to portland cement-treated material. It has also been used as an embankment material. Due to a diminishing supply and the rising cost of shell used in highway construction, there is a need to develop other locally available materials or construction techniques which could be used as alternates to shell in pavement structures or embankment construction. A technique which may prove to be feasible is chemical treatment of marginal soils with inexpensive additives as stabilizers.

Coal-burning power plants in Louisiana are producing a self-hardening, ASTM (American Society of Testing and Materials) Class C fly ash. This type of fly ash contains high calcium oxide (CaO) contents and has been found to be advantageous in the stabilization of soils with and without the use of added lime (1). The use of fly ash, while contributing environmentally in the disposal of a waste product, could provide monetary savings by reducing the amount of portland cement or hydrated lime currently required for the stabilization of soils in highway construction. Substitution of fly ash, where possible, reduces the need for other energy-intensive products.

Although fly ashes have been utilized for soil stabilization purposes in the United States since 1950, little definitive information is available which identifies soils which are most suitable for stabilization with fly ash, lime-fly ash, or cement-fly ash mixtures (2). ASTM Class C fly ashes have been shown to exhibit properties that would permit their use as a partial replacement of lime and portland cement in soil stabilization (3). The objective of this research is to evaluate laboratory test efforts for utilizing fly ash (ASTM Type C) as a singular additive, or as a replacement of hydrated lime or portland cement, in the modification or stabilization of common soils used in Louisiana highway construction. The ultimategoal of this research is to determine if such a system will reduce the plasticity or improve the support characteristics of a given soil type and allow its use in embankment or base construction.

The study was undertaken to provide the Louisiana Department of Transportation and Development (LDOTD) with a data base from which decisions concerning fly ash stabilization could be made. An interim report on the test results and initial findings for two soils used in base construction combined with cement, lime and/or fly ash from three local generating plants has been reported by Melancon, et al. (4). This report reviews and analyzes further the testing of the base materials and includes the tests conducted on soils used in subbase/embankment construction.

TYPES OF FLY ASH

Fly ash is produced in power plants from the combustion of ground coal or lignite. It is the very fine particulate matter that is collected from the flue gases. Fly ash is a pozzolan that consists mostly of amorphous components of siliceous or siliceous and aluminous material. As a finely divided material and in the presence of moisture, it chemically reacts with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties. The fly ash particles are generally spherical granules formed when the clay, pyrite and calcite in the coalare burned under high temperatures in the combustion chamber. The composition and properties of the resulting fly ash are determinedby the coal source burned, the equipment and procedure used and the methods in which the fly ash is handled.

ASTM specification C 593-76 addresses the use of fly ash as a pozzolan with lime, including requirements for the maximum allowable water soluble fraction and gradation requirements. Several physical and chemical characteristics of fly ash are used as indicators of how well a fly ash will perform as a pozzolan. Fineness, or specific surface, as determined in accordance with ASTMC 311-77, is the best physical indicator. The finer the fly ash particles, the greater the rate of pozzolanic reaction. An important chemical indicator is the carbon content of the fly ash, measured as a loss-on-ignition in accordance with ASTM C 311-77. A high carbon content (>10%) tends to inhibit the pozzolanic reactivity of a fly ash. High calcium oxide (CaO) content has also been found to be very advantageous in stabilization studies (1). Increased strength and durability have been reported with cohesive and cohesionless soils (5).

ASTM identifies two classes of fly ash based on the coal source; Class F fly ash originating from bituminous coals and Class C from subbituminous and lignite origins. Most bituminous coal is found in the eastern and north central states. Subbituminous and lignite coals are found in the western and southwestern areas of the United States and are referred to as western coals. There is a difference in chemical composition between the ashes produced from the different coals. The subbituminous and lignite coals produce a fly ash with a higher lime content, CaO, (10% to 32%) than is found in the bituminous fly ash (<1% to 5%) . The high-lime fly ashes can possess both pozzolanic and cementitious characteristics. The power plants operating in Louisiana use western coals and are producing Class C fly ashes.

Lime and portland cement are commonly used in combination with fly ash for the treatment or stabilization of soils. The fly ash acts as a pozzolan and/or as a filler in reducing the voids in coarse soil aggregates. Pozzolanic action occurs when the silica and alumina in the fly ash react chemically with slaked lime. Some Class C fly ashes, in addition to being pozzolanic, possess sufficient amounts of calcium silicates to exhibit cementitious properties similar to portland cement. The high CaO content has also been thought to be an indication of the presence of a substantial amount of free lime, providing a beneficial effect on the soil's physical properties and reacting with the siliceous and aluminous compounds in the fly ash to produce long-term pozzolanic cementation (1). If the fly ash has a high free-lime content, flocculation of the clay minerals and reduction of the soil plasticity for fine-grained soils is also possible.

There is a wide range in the chemical composition of western fly ashes. Joshi (6) observed that the calcium in these very reactive fly ashes when mixed with lime yields very high compressive strengths. However, he concluded that the strength is not a function of the amount of calcium hydroxide, but may be related to increased surface area and possibly the presence of hydraulic cement compounds. The need for research analyzing the variability of the Class C fly ashes and the behavior of the stabilized end product has been cited (3).

According to Ledbetter (7) and McKerall et al. (3), most ofthe CaO (lime) present in the Class C fly ash is not in a free, or available, state. Most of it is combined with the silicates and aluminates of the fly ash in a manner similar to the CaO in portland cement. Thus, the lime in the fly ash is not readily available as a soil stabilizing agent for use in modifying the behavior of plastic, fine-grained soils. In previous studies, it was determined that almost all of the lime present in fly ash is chemically combined- - as a result of the intenseheat in the furnace- - with thesilicates and aluminates. The free lime available has been found to be approximately 2 percent. Using X-ray analysis techniques, several Iowa Class C fly ashes were evaluated (8, 9) as having about 30 percent elemental calcium oxide, but only 1 to 2 percent of this existed as free calcium oxide. The cementitious character of the fly ashes was attributed to some of the calcium (6 to 7.5 %) being combined as tricalcium aluminate and tricalcium aluminate sulfate, producing a fast set and high early strength. The rest of the calcium consisted of other calcium crystalline components (5 to 10%) and calcium in a glassy phase (20 to 25 %). Some Class C fly ashes in the study were found to be high in calcium content but not cementitious. Attempts to find a technique that will liberate calcium and other elements necessary to form cementitious compounds from the glassy phase without the use of lime or portland cement and a means for retarding the flash set of the tricalcium aluminates present have been investigated (10). As evidenced by X-ray diffraction and scanning electron microscope data, the addition of ammonia phosphate appeared to break down the calcium-rich glassy phase as well as retard the rapid hydration of the tricalcium aluminates.

SOIL STABILIZATION WITH FLY ASH

Most applications of fly ash for soil stabilization have involved Class F fly ash due to the longer history of production and use of eastern coal. The results of many studies are summarized in an FHWA implementation package (2) and in an NCHRP synthesis report (11), both published in 1976. Very little published information is available on the use of ASTM Class C type fly ash used alone or in combination with lime and/or portland cement in soil treatment or stabilization. That which exists, (12, 13, 14, 15, 16, 17, 18) suggests a potential for the use of this fly ash in modification of the plasticity of clays in embankments and in combination with soils in bases and in subbases. These reports also indicate that the effectiveness does depend on soil types and the specific ash used, as well as field mixing techniques, degree of compaction and curing.

In soil stabilization, fly ashes act as pozzolans and/or fillers for the reduction of air voids in naturally or blended aggregate systems. In fine-grained soils the void space is usually smaller than the fly ash particle size. Thus, the role for the fly ash in the stabilization of fine-grained soils has been that of a pozzolan rather than filler material. Since most clays are pozzolanic in nature, silts are generally considered the most suitable fine-grained soil type for treatment with lime-fly ash or cement-fly ash mixtures. However, the cementitious or self-hardening characteristics of the Class C fly ashes provide incentive for possible consideration of a lime-Class C fly ash combination in a clay soil.

Several reactions take place when lime with water is added to a fine-grained soil. Cation exchange and flocculation reactions take place initially and produce immediate changes in soil plasticity, workability, and the initial uncured strength (load deformation properties). Depending on the characteristics of the soil, a soil-lime pozzolanic reaction may occur. In the pozzolanic reaction various cementing compounds are formed, increasing the mixture strength and durability. The pozzolanic reactions are time-dependent; therefore, strength development is gradual but continuous for long periods of time, amounting to several years in some instances. Temperature also affects the pozzolanic reaction. Temperatures less than 55^{o} F retard the reaction, and higher temperatures accelerate the reaction (19).

When used as a pozzolan additive, Class F fly ash is in most cases combined with lime since it is not self-hardening. In addition to being pozzolanic, many Class C fly ashes have been found to possess sufficient amounts of calcium silicates and exhibit cementitious properties similar to portland cement. In recent years an increasing number of stabilization projects have been performed using Class C fly ash alone (1). If the Class C fly ash has a high percentage of free-lime content, it would be possible to flocculate the clay mineral and reduce the soil's plasticity. In addition, the free lime could also react with the siliceous and aluminous compounds in the fly ash to produce a very effective cementing action with a long-term strength gain. Additional strength gain may result from hydration of tricalcium aluminate or other portland cement type compounds in some Class C ashes. However, lime may also be added to Class C fly ash to enhance its natural cementitious properties.

EVALUATION OF FLY ASH MIXTURE

The evaluation of fly ash mixtures in a base or subbase material is usually determined on the resulting strength, bearing value, or load/deflection characteristics and the durability, or the mixture's ability to withstand damaging freeze-thaw and wet-dry action. ASTM C 593-76a (1981), "Fly Ash and Other Pozzolans for Use with Lime", establishes minimum strength and durability requirements applied to coarse-grained soil. The unconfined compressive strength criterion is 400 psi (2760 kPa) at the end of a 7-day cure, with an elevated temperature of 100° F (38° C). The accelerated curing period is an attempt to produce an approximation of the 28-day strength of the mixture under ambient conditions. However, accelerated curing times equivalent to 28-day normal-cure strengths depend on several factors: curing temperature, soil type, and to a lesser degree, the lime content or the lime-to-fly ash ratio. Thus, a universal accelerated test that will duplicate a 28-day strength at normal temperatures under all conditions does not exist. ASTM C 593 specifications are also used with projects involving lime and fly ash for stabilizing fine-grained soils (1). Recommendations have been made for reducing the requirements of strength to as low as 100 psi (690 kPa) for subbase applications. A former Louisiana Department of Highways (LDH) Designation TR 433-70 (Determining the Minimum Lime Content for Lime-Soil Treatment) required a minimum lime content that provided unconfined compressive strengths of 100 psi (690 kPa) for a base course and 50 psi (345 kPa) for subbase courses, with a 7-day curing time. However, LADOTD TR 433-81 states only that for a base or subbase course, the liquid limit following lime treatment shall be a maximum of 40 with a maximum plasticity index of 10 and 15, respectively.

The vacuum saturation testing method replaces the freeze-thaw brush test in ASTM C 593. In this test, a sample is subjected to a vacuum for a specified period of time and then soaked. The unconfined compressive strength of stabilized soil samples at the end of this test corresponds to the strength of samples subjected to five or ten freeze-thaw cycles (but not brushed) (20). The required strength at the end of the vacuum saturation test is also 400 psi (2760 kPa). This durability criteria requires that no loss of compressive strength occurs.

FACTORS AFFECTING PERFORMANCE

The strengthand durability of a cement- or lime-fly ash-soil mixture is influenced by a number of factors. These include: soil type, type of fly ash, type of lime, percentage of stabilizers used, lime-to-fly ash ratio, dry density and moisture content of the compacted mixture, curing time or age, and temperatures.

Types and Proportions of Stabilzer to Soil

The optimum percentage of fly ash for granular soils can be selected on the basis of maximum dry density. The ultimate strength is closely related to the quality of the cementitious matrix of the mixture. The matrix includes the lime/cement plus fly ash plus fines. Only if there is sufficient matrix to "float" the coarser aggregate fraction is it possible to achieve a highly compacted density which is essential to good strength. The proportion of lime-to-fly ash in the matrix must also be sufficient to provide a good chemical reaction. In fine-grained soils the strength of the lime-fly ash-soil mixture increases as the total amount of stabilizer is increased (21), and the selection of a total stabilizer percentage is usually based on economic considerations. A number of lime-to-fly ash ratios will produce approximately the same mixture performance . The most economical combination which satisfies the specified criteria is selected. High plastic and expansive clays will require a larger lime-to-fly ash ratio to ensure that there is adequate lime for both the lime-clay and lime-fly ash reactions. Class C fly ashes having calcium oxide contents of 20 percent or greater have been reported as adequately stabilizing fine-grained plastic soils such as clay, as well as coarse-grained soils, without the use of lime (6, 12, 16). However, using the Class C fly ash requires that the complete operation of adding water to gain optimum moisture, thorough mixing of the soil and fly ash, and final compaction must be carried out quickly or a significant loss in strength will occur.

Dry Density and Moisture Content

The compressive strength of a lime-fly ash-soil mixture increases with increasing density to a point near the maximum dry density as determined by the standard Proctor test (AASHTO T 99-83, ASTM D 698). Experiments have shown that for most lime-fly ash-soil mixtures, maximum compressive strength is obtained at a moisture content slightly less than the optimum moisture content required for maximum dry density (21). Studies have shown that densities above those produced by the standard Proctor method result in greater durability for lime-fly ash-soil mixtures (22). Mixtures which are not durable at standard Proctor densities might be made durable through greater compactive effort.

The density of the mixture has a major effect on the strength and durability of cement- and lime-fly ash stabilized materials (20, 23 and 24). The pozzolanic reactions that are possible will be influenced by many factors, including in situ density. The aggregate gradation has a very significant effect on the density, strength and durability of the mix (24). The initial step required in developing the proper proportions of a cement- or lime - fly ash mixture is to estimate the fly ash required to fill the voids in the aggregate (23). Sand aggregates with single-sized particles and sands without minus 200-sized particles may require high fly ash content to serve as filler or void reducer, as well as a pozzolan in the mixture.

Curing Age of Mixture

The rate of strength development in lime-fly ash-soil mixtures is much slower than in soil-cement mixtures. There is a gain in strength over a longer period of time with lime-fly ash mixtures, however. The rate of the strength gain has been cited by Dumbleton et al. as being 10 percent of the ultimate strength occurring in 7 days (1) under normal curing conditions (moist cure at 70° F or 21° C) and about 50 percent at the end of 28 days (25). This rate of strength gain will vary with the soil, lime, and fly ash and can continue for a period of years.

Strength in a self-hardening fly ash develops rapidly when compacted immediately after mixing. The initial set times are sometimes faster than portland cement. A time delay between mixing and compaction of the fly ash-soil and water mixture can result in a significant reduction of the strength (15). A two-hour delay was found to produce a reduction in strength of one-third, with one-half of the strength lost in a four-hour delay. Adequate mixing and rapid compaction is necessary to achieve maximum benefits of the highly reactive Class C fly ashes.

Temperature

The cementation proceeds more rapidly at higher temperatures and ceases at temperatures below 40° F (4° C). However, warmer temperatures will reactivate the pozzolanic reaction. The reaction continues until the chemical compounds involved in the reaction are depleted.

ASTM C 593 specifications suggest a curing temperature of 100° F (38° C) for 7 days. An accelerated curing period of 7 days at 140° F (60° C) has also been used as an approximation of the condition of the mixture at the end of a 28-day cure at 73° F (21° C). However, certain pozzolanic reactions may occur at higher temperatures and not at lower temperatures. In addition, the relationship between age, temperature, and strength is not the same for all lime-fly ash-soil mixtures (9). Therefore, strength at the end of a 7-day, high-temperature curing period may not be a good approximation of strength after 28 days of curing at normal temperature for all lime-fly ash-soil mixtures. In the mix design process, it is recommended that all testing be based on a 28-day curing period at 73^{o} +/- 3^{o} F (21^{o} C) and100 percent relative humidity, where possible (1).

METHODOLOGY

SCOPE OF STUDY

The testing program was designed to evaluate stabilization efforts with portland cement, lime and locally available fly ash with typical soils used for base and subbase/embankment construction in Louisiana. Soils used in base courses were tested on the basis of strength gain with respect to percent additive, fly ash source and curing time. Fly ash alone and combinations of fly ash with hydrated lime were mixed with soils in various proportions and tested in unconfined compressiontests, along with cement-treated specimens as control. Vacuum saturation and indirect tensile tests were also conducted as an indication of durability. The stabilizing effects of the fly ash-lime/cement on embankment soils were tested with respect to percent additive, fly ash source, strength index, and curing time. Atterberg limits were conducted on the soils for evaluating the effects of the stabilizers on the soil plasticity and R-value tests as a measure of resistance to deformation and change in soil support values. Table 1 provides the scope and materials examined in the testing program (4).

Table 1 - MATERIALS AND TESTING PROGRAM

Soil Type	Treatment*	General Use	Laboratory Test**
Sand	C,F,F+L,F+C	Base	UC,VS,ST
Sandy Silt	C,F,F+L,F+C	Base	UC,VS,ST
Silty Clay	F,L,L+F	Embankment	A,R-value,UC,VS
Lt.-Med. Clay	F,L,L+F	Embankment	A,R-value,UC,VS

*Treatment:

C - Portland Cement
F - Fly Ash
L - Hydrated Lime

**Laboratory Tests:

UC - Unconfined Comp. (ASTM D 2166)
VS - Vacuum Saturation (ASTM C-593)
R-value - Resistance (ASTM D 2844)
A - Atterberg Limits (ASTM D 4318)

Fly Ash

Three fly ashes produced in Louisiana were included in the testing program. The power plants producing the fly ashes include Big Cajun, Nelson and Rodemacher, Figure 1. The physical and chemical properties were analyzed according to ASTM C 618. Table B-1 of Appendix B presents the test results on the fly ash used in the molded specimens of the testing program. All of the fly ashes are of type ASTM Class C and originate from the burning of subbituminous coal from the Gillette, Wyoming area. The calcium oxide present ranged from a low of 21.5 percent to a high of 27.2 percent. Table B-2 of Appendix B summarizes the properties of all three fly ashes as determined by the LADOTD Materials Testing Laboratory between April 1982 and March 1985.

Lime

A hydrated, high-calcium lime conforming to the LADOTD 1982 specifications (ASTM C 207) was used in the test specimens. Section 1018 of the specifications requires a minimum calcium oxide content of 90 percent by weight of total weight. The lime was acquired from Pelican State Lime of Morgan City, Louisiana. The properties of the lime are presented in Table B-3 of Appendix B.

Cement

Two sources of Type I portland cement conforming to the LADOTD 1982 specifications, Section 1001, were used in the study. One was produced by Lone Star Industries in New Orleans, Louisiana, and the other was from Blue Circle Inc. of Birmingham, Alabama. The physical and chemical properties are presented in Table B-4 of Appendix B.

The study was divided into two phases. Phase I included two soils commonly used in pavement bases. Phase II tested and analyzed soils used in embankment construction. Each of the soils was evaluated with respect to percent of stabilizing additive, fly ash source, strength index and curing time.

BASE SOILS AND TESTING METHODOLOGY

A nonplastic sand, A-3-0, and a slightly plastic A-2-4 sandy silt (LL=21, PI=7) were included in the testing program as two soils commonly used in base construction. Both of these soils were obtained from a borrow pit (Johnson Pit) located along the construction route of Interstate 49 in Evangeline Parish, Figure 1. The engineering properties of the untreated soils were determined using the following LADOTD test procedures:

1. TR-407-74 Mechanical Analysis of Soils (ASTM D 422 modified)
2. TR-418-81 Moisture Density Relationships (ASTM D698 modified)
3. TR-428-67 Atterberg Limits of Soils (ASTM D 4318 modified)
4. TR-430-67 pH Values of Soils

A summary of the above test results for the two base soils, the A-3 sand and the A-2-4 sandy silt, is presented in Appendix C, Table C-1 .

The experimental design for the study of the base soils is presented in Figure 2. The testing variables, materials and tests are as follows:

Soil Types:	1. A-3-0 sand 2. A-2-4 sandy silt
Fly Ash Sources:	1. Cajun Power Plant - NewRoads, La. 2. Nelson Power Plant - Westlake, La. 3. Rodemacher Power Plant - Boyce, La.
% Fly Ash:	5, 10, 15, 20, 25 and 30% by weight
% Cement:	4, 6 and 8% by weight
% Lime:	2, 4 and 6% by weight
Tests:	unconfined compression with and without vacuum saturation
Curing Periods:	7, 28, 56 days

The untreated soils were initially oven-dried at 140° F. The procedures of LADOTD TR-418-81 were used to determine the dry weight densities and corresponding optimum moisture content for all combinations of cement, lime and fly ash. These design values are summarized in Tables C-2 and C-3 of Appendix C. The combinations of soil plus cement, lime and/or fly ash were mechanically mixed dry for one minute and wet for two minutes. Afterwards, a one-minute slake time was followed by two more minutes of mixing. The standard Proctor procedure was used in molding the mixtures. In compacting the specimens, moisture contents were not allowed to vary beyond plus or minus one percent of optimum, and dry densities were maintained to within plus or minus three pounds of the theoretical dry weight density. Those specimens not meeting this criteria for moisture or density were discarded and new specimens were remolded.

The molded specimens were extruded and cured for 7, 28, and 56 days at a temperature of 73° +/- 3° F and at a relativehumidity of 90 percent or greater. At the end of a curing period the compressive strength and durability of the mix were tested according to ASTM C593. Sets of three specimens were soaked four hours and then tested in unconfined compression. An additional three specimens were conditioned in a vacuum saturation chamber followed by the unconfined compression test. ASTM C593 procedures were modified here in that the specimens were cured in the humidity room at an ambient temperature rather than the 100° F temperature called for by ASTM.

LADOTD TR 432-82 Method A or B was used in evaluating the performance of the cement stabilization of the base soils. This specification requires that the minimum cement content is that corresponding to a strength of 250 psi at a seven-day curing period. Minimum lime content for a base course is specified in LADOTD TR 433-81 as requiring a liquid limit less than 40 and a plastic index less than 10 after lime treatment. Both of the base soils, the A-3 sand and the A-2-4 sandy silt, met this requirement prior to any stabilization effort. A previous LADOTD specification TR (433-70) used a minimum strength of 100 psi as a criteria for specification of lime content. Phase I of the study used the cement-treated specimens as a control or benchmark for the base soils. Therefore, in the evaluation of the test results, Melancon et al. (4) used the 250 psi at a seven-day cure criteria in evaluating all stabilizing combinations; fly ash alone, fly ash plus lime and fly ash plus cement.

EMBANKMENT/SUBBASE SOILS AND TEST METHODOLOGY

A silty-clay, A-6(9), and a clay of medium plasticity, A-7-6(20), were tested with fly ash and blends of lime-fly ash. These mixtures were evaluated by comparing the resulting plasticity (Atterberg limits) and resistance to deformation or change in soil support values (R-value). The A-6 silty clay was obtained from a site near Baton Rouge, and the A-7-6 clay from Lake Charles, as shown in Figure 1. The testing procedures used in evaluating the raw soil included the following LADOTD methods:

1. TR-407-74 Mechanical Analysis of Soils (ASTM D422 modified)
2. TR-418-81 Moisture Density Relationships (ASTM D698 modified)
3. TR-428-67 Atterberg Limits of Soils (ASTM D4318 modified)

The results of tests conducted on the natural soil are summarized in Table D-1 of Appendix D.

The experimental design for the study of the embankment soils is presented in Figure 3. The testing variables, materials and tests included the following:

Soil Types:	1. A-6 silty clay
	2. A-7-6 medium plastic clay
Fly Ash Sources:	1. Cajun Power Plant - New Roads, La.
	2. Nelson Power Plant - Westlake, La.
	3. Rodemacher Power Plant - Boyce, La.
% Fly Ash:	15, 20, 25, and 30 % by weight
% Lime:	2, 3, 4, and 5 % by weight
Tests:	Atterberg Limits, R-Value Tests, Unconfined Compression, and Vacuum Saturation Strength Tests

The means of evaluating the minimum lime content for treatment of soil is designated in LADOTD TR 433-81. For soils to be used as a base course, the liquid limit and plastic index following lime treatment shall not exceed 40 and 10, respectively. For soils to be used as a subbase, the post-lime treatment values for the liquid limit and plastic index shall not exceed 40 or 15, respectively. This same criterion is used as a gauge in evaluating fly ash as a replacement or partial replacement of lime. The LADOTD TR 416-81 procedure, Determination of Quantity of Lime Needed For Conditioning Plastic Soils, was used to obtain the resulting liquid and plastic limit of the treated soils. This method requires that the raw soil be conditioned to a moisture content approximately equal to its plastic limit prior to mixing with the lime. It is broken up into soil clods small enough to pass the 1/2 - in. (12.5 mm) screen. The lime is added and is gently mixed to assure that each clod is coated. Approximately 500 grams is compacted in one layer in a 1/30 - cu ft (1 cu dm) mold using 15 blows of a 5.5 lb (2.5 kg) hammer. The specimen is extruded, sealed in a plastic bag and placed in a moist room to age for 72 hours. At the end of the curing period, the sample is removed from the moist room, broken up, dried and tested for liquid limit, plastic limit and plastic index according to LADOTD Designation TR 428.

As a means of predicting stabilization efforts on the performance of these two soils when used in an embankment or subbase, the R-value (resistance value - ASTM D 2844) was determined for the different mixtures. This test provides a measure of the resistance to plastic deformation of the compacted materials using the Hveem stabilometer. The method for determining the R-value consists of five parts:

1. Sample preparation
2. Compaction of test specimen
3. Measurement of exudation pressure
4. Measurement of expansion pressure
5. R-value determination

Mixtures of the soil plus lime and/or fly ash were initially conditioned by slaking at a moisture content slightly below optimum moisture content for a period of 72 hours. At the end of the slake period, the test specimens were molded with a mechanical kneading compactor in a 4- in. (10.16 cm) inside diameter by 5-in. (12.7 cm.) high steel mold. Specimens 2.5 in. (6.35 cm) thick were fabricated applying 100 tamps with a foot pressure of 350 psi.

The exudation pressure required to squeeze moisture from the soil was determined on the molded specimens. This was followed by the test for expansion pressure developed in the presence of free water. This portion of the test procedure requires approximately 24 hours. The last measurements involve the determination of the R-value with the Hveem stabilometer. A few specimens were subjected to an R-value retest at the end of an additional 7 days to observe any increase that may have occurred.

A limited number of unconfined compression tests and vacuum saturation strength tests were conducted on the A-6 soil. The unconfined compression tests were conducted after a curing period of 7 and 28 days in order to obtain an approximate measure of the magnitude of compressive strength and the gain in strength with respect to time. The strength tests after the specimens are conditioned with the vacuum saturation chamber provides a measurement of durability in terms of strength loss.

DISCUSSION OF RESULTS

EVALUATION OF TEST RESULTS FOR BASE MATERIALS

The test results on the two base soils, an A-3 sand and an A-2-4 sandy silt, with the four combinations of additives and sources of fly ash (fly ash alone, lime plus fly ash, and cement plus fly ash) are summarized in Appendix C. Using the acceptance criteria cited above (250 psi at a 7-day cure), the test results were presented in an interim report (4). The fly ashes used as the only stabilizer were found in general to be ineffective except at levels exceeding 20 percent. The lime plus fly ash "acting as a pozzolan was not notably effective." The A-3 sand met the acceptance criteria when mixed with 4 to 6 percent lime and 15 percent fly ash. The A-2-4 sandy silt did not reach the minimum strength required at any of the mix combinations used. The percentages of cement alone required to produce the 250 psi unconfined strength was 8 percent with the A-3 sand and 4 percent with the A-2-4 sandy silt. The cement-plus-fly ash combinations were also found to produce acceptable strength except at low percentages (4 percent cement and 5 percent fly ash). The concluding statements of the interim report stressed the inconsistency and variance of the test results between and within the three sources of fly ash tested. This report will review and evaluate the test results in an attempt to identify or explain some of the trends or inconsistencies that were present.

Moisture-Density - Base Soils

The addition of fly ash to the A-3 sand significantly increased the maximum dry density and decreased the optimum moisture content throughout the range of fly ash percentages tested, Table C-2. The curves for maximum density and optimum moisture content versus the percentage of fly ash additive for all fly ash sources used in this study are similar, Figures 4, 5 and 6. Overall, the variation of maximum dry density with percent fly ash addition in the A-3 sand is almost a linear relationship (approximately 0.95

pcf/1% fly ash). There was, however, a slight reduction in the gain in density at a specific fly ash percentage (10% Rodemacher, 15% Nelson, and 20% Cajun). The reduction in moisture content is attributed to the spherical shape of the fly ash particles in the voids, which "lubricates" the mix and aids in the densification efforts.

Other mix combinations of cement plus fly ash and lime plus fly ash also produced gains in density, Table C-2 and Figures 7 through 12. However, the addition of lime or cement affect the density differently. Both lime and cement seem to contribute or aid in increasing the density at the lower percentage levels. As the stabilizing mix approaches the point where the voids are becoming filled (20 to 25% fly ash), increases in density with additional cement or lime seem to peak. For example, in Figures 13 and 14, a maximum density is produced with a specific combination of lime and Cajun fly ash as opposed to that for cement combined with fly ash. The optimum moisture content for the lime-plus-fly ash and cement-plus-fly ash additives is also less than that required by the raw soil but is slightly higher than that required by the A-3 sand and fly ash alone, Table C-2.

The addition of fly ash to the A-2-4 sandy silt produced a slight increase in dry density of between 3.3 and 6.9 pcf with the different fly ashes, Table C-3 and Figures 15, 16 and 17. The maximum density occurred at a fly ash percentage of about 20 percent for all three fly ashes. Additional percentages beyond that point produced a decrease in density. The maximum density seems to coincide approximately with the minimum value of the optimum moisture content. A decrease in density occurs with the addition of lime to the mix, i.e., less than the maximum dry density of the raw soil, Figures 18, 19 and 20. There is also a decrease in the maximum dry density of cement- fly ash combinations with respect to those of the A-2-4 sandy silt and fly ash alone. There is not as great a reduction in density as produced with the addition of lime, however. Shown in Figures 18, 19 and 20, dry densities smaller than that of the raw soil occur for low percentages of fly ash in the cement-fly ash combinations, i.e., higher ratios of cement-to-fly ash.

A slight decrease in optimum moisture content also occurs with the addition of fly ash alone to the A-2-4 sandy silt, Figures 15, 16 and 17. More water is required with the cement-plus-fly ash and lime-plus-fly ash combinations. The cement-plus-fly ash moisture requirements are plus or minus one percent that of the raw soil, Table C-3. The lime-plus-fly ash combinations required 2 to 3 percent more water.

Using Talbot's relationship for the ideal gradation required for maximum density, a grain-size analysis was made.

$$P = (d/D)^{0.5}$$

where P = percent finer by weight for grain size

d = grain size being considered

D = maximum grain size

The original gradation curves for Rodemacher fly ash, the two soils (A-3 sand and A-2-4 sandy silt) and the resulting gradation curves for the blended soils-plus-Rodemacher fly ash combinations are shown with reference to the ideal gradation curve (for maximum density) in Figures 21 and 22, respectively. Note in Figure 21 that additional fly ash appears to bring the resulting gradation curve closer to the ideal curve for maximum density of the A-3 sand; i.e., maximum density is achieved at 25 percent fly ash.

Figure 22 indicates that the addition of approximately 15 percent fly ash would maximize the dry density of the A-2-4 sandy silt compacted with the Rodemacher fly ash. Test results showed that 20 to 25 percent fly ash actually provides the maximum dry density, Figure 17. However, considering the assumption of an exponent of 0.5 in Talbot's gradation expression for maximum density (values between 0.35 and 0.5 have been suggested), incomplete information concerning the full range of fly ash particle sizes and experimental variations, this analysis of the grain-size distribution does provide insight into the resulting densities and the test performance of the mixture. This supports the requirement and need for conducting a mechanical stability analysis with coarse-grain soils, including medium-to-fine sands and coarse silts.

Unconfined Compressive Strength - Base Soils

Strength variation with increasing fly ash reflects the changes observed in density for both of the base soils, Figure 23. Greater strengths were developed in the A-3 sand with increasing fly ash, Table B-4. The rate of strength gain exceeded that of the rate of increase in density, as shown in the figure. The fly ash forms a matrix for the coarser particles in the A-3 sand. With sufficient matrix to float the coarser aggregate fraction, a higher compacted density of the matrix was achieved and produced higher strengths with greater durability. The quality of the stabilized soil, as measured by its strength and durability tests, is closely related to the quality of the cementitious matrix of the mixture. A comparison of the strengths and densities measured for the three fly ashes, Cajun, Nelson and Rodemacher, is shown in Figure 24. The data points of each curve represent sequential increases in fly ash, i.e., 10, 15, 20 and 25%.

The strength variation of the A-2-4 sandy silt with fly ash parallels that of its variation in dry density, Figure 23. Additional fly ash beyond 20 to 25 percent produces a reduction in density with little or no increase in strength. The silt and clay particles present do not seem to be reactive with the fly ash or possibly even interrupt the cementitious matrix that is partially formed by the self-hardening fly ash. The amount of lime in the fly ash that is free to react with the fines of the

A-2-4 soil may be insufficient.

The strengths developed with the base soils and portland cement are presented in Table C-5. There was generally a gain in strength corresponding to the addition of lime or portland cement with fly ash and the two soils, Tables C-6 and C-7 and Figures 25 through 30. However, there are some discrepancies observed in the test results for the lime-plus-fly ash mixtures and the A-2-4 soil, Figure 28. In comparing the variation in strength for the lime-plus-fly ash mixes of both soils, they were not always consistent. The strength gain with addition of lime was not as significant in the A-2-4 sandy silt as that occurring in the A-3 sand. For example, in Figure 28 the addition of lime in the series of tests with the Cajun fly ash did not produce expected results. There appears to be a drop in strength with some percentages of lime and a reversal between the 5% Cajun fly ash strength and that of the 10% test series. This inconsistent behavior may possibly be due to a bad series of test specimens. However, the A-2-4 sandy silt in general did seem to have more erratic test measurements than did the A-3 sand. The clay and silt-size fraction of the A-2-4 sandy silt did not appear to have strong pozzolanic characteristics. The fines of the natural soil may interfere and result in discontinuities within the fly ash and fly ash combinations of additives. In the A-3 sand the density, i.e., increased percentages of fly ash, seems to be more critical for strength gain than does the added lime.

The variation of the 28-day unconfined compression strength for the cement plus fly ash mixtures and the A-3 and A-2-4 soils can be seen in Figures 31 through 36. The curves seen in these figures appear to take a shape formed by shifting and adding the strength curves for the cement and the fly ash used as lone stabilizers. Note that for the A-3 sand, Figures 31, 32, and 33, the unconfined compression strength of the different cement-plus-fly ash mixtures branch off of the cement strength curve (0% fly ash) and run in a direction more or less parallel to the fly ash strength curve (0% cement). The cement-plus-fly ash strength tests demonstrate that the combination of both materials, portland cement and fly ash, contribute greatly to the cementing properties of the matrix. Similar trends are seen in the cement plus-fly ash mixtures of the A-2-4 sandy silt. However, the magnitude of the developed strengths is somewhat more erratic. Also, the cement-plus-fly ash strength branch curves for constant percentages of cement show very little strength increase with additional fly ash, which is consistent with the performance of the A-2-4 soil with fly ash alone.

Curing Time - Base Soils

The molded specimens were cured for periods of 7, 28 and 56 days at 73^{o} +/- 3^{o} F. Plots of the strength performance for the different curing periods and fly ashes are presented in Figures 37, 38 and 39. The results are similar, but variations between the measured strengths of the different fly ashes and among the test results of the individual fly ashes do occur. As can be seen from Figures 37 and 39, there is not a great difference between the 7-day strength and the 56-day strength that occurs with the A-2-4 sandy silt. In some cases, specimens with longer curing times produced lower strengths. This again is credited to the incompatibility of the fines in the A-2-4 soil with the fly ash.

Only at higher fly ash percentages, i.e., 20% and 25%, was any significant gain in unconfined strength achieved for the A-3 sand, Figures 37, 38 and 39. This is demonstrated again in Figure 40 for the Cajun fly ash. Note that in this figure the ultimate strength achieved in 56 days is approximately that of the 7-day strength. This is especially true for the lower fly ash percentages of 10 and 15%, Figures 37, 38 and 39. Greater gains in strength with longer curing times take place at the higher fly ash percentages of 20 and 25%. The initial strength gain at 7 days is attributed to the self-hardening characteristics of the calcium silicates present. That additional strength with longer curing periods is credited to pozzolanic activity. In Figure 40 the initial slope of the curve is an indication of the immediate self-hardening characteristics of the particular fly ash. This would depend on the quantities of calcium silicates present in the fly ash and the overall quality of the mixture; i.e., the continuity and compacted density. The spread or separation between the 7-, 28- and 56-day curing curves would indicate the pozzolanic reaction or potential of the individual fly ash, i.e., the amount of free lime present and the pozzolanic characteristics of the fly ash particles and the fines of the soil.

Comparisons of the A-3 sand with cement alone and with the Rodemacher fly ash alone are shown in Figure 41. The 25% fly ash mixture compares quite well with the 8% cement and provides a denser mixture which may be more durable. With the current cost of cement being approximately three times that of fly ash, fly ash would appear to be competitive on a materials cost basis. Figure 42 provides the strength variation with time for the A-3 sand mixed with 4% lime or cement and 10% fly ash for the three fly ashes. Except for the 56-day strength with the Rodemacher fly ash and lime mixture, the strength-time curves are consistent for the three fly ashes. Noting also the curing times used (a maximum of 56 days), the cement mixture's rate of strength gain is greater than the lime mixture's. Additional testing using longer curing durations or accelerated curing with elevated temperatures for determing long-term strengths should possibly be considered in the future.

Durability - Base Soils

Specimens were conditioned in a vacuum saturation chamber and tested for compressive strength according to ASTM C 593 specifications with the exception that they were cured in a humidity room at 73^{o} +/- 3^{o} F instead of the 100^{o} F specified in the ASTM procedure, Tables C-8 and C-9. A comparison of the loss or gain in strength between specimens subjected to this procedure and those not conditioned provides a relative measure or indication of the durability of the mix. Figures 43 and 44 provide the comparison of durability for all mixtures on the basis of loss in strength. As can be noted in Figure 43, there does not appear to be any consistent loss of strength of the A-3 sand beyond what might be expected as experimental variation. However, the A-2-4 sandy-silt demonstrates a consistent loss in strength for the vacuum saturation test, Figure 44. Loss of strength with vacuum saturation for cement alone and Cajun fly ash alone is presented in Figure 45 for the A-3 sand. The vacuum saturation tests for the A-2-4 sandy silt with cement or fly ash alone are shown in Figures 46 and 47.

EVALUATION OF TEST RESULTS FOR EMBANKMENT SOILS

The test results of the two embankment soils, the A-7-6(20) silty clay and the A-6(9) silty clay with the combinations of additives and three sources of fly ashes (lime alone, fly ash alone and lime-plus-fly ash), are summarized in Tables D-1 through D-7 of Appendix D. As previously noted, the group index for the A-7-6 and A-6 silty clays was 20 and 9, respectively. The group index, GI, provides an approximate evaluation within a group of the "clayey granular materials" and the "silty clay materials." The magnitude of the group index (ranging from 0 to 20) is inversely related to the supporting value of the soil; i.e., a GI of 0 is a "good" subgrade material, and a GI of 20 indicates a "very poor" subgrade material. A linear correlation is assumed to exist between the soil support value, S, and the group index as shown in Figure 48. The soil support values, S, for the A-7-6 and A-6 soils on the basis of their respective GI numbers are approximately 2.5 and 4.5.

The properties of both soils were improved with the addition of the stabilizing agents in terms of measurements of support and in modification of their plastic properties. The difference in the material properties brought about by the addition of the lime or fly ash, alone or in combination, can be seen in the test results. However, variation does exist within the test results of the various stabilizing groups and combinations. This can be attributed to the inherent variation of the materials and to testing errors and operator judgment.

Effects on Dry Density - Embankment Soils

Addition of lime as a lone stabilizing agent produces a less dense mix. This was true for both soils, A-7-6 and A-6. A somewhat greater reduction in the maximum dry density occurs with the A-7-6 silty clay. Modification of the texture of the A-7-6 soil as a result of the soil-lime reaction probably accounts for these changes. This textural modification also takes place in the A-6 silty clay but not to the extent of the more active A-7-6 clay. The densities obtained when the fly ashes are mixed with the soil vary between being somewhat greater to somewhat less than that for the untreated natural soils, A-7-6 and A-6. Though not necessarily conclusive, from Figure 49 it appears that the resulting densities obtained by adding the fly ashes to the A-7-6 soil are equal to or less than the natural soil. However, the fly ashes mixed with the A-6 soil produce approximately the same density as the untreated soil, Figure 50. A plot of the gradation curve for the two soils and two of the fly ashes, Rodemacher and Nelson, is presented in Figure 51.

Combining the lime and fly ash with the A-6 soil reduces further the density of the mix, Figures 52, 53 and 54. In addition to the effects of the grain size variation existing between the components, further flocculation and agglomeration occur not only with the soil particles but also with the reactive fly ash.

Soil Plasticity - Embankment Soils

In a lime-reactive soil, immediate changes in soil plasticity, workability, and an initial gain in uncured strength and load deformation properties occur with the addition of lime. The success with which a soil can be modified is measured in terms of the resulting plasticity index (maximum PI of 10 for bases and 15 for subbases) and the liquid limit (maximum LL of 40 for bases and subbases), LADOTD TR 433-81. A review of the Atterberg tests for the two soils in general seems to demonstrate a distinct advantage in the use of lime, either alone or as a partial component of stabilizing additives used with clay soils. The response of the two soils, the A-7-6 and the A-6, is somewhat different and will be discussed separately.

A-7-6 Silty Clay

Lime added to the A-7-6 silty clay produces a dramatic decrease in the liquid limit and increases greatly the plastic limit, resulting in a much reduced plasticity index, Figure 55. The "lime fixation" for this soil, i.e., the amount of lime required to produce a constant value of the plastic limit, is 4%. This also corresponds to a PI of 4 (raw soil PI=40) and liquid limit of 38 (raw soil LL=60). The percentage of lime required to meet LADOTD TR 433-81 specifications would be 3%.

The addition of fly ash as a lone stabilizer and in the quantities used , 15 to 30% by dry weight, produced a significant drop in the liquid limit of the A-7-6 soil, Figure 55. Some of the liquid limit values were less than those obtained with lime alone at the highest percentages (30%) of the fly ashes used. However, surely the constituents of the basic soil are diluted at these high additive levels, i.e., the soil is really a blend of two materials which produce a third with different characteristics, as opposed to just the chemical alteration brought about by the addition of the fly ash. The plastic limit of the A-7-6 soil was only slightly increased (5% or less). As previously discussed, the spherical-shaped fly ash seems to act as a lubricant, reducing the amount of moisture required in compaction of soil-fly ash mixtures. This theory could possibly explain the resulting Atterberg limits. The addition of fly ash makes the soil more "fluid acting" and counters the effects of the CaO in the Class C fly ashes. The fly ash-soil mixture, being more fluid, produces a lower liquid limit but makes it more difficult to obtain a higher plastic limit. The performance of all three fly ashes, Cajun, Nelson and Rodemacher, were similar. A plot of plasticity versus the percent stabilizer for the three fly ashes and the lime is shown in Figure 56. The relation between the percent fly ash and PI is almost linear. Twenty-five to thirty percent fly ash is required to satisfy the LL and PI specifications of LADOTD TR 433-81.

Many if not most of the changes occuring in the plastic properties can be attributed to the diluting effects of the clay constituents by the addition of a non-plastic material, i.e., the fly ash. Theoretical relationships between the liquid and plastic limits and the clay content were developed by Seed, et al. (26). It was shown that the variation of the liquid limit with the clay content can be expressed as

$$w_{LL} = (C/100)\ w_{CLL}$$

where w_{LL} = liquid limit of the soil mixture

w_{CLL} = liquid limit of clay fraction

C = percent of clay particles

Assuming that the fly ash consists of particle sizes in the fine sand to silt range (27, 28), Table B-1, and that the only source of clay-size particles is from the A-7-6 clay alone, then

$$C = (1 - (FA/100))(\%\ \text{clay in raw soil})$$

where FA = percent fly ash

Table 2 compares the liquid limits measured in laboratory tests with the three fly ashes to those predicted on the basis of variation in the clay content. Average values of the percent clay in the raw soil were assumed to vary between 55 and 60% and the mean value for the liquid limit of the raw soil was used. One should also consider that the values reported are based on average values, and that the predicted liquid limits will vary somewhat by the selection of these numbers.

TABLE 2

LIQUID LIMIT: PREDICTED AND MEASURED

Percent		Liquid Limit				
FA	C	Raw Soil	Cajun	Nelson	Rodemacher	Predicted
0	55	60				
15	47		47	44	41	47-51
20	44		42	43	39	44-47
25	41		35	41	37	41-44
30	39		34	39	31	39-43

Similarly, Table 3 can be composed for comparing the outcome of the laboratory plastic limits with theoretical predictions on the basis of clay content. The relationship used includes (25):

$$w_{PL} = (C/100)\ w_{CPL}$$

where w_{PL} = plastic limit of soil mixture

w_{CPL} = plastic limit of clay fraction

or

$$w_{PL} = 0.5C \qquad \text{for } C > 40$$

TABLE 3

PLASTIC LIMIT: PREDICTED AND MEASURED

Percent FA	C	Raw Soil	Plastic Limit Cajun	Nelson	Rodemacher	Predicted
0	55	20				
15	47		23	18	18	16-24
20	44		23	24	21	15-22
25	41		20	23	19	14-21
30	39		21	27	21	13-20

The above analysis does involve a number of assumptions. However, the outcomes of the laboratory Atterberg tests do not vary far from what would be expected by blending a fine-grained, non-plastic soil to the A-7-6 clay. Thus, it appears that the changes in the plastic properties of the fly ash-soil mixtures for the fly ashes used can be credited more to a decrease in the clay fraction than to chemical alteration of the clays. This is not to say that there isn't some effect or change due to whatever percentage of free lime is available. There does not appear to be enough free lime to provide substantial changes, however.

By combining lime and fly ash, liquid limit values as low as and lower than those produced by lime or fly ash alone were obtained, and higher plastic limits than those with only the fly ash resulted. The plastic limit values varied but were almost equal to those corresponding to the same lime content when used as a lone additive, i.e., 2% lime in a lime-plus-fly ash mix versus 2% lime only, Figures 57, 58, and 59. The increase in the plastic limit is mostly influenced by the amount of lime in the combined lime-plus-fly ash mix. The lime-plus-fly ash combination meeting the LATR 433-81 requirements for a subbase material is 2% lime with 10% fly ash for all fly ashes. The variation of the plasticity index with respect to percent of lime (2% and 4%) plus fly ash are shown in Figure 60. Note that in order to duplicate the PI as determined by the lime fixation (PI=4) for the A-7-6 clay, the same percentage of lime (4%) is required in the lime-plus-fly ash mix as that with lime alone. Again, the performance of the individual fly ashes varied to some extent but was, in general similar.

A-6 Silty Clay

The liquid limit and the plasticity index of the natural, untreated A-6 soil (LL=31 and PI=13) were less than the maximum values specified by LADOTD TR 433-81 for subbases. However, the soil was somewhat lime-reactive, and modification of the plastic properties of the soil did occur with the addition of lime. The plasticity index was reduced from 13 to 7 with 2% lime and to a PI of 4 at 5% lime. The lime produced a slight increase (1 to 3%) in the liquid limit and increased the plastic limit (8 to 10%), Figure 61.

There is little variation in the results between the Atterberg tests conducted using the different fly ashes, Cajun, Rodemacher and Nelson. A comparison of the liquid limit plots for the three fly ashes, Figure 61, seems to indicate no change from that of the untreated soil. There may have been a slight increase in the plastic limit brought about by the addition of the fly ash. Overall, the quantity of fly ash used, 15% to 30%, doesn't seem to have much effect on this soil's liquid or plastic limit. The addition of fly ash to the soil didn't have much more of an effect than would the addition of more A-6 silty clay to the original sample.

The addition of a lime-fly ash mixture to the soil produced a response in the soil's plasticity that was similar to that resulting with the addition of lime only. The lime-fly ash Atterberg test results, however, were more erratic, Figures 62 and 63. Again, the amount of fly ash doesn't seem to affect the outcome, but the amount of lime does. The fly ash did not improve the plastic properties or modify this A-6 silty clay.

<u>Soil Support Resistance Value - Embankment Soils</u>

The resistance or R-values of the A-7-6 and A-6 silty clays were greatly improved with addition of the lime, fly ash and the lime-fly ash combination. In comparing the R-values of the various stabilizing agents and combinations used in the testing program, Table D-6 and D-7 of Appendix D, with the soil support value correlations of Figure 48, it can be seen that the A-7-6 clay goes from a very low soil support value, S=2.5 (untreated R-value < 5), to a soil support value ranging from 6 to 9.5. The A-6 silty clay soil support value was improved from 5 to values that range between 7.5 and 9.5. However, the addition of lime or lime in combination with fly ash appears to produce the greatest gain in the measured R-values for both soils.

The testing procedures may have had some effect and should be considered in evaluating the results. As previously discussed, the test specimens were compacted after being mixed with the stabilizing agents and slaked with water for a 72-hour period. Within the following 24 hours, they were tested for exudation pressure, followed by the determination of expansion pressures. The R-values were then measured with the Hveem stabilometer approximately 24 hours after compaction or 4 days after being initially mixed with the stabilizing agents and slaking water. This period of time is insufficient to assess the pozzolanic gain in strength, but the stabilometer test does and has been used to characterize uncured soil-lime mixtures. Studies by Alexander, 1976, have indicated that the R-values of the uncured soil-lime mixture are not necessarily indicative of the ultimate or cured support value of the treated soil (<u>2</u>). It is also possible that the slaking or aging period used is incompatible with the set time of the Class C fly ashes. The initial set times of this type of fly ash have been reported in other studies to occur very rapidly (7, 15). Laboratory tests conducted by the Louisiana Transportation Research Center with slurries of sand mixed with Cajun, Nelson and Rodemacher fly ashes measured short initial set times ranging from 21 minutes to 3 hours and 20 minutes. In this particular series of tests, Cajun and Rodemacher fly ashes had the shorter times for initial set, and the Nelson fly ash had the longest. The rapid initial set has been attributed to some portion of the CaO found in the Class C fly ash existing in the form of tricalcium silicates similar to portland cement (7, 9). Thus, to get the full advantage of the Class C fly ash, the soil should be quickly mixed and compacted (15). Mixing the fly ash, soil and water with a 72-hour slake period in the R-value test procedure is probably counterproductive with respect to the initial set of the Class C fly ashes.

The uncured lime-soil and fly ash-soil mixtures produced an immediate improvement in the strength and deformation properties of both soils, with the most significant gain occurring in the A-7-6 clay, Figures 64 through 69. Treating the A-7-6 soil with lime or with a lime-fly ash mix produced final R-values almost as high as those of the treated A-6 silty clay (approximate R-values of 77 vs 85), the A-7-6 being the most lime-reactive of the two soils. The performance of the individual fly ash can be seen and compared from plots in Figures 64 through 70. Lime was the most effective stabilizing agent, although some small gains in the R-values are achieved by combining the lime and fly ash in some cases, i.e., Cajun fly ash in Figures 64 and 67. Larger quantities (30%) of fly ash added to the lime did not show any improvement over the smaller proportions (10%).

Some of the A-7-6 specimens were retested for their R-value after an additional 7-day curing period. The test results are shown in Table D-6, with the most complete group graphed in Figure 70. The results varied but consistently showed a significant gain in a relatively short time. Since these test specimens had been previously subjected to loading in the stabilometer in the initial R-value test, the second R-value determined at the end of an additional 7-day curing included some autogeneous healing in addition to further development of the pozzolanic reaction. Perhaps test specimens that had not been subjected to a previous test might have developed an even greater R-value than those shown.

CONCLUSIONS

Class C fly ash has been shown in the literature to be a beneficial stabilizing agent for fine- and coarse-grained soils. However, as previously noted (4), a significant amount of variation exists in comparing test results between and among fly ash sources under "identical" test conditions. In using this material, it is important that one understand the general character of fly ash and those factors, i.e., soil types, construction techniques, etc., that will influence the outcome of the end product. With that in mind, the following conclusions, both general and specific, are made based on the laboratory testing program, the materials employed and the evaluation criteria as presented.

BASE SOILS

The density and moisture requirements for achieving the desired compaction have a unique relationship with the soil's gradation and that of the fly ash. There is some indication that the quantity of fly ash required for the maximum density corresponds to a minimum optimum moisture. Significant gains in density occur in granular soils that are void of finer particles, i.e., the A-3 sand. However, larger quantities of fly ash are required in order to maximize the density. Since the fly ash consists of particles in the fine sand and silt range, it may not be possible to greatly improve the density of silty sands or sandy silts, i.e., the A-2-4 soil in this study. Blending techniques using the individual gradation curves of the soil and fly ash can produce a well-graded mix that minimizes the porosity and provides maximum density. The addition of lime and cement reduces the density of the soil-fly ash mix, decreasing the maximum density below that of the compacted raw soil in the A-2-4 sandy silt.

Using the stated acceptance criteria (250 psi with a 7-day cure), the following percentages of stabilizing agents were required for the selected soils:

A-3 Sand Fly Ash: 20 to 25%

Lime + Fly Ash : 4 to 6% lime + 15% fly ash

Cement + Fly Ash: all percentages tested except 4% cement + 5% fly ash

Cement: 8%

A-2-4 Sandy Silt Fly Ash: none meeting criteria

Lime + Fly Ash: none meeting criteria

Cement + Fly Ash: all percentages tested

Cement: all percentages tested

Proportioning the A-3 sand and fly ash for maximum density produces a corresponding gain in strength that is greatly acceleratedas the blend approaches its maximum density. A larger percentage of fly ash (20 to 25%) is required to produce the strength level within the specified curing period but may be economically competitive with that percentage of cement required (8%). The same is also true for the A-3 sand-lime-fly ash combination. Fly ash and the A-2-4 sandy silt did not perform well. However, fly ash did increase the strength when used in combination with cement. Success in stabilizing a silty sand or sandy silt with fly ash will depend on the quantity and reactive properties of the fine material (< #200 sieve).

The unconfined compression tests on the A-3 sana specimen subjected to vacuum saturation indicate a very durable material. The addition of the self-hardening fly ash produced a continuous cementitious matrix. The non-reactive fines of the A-2-4 sandy silt, however, seem to diminish the stability and continuity of the fly ash matrix or that formed with the cement when used alone. As a result, the strengths measured after the specimens were subjected to vacuum saturation conditions were consistently lower.

These Class C fly ashes seem to work best when used in the dual role as a matrix filler and cementing agent.

EMBANKMENT SOILS

The blending of fly ash with the two soils tested (A-6 and A-7-6 clays) produced mixed results with respect to density. The measured densities of the soil-fly ash mixture ranged from values that slightly exceeded to values that were somewhat smaller than the maximum density determined for the untreated raw soil. The addition of lime, either alone or in combination with fly ash, consistently produced less dense specimens.

LADOTDTR 433-81 was used as a means for evaluating themodifying effects of the clay's plasticity, i.e., maximum LL of 40 and maximum PI of 10 and 15 for bases and subbases, respectively. The Atterberg limits of the untreated A-6(9) silty clay satisfy the subbase criteria. The results of modification efforts on the plasticity of both soils produced the following proportions:

<u>A-6(9) Clay</u>	Fly Ash:	20% (base requirements)
	Lime:	2%
	Lime + Fly Ash:	< 2% lime + fly ash
<u>A-7-6(20) Clay</u>	Fly Ash:	30% (base criteria, Rodemacher only)
		25-30% (subbase criteria)
	Lime:	3% (base and subbase criteria)
	Lime + Fly Ash:	varies, approximately 2% lime with 10% fly ash

Alteration of the plastic properties of the clays with fly ash alone is mostly credited to a decrease in clay content corresponding to the high percentages of fly ash used. The amount of free calcium available in the fly ash for the cation exchange and flocculation-agglomeration reactions is insufficient to effectively compete as a substitute for lime.

The soil support resistance as measured by the stabilometer test was greatly improved with the addition of fly ash and lime-fly ash to both soils. The most dramatic improvements in soil support were achieved in the poorest soil, i.e., the A-7-6(20) clay. Both soils improved greatly; there was more than a 200% increase in the R-value for the A-6(9) clay, and the A-7-6(20) clay went from an R-value of less than 5 to as much as 81 with lime plus fly ash. However, lime alone performed almost as well without the fly ash.

Unless proven economically advantageous, there doesn't appear to be any advantage for using fly ash over lime in thestabilization of clays.

RECOMMENDATIONS

1. Where proven economically advantageous, soil stabilization with locally produced fly ash should be considered as an alternate method for improving poor soils.

2. The variations of the Class C fly ash properties (among and between sources) and the significance of construction operations should be understood as they affect the end product. It is essential that each instance of using fly ash alone or with another additive be evaluated on a case-by-case basis. Costs should be considered along with engineering properties.

3. Better testing techniques and criteria for evaluating the performance of fly ash as a stabilizing agent should be developed and adopted. Longer curing tests or accelerated testing techniques should be used to evaluate the potential gain in pozzolanic strength. Mixture design criteria similar to that used for lime (29) in the stabilization of fine-gained soils should be considered. For coarse-gained soils, initial considerations should involve a gradation analysis in an attempt to achieve maximum density. The mixture proportioning concepts presented in FHWA-DP-59-8 (31) provide an approach consistent with the findings for the sands in this study and should be used as a partial guide for mix design. The steps involved are presented in Table C-10 of Appendix C .

4. Field applications with base courses should be evaluated to determine performance and construction criteria. The Procedures required to incorporate two additives into a base course of acceptable quality should be determined.

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APPENDIX A

FIGURES

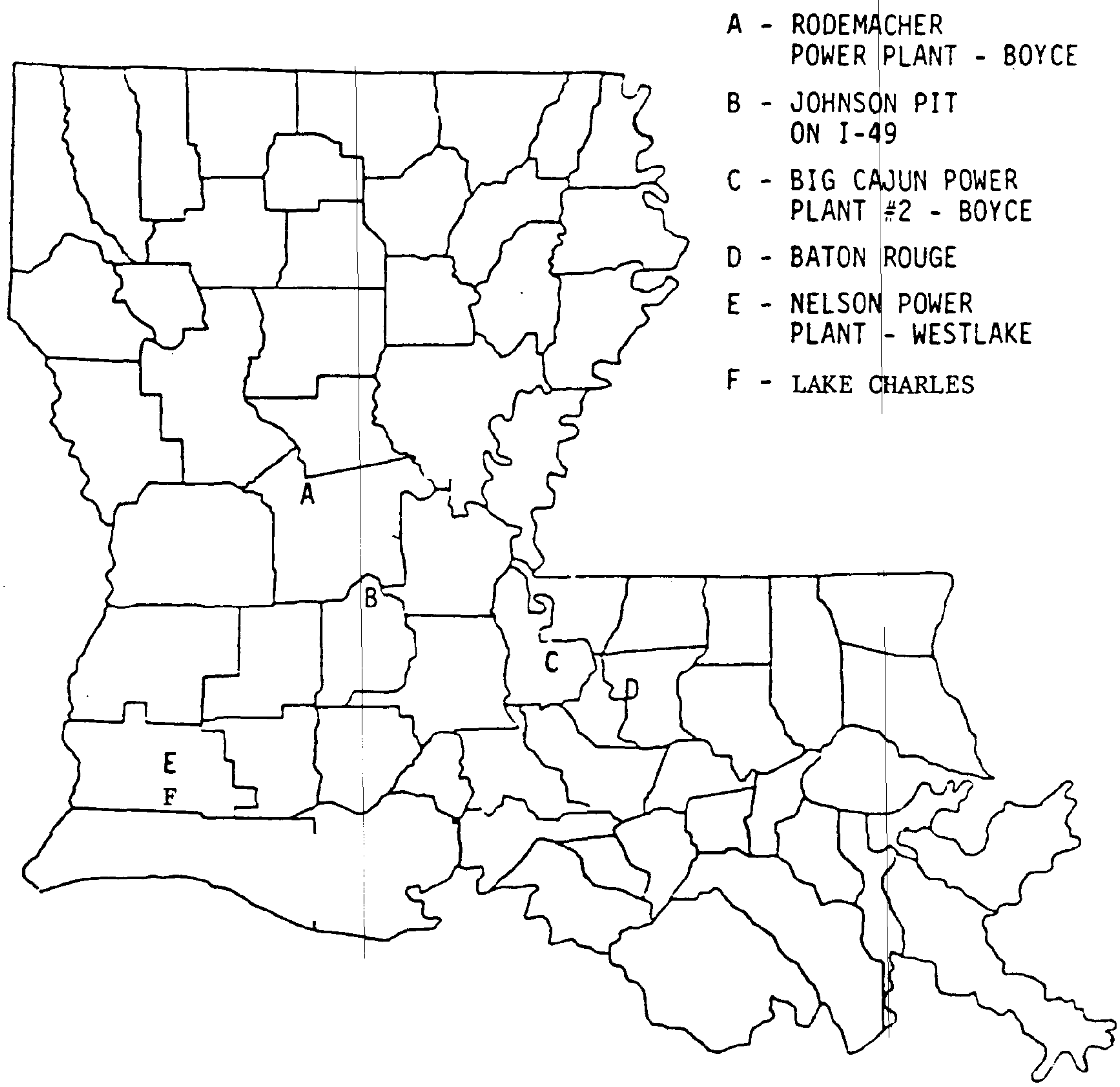
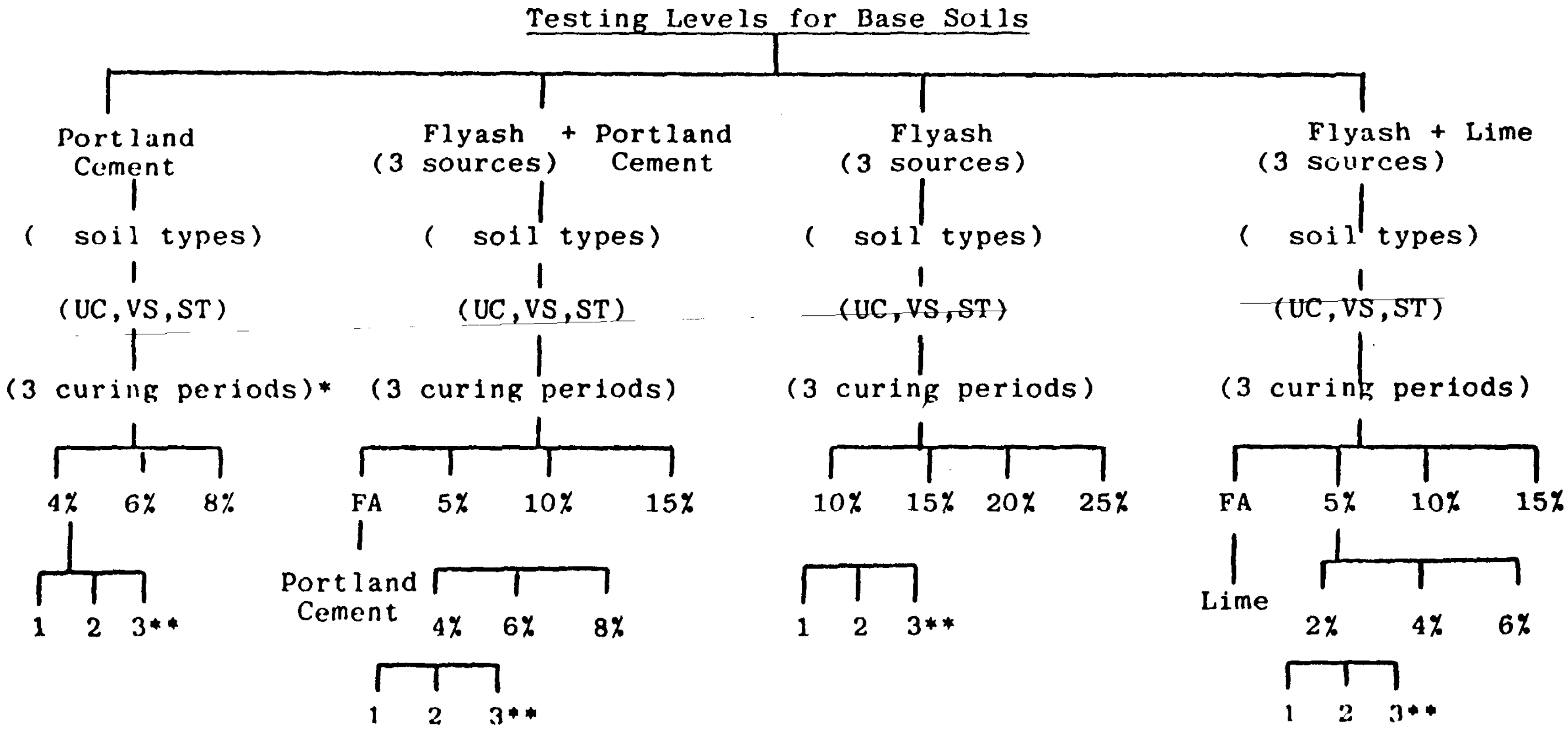


FIGURE 1. Louisiana state map - material locations



*Specimens tested after 7, 28, 56 days curing.

** Replicates.

FIGURE 2. Experimental design - base soils

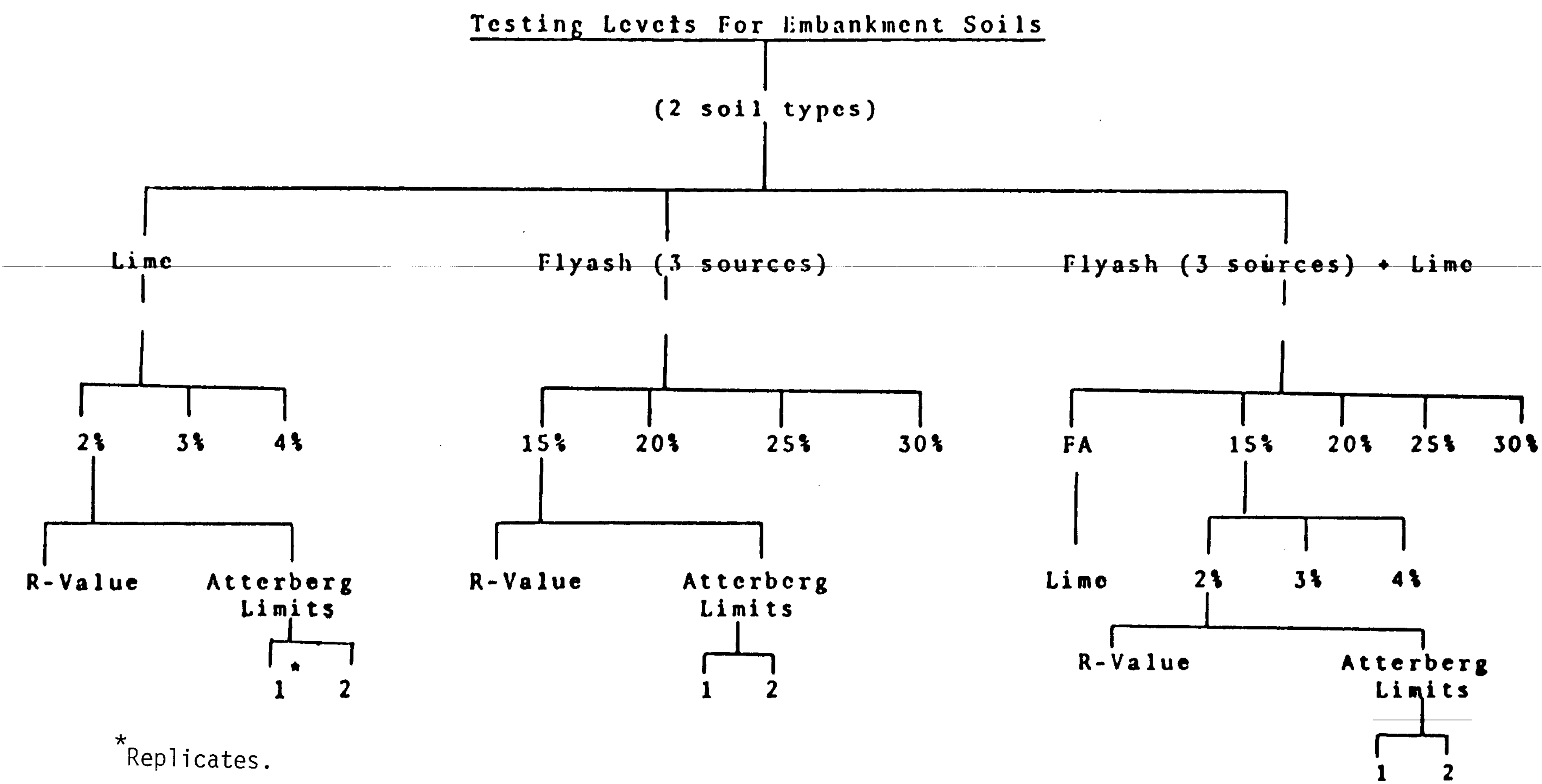


FIGURE 3. Experimental design - embankment soils

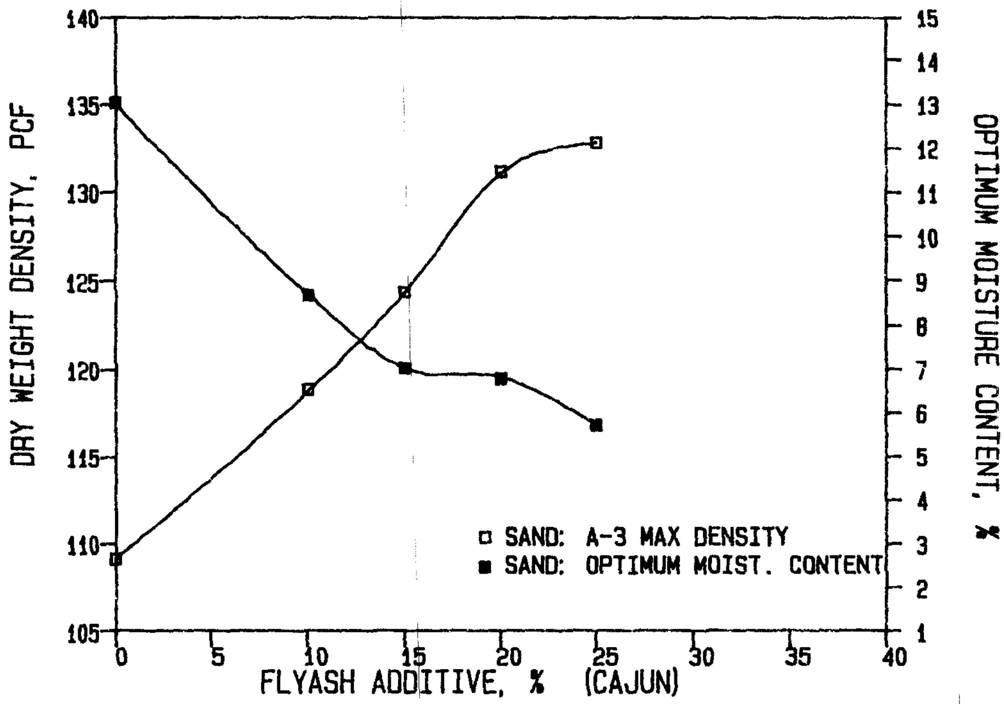


FIGURE 4. Moisture content/dry density vs flyash additive

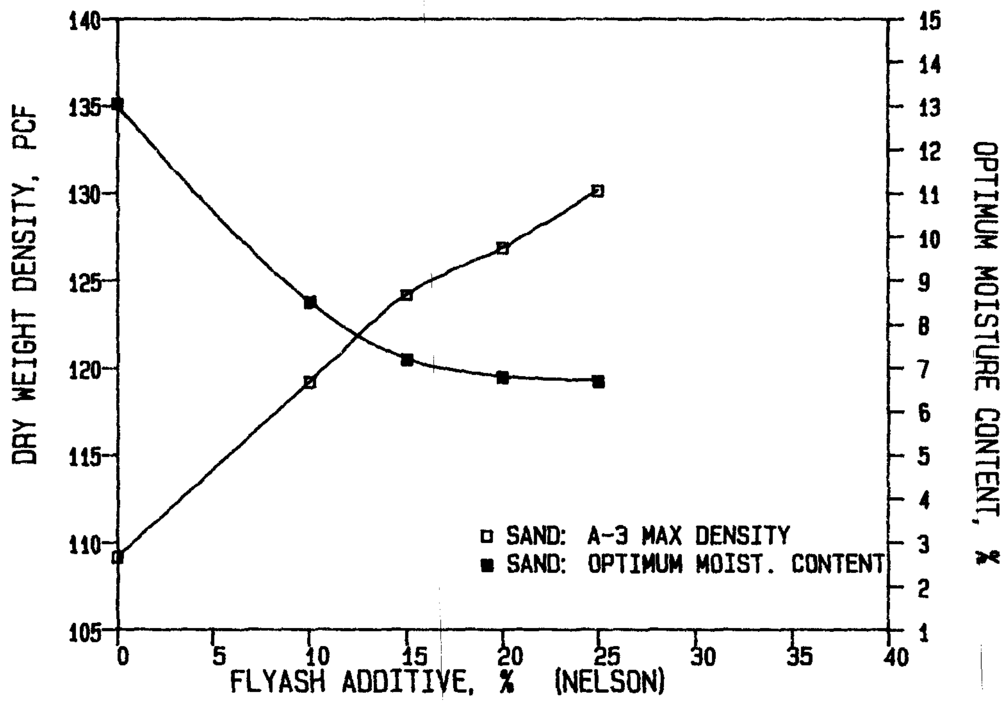


FIGURE 5. Moisture content/dry density vs flyash additive

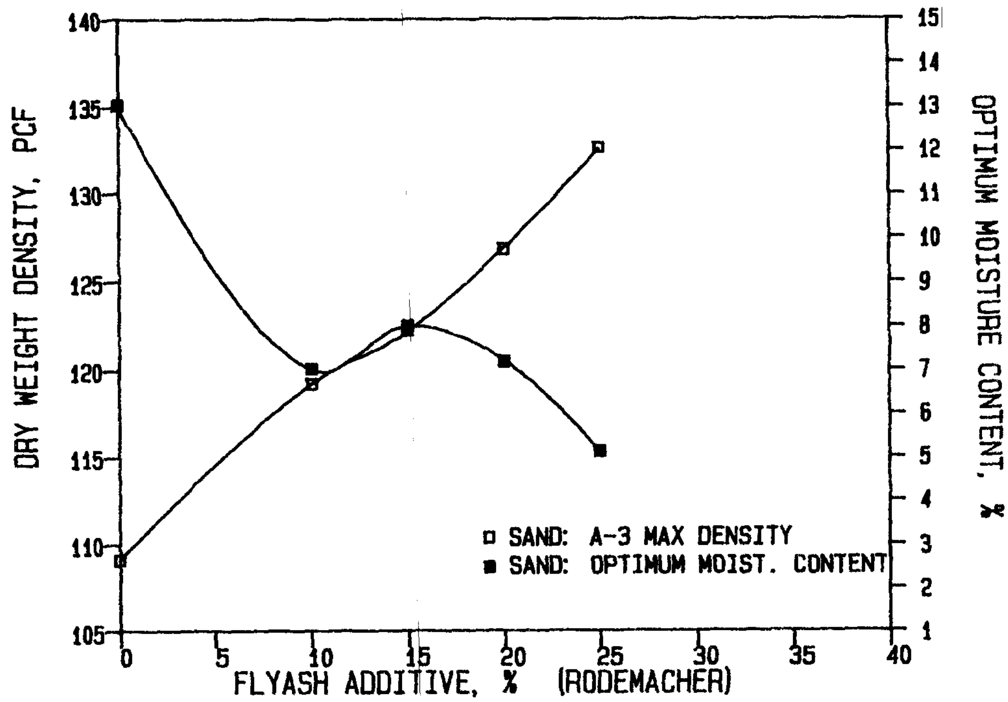


FIGURE 6. Moisture content/dry density vs flyash additive

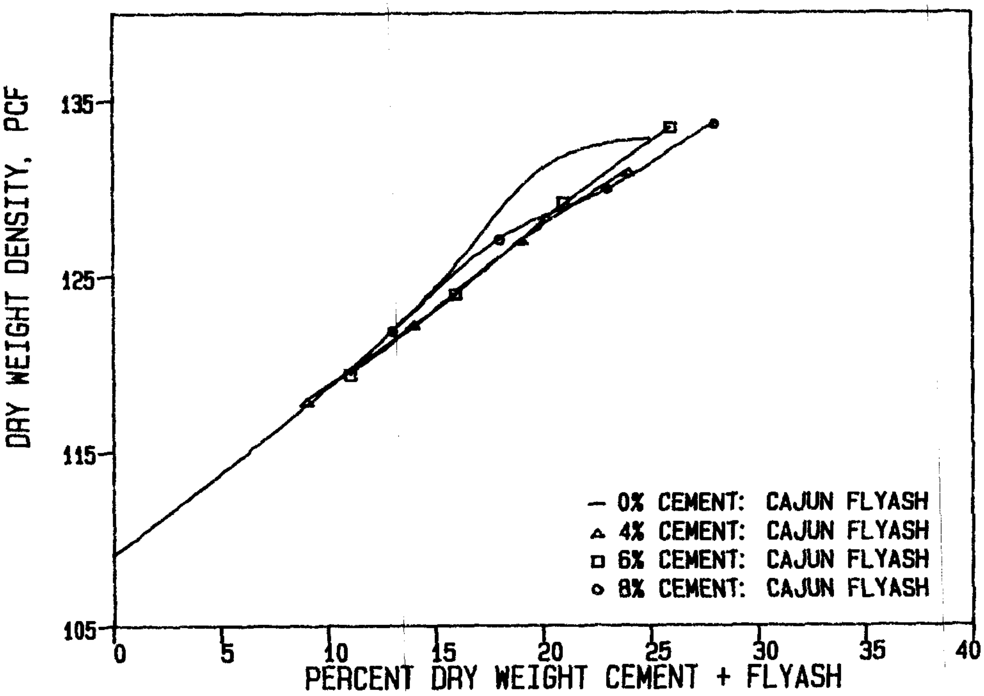


FIGURE 7. Sand A-3: Max dry density vs cement + flyash

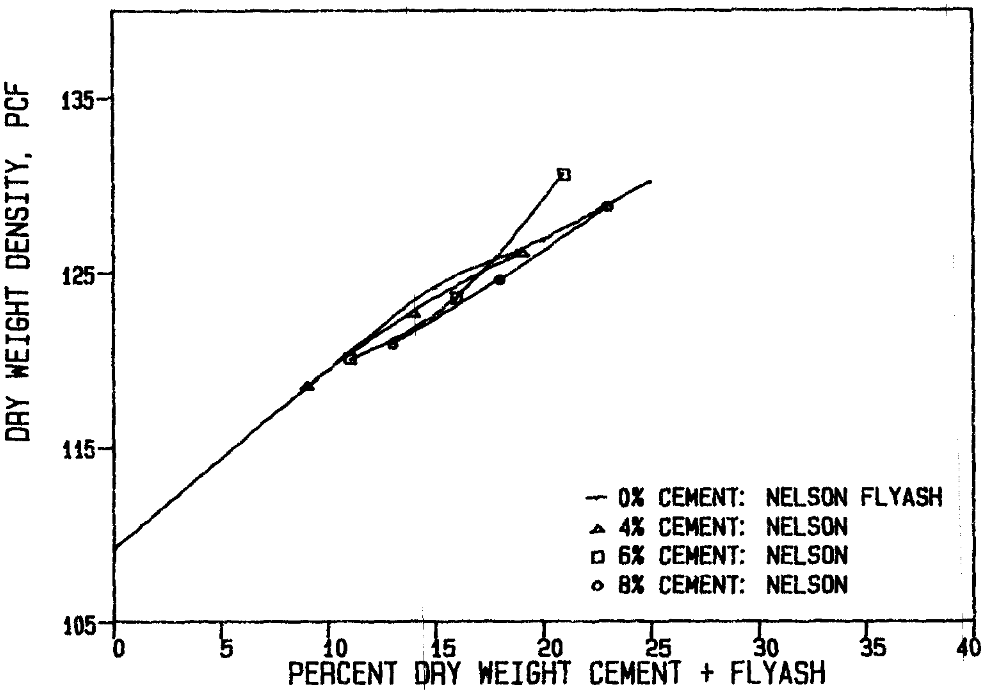


FIGURE 8. Sand A-3: Max dry density vs cement + flyash

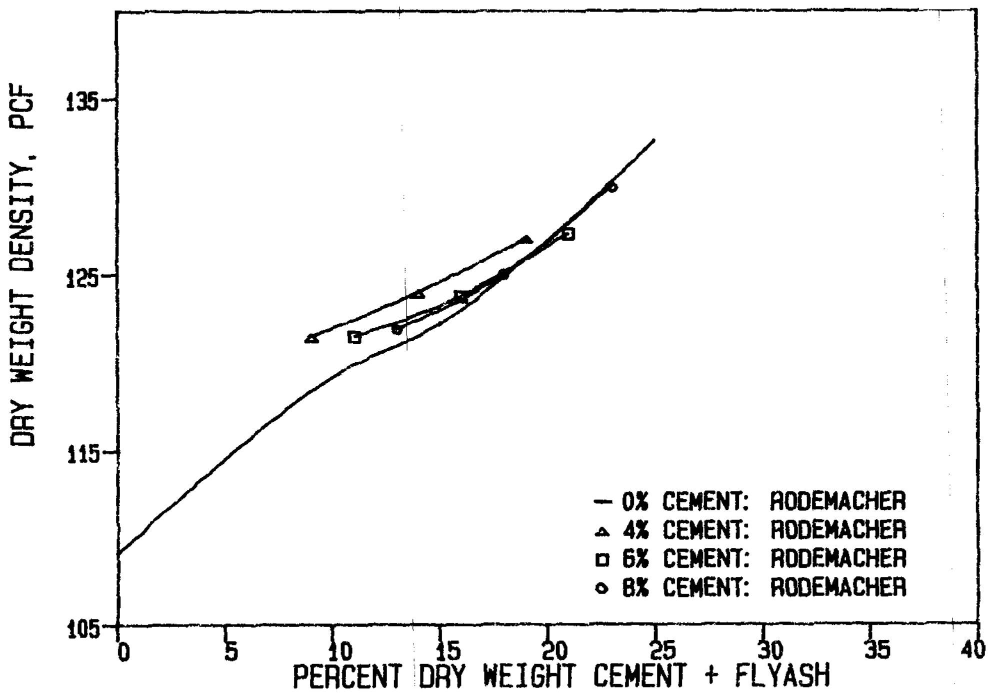


FIGURE 9. Sand A-3: Max dry density vs cement + flyash

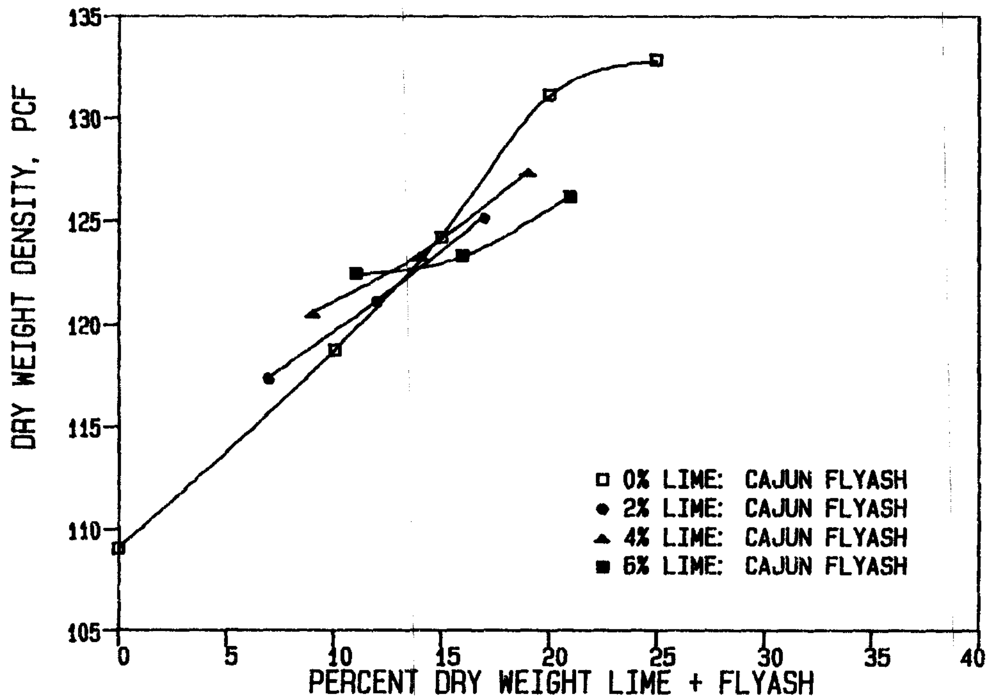


FIGURE 10. Sand A-3: Dry density vs lime + cajun flyash

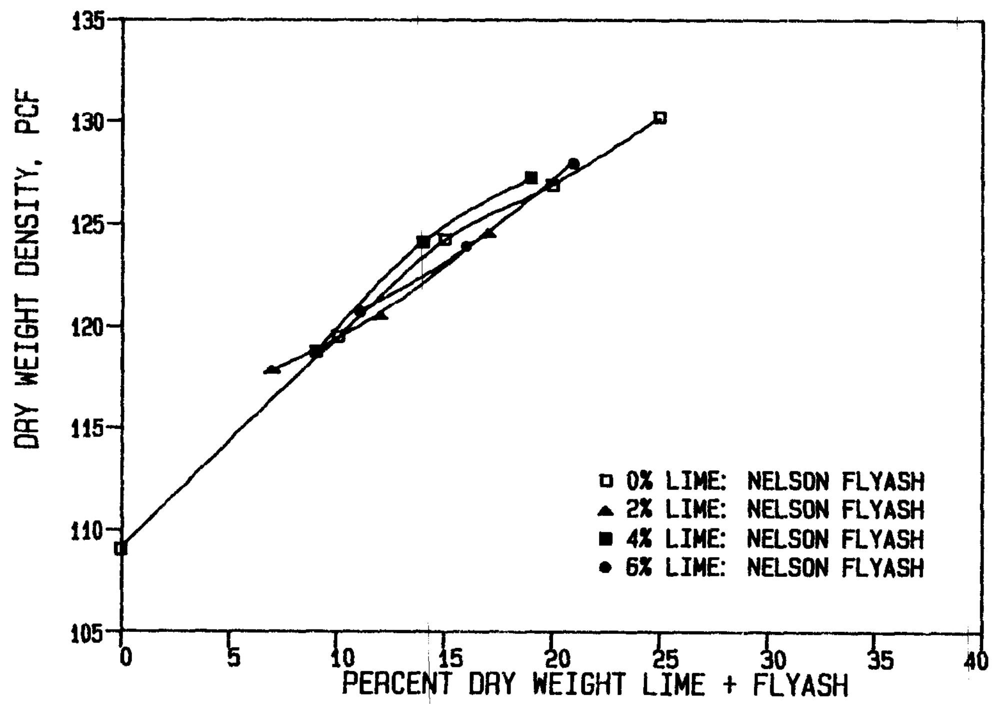


FIGURE 11. Sand A-3: Dry density vs lime + Nelson flyash

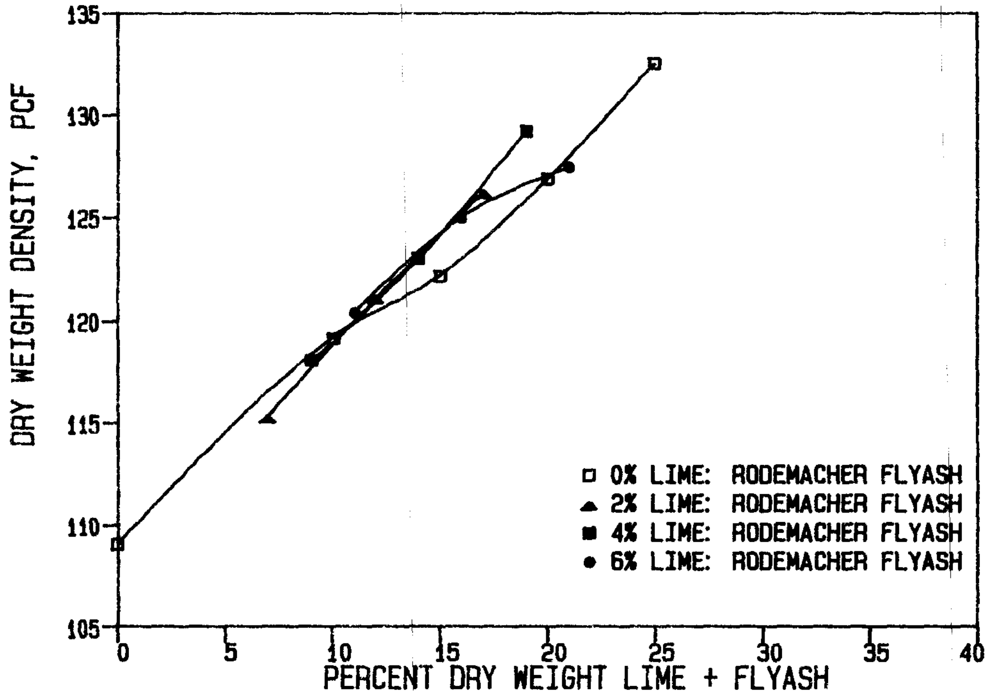


FIGURE 12. Sand A-3: Dry density vs lime + Rodemacher flyash

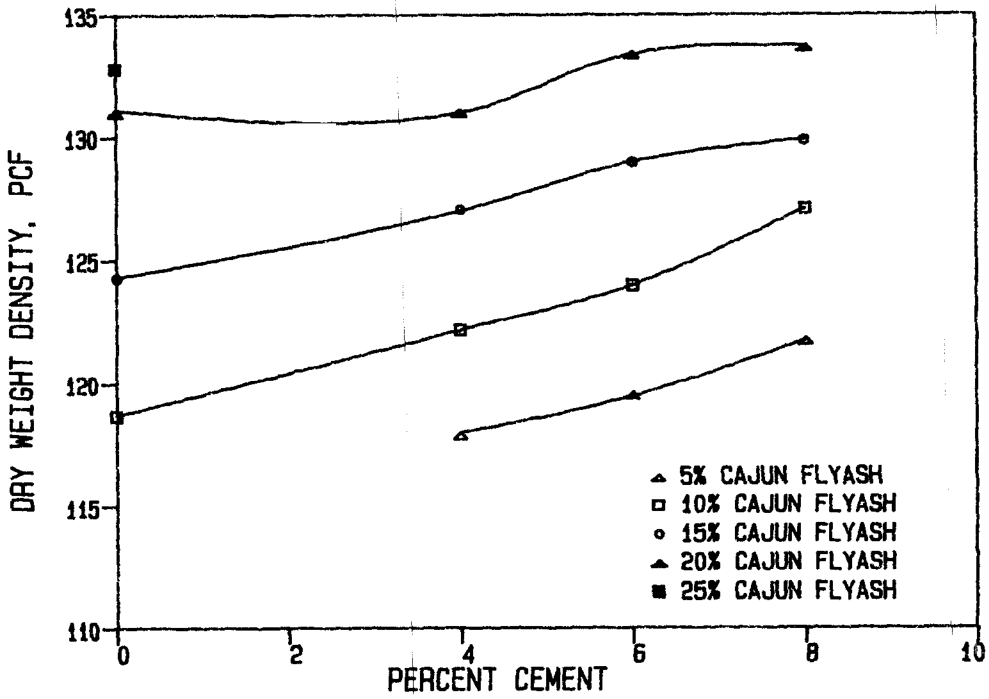


FIGURE 13. Sand A-3: Dry weight density vs percent cement

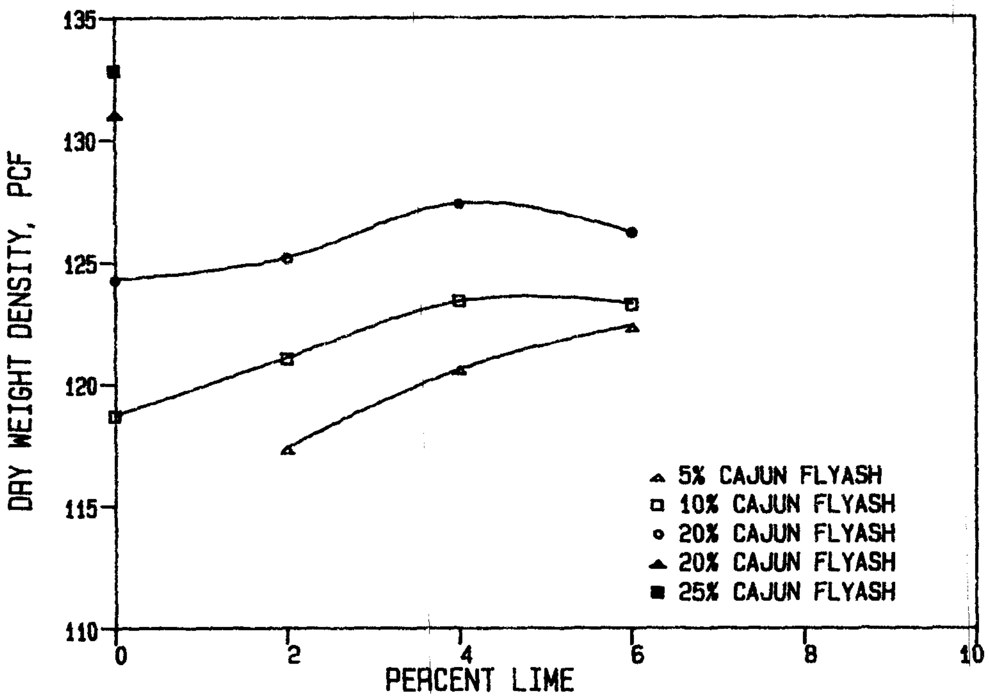


FIGURE 14. Sand A-3: Dry weight density vs percent lime

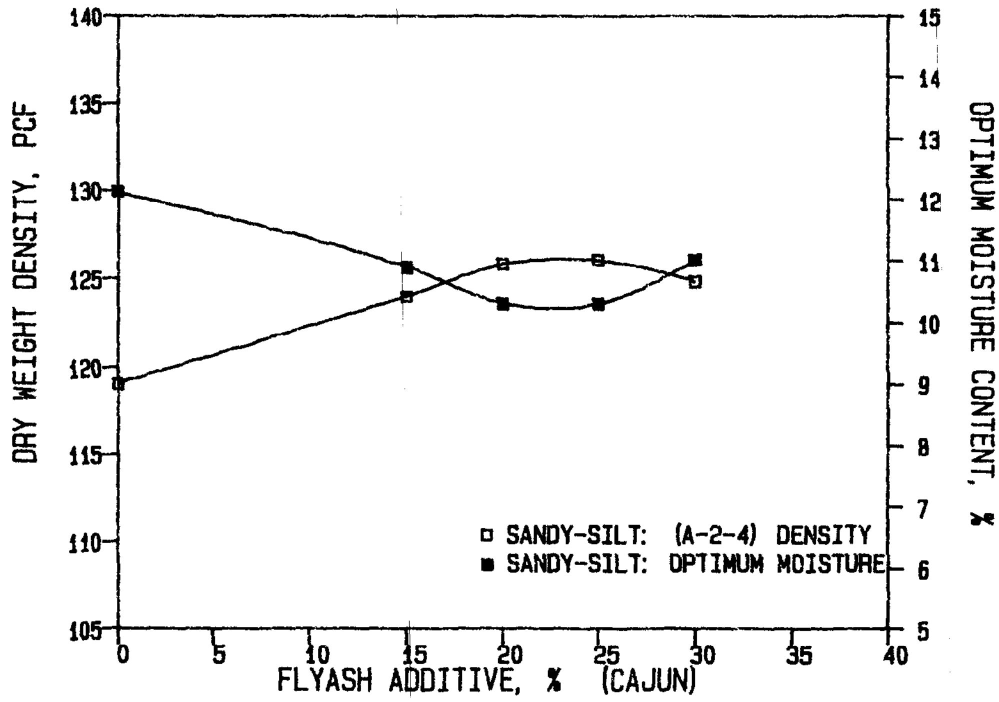


FIGURE 15. Optimum moist content/max dry density vs flyash

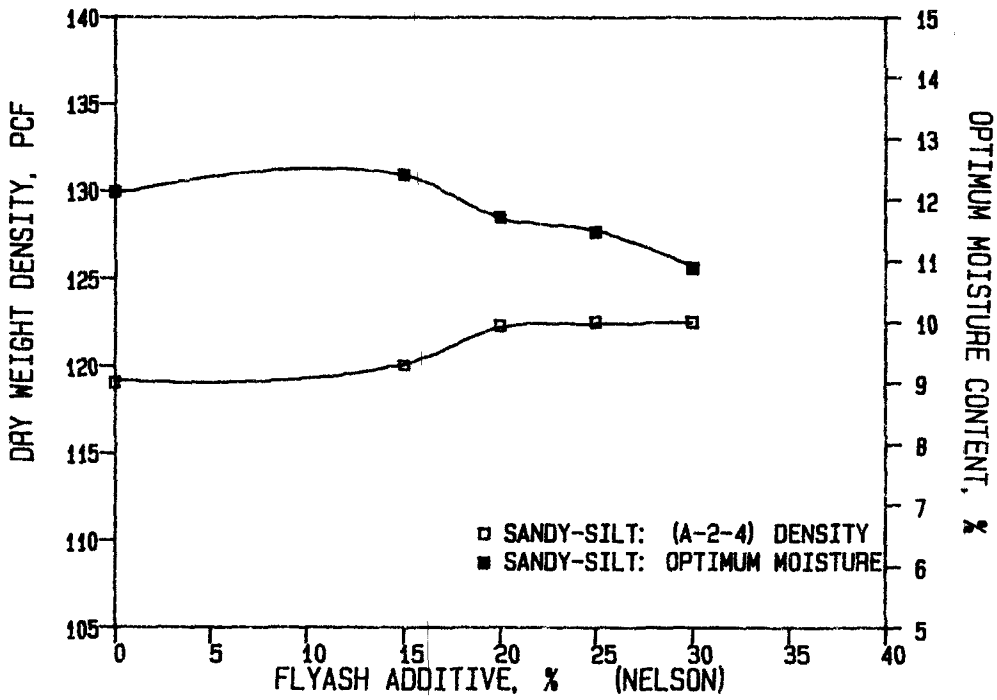


FIGURE 16. Optimum moist content/max dry density vs flyash

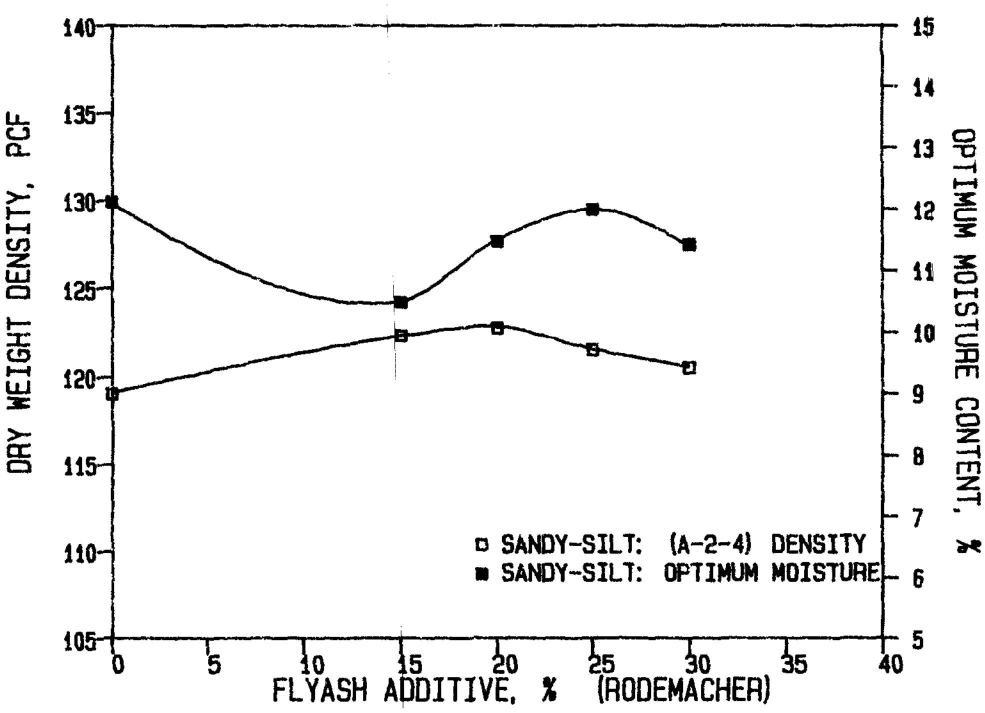


FIGURE 17. Optimum moist content/max dry density vs flyash

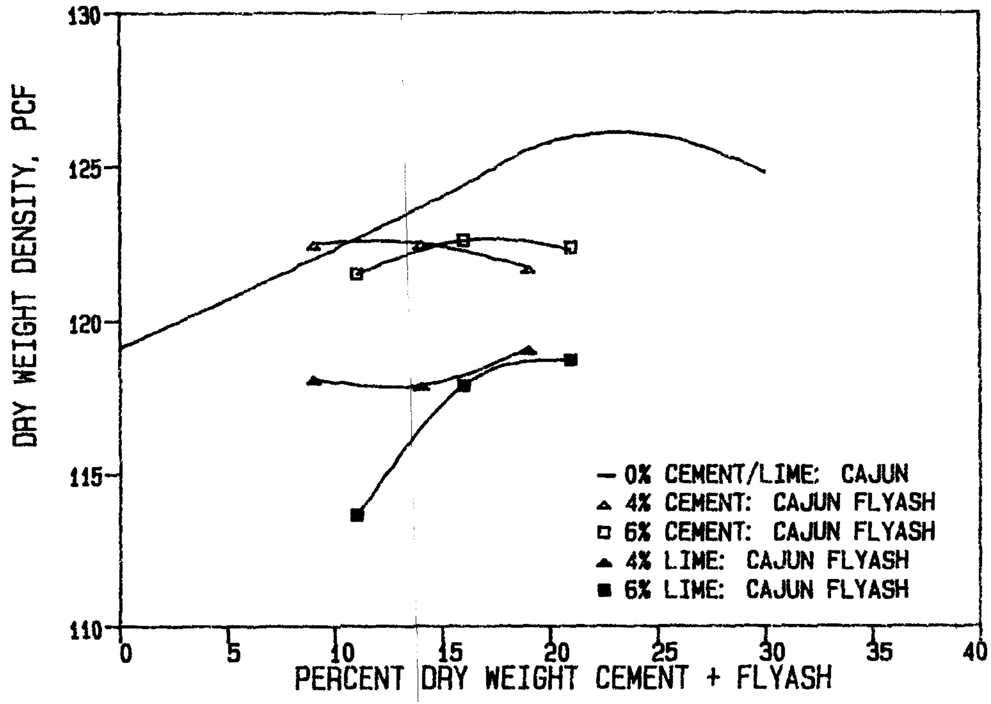


FIGURE 18. Sandy-silt A-2-4: Density vs lime/cement + flyash

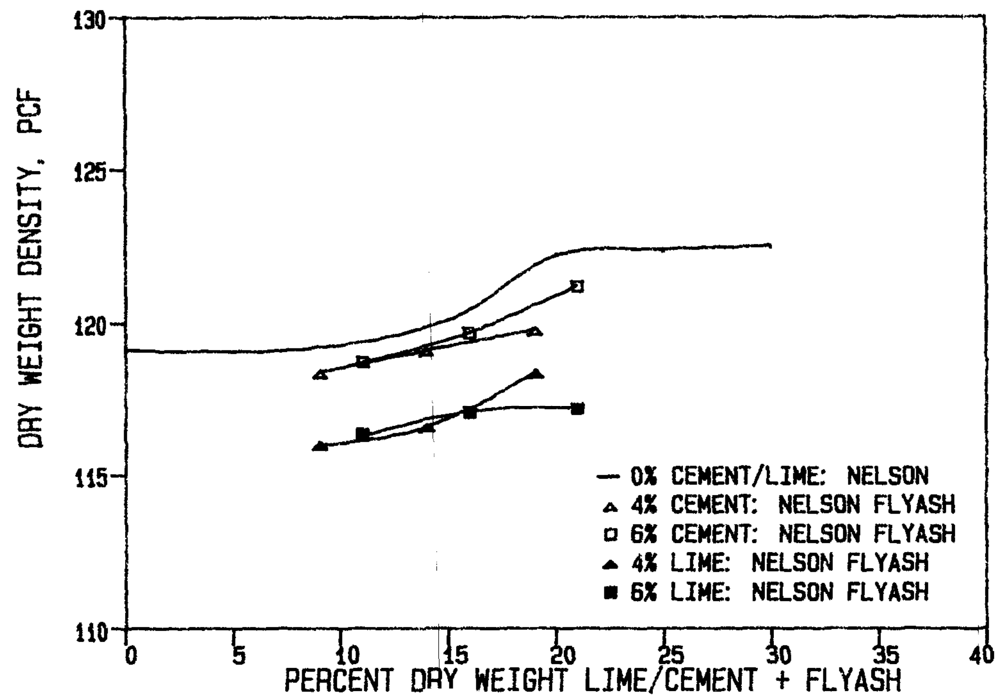


FIGURE 19. Sandy-silt A-2-4: Density vs lime/cement + flyash

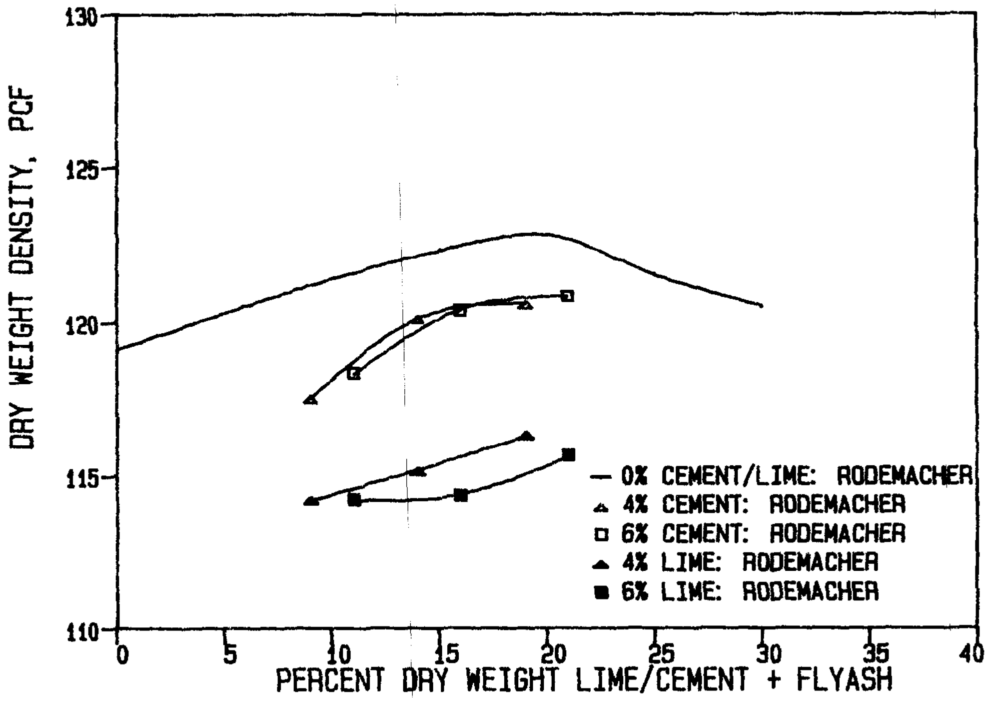


FIGURE 20. Sandy-silt A-2-4: Density vs lime/cement + flyash

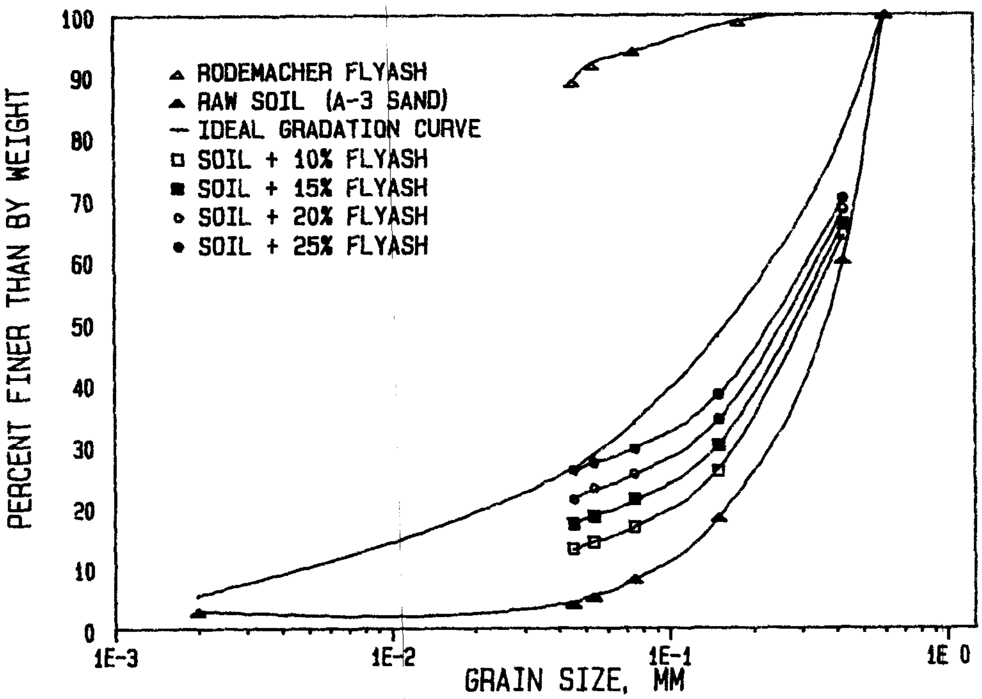


FIGURE 21. A-3 sand & Rodemacher flyash - gradation curves

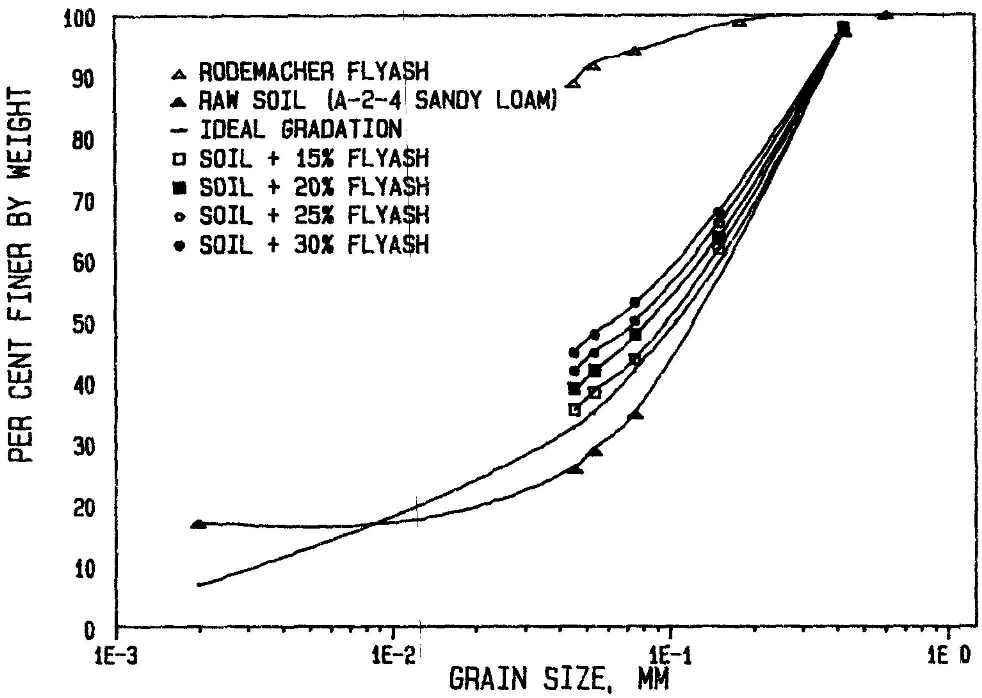


FIGURE 22. A-2-4 sand & Rodemacher flyash - gradation curves

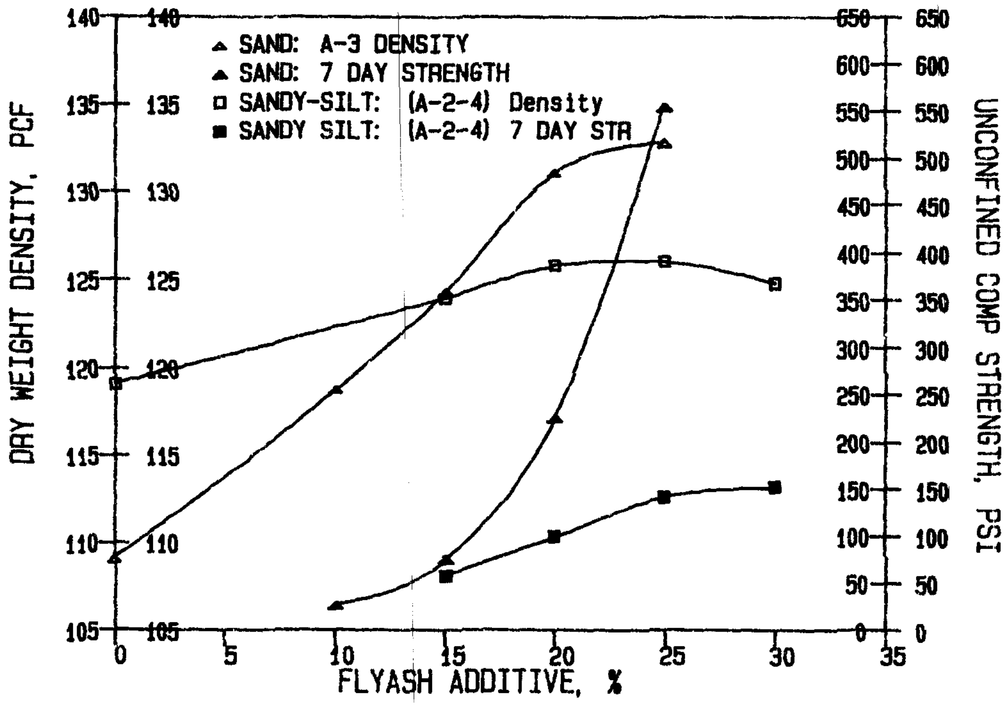


FIGURE 23. Strength/dry density vs Cajun flyash additive

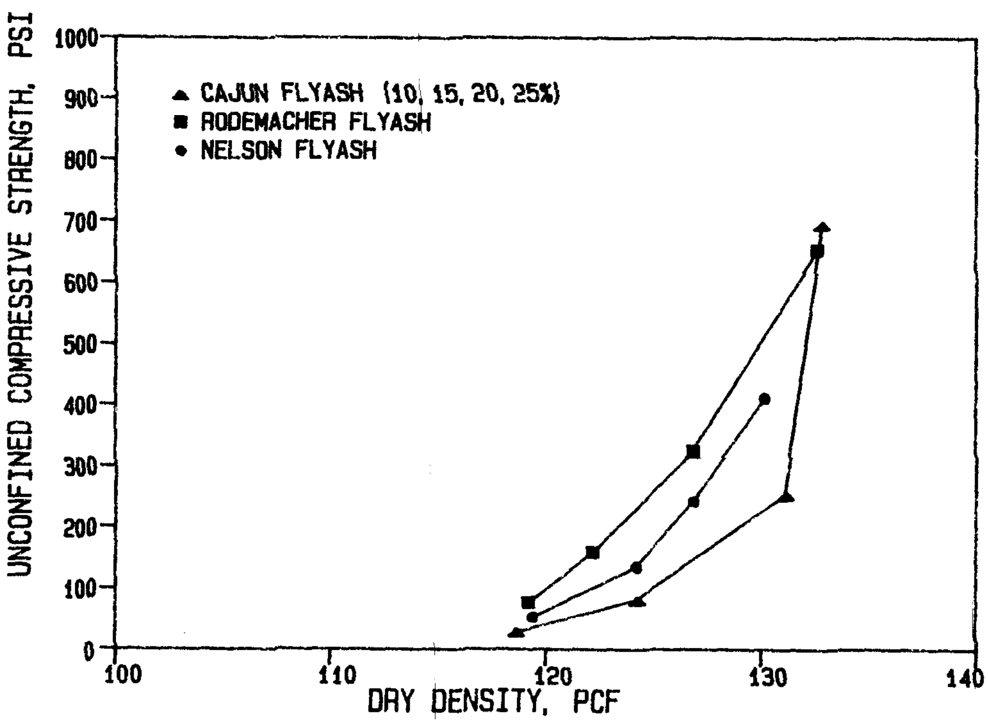


FIGURE 24. A-3 sand: 28 day strength vs dry density

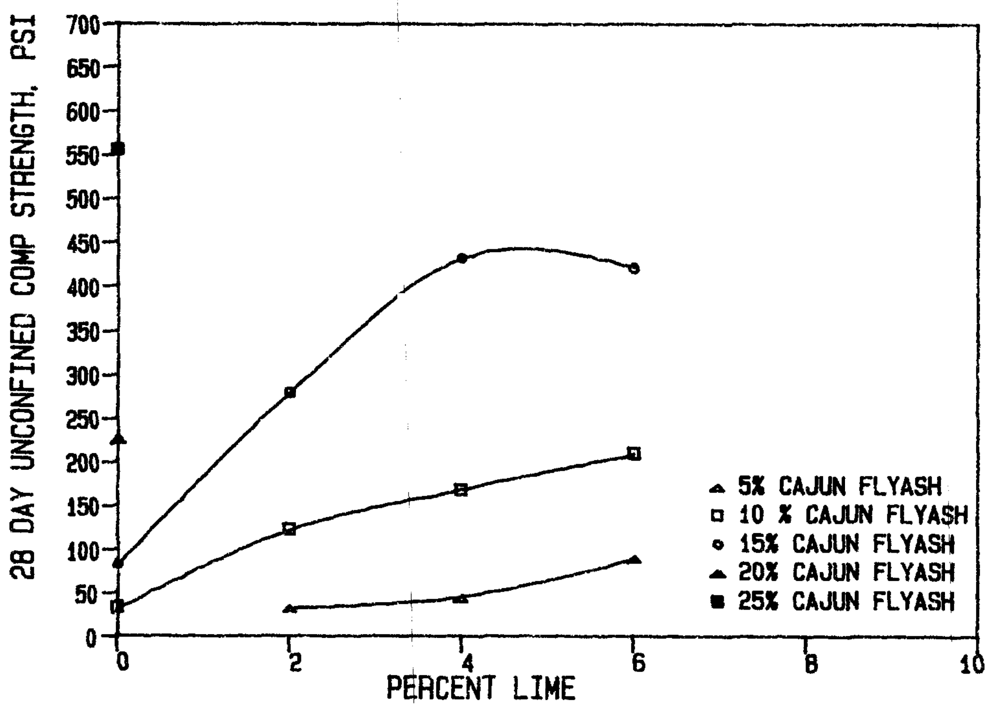


FIGURE 25. Sand A-3: LFA unconfined comp strength vs % lime

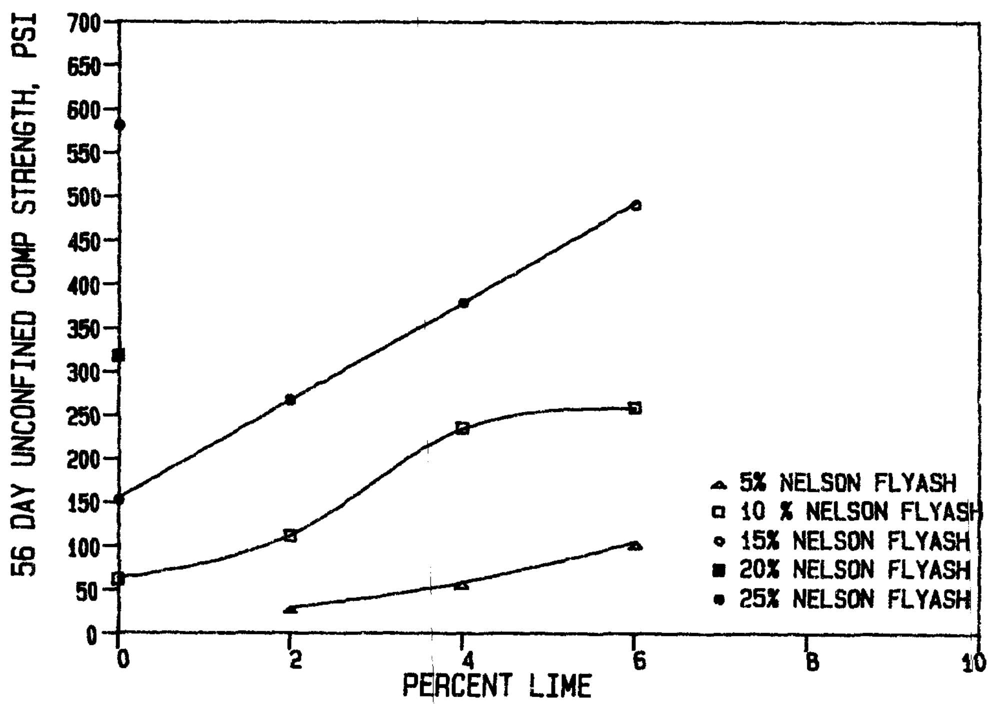


FIGURE 26. Sand A-3: LFA unconfined comp strength vs % lime

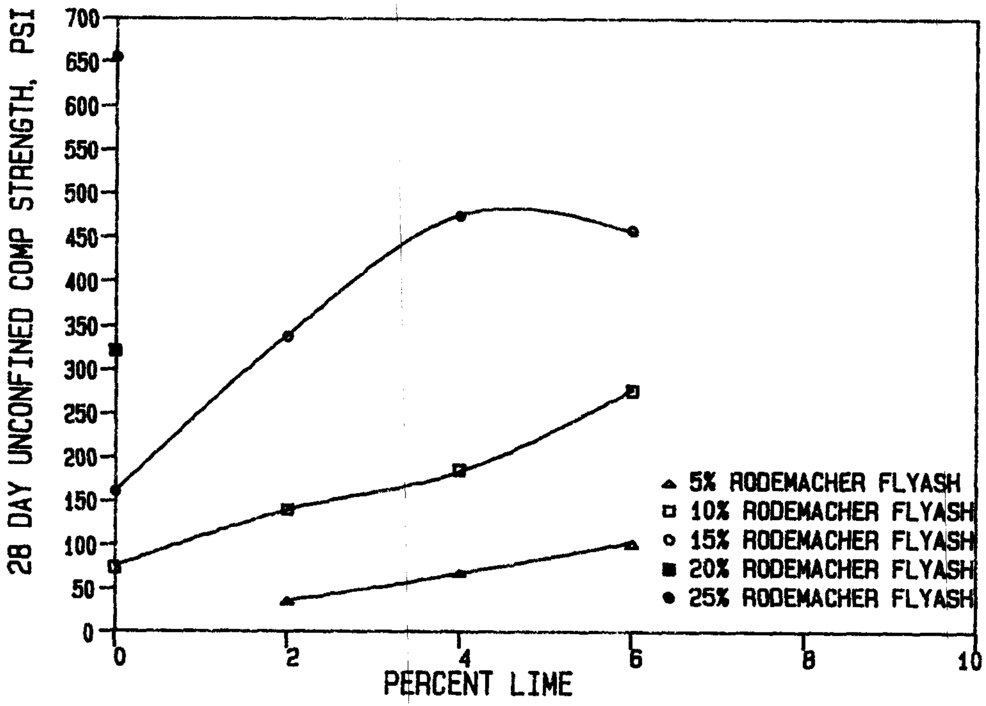


FIGURE 27. Sand A-3: LFA unconfined comp strength vs % lime

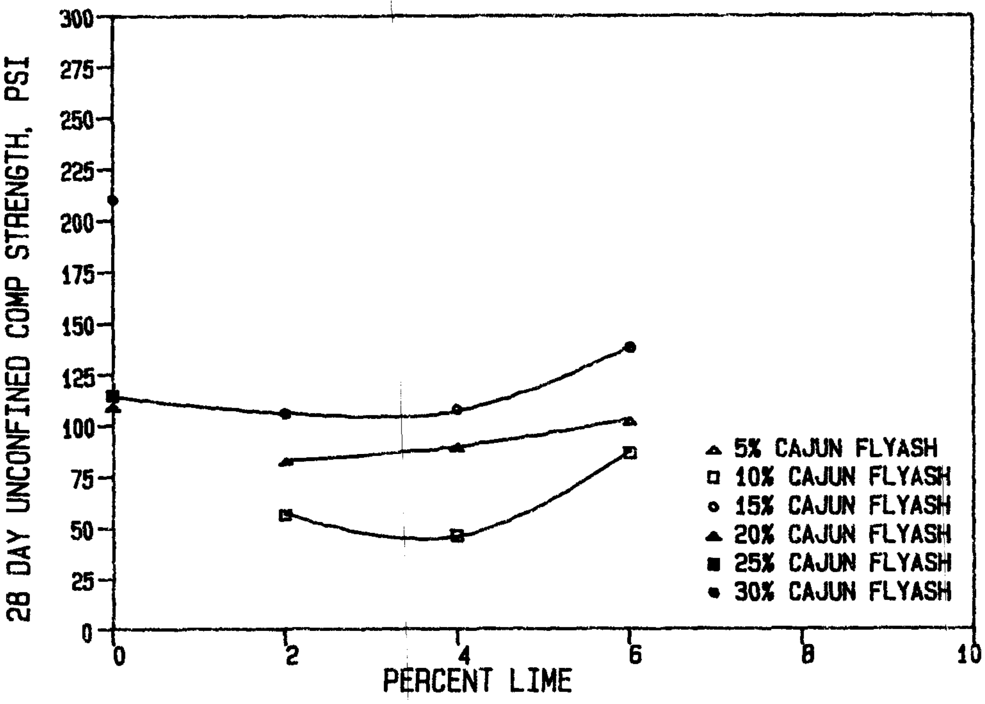


FIGURE 28. SA-Silt A-2-4: Unconfined comp strength vs % lime

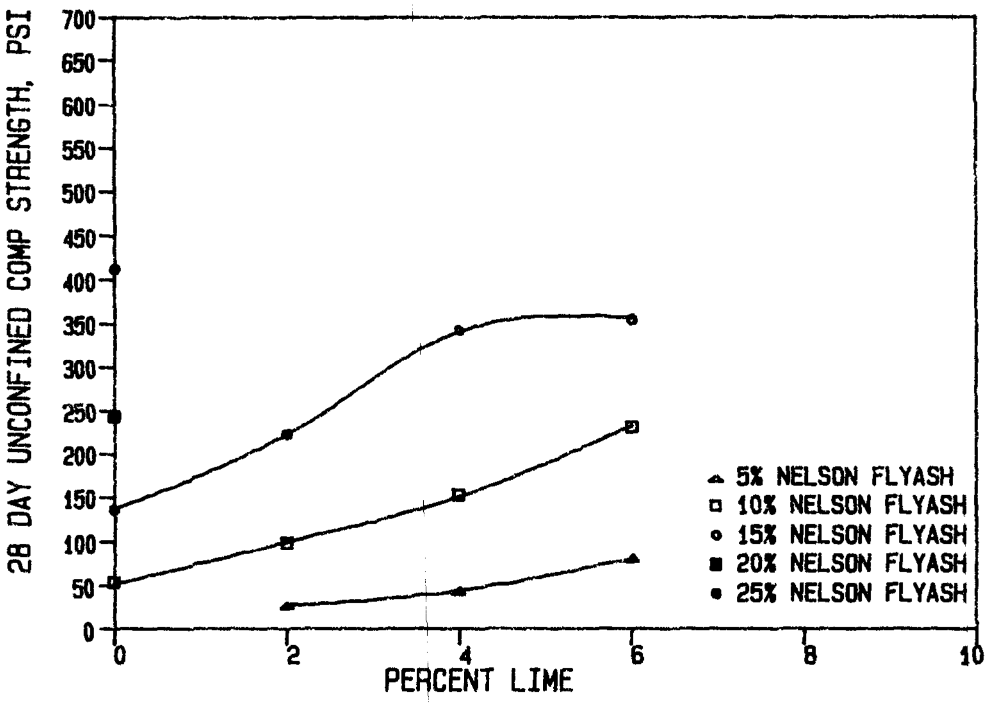


FIGURE 29. SA-Silt A-2-4: Unconfined comp strength vs % lime

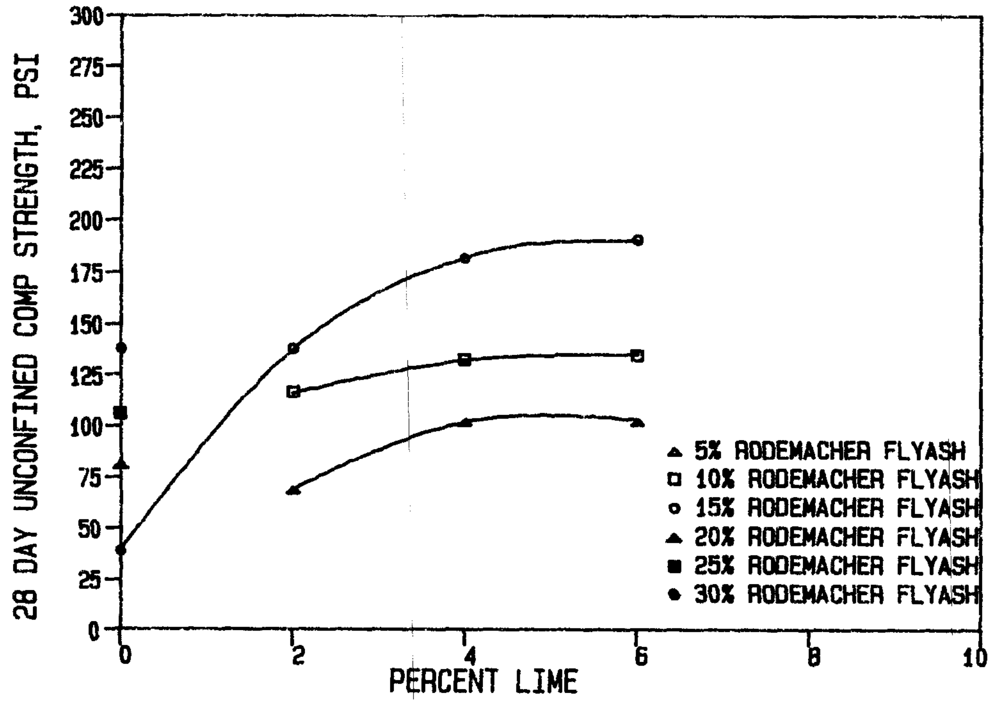


FIGURE 30. SA-Silt A-2-4: Unconfined comp strength vs % lime

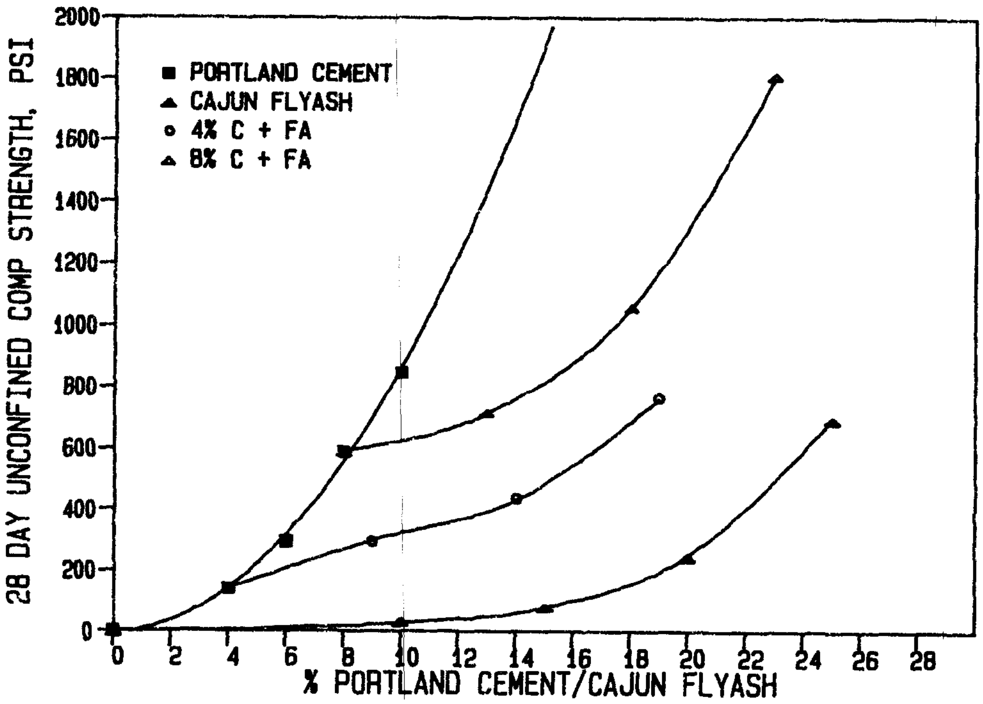


FIGURE 31. A-3 Sand: Qu vs % Portland cement/Cajun flyash

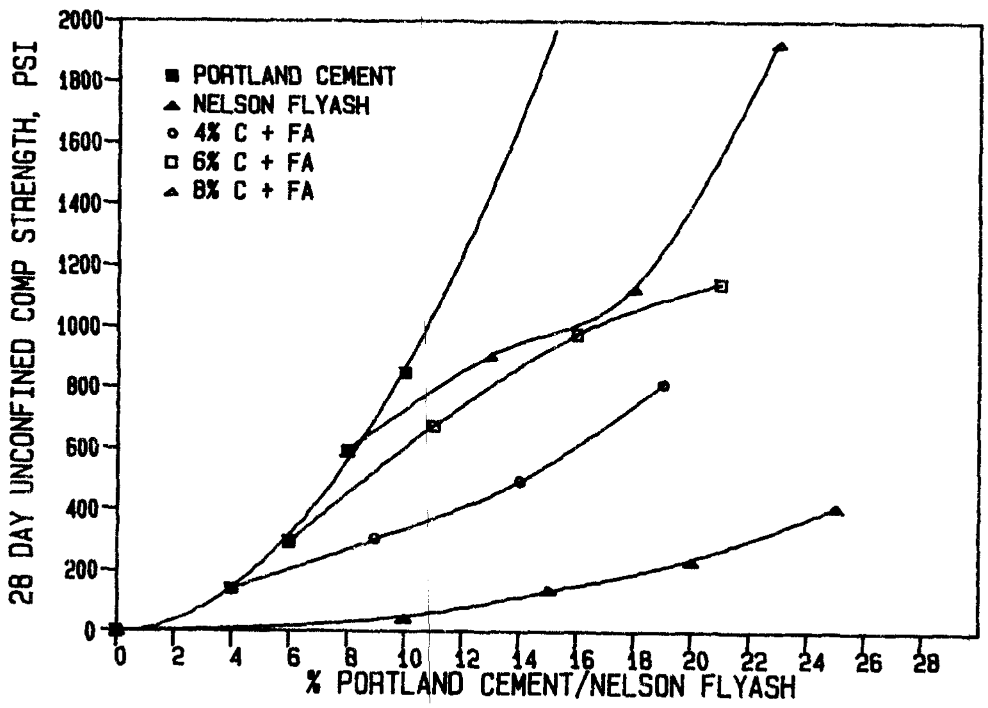


FIGURE 32. A-3 Sand: Qu vs % Portland cement/Nelson flyash

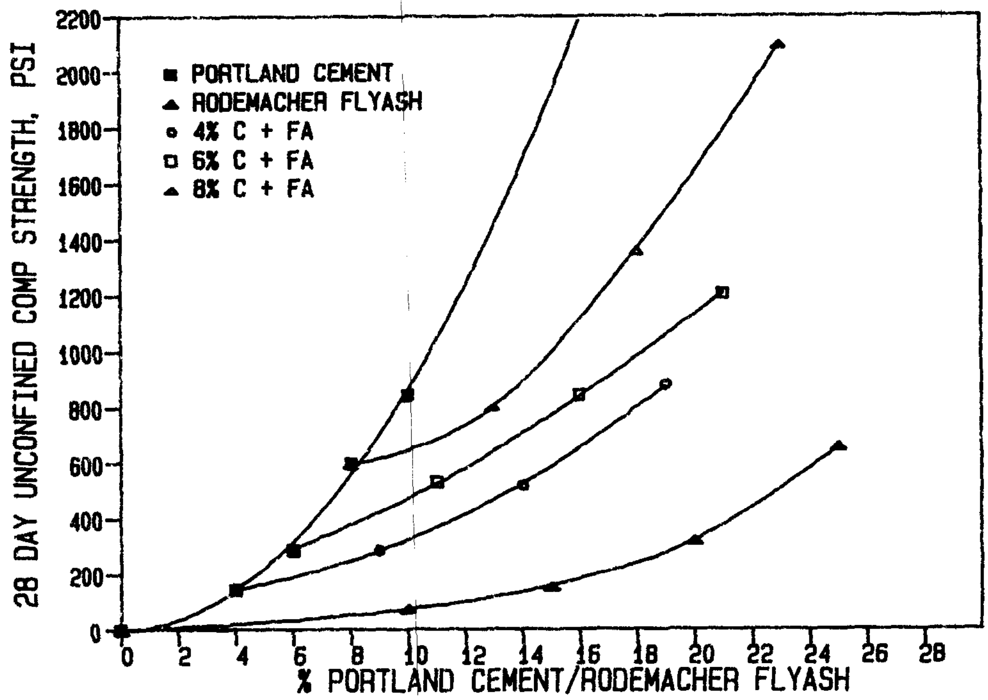


FIGURE 33. A-3 Sand: Qu vs % Portland cement/Rodemacher flyash

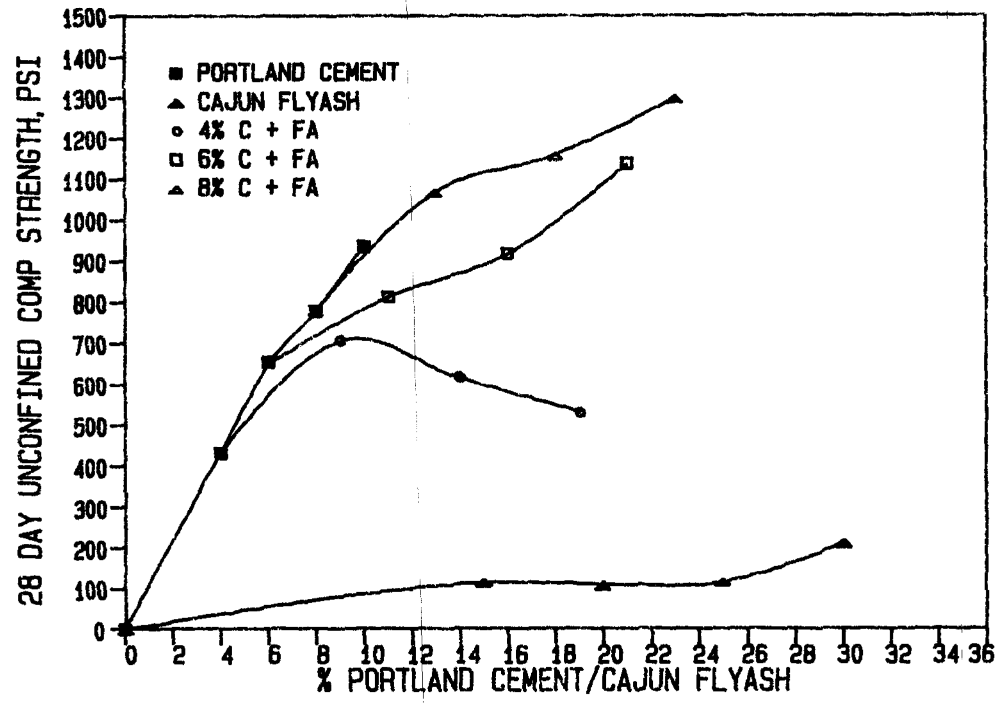


FIGURE 34. A-2-4 Sandy loam: Qu vs cement/Cajun flyash

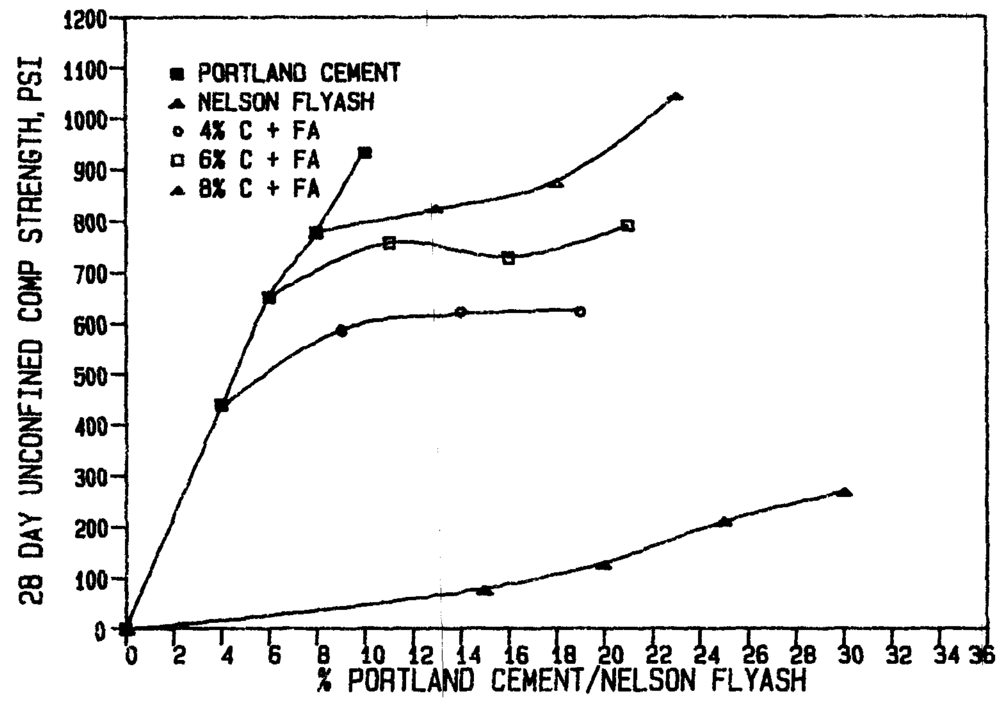


FIGURE 35. A-2-4 Sandy loam: Qu vs cement/Nelson flyash

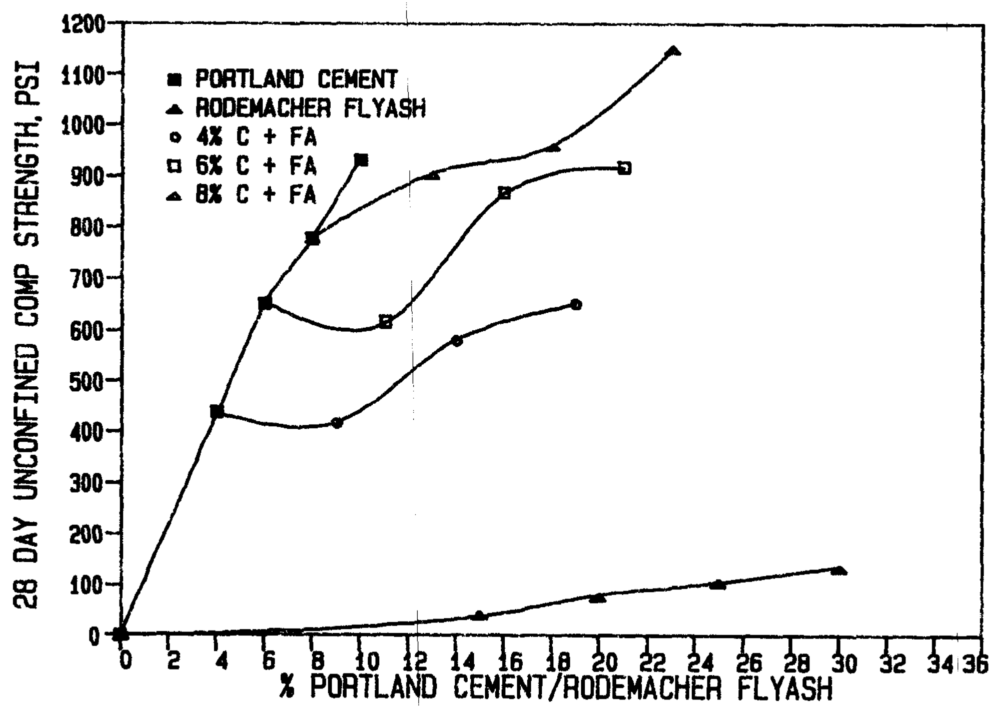


FIGURE 36. A-2-4 Sandy loam: Qu vs cement/Rodemacher flyash

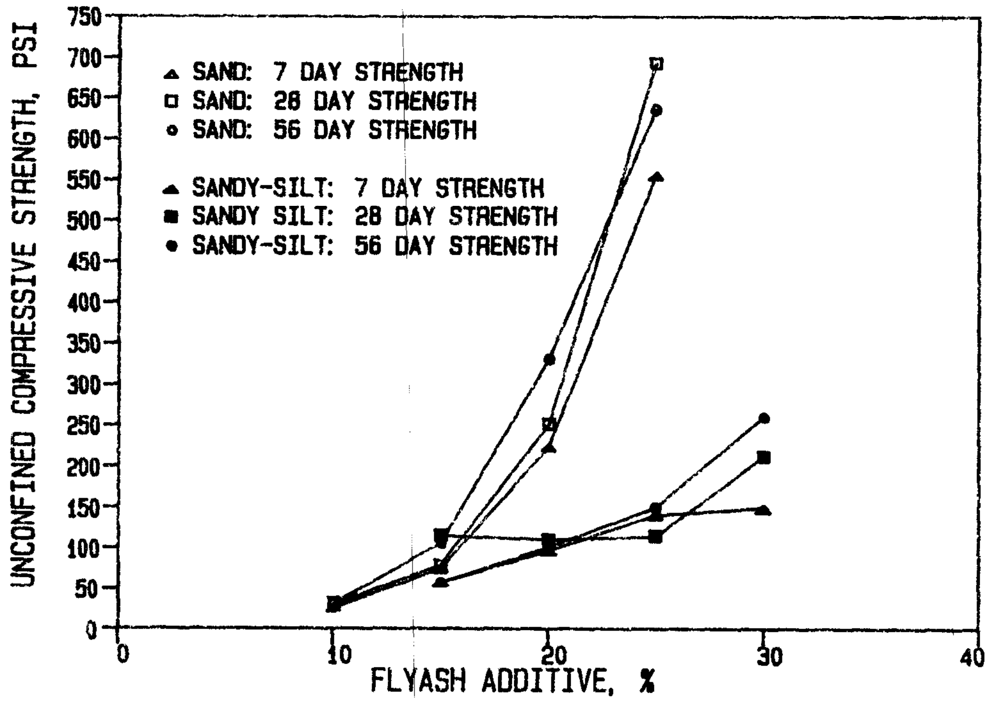


FIGURE 37. Unconfined compressive strengths - Cajun flyash

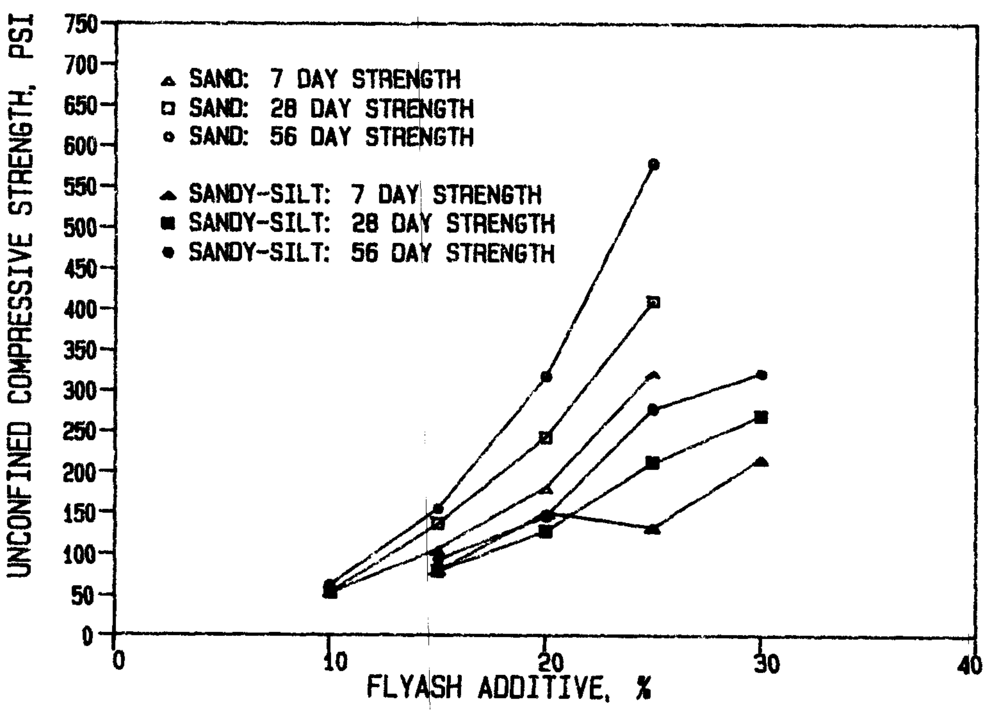


FIGURE 38. Unconfined compressive strengths - Nelson flyash

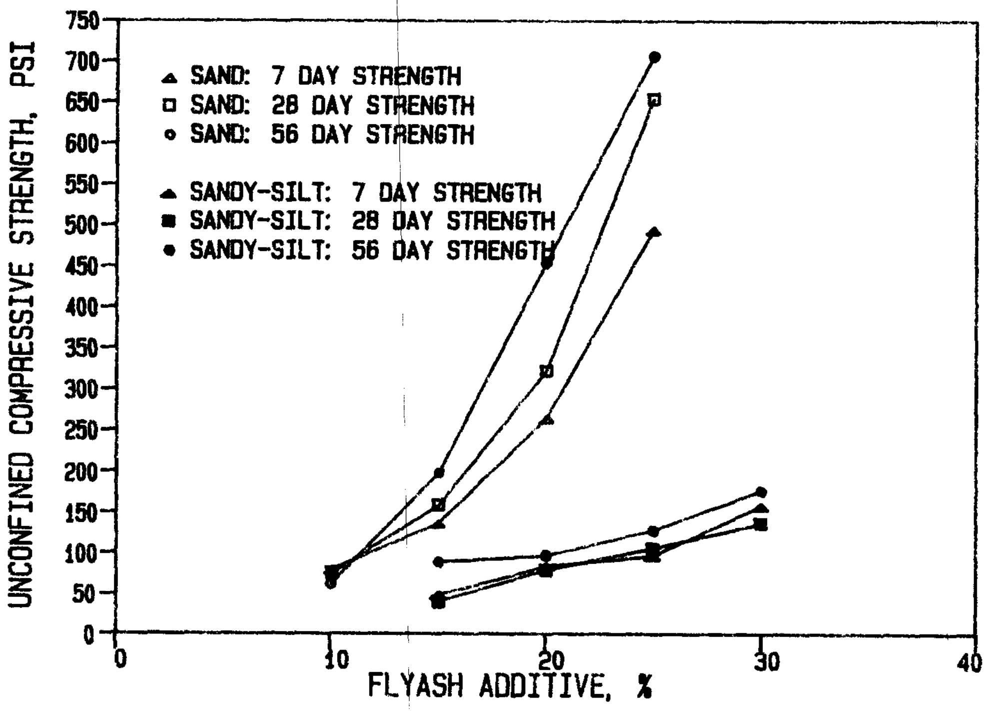


FIGURE 39. Unconfined compressive strengths - Rodemacher

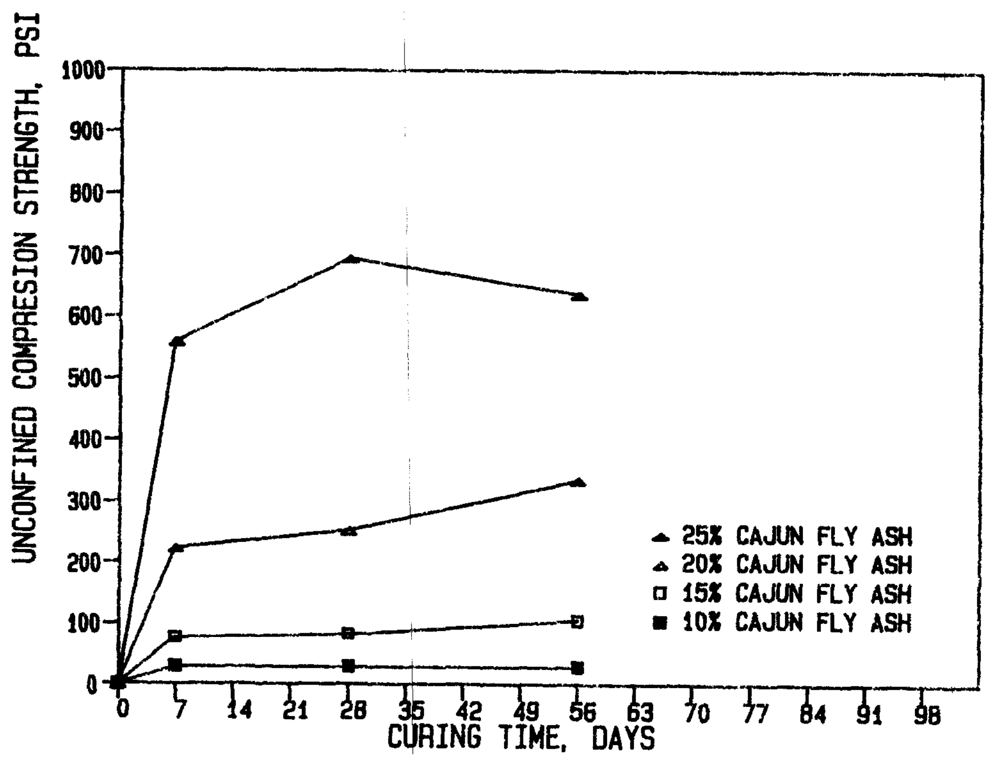


FIGURE 40. Strength gain and curing time: A-3 sand

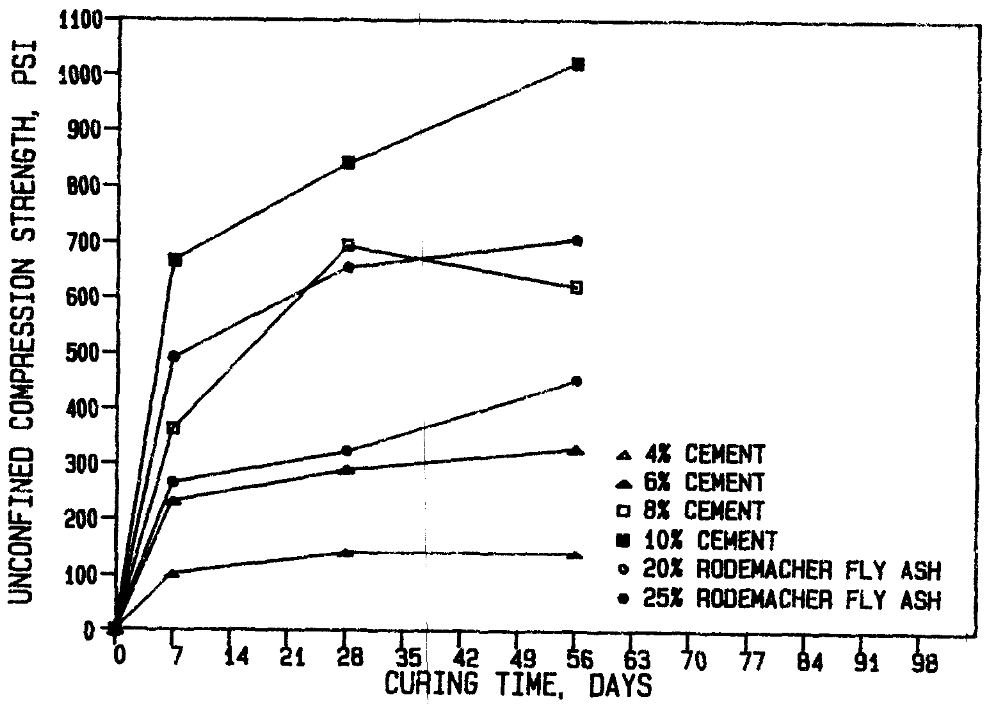


FIGURE 41. Strength gain vs time: A-3 sand/cement/flyash

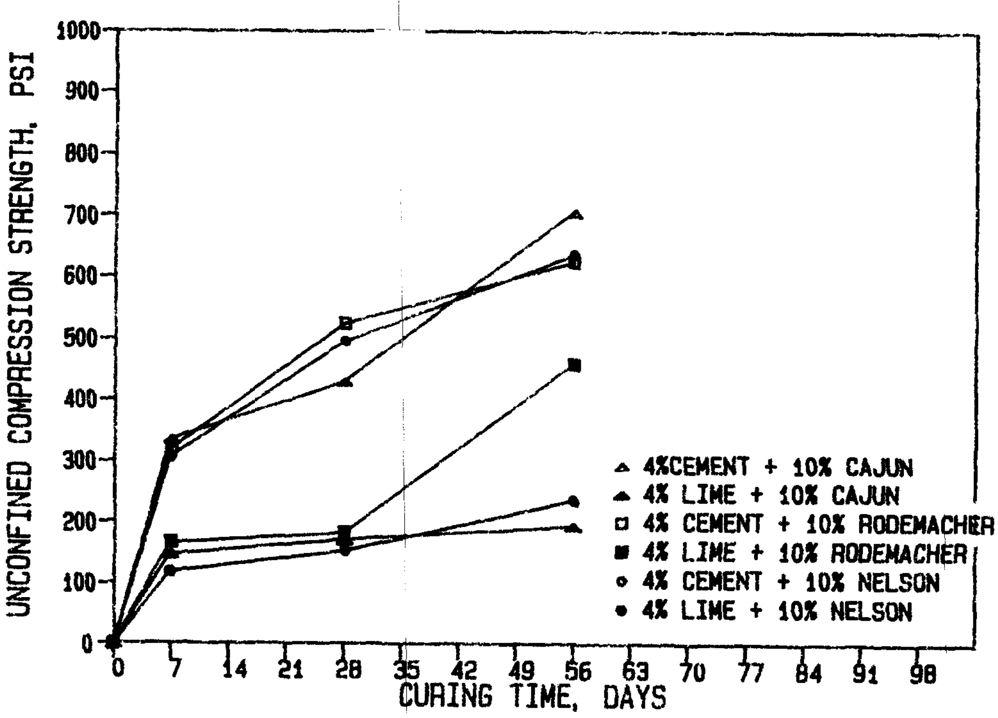


FIGURE 42. Strength gain vs time: A-3 sand/cement/lime/flyash

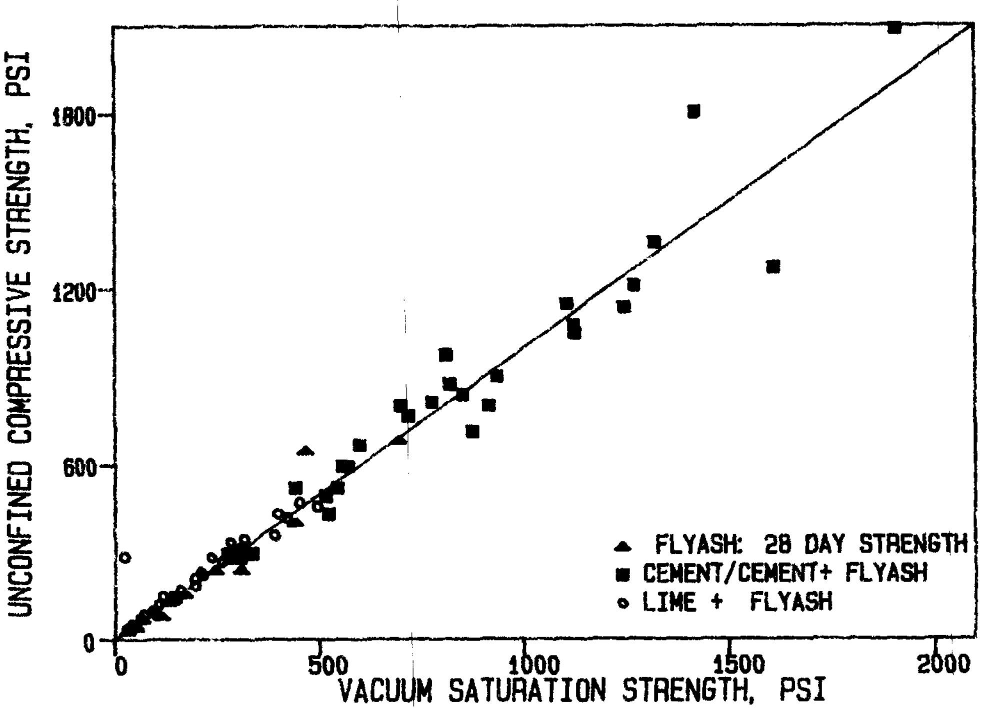


FIGURE 43. Sand A-3: Unconfined vs vacuum saturated strength

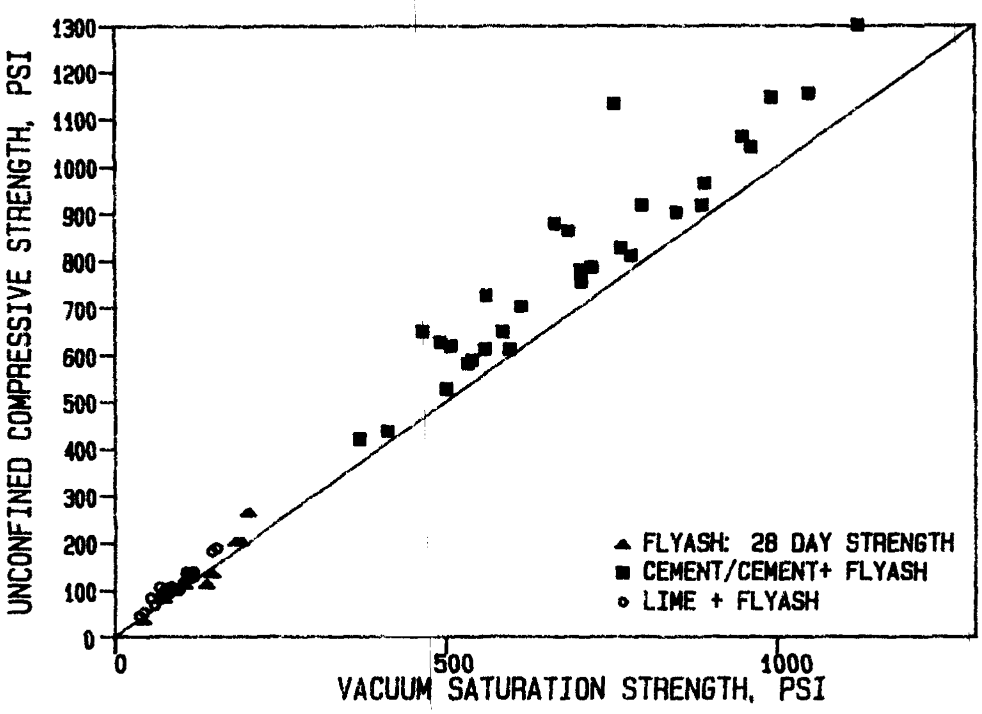


FIGURE 44. Sandy-silt A-2-4: Unconfined vs vac sat strength

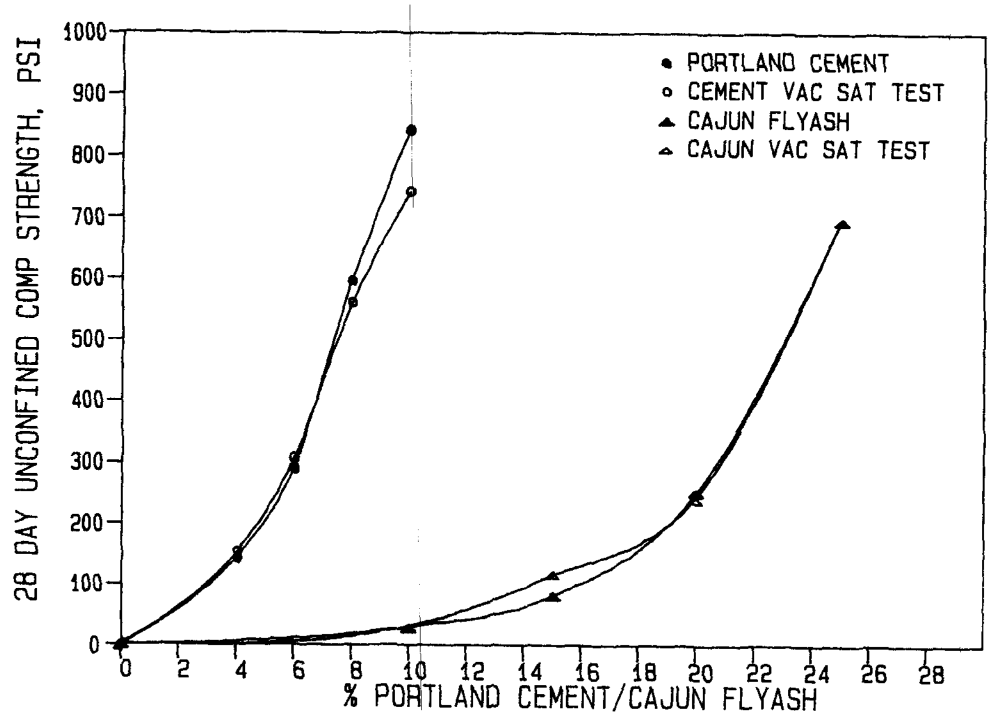


FIGURE 45. A-3 Sand: Durability - Qu vs cement/flyash

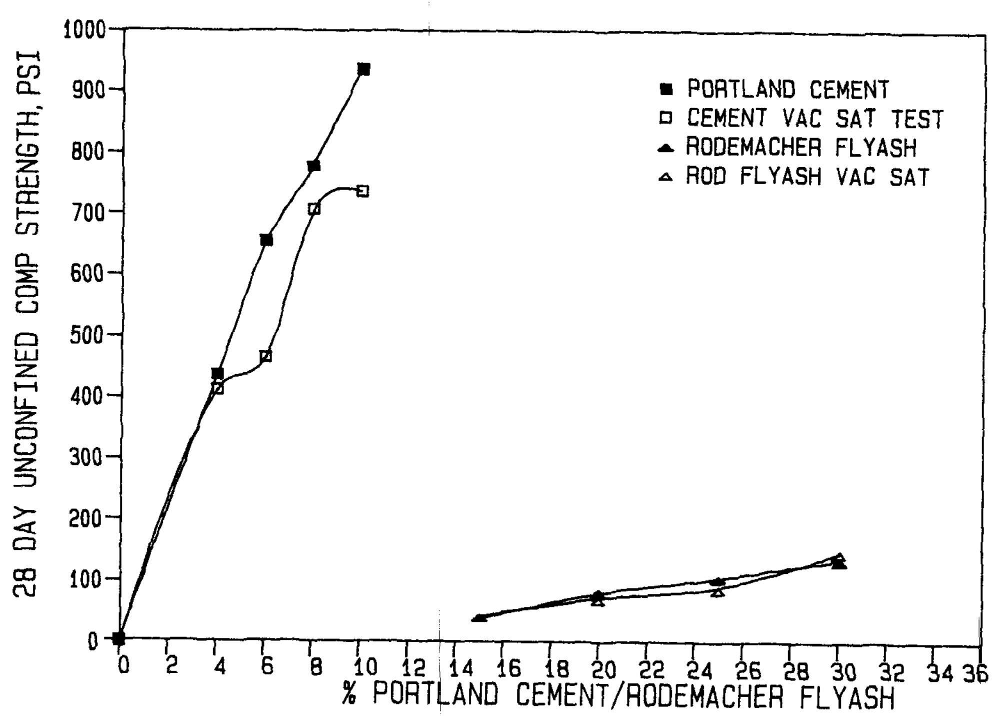


FIGURE 46. A-2-4 Sandy loam: Qu vs cement/Rodemacher flyash

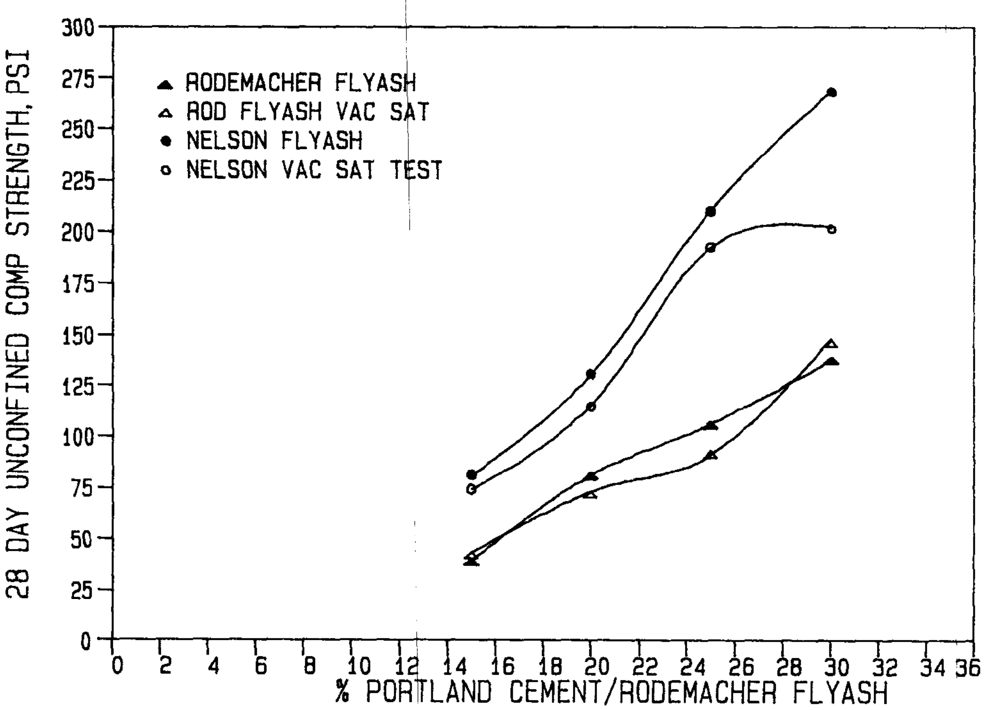


FIGURE 47. A-2-4 Sandy loam: Qu vs percent flyash

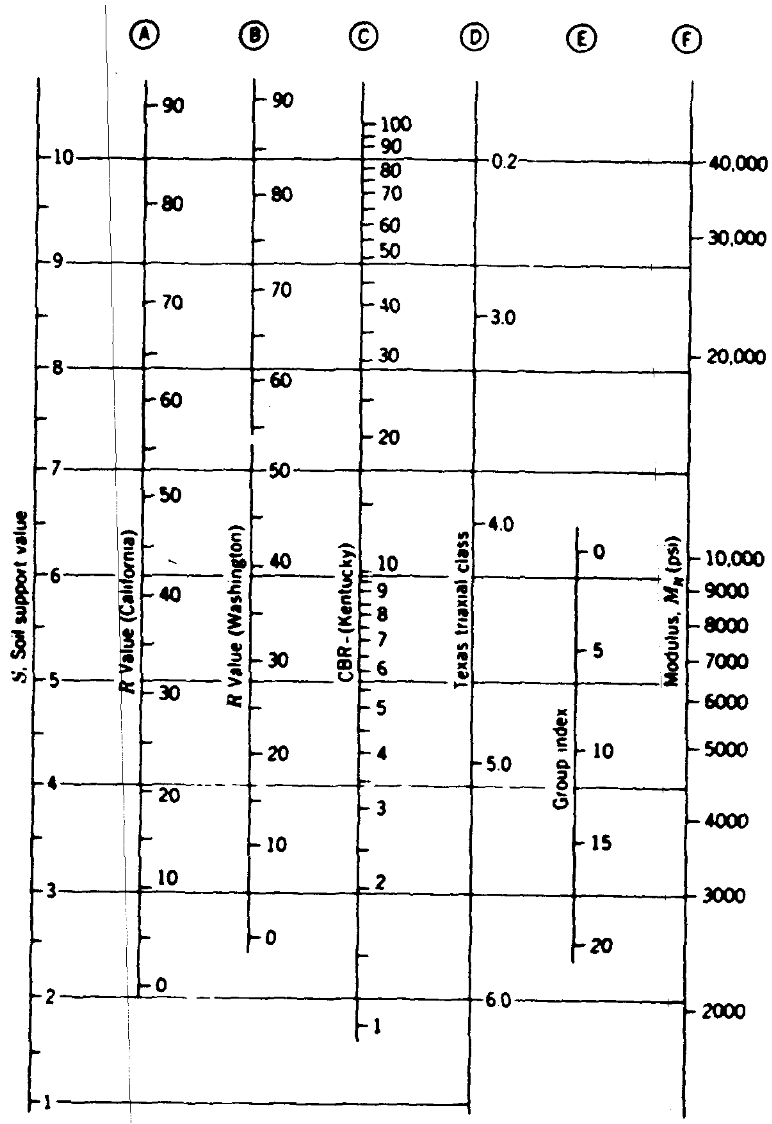


FIGURE 48. Soil support correlations (from Van Til et al., NCHRP 128)

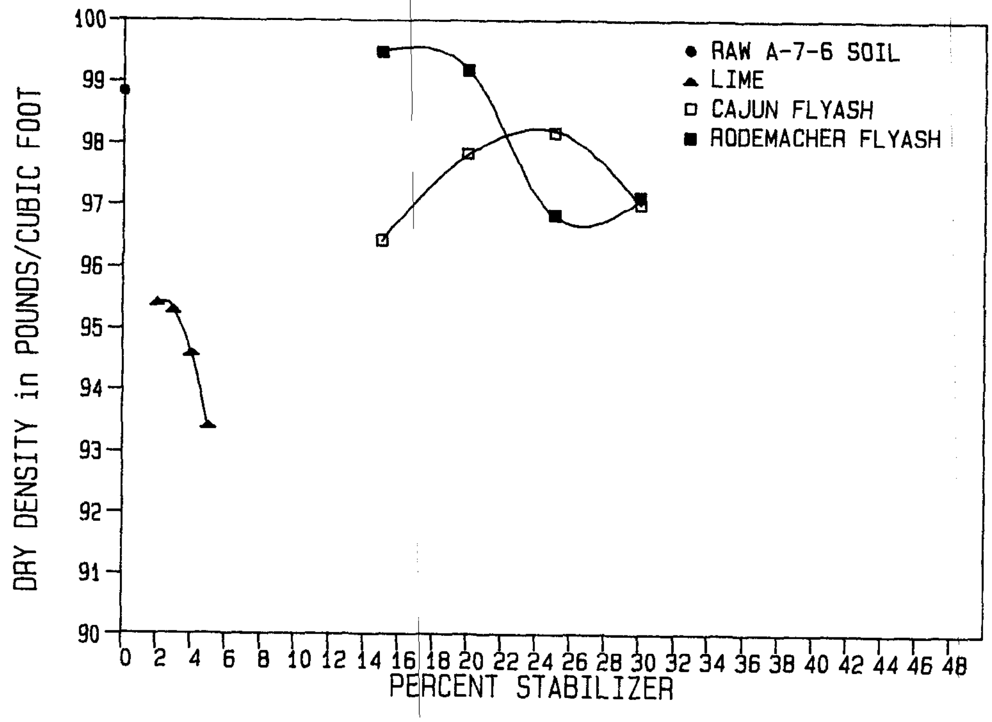


FIGURE 49. Dry density vs percent flyash - A-7-6(20) soil

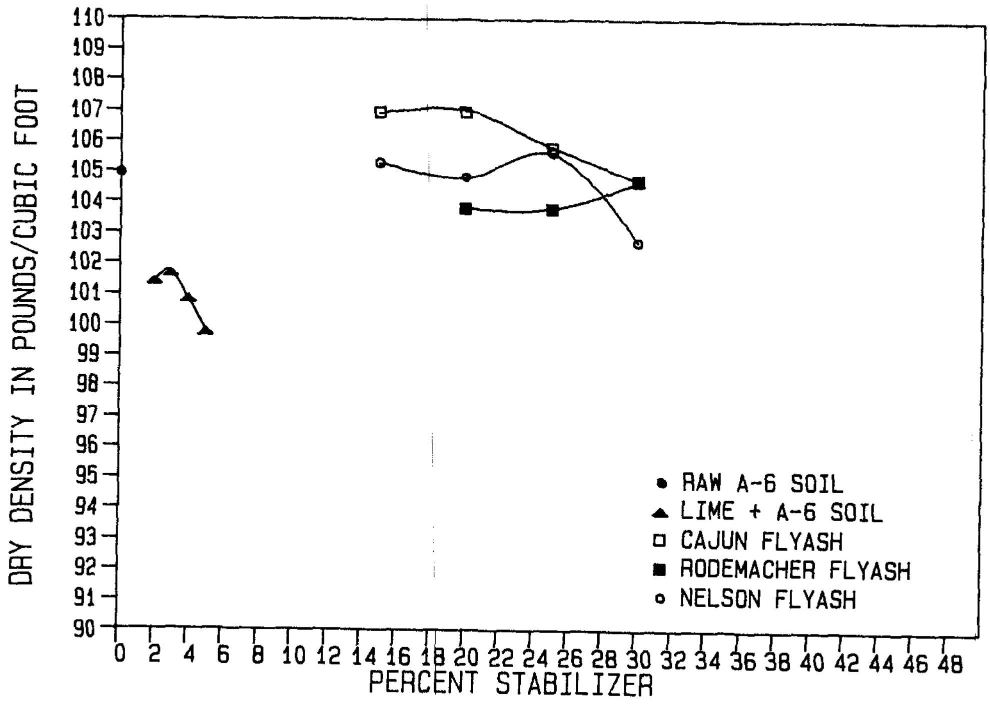


FIGURE 50. Maximum dry density vs percent flyash - A-6 soil

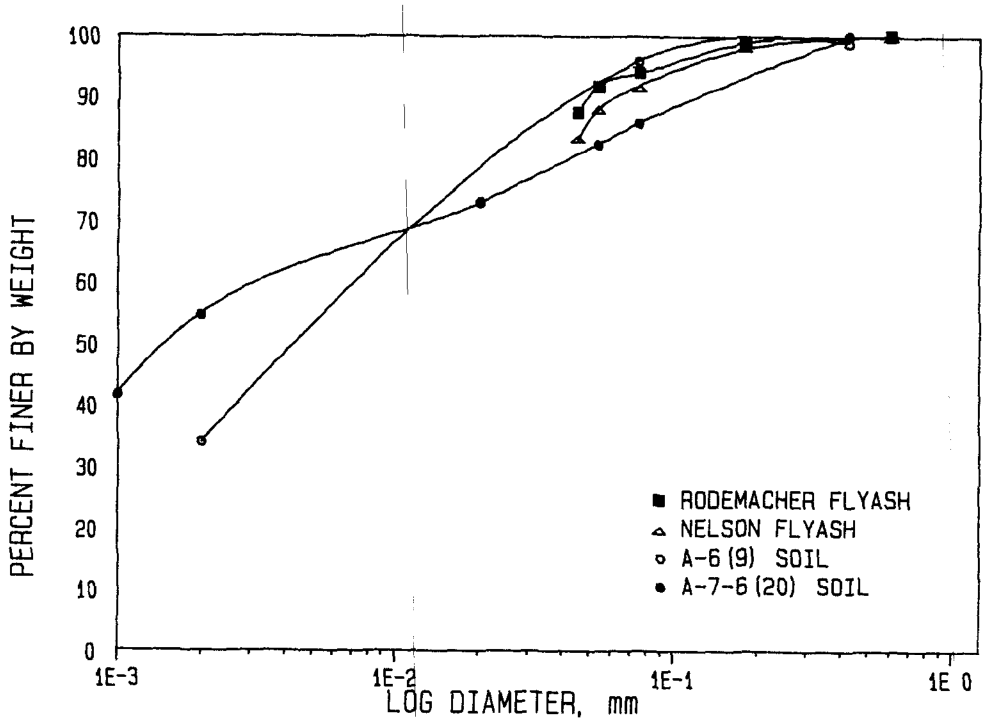


FIGURE 51. Grain size analysis - flyash & sub base materials

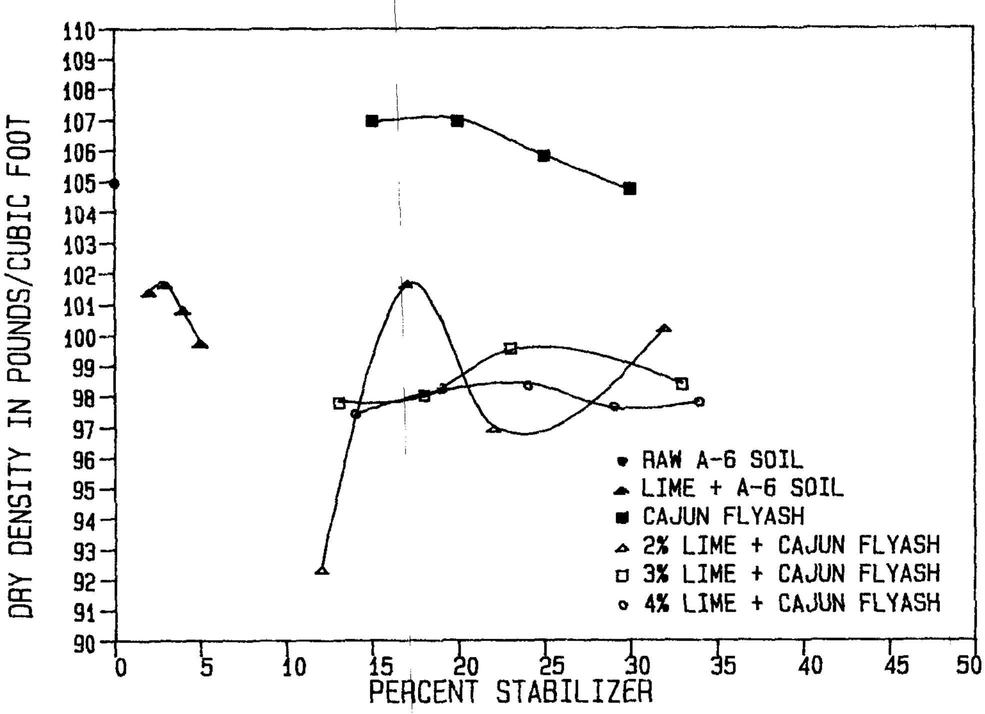


FIGURE 52. Maximum dry density vs percent flyash - A-6 soil

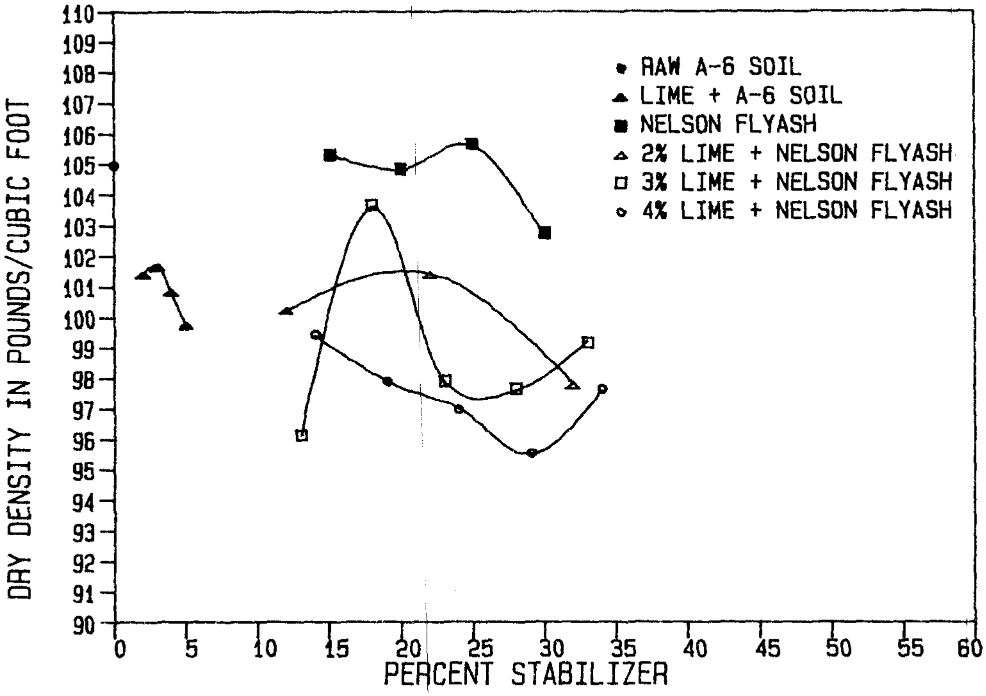


FIGURE 53. Maximum dry density vs percent flyash - A-6 soil

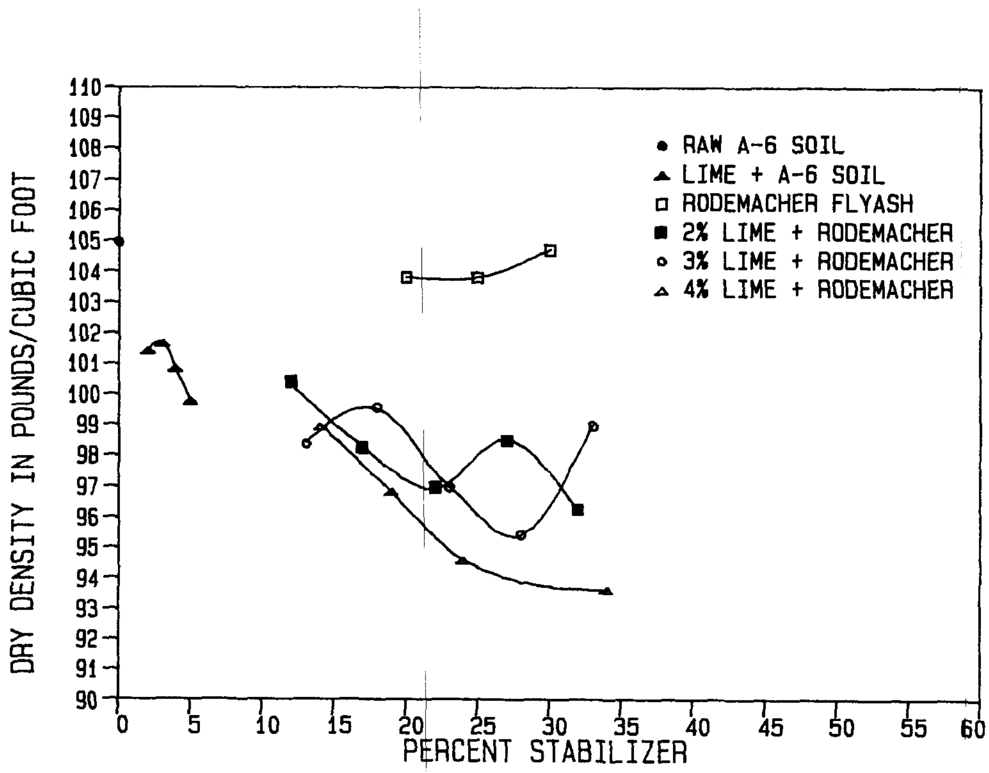


FIGURE 54. Maximum dry density vs percent flyash - A-6 soil

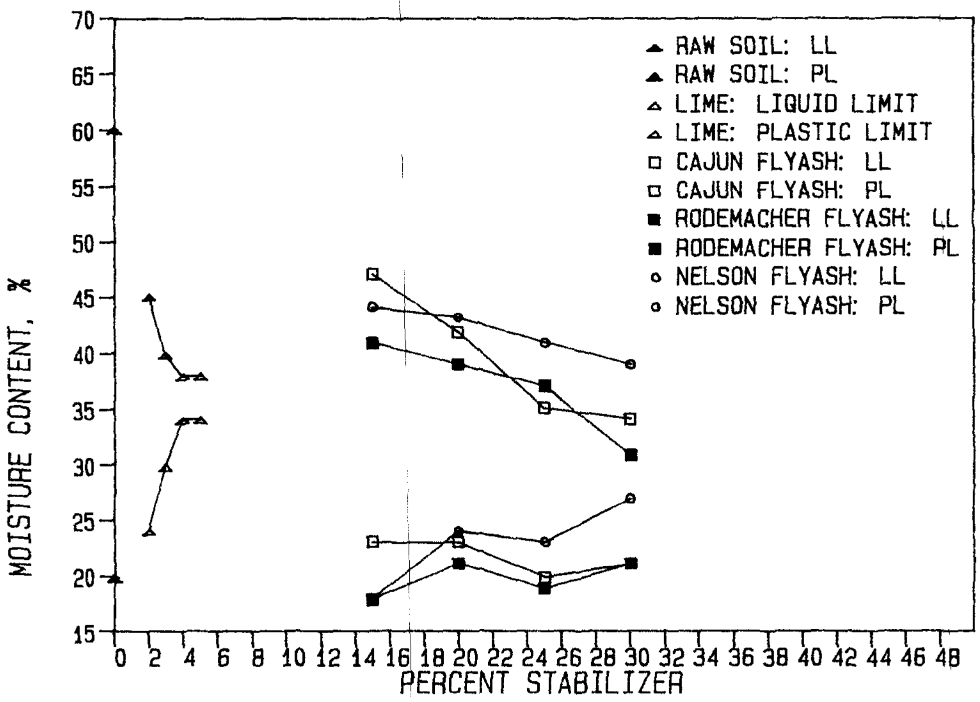


FIGURE 55. Atterberg tests vs stabilizer - A-7-6(20) clay

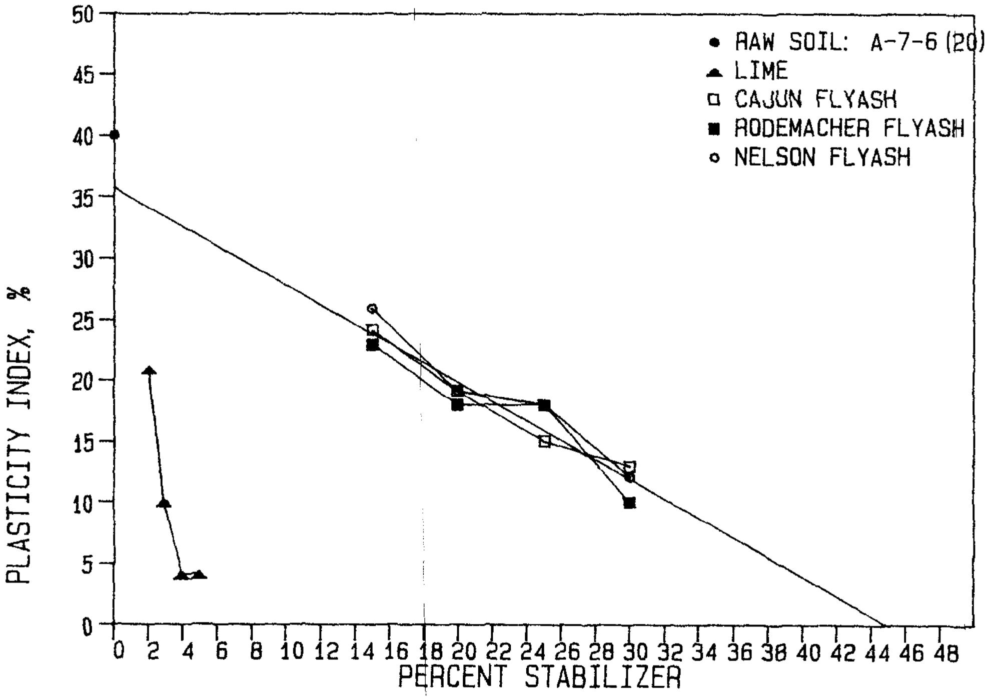


FIGURE 56. Plastic index vs stabilizer mix - A-7-6(20) soil

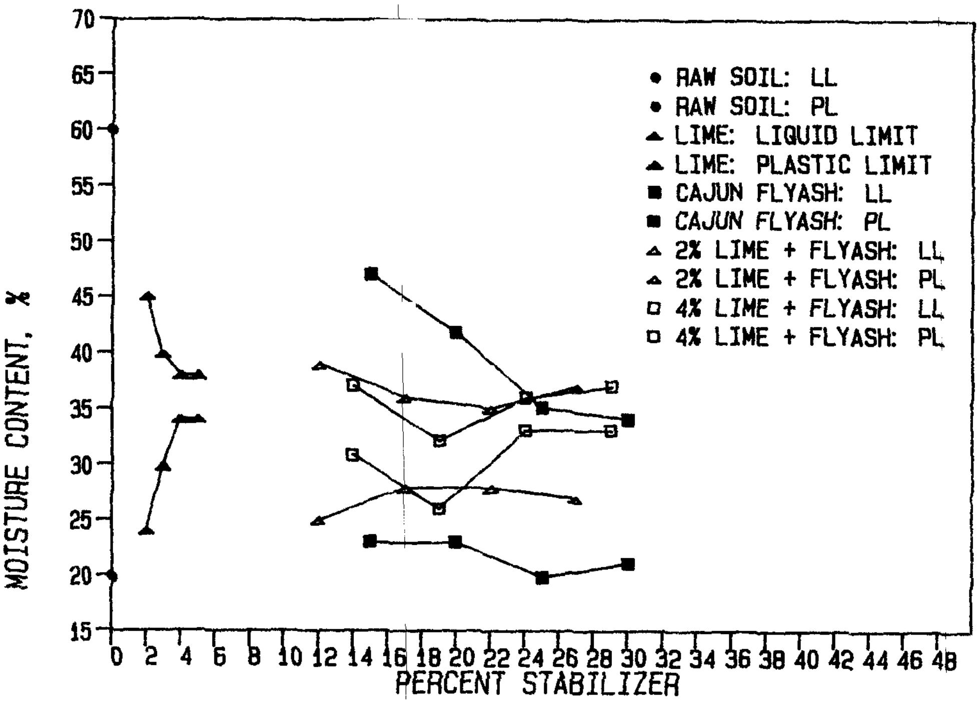


FIGURE 57. Atterberg tests - Lake Charles soil - A-7-6(20)

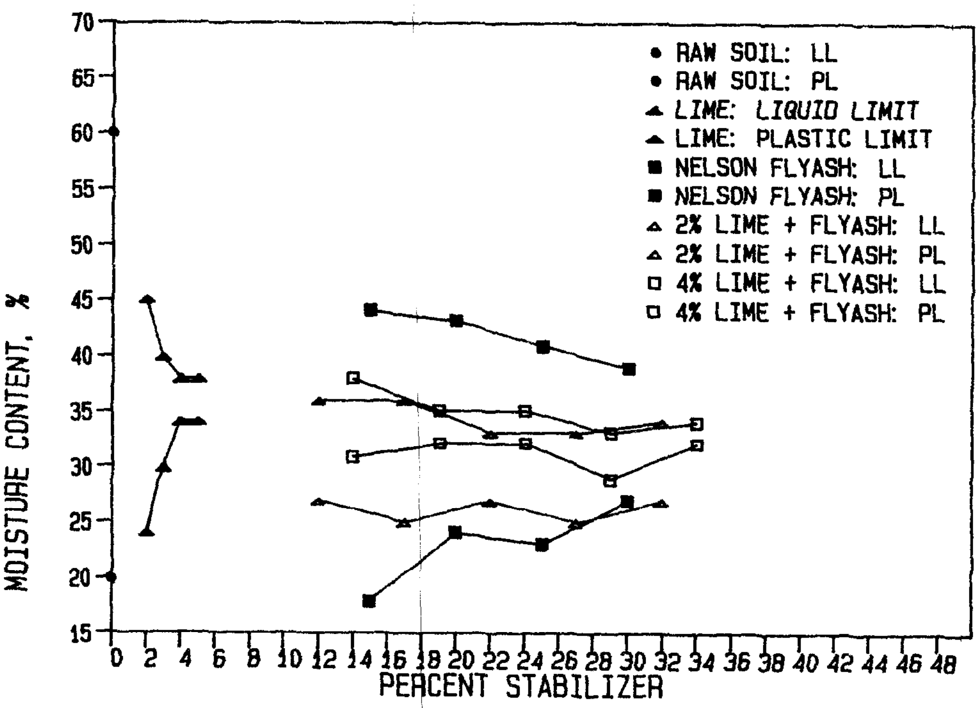


FIGURE 58. Atterberg tests - Lake Charles soil - A-7-6(20)

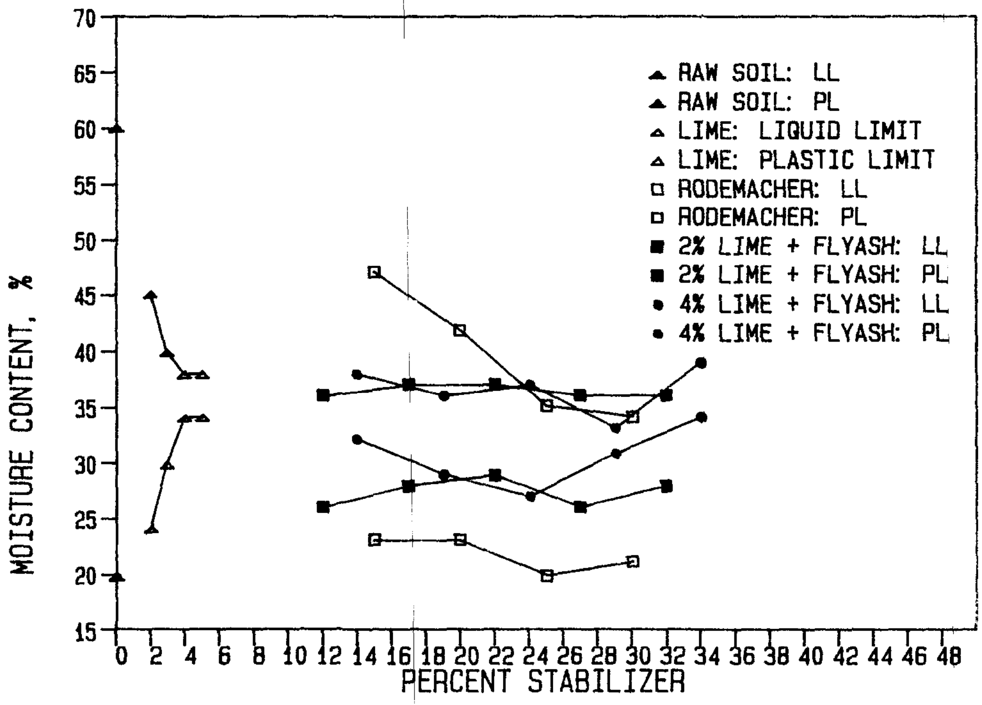


FIGURE 59. Atterberg tests - Lake Charles soil - A-7-6(20)

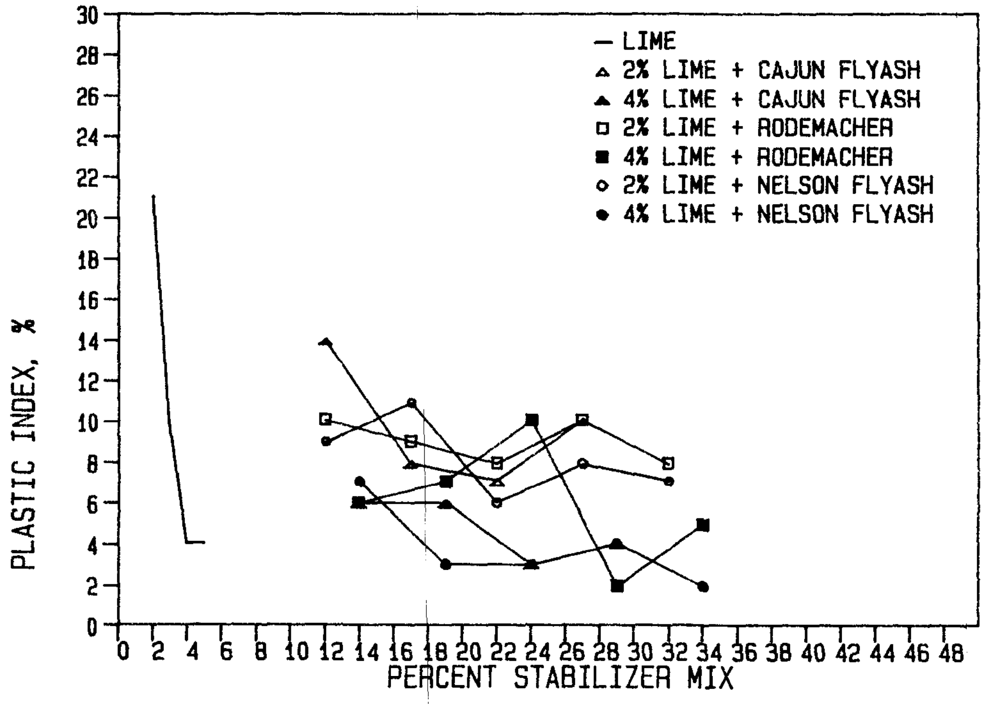


FIGURE 60. Plasticity index vs stabilizer mix - A-7-6(20)

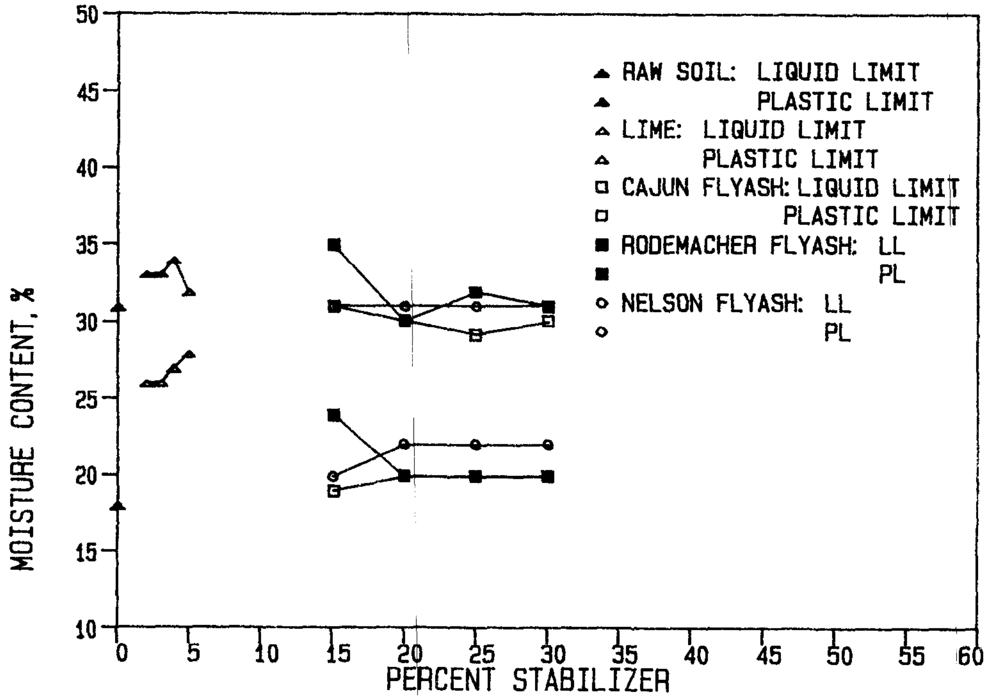


FIGURE 61. Silt-clay A-6: Atterberg limits vs stabilizer

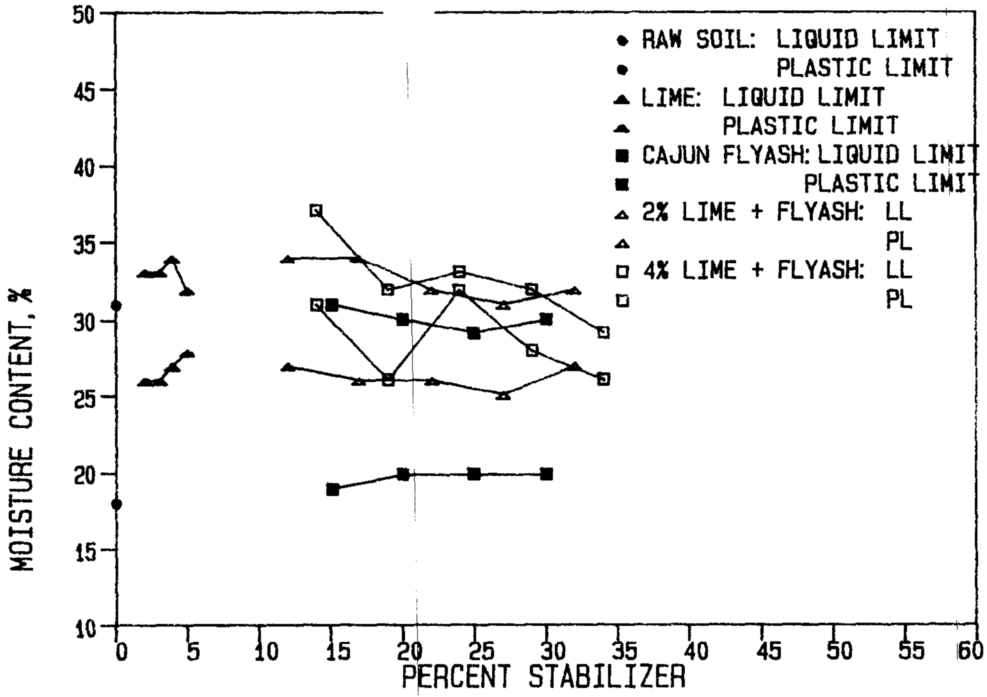


FIGURE 62. Silt-clay A-6: Atterberg limits vs stabilizer

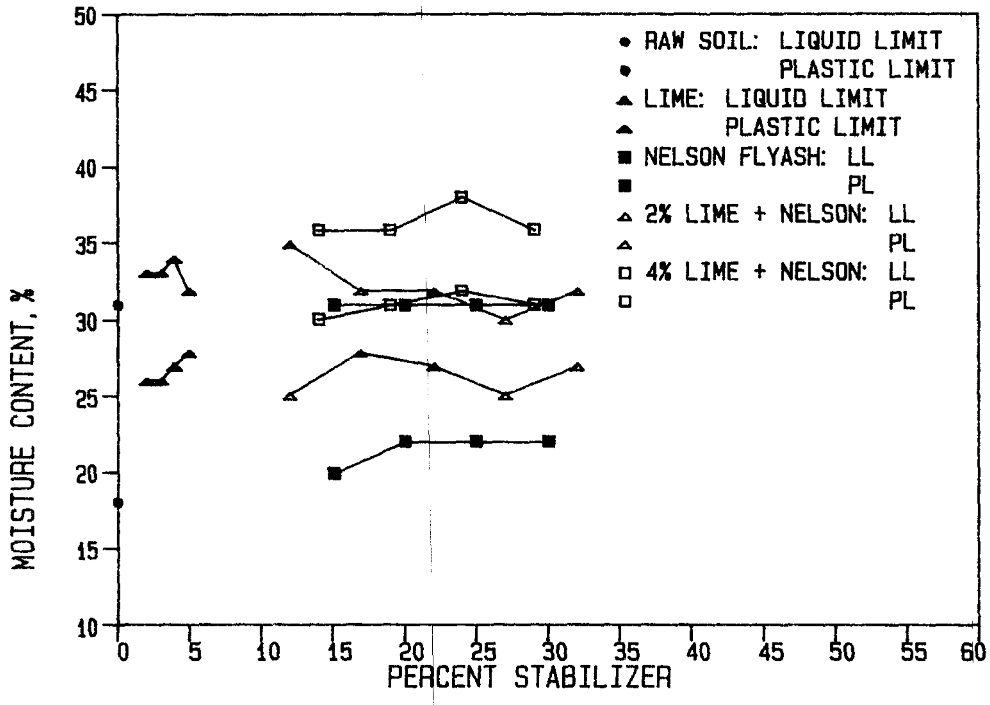


FIGURE 63. Silt-clay A-6: Atterberg limits vs stabilizer

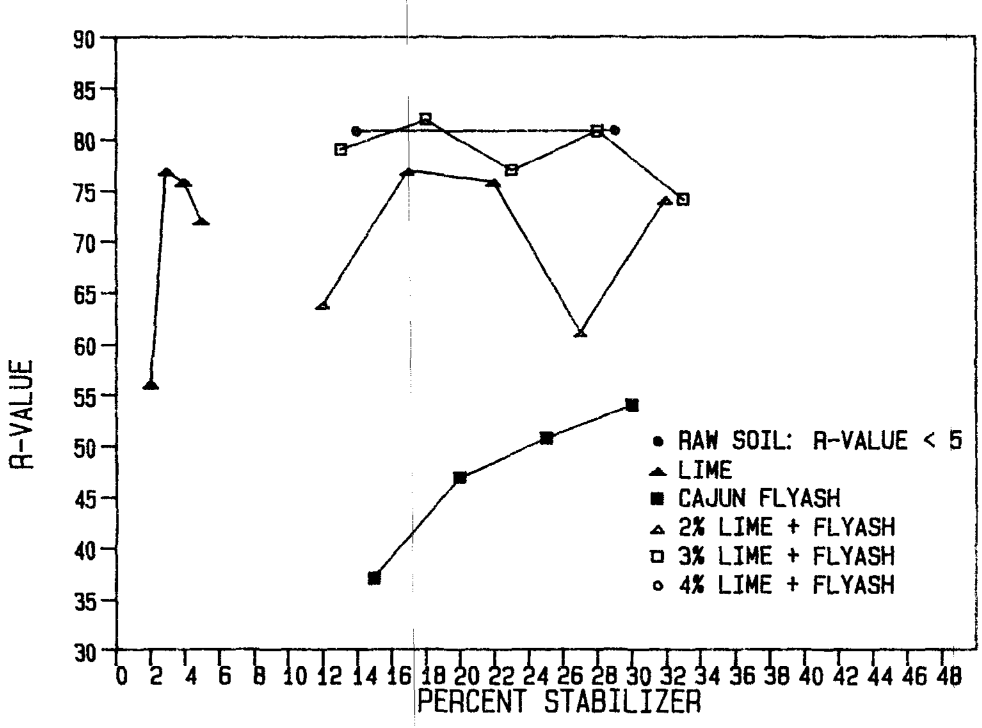


FIGURE 64. Stabilometer tests - Lake Charles soil - SR2821

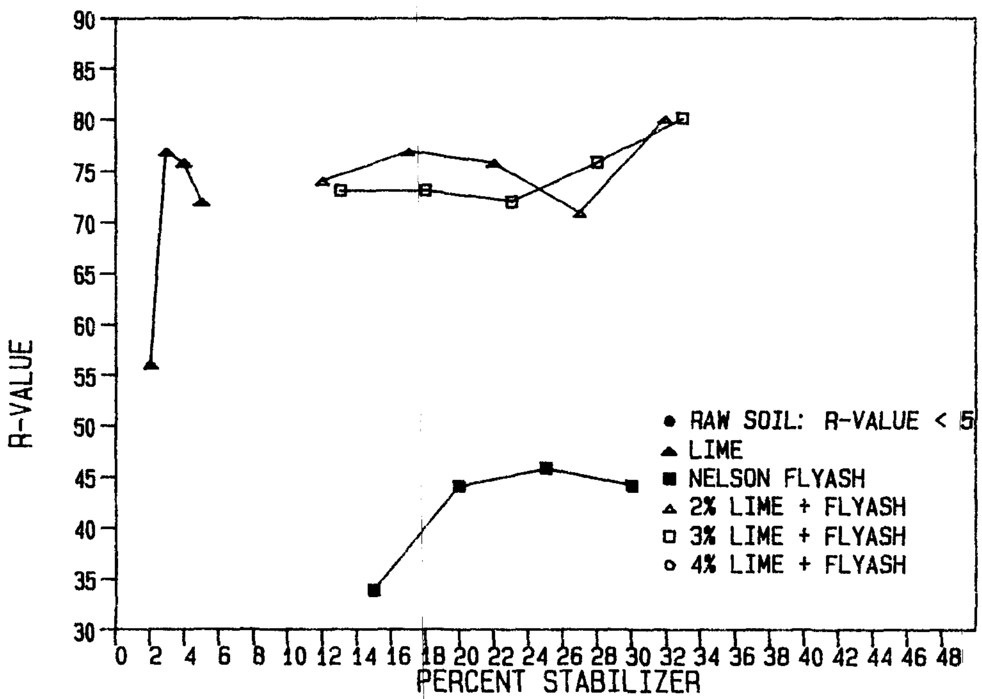


FIGURE 65. Stabilometer tests - Lake Charles soil - A-7-6(20)

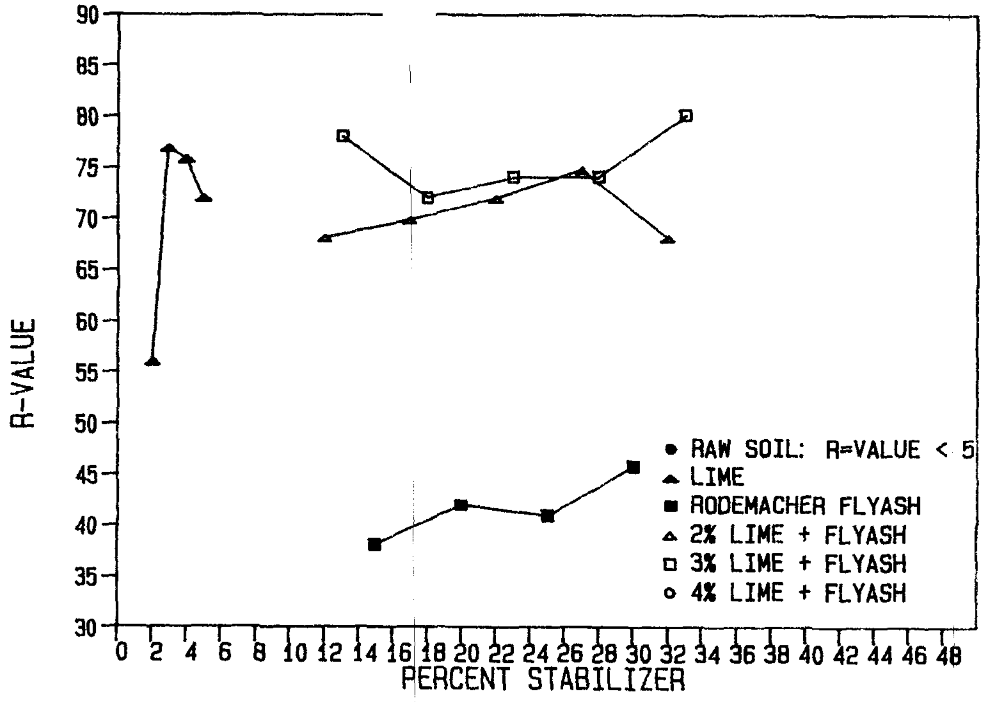


FIGURE 66. Stabilometer tests - Lake Charles soil - A-7-6(20)

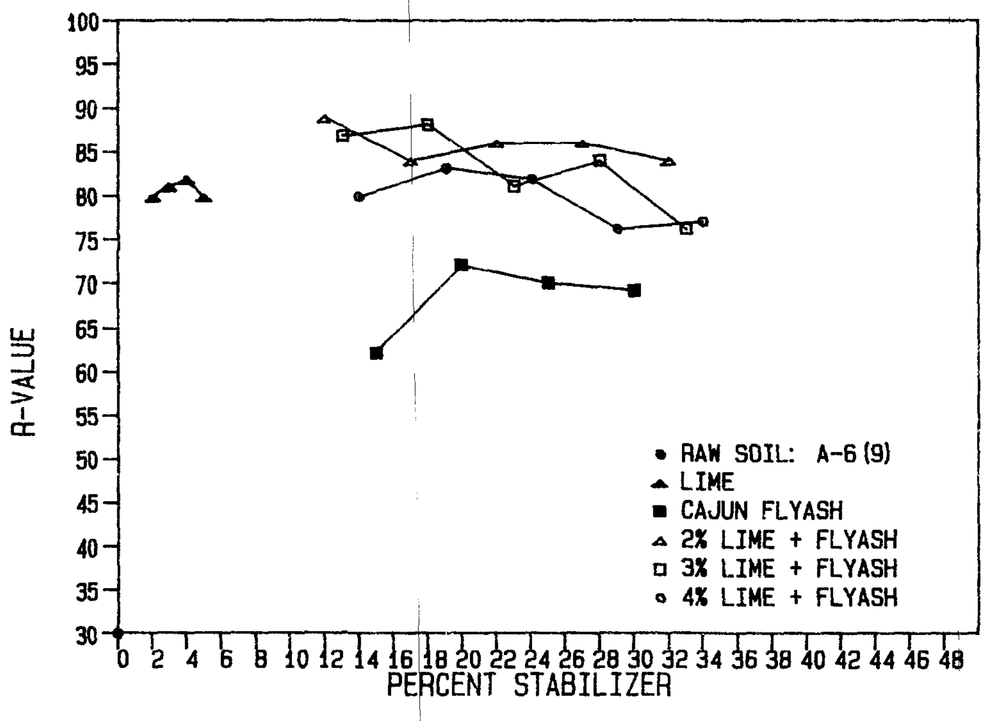


FIGURE 67. Silt-clay A-6(9):Stabilometer tests (Cajun flyash)

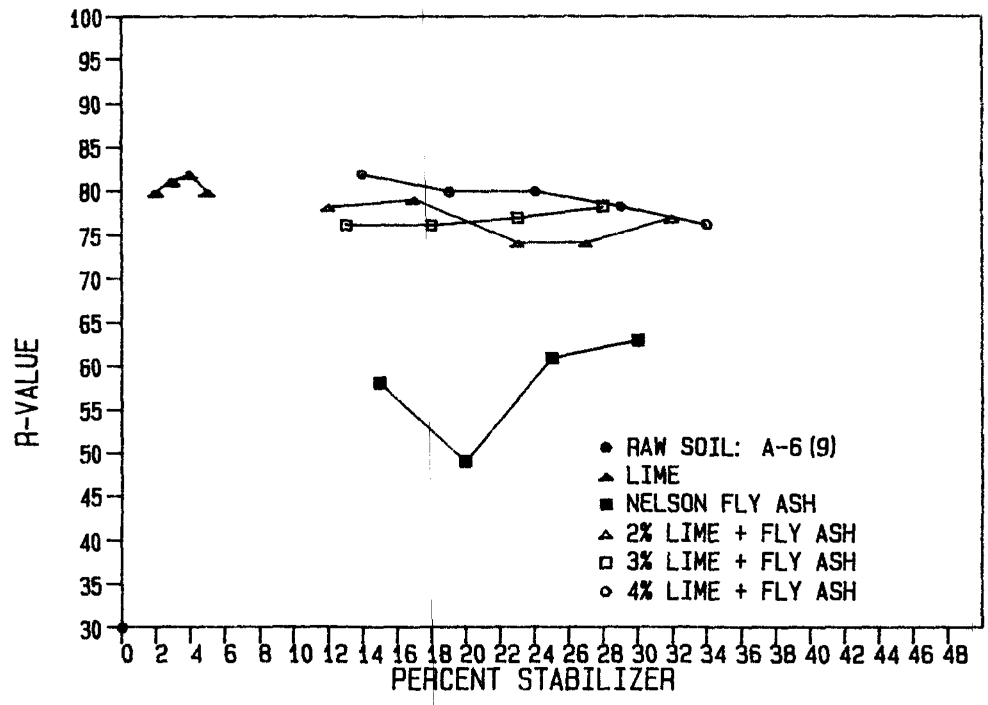


FIGURE 68. Silt-clay A-6(9): Stabilometer tests (Nelson)

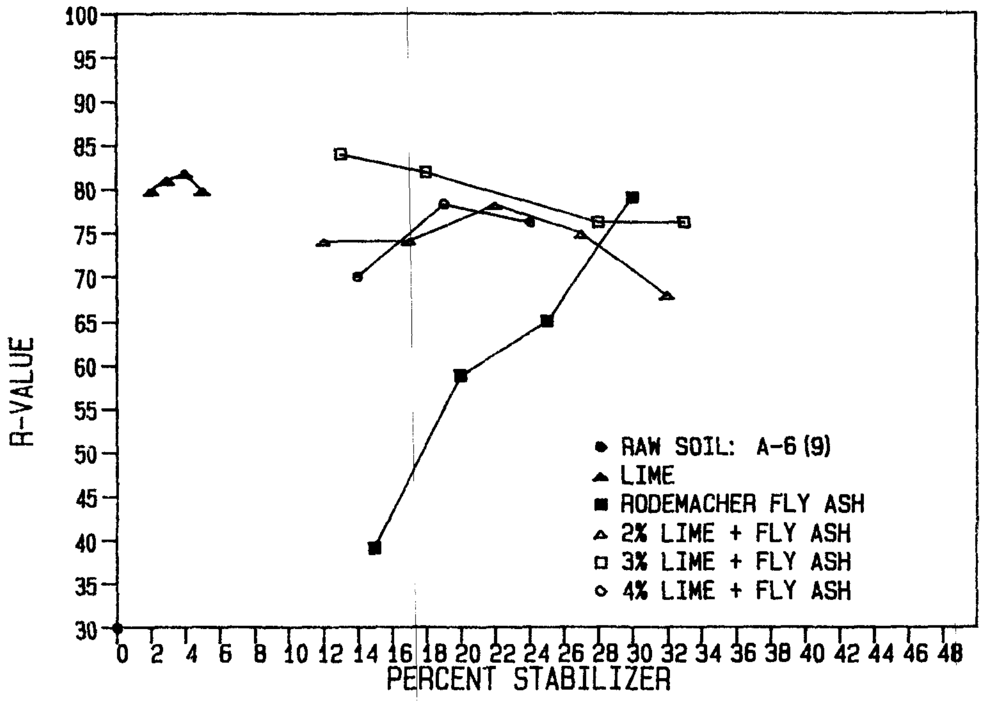


FIGURE 69. Silty-clay A-6(9): Stabilometer tests (Rodemacher)

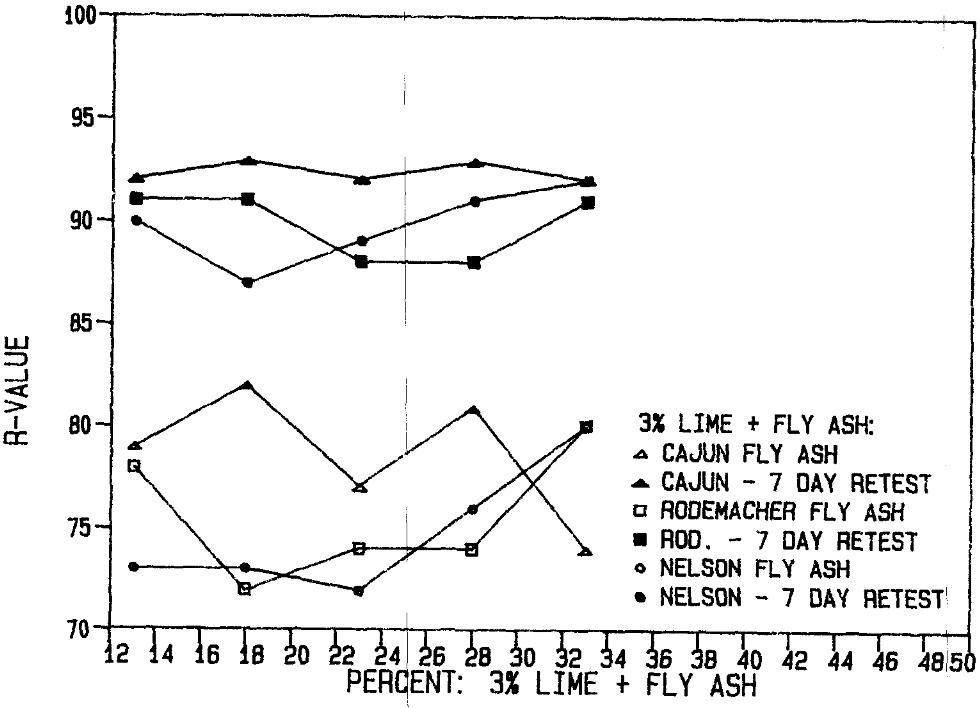


FIGURE 70. 7 day R-value retest - A-7-6 soil + lime + flyash

APPENDIX B

PHYSICAL AND CHEMICAL PROPERTIES OF
FLY ASH
LIME
PORTLAND CEMENT

Table B-1

Chemical and Physical Analysis of Fly Ashes *

Flyash Source	Cajun		Rodemacher		Nelson	
% Retained #325	9.2	7.6	12.0	11.0	13.9	18.3
Loss On Ignition	1.30	0.50	0.50	1.90	0.70	1.00
Total Oxides **	65.80	66.50	62.30	64.70	51.50	62.90
Calcium Oxide	21.50	24.50	27.20	24.00	25.20	25.80
Magnesium Oxide	4.40	4.70	4.90	4.50	4.90	2.90
Sulfer Trioxide	2.80	2.80	2.70	2.90	3.10	3.30
Alkalis	1.34	0.66	1.40	1.06	1.45	1.74

* - Material tested accordingto ASTM Designation : C 311.

** - $SiO_2 + Al_2O_3 + Fe_2O_3$

Table B-2

RODEMACHER

VARIABLE	N	MEAN	RANGE	CV	STD	MIN	MAX
% RETAINED ON THE #325	37	15.8838	10.50	17.354	2.75646	11.00	21.5
POZZALANIC ACTIVITY INDEX	31	87.0032	10.00	2.359	2.05272	83.00	93.0
AUTOCLAVE EXPANSION	31	0.0874	2.01	408.970	0.35752	-0.01	2.0
SPECIFIC GRAVITY	29	2.5590	0.35	2.815	0.07203	2.35	2.7
LOSS ON IGNITION	37	1.3730	2.70	55.593	0.76327	0.30	3.0
SULFUR TRIOXIDE CONTENT	37	2.7784	4.00	36.480	1.01356	1.50	5.5
TOTAL OXIDES	37	66.1081	15.60	4.931	3.25939	55.90	71.5
CALCIUM OXIDES	37	22.9297	12.20	13.781	3.15998	18.00	30.2
MAGNESIUM OXIDES CONTENT	37	4.1054	2.70	16.269	0.66789	3.10	5.8
MOISTURE CONTENT	31	1.3226	3.00	61.137	0.80858	0.00	3.0

CAJUN

VARIABLE	N	MEAN	RANGE	CV	STD	MIN	MAX
% RETAINED ON THE #325	45	11.2978	16.00	30.114	3.40217	5.00	21.00
POZZALANIC ACTIVITY INDEX	30	86.8300	11.50	2.497	2.16829	81.50	93.00
AUTOCLAVE EXPANSION	30	0.0370	0.11	118.160	0.04372	0.00	0.11
SPECIFIC GRAVITY	30	2.7010	0.20	1.848	0.04992	2.58	2.78
LOSS ON IGNITION	47	1.3277	4.70	66.349	0.88088	0.20	4.90
SULFUR TRIOXIDE CONTENT	48	2.5437	6.30	35.403	0.90057	1.50	7.80
TOTAL OXIDES	48	66.3729	13.40	3.924	2.60423	58.10	71.50
CALCIUM OXIDES	48	23.0250	12.70	10.292	2.36971	18.00	30.70
MAGNESIUM OXIDES CONTENT	47	4.0915	4.30	20.256	0.82878	1.10	5.40
MOISTURE CONTENT	37	1.0459	3.00	79.532	0.83186	0.00	3.00

NELSON

VARIABLE	N	MEAN	RANGE	CV	STD	MIN	MAX
% RETAINED ON THE #325	41	18.1561	20.00	23.358	4.24088	11.60	31.6
POZZALANIC ACTIVITY INDEX	26	85.3885	4.50	1.692	1.44453	81.40	85.9
AUTOCLAVE EXPANSION	28	0.0314	0.12	137.119	0.04309	-0.02	0.1
SPECIFIC GRAVITY	~~28~~	~~2.5761~~	~~0.30~~	~~3.046~~	0.07847	2.40	2.7
LOSS ON IGNITION	41	1.6317	3.20	45.625	0.74446	0.10	3.3
SULFUR TRIOXIDE CONTENT	41	3.4756	31.50	138.596	4.81704	1.50	33.0
TOTAL OXIDES	41	65.6829	25.20	8.164	5.36218	50.00	75.2
CALCIUM OXIDES	41	22.8780	10.30	11.966	2.73756	18.40	28.7
MAGNESIUM OXIDES CONTENT	41	3.7707	4.80	24.957	0.94107	1.00	5.8
MOISTURE CONTENT	38	1.6263	2.80	41.794	0.67970	0.10	2.9

Table B-3

Chemical Properties of Lime

Lab Number	22-422122	22-394023
Total Calcium plus Magnesium Oxides %	95.9 %	95.6 %

Table B-4

Portland Cement
Chemical and Physical Properties

Variable	SPECIFICATION	LONE STAR	BLUE CIRCLE
Loss on Ignition	3.0 max. %	1.3	1.5
Sulfur Trioxide	3.0 max. %	2.0	2.7
Iron & Alum. Oxide	12.0 max. %	8.0	5.9
Magnesium Oxide	5.0 max. %	0.4	1.0
Insoluble Residue	0.75 max. %	0.49	0.23
Tricalcium Aluminate	15.0 max. %	4.3	6.0
Ferric Oxide		3.9	2.2
Aluminum Oxide		4.1	3.7
Alkalis	0.6 max. %	0.44	0.25
Set Time Vicat Init.	0.75 hr. min.%	1.15	1.50
Set Time Vicat Final	8.0 hr. max.%	3.55	3.50
Autoclave Expansion	0.8 max.	0.03	0.04
Air Permeability	2600min 4200max	3310.00	3960.00
Air Content	12.0 max.%	10.40	7.50
Comp. Stength 72hrs	1800 min. PSI	3330.00	4310.00
Comp. Strength 7days	2800 min. PSI	5100.00	5220.00

APPENDIX C

PHYSICAL PROPERTIES OF BASE SOILS

Table C-1

Soil Properties

Variable	Sand	Sandy Silt
% Coarse Sand (Ret # 40)	40	3
% Fine Sand (Ret # 200)	52	62
% Silt	5	18
% Clay & Colloids	2	17
Liquid Limit	NP	21
Plasticity Index	NP	7
Max. Dry Wt. Den. (pcf)	109.1	119.1
Optimum Moisture (%)	13.0	12.1
Specific Gravity	2.62	2.63
Ph	6.6	5.4
Soil Classification	Sand	Sandy Loam
AASHTO Classification	A-3 (0)	A-2-4 (0)

Table C-2

Theoretical Dry Weight Densities and Optimum Moisture Contents

Soil Type -- Sand A-3(0)

Flyash Source	% Flyash	Percent Cement 4	6	8
Cajun	5	118.0 @ 9.2	119.5 @ 9.1	121.8 @ 8.5
	10	122.2 @ 8.4	124.0 @ 8.0	127.1 @ 7.6
	15	127.0 @ 7.2	129.0 @ 7.4	129.9 @ 7.2
	20	131.0 @ 6.6	133.4 @ 7.0	133.7 @ 6.0
Rodemacher	5	121.5 @ 8.6	121.5 @ 8.2	121.9 @ 8.5
	10	124.0 @ 8.0	123.8 @ 8.0	125.0 @ 8.0
	15	127.9 @ 7.6	127.3 @ 7.6	130.0 @ 6.7
Nelson	5	118.5 @ 8.9	120.0 @ 8.5	120.9 @ 8.3
	10	122.8 @ 7.9	123.6 @ 7.6	124.6 @ 7.7
	15	126.2 @ 7.2	130.6 @ 6.6	128.7 @ 7.3

Flyash Source	% Flyash	Percent Lime 2	4	6
Cajun	5	117.4 @ 9.2	120.6 @ 9.6	122.4 @ 8.6
	10	121.1 @ 8.3	123.4 @ 7.8	123.3 @ 9.0
	15	125.2 @ 7.5	127.4 @ 7.7	126.2 @ 8.4
Rodemacher	5	115.3 @ 9.9	118.0 @ 9.0	120.4 @ 8.5
	10	121.0 @ 7.8	123.0 @ 8.1	125.0 @ 8.5
	15	126.2 @ 8.1	129.2 @ 7.9	127.4 @ 8.5
Nelson	5	117.8 @ 9.5	118.7 @ 9.4	120.7 @ 9.0
	10	120.6 @ 8.9	124.1 @ 8.6	123.8 @ 8.5
	15	124.6 @ 8.4	127.2 @ 7.7	128.0 @ 7.7

% Flyash	Flyash Source Cajun	Rodemacher	Nelson
10	118.7 @ 8.7	119.2 @ 7.0	119.4 @ 8.5
15	124.3 @ 7.0	122.2 @ 8.0	124.2 @ 7.2
20	131.1 @ 6.8	126.9 @ 7.2	126.9 @ 6.8
25	132.8 @ 5.7	132.6 @ 5.1	130.2 @ 6.7

Table C-3

Theoretical Dry Weight Densities and Optimum Moisture Contents

Soil Type -- Sandy Silt A-2-4(0)

Flyash Source	% Flyash	Percent Cement		
		4	6	8
Cajun	5	122.5 @ 11.5	121.5 @ 11.9	121.8 @ 11.9
	10	122.5 @ 11.8	122.6 @ 11.8	122.3 @ 11.8
	15	121.7 @ 11.9	122.3 @ 11.1	122.6 @ 11.4
Rodemacher	5	117.5 @ 14.0	118.3 @ 13.8	117.3 @ 13.5
	10	120.1 @ 12.4	120.4 @ 12.4	121.1 @ 12.3
	15	120.6 @ 12.0	120.8 @ 12.2	120.9 @ 12.2
Nelson	5	118.4 @ 12.6	118.7 @ 12.7	119.0 @ 13.1
	10	119.1 @ 12.7	119.7 @ 13.1	119.2 @ 13.2
	15	119.8 @ 13.1	121.2 @ 13.0	121.4 @ 12.6

Flyash Source	% Flyash	Percent Lime		
		2	4	6
Cajun	5	119.2 @ 13.0	118.1 @ 13.2	113.6 @ 15.2
	10	118.6 @ 13.3	117.9 @ 13.8	117.9 @ 14.2
	15	118.0 @ 13.2	119.1 @ 12.8	118.7 @ 13.4
Rodemacher	5	114.0 @ 14.4	114.2 @ 14.6	114.2 @ 14.6
	10	115.8 @ 14.2	115.2 @ 14.1	114.4 @ 14.6
	15	116.0 @ 14.4	116.3 @ 14.0	115.6 @ 14.0
Nelson	5	116.4 @ 14.0	116.0 @ 14.2	116.3 @ 14.0
	10	118.4 @ 13.0	116.6 @ 14.0	117.1 @ 13.8
	15	119.0 @ 13.2	118.4 @ 13.1	117.2 @ 14.0

% Flyash	Flyash Source		
	Cajun	Rodemacher	Nelson
15	124.0 @ 10.9	122.3 @ 10.5	120.1 @ 12.4
20	125.8 @ 10.3	122.8 @ 11.5	122.2 @ 11.7
25	126.0 @ 10.3	121.5 @ 12.0	122.4 @ 11.5
30	124.8 @ 11.0	120.5 @ 11.4	122.5 @ 10.9

Table C-4

Unconfined Compressive Strengths -PSI

Additive -- Flyash

% Fly Ash	Soil Type	Cajun 7	Cajun 28	Cajun 56	Rodemacher 7	Rodemacher 28	Rodemacher 56	Nelson 7	Nelson 28	Nelson 56
10	S	28	31	31	79	76	62	53	52	63
15	A	75	81	107	136	160	199	105	136	154
20	N	226	251	333	265	323	454	183	241	319
25	D	556	693	637	494	654	705	322	410	580
	S									
15	A S	59	114	58	49	39	90	80	81	91
20	N I	99	109	103	85	81	96	150	130	146
25	D L	142	115	148	98	106	127	131	210	280
30	Y T	152	210	260	160	137	178	216	268	323

Vacuum Saturation Strengths - PSI

Additive -- Flyash

% Fly Ash	Soil Type	Cajun 7	Cajun 28	Cajun 56	Rodemacher 7	Rodemacher 28	Rodemacher 56	Nelson 7	Nelson 28	Nelson 56
10	S	27	32	35	63	69	63	42	54	51
15	A	72	115	83	146	172	177	108	133	136
20	N	188	244	244	239	302	424	197	308	272
25	D	582	694	653	457	464	714	264	442	488
	S									
15	A L	45	139	62	64	43	75	72	74	80
20	N O	71	77	86	63	73	100	131	115	158
25	D A	108	106	158	90	91	123	159	192	250
30	Y M	113	182	191	150	147	150	202	202	271

Table C-5

Unconfined Compression and Vacuum Saturation Strengths-PSI

Additive -- Portland Cement

Test	Cement	SAND			SANDY LOAM		
		Age in Days			Age in Days		
COMPRESSION	%	7	28	56	7	28	56
	4	104	143	142	321	435	535
	6	231	290	331	496	651	719
	8	363	593	619	507	779	913
	10	665	843	1021	718	934	1032
VACUUM		7	28	56	7	28	56
	4	90	151	138	283	411	375
	6	222	304	325	378	464	604
	8	338	556	597	509	704	732
	10	647	742	936	569	735	716

Table C-6

Unconfined Compressive Strengths

Soil Type -- Sand A-3 (0)

Additives--- Flyash and Portland Cement

Flyash Source	% Fly ASH	4 % Cement			6 % Cement			8 % Cement		
		7	28	56	7	28	56	7	28	56
none	0	104	143	142	231	290	331	363	593	619
Cajun	5	190	298	404	330	597	702	498	714	1019
	10	333	431	704	447	808	954	810	1053	1552
	15	477	761	1093	741	1069	1233	1212	1806	2149
Rod	5	228	284	445	396	525	716	520	801	789
	10	320	524	621	515	838	944	784	1363	1554
	15	434	875	1198	657	1205	1359	1150	2093	2101
Nel	5	192	302	421	387	670	593	639	907	993
	10	306	492	638	523	976	1013	748	1133	1273
	15	337	813	1215	831	1144	1464	981	1935	2082

Fly Ash and Lime Combinations

Flyash Source	% Fly ASH	2 % Lime			4 % Lime			6 % Lime		
		7	28	56	7	28	56	7	28	56
Cajun	5	28	33	31	41	46	64	75	89	121
	10	106	122	150	148	169	196	169	209	207
	15	242	278	349	349	431	469	375	421	454
Rod	5	34	38	50	53	68	96	105	104	142
	10	128	140	324	166	184	459	232	277	452
	15	254	339	383	367	475	886	379	456	909
Nel	5	27	28	30	38	45	59	66	81	105
	10	81	100	112	116	151	234	172	232	259
	15	160	222	267	277	341	378	323	356	492

Table C-7

Unconfined Compressive Strengths - PSI

Soil Type --- Sandy Silt A-2-4(0)

Additives -- Flyash and Portland Cement

Flyash Source	% Fly ASH	4 % Cement			6 % Cement			8 % Cement		
		7	28	56	7	28	56	7	28	56
none		321	435	535	496	651	719	507	779	913
Caj	5	473	705	753	599	812	1021	852	1066	1237
	10	537	615	910	743	917	1296	906	1158	1402
	15	530	531	676	623	1135	1413	1024	1298	1459
Rod	5	393	418	522	462	613	684	656	905	1021
	10	489	581	684	619	866	1061	764	960	1199
	15	500	650	629	694	918	1178	870	1148	1313
Nel	5	281	587	609	372	757	641	504	823	1119
	10	335	618	659	467	729	888	562	877	1143
	15	363	624	698	528	791	1095	632	1042	1332

Fly Ash and Lime Combinations

Flyash Source	% Fly ASH	2 % Lime			4 % Lime			6 % Lime		
		7	28	56	7	28	56	7	28	56
Caj	5	19	83	90	35	90	149	32	103	176
	10	33	57	150	49	46	142	75	87	138
	15	82	106	152	108	107	192	101	138	173
Rod	5	51	69	85	57	102	100	70	103	154
	10	85	116	136	93	132	156	97	135	155
	15	102	137	143	118	182	223	117	190	236

Table C-8

Vacuum Saturation Strengths - PSI

Soil Type -- Sand A-3(0)

Additives -- Flyash and Portland Cement

Flyash Source	% Fly ASH	4 % Cement			6 % Cement			8 % Cement		
		7	28	56	7	28	56	7	28	56
none		90	151	138	222	304	325	338	556	597
Cajun	5	178	279	345	303	569	718	495	869	997
	10	308	522	747	515	913	1044	833	1122	1615
	15	438	719	1008	693	1120	1395	1294	1419	2295
Rodemacher	5	179	281	398	426	544	695	554	696	894
	10	310	439	503	531	851	1020	764	1320	1745
	15	437	816	1247	647	1267	1519	1148	1907	2208
Nelson	5	184	334	281	394	598	589	597	933	964
	10	274	516	694	646	809	934	755	1243	1292
	15	316	775	1028	718	1104	1283	917	1609	2135

Additives -- Flyash and Lime

Flyash Source	% Fly ASH	2 % Lime			4 % Lime			6 % Lime		
		7	28	56	7	28	56	7	28	56
Cajun	5	20	27	32	33	39	54	63	97	132
	10	90	106	136	143	160	187	155	195	207
	15	235	233	328	338	398	430	322	421.	465
Rodemacher	5	34	40	45	46	63	79	94	94	120
	10	128	146	175	165	195	439	217	262	504
	15	238	282	414	381	451	846	388	495	814
Nelson	5	28	23	29	34	44	55	53	71	102
	10	73	96	122	108	119	243	163	211	284
	15	171	212	294	231	316	489	322	390	483

Table C-9

Vacuum Saturation Strengths - PSI

Soil Type -- Sandy Silt A-2-4(0)

Additives -- Flyash and Portland Cement

Flyash Source	% Fly ASH	4 % Cement			6 % Cement			8 % Cement		
		7	28	56	7	28	56	7	28	56
none	0	283	411	375	378	464	604	509	704	732
Cajun	5	445	613	650	562	781	950	741	947	1185
	10	461	559	803	641	887	1148	799	1050	1417
	15	488	501	719	574	755	1395	859	1123	1426
Rodemacher	5	379	369	469	462	597	631	560	849	862
	10	412	535	631	550	687	926	707	890	1210
	15	465	586	602	615	796	1146	759	993	1174
Nelson	5	239	538	512	363	703	600	453	764	955
	10	281	505	583	422	560	901	509	664	1074
	15	305	490	668	475	721	952	576	961	1194

Additives -- Flyash and Lime

Flyash Source	% Fly ASH	2 % Lime			4 % Lime			6 % Lime		
		7	28	56	7	28	56	7	28	56
Cajun	5	13	69	68	23	71	127	22	81	147
	10	34	45	128	32	38	124	48	56	118
	15	107	86	132	73	69	143	79	108	131
Rodemacher	5	45	60	68	39	88	81	39	97	131
	10	72	103	115	79	111	125	83	119	129
	15	85	115	128	101	147	174	111	154	211

TABLE C-10

FHWA-59-8

MIXTURE PROPORTIONING CONCEPTS
LIME/FLY ASH/AGGREGATE BASES*

Note:	This procedure presents basic concepts and is not intended to serve as a complete guide to mixture proportioning.
Step 1:	Determine the total quantity of lime plus fly ash required to produce the maximum dry density when combined with the proposed project aggregate (optimum fines content).
Step 2:	Select the proportion of lime to fly ash*. This ratio varies from one source to another. Initial proportions should be based on past experience where possible. Best results can be obtained by preparing trial batches with several ratios and selecting the most economical one that produces a mix exceeding performance requirements per ASTM C-593.
Step 3:	Adjust the mix to compensate for construction variability by increasing the designated lime content by about 1/2 percent and the fly ash by about 1 1/2 percent.

* An ASTM Class C fly ash could be used alone or in combination with lime or cement.

APPENDIX D

PHYSICAL PROPERTIES OF EMBANKMENT SOILS

TABLE D-1

EMBANKMENT SOILS

Variable	Lake Charles Clay	Baton Rouge Clay
% Coarse Sand (Ret # 40)	0	1
% Fine Sand (Ret # 200)	14	3
% Silt	31	62
% Clay & Collids	55	34
Liquid Limit	60	31
Plasticity Index	40	13
Activity	0.47	0.14
Max. Dry Wt. Den. (pcf)	98.8	104.9
Optimum Moisture (%)	23.1	19
R-Value	<5	30
AASHTO Classification	A-7-6 (20)	A-6 (9)

TABLE D-2

Theoretical Dry Weight Densities and Optimum Moisture Contents

Soil Type - A-7-6 (20) Clay

FLYASH SOURCE	% FLYASH	Percent Lime				
		0	2	3	4	5
NONE	0	98.8 @ 23.1	95.4 @ 25.5	95.3 @ 26.1	94.6 @ 26.4	93.4 @ 27.4
CAJUN	10					
	15	96.4 @ 23.6				
	20	97.8 @ 24.7				
	25	98.2 @ 23.2				
	30	97.0 @ 24.0				
NELSON						
RODEMACHER	10					
	15	99.5 @ 22.6				
	20	99.2 @ 23.8				
	25	96.8 @ 24.2				
	30	97.1 @ 21.8				

TABLE D-3

Theoretical Dry Weight Densities and Optimum Moisture Contents

Soil Type - A-6 (9) Silty Clay

FLYASH SOURCE	% FLYASH	Percent Lime				
		0	2	3	4	5
NONE	0	104.9 @ 19.0	101.4 @ 20.8	101.7 @ 19.6	100.8 @ 20.8	99.8 @ 21.5
CAJUN	10		92.4 @ 22.0	97.8 @ 21.5	97.4 @ 20.0	
	15	106.9 @ 17.2	101.6 @ 22.2	98.0 @ 21.8	98.2 @ 21.0	
	20	107.0 @ 17.8	97.0 @ 22.0	99.5 @ 21.0	98.4 @ 21.7	
	25	105.8 @ 17.5			97.6 @ 21.9	
	30	104.7 @ 18.3	100.2 @ 21.7	98.4 @ 22.4	97.8 @ 21.5	
NELSON	10		100.2 @ 21.2	96.1 @ 21.5	99.4 @ 21.2	
	15	105.3 @ 18.5	-	103.6 @ 22.3	97.9 @ 22.7	
	20	104.8 @ 16.5	101.4 @ 18.4	97.9 @ 21.2	97.0 @ 23.2	
	25	105.6 @ 18.3		97.6 @ 22.0	95.5 @ 20.6	
	30	102.7 @ 21.0	97.8 @ 22.0	99.2 @ 20.9	97.6 @ 21.6	
RODEMACHER	10		100.3 @ 20.3	98.4 @ 22.2	99.0 @ 21.8	
	15		98.2 @ 21.6	99.5 @ 21.4	96.8 @ 22.5	
	20	103.8 @ 19.2	96.9 @ 22.6	97.0 @ 19.9	94.6 @ 23.0	
	25	103.8 @ 19.2	98.5 @ 21.5	95.4 @ 22.4		
	30	104.7 @ 18.9	96.2 @ 22.7	99.0 @ 21.1	93.6 @ 24.2	

TABLE D-4

Liquid and Plastic Limits

Soil Type: A-7-6 (20) Clay

FLYASH SOURCE	% FLYASH	Percent Lime				
		0	2	3	4	5
NONE	0	60* 40**	45 21	40 10	38 4	38 4
CAJUN	10		39 14	36 8	37 6	
	15	47 24	36 8	36 9	32 6	
	20	42 19	35 7	38 9	36 3	
	25	35 15	37 10	36 8	37 4	
	30	34 13				
NELSON	10		36 9	34 5	38 7	
	15	44 26	36 11	34 8	35 3	
	20	43 19	33 6	35 6	35 3	
	25	41 18	33 8	33 4	33 4	
	30	39 12	34 7	34 2	34 2	
RODEMACHER	10		36 10	36 8	38 6	
	15	41 23	37 9	35 10	36 7	
	20	39 18	37 8	39 8	37 10	
	25	37 16	36 10	38 8	33 2	
	30	31 10	36 8	36 9	39 5	

*Liquid Limit

**Plasticity Index

TABLE D-5

Liquid Limit and Plasticity Index

Soil Type: A-6 (9) Clay

FLYASH SOURCE	% FLYASH	Percent Lime				
		0	2	3	4	5
NONE	0	31* 13**	33 7	33 7	34 7	32 4
CAJUN	10		34 7	34 5	37 6	
	15	31 12	34 8	36 7	32 6	
	20	30 10	32 6	33 4	33 1	
	25	29 9	31 6	32 2	32 4	
	30	30 10	32 5	35 6	29 3	
NELSON	10		35 10	31 5	36 6	
	15	31 11	32 4	35 7	36 5	
	20	31 9	32 5	31 5	38 6	
	25	31 9	30 5	34 4	36 5	
	30	31 9	32 5	32 5		
RODEMACHER	10		34 7	34 5	31 4	
	15	35 11	32 6	35 5	35 5	
	20	30 10	33 7	32 5	32 3	
	25	32 12	33 6	31 1	34 5	
	30	31 11	33 7	36 9	39 5	

*Liquid Limit

**Plasticity Index

TABLE D-6
R-Values of 240 psi Exudation Pressure
72 Hr. Slake at 20%

Soil Type: A-7-6 (20) Clay

FLYASH SOURCE	% FLYASH	Percent Lime				
		0	2	3	4	5
NONE	0	<5	57	71	68 (96)	72
CAJUN	10	-	64	79 (92)	81	
	15	37	77	82 (93)	-	
	20	47	76	77 (92)	-	
	25	51	61	81 (93)	81 (92)	
	30	54	74 (92)	74 (92)	-	
NELSON	10	-	74	73 (90)	-	
	15	34	77	73 (87)	-	
	20	44 (66)	76	72 (89)	-	
	25	46	71	76 (91)	-	
	30	44	80 (90)	80 (92)	-	
RODEMACHER	10	-	68	78 (91)	-	
	15	38 (58)	70	72 (91)	-	
	20	42 (61)	72	74 (88)	-	
	25	41	75	74 (88)	-	
	30	46	68 (86)	80 (91)	-	

( ) 7 day retest

TABLE D-7

R-Value at 240 psi Exudation Pressure
72 Hr. Slake at 20%

Soil Type: A-6 (9) Clay

FLYASH SOURCE	% FLYASH	Percent Lime				
		0	2	3	4	5
NONE	0	30	80	81	82	80
CAJUN	10		89	87	80	
	15	62	84	88	83	
	20	72	86	81	82	
	25	70	86	84	76	
	30	69	84	76	77	
NELSON	10		78	76	82	
	15	58	79	76	80	
	20	49	74	77	80	
	25	61	74	78	78	
	30	63	77	-	76	
RODEMACHER	10		74	84	70	
	15	39	74	82	78	
	20	59	78	-	76	
	25	65	75	76	-	
	30	79	68	76	-	

APPENDIX E

LOUISIANA DOTD TESTS CITED

Method of Test for

DETERMINATION OF QUANTITY OF LIME NEEDED FOR CONDITIONING PLASTIC SOILS

DOTD Designation: TR 416 - 81

Scope

1. This method describes a procedure for the determination of the minimum percentage of hydrated lime or quicklime to be added to a plastic soil to reduce the plasticity index and to increase its friability.

(NOTE: Some soils do not react satisfactorily with lime. In such cases the reduction of the plasticity index will be slight.)

Apparatus

2. (a) *Mold*- A cylindrical metal mold having a capacity of 1/30 or 0.0333 ft^3 (1 dm^3) with an internal diameter of 4.000 in. (101.6 mm) and a height of 4.584 in. (116.43 mm), and with a detachable collar approximately 2 1/2 in. (64 mm) in height. The mold and collar assembly shall be so constructed that it can be fastened firmly to a detachable base plate. Molds shall be replaced if any diameter is more than 4.01 in. (101.9 mm) or the height is less than 4.55 in. (115.6 mm) on any side.

(b) *Compactive device* - A metal rammer having a 2.00 ± 0.01 in. (50.8 ± 0.3 mm) diameter circular face or a segment of a 2 in. radius circle with an equivalent area and weighing 5.50 ± 0.05 lb (2.5 ± 0.02 kg). The rammer shall be equipped with an arrangement to control the height of drop to 12.0 ± 0.1 in. (305 ± 3 mm).

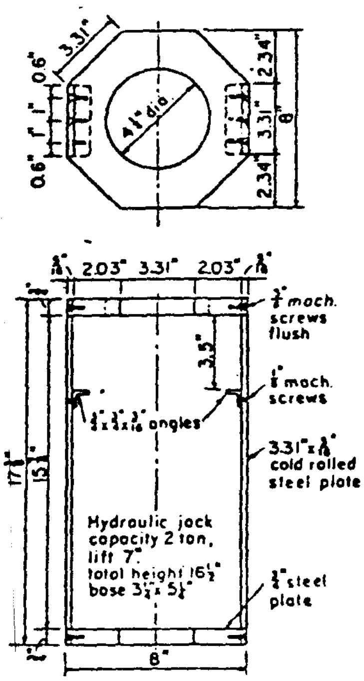
(c) *Balance* - A balance of 10 kg capacity sensitive to 1 g.

(d) *Containers* - Two 1-gallon (3.79-litre) cans with friction tops.

(e) *Plastic bags* of a suitable size.

(f) *Sieve* - 1/2 in. (12.5 mm) sieve.

(g) *Extrusion apparatus* - An extrusion apparatus similar to the illustration below, used to remove specimen from mold.



(Courtesy of Portland Cement Association)
Figure 1

Sample

3. The sample shall consist of approximately 5000 g of soil. The soil shall be placed and sealed in plastic bags, and then the plastic bags placed in friction top cans to keep it in its field condition. However, other methods that keep the soil as nearly as possible in its field condition may be used.

Procedure

4. (a) If the soil is above its plastic limit, air dry to approximately this moisture content.

(b) If the soil is below its plastic limit, add sufficient water to bring to approximately its plastic limit and allow to age until equilibrium has been reached.

(c) After the soil has been brought to a moisture content approximately equal to its plastic limit, select a representative portion of approximately 500 g for each percentage of lime to be used.

(d) Break up soil clods with fingers, being careful to use *minimum pressure so the broken faces will* not be sealed. The sample shall have 100% passing the 1/2 in. (12.5 mm) screen.

(e) Determine moisture content in accordance with *DOTD Designation:* TR *403,* Method *C,* and calculate dry weight of sample.

(f) Estimate the percentage of hydrated lime or quicklime that is required to reduce the soil plasticity index to the desired value.

(g) Calculate the amount of lime to be added by following paragraph 5 (a) of this method.

(h) Add the calculated amount of lime to the prepared sample, mixing gently to assure coating of each clod.

CAUTION: If quicklime is used, care must be exercised not to inhale its reaction fumes, or to let the quicklime contact skin.

(i) Place sample in 1/30 ft^3 (1 dm^3) mold in one layer and compact using 15 blows of the standard 5.5 lb (2.5 kg) rammer. Jack sample out and place in a plastic bag. Seal plastic bag to make it airtight.

(j) Set plastic bag with sample in moist room, and age undisturbed for 72 hours.

(k) At the end of the curing period, take sample out of the plastic bag and break it up. Dry and prepare sample in accordance with DOTD Designation: TR 411.

(l) Determine liquid limit, plastic limit and plasticity index in accordance with DOTD Designation: TR 428.

(m) Repeat this procedure for a percentage of lime above and a percentage below the estimated percentage. This will verify the accuracy of, and provide for a correctional adjustment to, the estimated percentage of lime required to condition the soil.

Calculations

5. (a) Weight of lime to be added for specimen preparation:

Wt. of lime = Dry wt. of sample x % lime by wt.

(b) Percent of volume of lime required for conditioning:

$$\% \text{ lime by vol.} = \frac{D - \frac{D}{L}}{W} \times 100$$

where:

D = maximum dry density of soil-lime, as determined by DOTD TR 418

L = 100 + *% lime* by weight, *the quantity* divided by 100

W = unit weight of lime

NOTE: For hydrated lime use 35 lb/ft^3 (561 kg/m^3) as the unit weight, and for quicklime use the actual unit weight as maintained by the Materials Section.

NOTE: Chart 1 may be used for converting percentage of hydrated lime by weight to percentage by volume.

Discussion

6. (a) Normally lime conditioning is used to assure a friable soil for use in the top course of an embankment or as a preliminary step prior to stabilization with cement.

(b) A soil that will not condition with 5% lime by weight may condition with a double treatment. This double treatment should consist of 3% lime by weight and then a second treatment with 2, 3 or 4% lime by weight. The procedure given above will be used for each of the treatments.

Report

7. Report percentage of lime needed as a percentage by volume, and the source and type of lime (hydrated or quicklime) used in testing.

Normal testing time is 7 days.

Method of Test for

MOISTURE - DENSITY RELATIONSHIPS

DOTD DESIGNATION: TR 418-81

INTRODUCTION

These methods of test are intended for determining the relationship between the moisture content of the materials listed below and the resulting dry density when the material is compacted in the Laboratory as specified herein.

TABLE OF PROCEDURES

Method A -	Raw soils with less than 5% by weight of aggregate retained on a No. 4 (4.75 mm) sieve.
Method B -	Soil cement, lime conditioned soil cement and cement stabilized mixtures all with less than 5% by weight of aggregate retained on a No. 4 sieve. Lime treated mixtures with less than 5% by weight of aggregate retained on a No. 4 sieve.
Method C -	Shell and sand-shell.
Method D -	Cement treated shell and sand-shell.
Method E -	Raw soil-aggregate mixtures with 5% or more by weight of aggregate retained on a No. 4 sieve.
Method F -	Soil cement, lime conditioned soil cement and cement stabilized soil-aggregate mixtures all having 5% or more by weight of aggregate retained on a No. 4 sieve. Lime treated soil-aggregate mixtures with 5% or more by weight of aggregate retained on a No. 4 sieve.

DOTD DESIGNATION: TR 418 - 81
METHOD A

Scope

1. This method of test is intended for determining the optimum moisture content and maximum dry density of raw soils with less than 5% by weight of aggregate retained on a No. 4 (4.75 mm) sieve, when the soil is compacted in the Laboratory as specified herein. For raw soils with 5% or more by weight of aggregate retained on the No. 4 sieve, refer to Method E.

NOTE: For determinations of the field moisture-density relationships of the above materials, it is permissible to use DOTD Designation: TR 415, Method A.

Apparatus

2. (a) *Mold* - A cylindrical metal mold having a capacity of 1/30 or 0.0333 ft^3 (1 dm^3) with an internal diameter of 4.000 in. (101.60 mm) and a height of 4.584 in. (116.43 mm), and with a detachable collar approximately 2½ in. (64 mm) in height. The mold and collar assembly shall be so constructed that it can be fastened firmly to a detachable base plate. Molds shall be replaced if any diameter is more than 4.01 in. (101.9 mm) or the height is less than 4.55 in. (115.6 mm) on any side.

(b) *Compactive device* - A metal rammer having a 2.00 ± 0.01 in. (50.8 ± 0.3 mm) diameter circular face or a segment of a 2 in. radius circle with an equivalent area and weighing 5.50 ± 0.05 lb (2.5 ± 0.02 kg). The rammer shall be equipped with an arrangement to control the height of drop to 12.0 ± 0.1 in. (305 ± 3 mm).

(c) *Straightedge* - A steel straightedge, 12 in. (305 mm) length.

(d) *Balance* - A balance or scale of 20 lb or more capacity sensitive to 0.01 lb or a 10 kg or more capacity balance sensitive to 1 g.

(e) *Sieve* - A No. 4 (4.75 mm) sieve conforming to the requirements of the Standard Specification for Wire Cloth Sieves for Testing Purposes (AASHTO Designation: M 92).

(f) *Mixing tools* - Miscellaneous tools such as mixing pan, spoon, trowel, spatula or a suitable mechanical device for thoroughly mixing the sample of soil with water.

(g) *Graduated cylinder (250 ml)* - For measuring water to be added to sample.

NOTE: It is convenient, but not essential, to have a mechanical device for removing the compacted soil from the mold. Such a device may consist of a closed cylindrical sleeve slightly less than 4.0 in. (102 mm) in diameter or a piston of like diameter actuated mechanically or by hydraulic or air pressure. It is also permissible to use a mold, the inside upper diameter of which is slightly larger than the inside lower diameter provided the volume of the mold is 1/30 ft^3 (1 dm^3).

Sample

3. A representative sample of soil weighing approximately 6 lb (2.7 kg) shall be taken from the thoroughly mixed portion of the dried material passing the No. 4 (4.75 mm) sieve which has been prepared in accordance with DOTD Designation: TR 411.

Procedure

4. (a) The 6 lb (2.7 kg) sample shall be thoroughly mixed with sufficient water to make it slightly damp so that when a portion is tightly squeezed in the palm of the hand, it will form a cake which will bear very careful handling without breaking. A compacted specimen shall then be formed by compacting the thoroughly mixed soil in the mold (with collar attached) in three equal layers to give a total compacted depth of about 5 in. (127 mm), each layer being compacted by 25 blows of the rammer dropping free from a height of 12 in. (305 mm), or 12 in. above the approximate elevation of each finally compacted layer when a stationary mounted type of rammer is used. During compaction the mold shall rest on a uniform, rigid foundation. The blows shall be uniformly distributed over the surface of the layer being compacted. After the specimen has been compacted, the collar shall be removed from the mold and the compacted soil carefully trimmed even with the top of the mold by means of the straightedge.

(b) The mold containing the compacted soil specimen shall be weighed. This weight minus the weight of the mold shall then be divided by the volume of the mold and the result recorded as the wet density of the compacted soil.

(c) The base plate shall be detached and the specimen removed from the mold. A representative sample shall be taken from a location near the center of the specimen and tested in accordance with DOTD Designation: TR 403, Method C.

(d) The remainder of the specimen shall be combined with the material left from the molding and broken up such that it will pass through a No. 4 (4.75 mm) sieve as judged by eye. Successive increments of water in sufficient amount to increase the moisture content of the soil sample by approximately 2% shall be added, and the above procedure repeated for each increment of water added. This series of determinations shall be continued until the soil becomes very wet or there is a substantial decrease in the wet weight of the compacted soil. Three points on the dry side of curve and two points on the wet side of curve should give an adequate curve.

Calculations

5. $$\text{M.C.} = \frac{(\text{W.W.})\,(100)}{\text{D.W.}}$$

$$\text{D. D.} = \frac{(\text{W.D.})\,(100)}{100 + \text{M.C.}}$$

where:

M.C. = moisture content in percent
W.W. = weight of water
D.W. = weight of dry soil
D.D. = dry density
W.D. = wet density

NOTE: Record and calculate values to the same degree of accuracy shown in the example on the Moisture-Density Relationship Work Sheet. (See Method B, Figure 1.)

Moisture-Density Relationship

6. (a) Compute the moisture content for each point.

(b) Compute the corresponding dry density.

(c) Plot the wet and dry densities as ordinates with the corresponding moisture contents as abscissas.

(d) *Optimum Moisture Content* - When the moisture density relations have been determined for a soil and the results plotted as indicated in paragraph 6(c), it will be found that by connecting the plotted points with a smooth line, a curve is produced which is generally parabolic in form. (See Method B, Figure 2.) The moisture content corresponding to the peak of the dry density curve shall be the "optimum moisture content" for the compaction specified herein.

(e) *Maximum Density* - The dry density of the soil at "optimum moisture content" shall be termed "maximum density" for the compaction specified herein.

Report

7. The curve plotted in paragraph 6(d) is the report. Record the maximum density and optimum moisture on the curve.

NOTE: When these results are used on the Density and Moisture Content Work Sheet, record the maximum density as (G) and the optimum moisture as (J).

Normal testing time is 3 days.

DOTD DESIGNATION: TR 418-81

METHOD B

Scope

1. This method of test is intended for determining the optimum moisture content and maximum dry density of soil cement, lime conditioned soil cement, cement stabilized and lime treated soils, all containing less than 5% by weight of aggregate retained on a No. 4 (4.75 mm) sieve, when compacted in the Laboratory as specified herein. When the above materials contain 5% or more by weight of aggregate retained on a No. 4 sieve, refer to Method F.

NOTE: For determinations of field moisture-density relationships of soil cement, lime conditioned soil cement and cement stabilized mixtures, DOTD Designation: TR 415, Method B, may be used in conjunction with this procedure.
NOTE: For determinations of field moisture-density relationships of lime treated soils, it is permissible to use DOTD Designation: TR 415, Method A.

Apparatus

2. Same as Method A.

Sample

3. Same as Method A.

Procedure

4. (a) The dried soil first shall be pulverized to pass a No. 4 (4.75 mm) sieve so as to separate the soil particles without reducing the particle size. Only the material passing the No. 4 sieve will be used in the test.

(b) The required percent by volume of cement or lime as specified shall be added to the pulverized soil. For this purpose it is necessary to convert the specified percent by volume to percent by weight using the nomogram of Chart 1 or the graph in Chart 2. The maximum dry density of these charts may be obtained by performing a one-point Proctor test on the raw material in accordance with DOTD Designation: TR 415, Method A.

NOTE: As an alternate to determining the one-point Proctor test above, the maximum dry density may be estimated by one of the following methods:

A. Estimate the maximum dry density based on past experience with similar materials.

B. Obtain the plastic limit of the material passing the No. 40 (0.425 mm) sieve in accordance with DOTD Designation: TR 428 and, assuming this value to be equal to the optimum moisture, enter the Family of Curves chart of DOTD Designation: TR 415, Method A (Figure 1), and obtain either directly or through interpolation the corresponding maximum dry density from the numbered curves.

When an estimated dry density is used for converting percent by volume of cement or lime to percent by weight, the calculated maximum dry density of the test method, paragraph 6(e), should be within ± 5 lb/ft^3 of the estimated dry density. Otherwise, the procedure shall be repeated with the calculated maximum dry density used to make the conversion from percent by volume to percent by weight.

(c) A quantity of water sufficient to bring the mixture to approximately 5% below the plastic limit of the soil shall be thoroughly mixed with the soil mixture. In the case of soil-lime stabilization or soil-lime conditioning, a slaking period is required. When hydrated lime is used, a slaking period of a minimum 15 hours and a maximum of 24 hours is required; when pulverized quicklime is used, a minimum slaking period of 24 hours is required; when pelletized or coarse sized quicklime is used, a minimum slaking period of 48 hours is required. When the lime conditioned soil is to be cement stabilized, the lime conditioned soil, after slaking, shall be mixed with the required cement and cured an additional two hours prior to molding. Cement stabilized raw soil is cured two hours.

CAUTION: If quicklime is used, care must be exercised not to inhale its reaction fumes, or to let the quicklime contact skin.

(d) The cured soil mixtures shall be compacted in the mold in three equal layers to give a total compacted depth of about 5 in. (127 mm), each layer being compacted by 25 blows of the rammer dropping free from a height of 12 in. (305 mm), or 12 in. above the approximate elevation of each finally compacted layer when a stationary mounted type rammer is used. During compaction the mold shall rest on a uniform, rigid foundation. The blows shall be uniformly distributed over the surface of the layer being compacted. After the specimen has been compacted, the collar shall be removed from the mold and the compacted soil carefully trimmed even with the top of the mold by means of the straightedge.

(e) The mold containing the compacted soil specimen shall be weighed. This weight minus the weight of the mold shall then be divided by the volume of the mold and the result recorded as the wet density of the compacted soil.

(f) The base plate shall be detached and the specimen removed from the mold. A representative sample shall be taken from a location near the center of the specimen and tested in accordance with DOTD Designation: TR 403, Method C.

(g) The remainder of the specimen shall be combined with the material left from the molding and broken up such that it will pass through a No. 4 (4.75 mm) sieve as judged by eye. Successive increments of water in sufficient amount to increase the moisture content of the soil sample by approximately 2% shall be added, and the above procedure repeated for each increment of water added. This series of determinations shall be continued until the soil becomes very wet or there is a substantial decrease in the wet weight of the compacted soil. Three points on the dry side of curve and two points on the wet side of curve should give an adequate curve.

Calculations

5. $$\text{M. C.} = \frac{(\text{W.W.})\,(100)}{\text{D.W.}}$$

$$\text{D. D.} = \frac{(\text{W.D.})\,(100)}{100 + \text{M.C.}}$$

where:

M.C. = moisture content in percent
W.W. = weight of water
D.W. = weight of dry soil
D.D. = dry density
W.D. = wet density

NOTE: Record and calculate values to the same degree of accuracy shown in the example on the Moisture-Density Relationship Work Sheet. (See Figure 1.)

Moisture-Density Relationship

6. (a) Compute the moisture content for each point.

(b) Compute the corresponding dry density.

(c) Plot the wet and dry densities as ordinates with the corresponding moisture contents as abscissas.

(d) *Optimum Moisture Content* - When the moisture-density relations have been determined for a soil and the results plotted as indicated in paragraph 6 (c), it will be found that by connecting the plotted points with a smooth line, a curve is produced which is generally parabolic in form. (See Figure 2.) The moisture content corresponding to the peak of the dry density curve shall be the "optimum moisture content" for the compaction specified herein.

(e) *Maximum Density* - The dry density of the soil at "optimum moisture content" shall be termed "maximum density" for the compaction specified herein.

Report

7. (a) In soil cement and lime conditioned soil cement, the optimum moisture as determined in paragraph 6 (d) only will be reported. For areas in which the materials are considered sufficiently uniform, the Laboratory Engineer may determine the maximum density from this test to be used as a basis for determining percent compaction in the field. In this case the maximum density shall be reported.

(b) In lime mixtures the curve plotted in paragraph 6 (d) will be reported. Record the maximum density and optimum moisture on the curve.

NOTE: When these results are used on the Density and Moisture Content Work Sheet, record the maximum density as (G) and the optimum moisture as (J).

Normal testing time is 2 to 4 days.

Method of Test for

THE DETERMINATION OF THE PH VALUE OF SOIL

LDH DESIGNATION: TR 430-67

Scope

This method of test covers the procedure for determining the pH of water and soil samples.

Apparatus and Materials

(1) A 2 oz. or larger wide mouth container; e.g. beaker or waxed paper cup.

(2) A pH meter, suitable for laboratory analysis, with either one or two electrodes.

(3) Standard solutions of known pH values - use values of approx. 4.0, 7.0, and 9.0.

(4) Solution of potassium chloride and silver chloride mixture.

(5) Distilled or demineralized water with a pH value between 6.5 - 7.0 that has been freshly boiled and cooled to room temperature.

(6) Teaspoon or small scoop.

(7) Thermometer - 0° to 50°C.

(8) No. 10 sieve.

(9) Round tin pans.

(10) A Balance sensitive to the nearest tenth of a gram.

Procedure

(a) Water Determination

(1) Stir the water sample vigorously by means of a clean glass stirring rod.

(2) Pour a sample of approximately 50 cc ± 5cc into either a clean beaker or a waxed cup.

(3) Record the temperature of the sampled water, and adjust the temperature controller of the pH meter accordingly.

(4) *Standardize the pH meter by means of the three standard solutions provided.

(5) Immerse the electrodes of the pH meter into the water sample and swirl the container slightly so as to obtain good contact between the water and the electrodes.

* Use the standard solution to standardize the pH meter which is nearest the pH value of the sample to be tested. (i.e. if you assume your test sample to have an approximate pH value of 4.0 and you standardize the meter with a standard solution 4.0 and upon actually testing your sample, a value of 9.5 is indicated on the meter, you have used the wrong standard solution. Repeat the standardization of the meter, this time using the 9.0 standard solution.)

(6) Allow electrodes to stand for 15 seconds in sample before reading.

(7) Read and record the pH value to the nearest tenth of a whole number.

(8) Rinse electrodes well while wiping lightly with a soft cloth to remove any film formed on electrodes.

(b) Soil Determination:

(1) Prepare the soil in accordance with L.D.H. TR-411, Method A, (Mechanical Analysis).

(2) Weigh approximately 10.0 ± 0.1 gram of soil into a 2 oz. waxed cup or glass beaker.

(3) Add 50.0 ± 5 cc of distilled or demineralized water to the soil sample.

(4) Stir the sample vigorously and disperse soil well in water. Stir the sample every 15 minutes in order to disperse the soil and make sure all soluble material is in solution.

(5) Allow sample to stand for a period of one hour after addition of water. Record the temperature of the mixture and adjust pH meter.

(6) Immediately before immersing electrodes into sample stir and remove glass stirring rod. Place electrodes into solution and gently swirl so as to make good contact between the solution and the electrodes.

(7) Allow electrodes to stand for 15 seconds in sample before reading.

(8) Read and record the pH value to the nearest tenth of a whole number.

(9) Rinse electrodes well while wiping lightly with a soft cloth to remove any film formed on the electrodes.

Precautions

Carefully follow the above procedure and manufacturer's instructions.

If the pH reading appears unstable when the electrodes are immersed in the soil slurry, leave the electrode immersed until the pH reading has stabilized. In some cases the waiting period for the stabilization of the pH reading may take 5 minutes or more.

Method of Test for
DETERMINING THE MINIMUM CEMENT CONTENT FOR SOIL CEMENT STABILIZATION
DOTD Designation: TR 432 - 82
METHOD A

Scope

1. This method covers the procedure for determining the minimum cement content for soil stabilization on the basis of historical data using Type I or I(B) cement. When other cement types are to be used for stabilization this procedure shall not apply and the tests of Method B shall be used as a basis for determining minimum cement content. For soils which require lime conditioning prior to cement stabilization, the name classification classified in accordance with DOTD Designation: TR 423 prior to lime conditioning shall be used, and the A-Group shall be ignored. When this is the case, proceed to paragraph 2(b) of this method.

Procedure

2. (a) Soils for stabilization under this method shall be classified in accordance with DOTD Designation: TR 423.

(b) Cement content determinations for the various soil classifications shall be made in accordance with the table which follows. Soils not covered by the table shall be tested in accordance with DOTD Designation: TR 432, Method B. Soils which have a borderline classification, or which are suspect due to past performance or uniform particle size, may be tested in accordance with DOTD TR 432, Method B, in lieu of this method for determination of cement content.

Report

3. The minimum cement content as determined above shall be reported as the minimum percentage by volume required for cement stabilization.

NOTE: Should there be reason to believe that the soil is contaminated because of environment, the material, regardless of classification, shall be tested in accordance with DOTD Designation: TR 432, Method B. An example of suspect material would be soil from a sugar cane haul road. It would take very little cane juice in the soil to prevent satisfactory cement stabilization.

Normal testing time is 4 days.

DOTD Designation: TR 432 - 82

METHOD B

Scope

1. This method covers the procedure for determining the minimum cement content for soil stabilization on the basis of physical testing. The type of cement used in testing shall correspond to that to be used in construction.

Test Methods

2. (a) The soil samples shall be prepared in accordance with DOTD Designation: TR 411.

(b) The soils shall be classified in accordance with DOTD Designation: TR 423.

(c) The moisture-density relations of the soil cement mixture shall be determined in accordance with DOTD Designation: TR 418 or DOTD Designation: TR 415, Method A.

(d) Specimens for unconfined compressive strength determinations shall be made in accordance with the DOTD Designation: TR 434 molding procedure.

(e) The compressive strength specimens shall be tested in accordance with ASTM Designation: D 1633 with the following exceptions:

(1) Test specimens shall have a nominal diameter of 4.0 in. (102 mm) and a nominal height of 4.6 in. (117 mm).

(2) Specimens shall be moist room cured at not less than 90% relative humidity and at a temperature of 23 ± 1.7 C (73.4 ± 3 F) for a period of seven days.

(3) Upon removal from the moist room, cap the ends of all compression specimens that are not plane within 0.005 in. (0.13 mm). The capped surfaces shall meet the same tolerance. Use a commercial capping compound (such as Vitrobond or gypsum plaster) when capping is required. The specimens are then immersed in clean water for a period of four hours, at the end of which they are immediately measured for diameter and height and tested.

(f) Wetting and drying tests shall be conducted in accordance with AASHTO Designation: T 135 except that the test specimens shall be molded and compacted by the same procedure specified for compressive strength specimens in paragraph (d) above.

Procedure

3. (a) After the soil is prepared as in paragraph 2(a), classified as in paragraph 2(b), and moisture-density relations established (using the median cement content) as in paragraph 2(c), a minimum of three compressive strength specimens shall be molded as in paragraph 2(d) at each of four different cement contents. The median cement content is selected as that which, based on experience, would be expected to result in satisfactory stabilization. These specimens are to be cured and tested for compressive strength in accordance with paragraph 2(e).

(b) The minimum cement content which will produce a minimum compressive strength of 250 psi (1725 kPa) in the test specimens will be considered the amount required for stabilization.

For materials prepared in accordance with DOTD Designation: TR 434, Method A, the amount of cement required shall not be less than 8% nor more than 14% by volume prior to correcting for the amount of plus No. 4 aggregate. Material which will not attain at least 250 psi at 14% cement by volume is considered not suitable for use as a cement stabilized mixture.

For materials prepared in accordance with DOTD Designation: TR 434, Method B, the cement content for attaining a minimum compressive strength of 250 psi shall not be less than 8% nor more than 14% by volume prior to correcting for the amount of plus No. 4 aggregate. For mixtures meeting these requirements, the volume of cement required shall be reported based on the volume of the total material to be stabilized.

(c) When the unconfined compressive strength is at least 250 psi but the durability of the mixture is questionable, the soil cement mixture shall be tested further by being subjected to twelve cycles of wetting and drying in accordance with paragraph 2(f) above. The criteria for acceptance of this material shall be soil cement losses not exceeding the following limits:

SOILS GROUPS	MAXIMUM LOSS % BY WEIGHT
A - 1, A - 2 - 4, A - 2 - 5, A - 3	14
A - 2 - 6, A - 2 - 7, A - 4, A - 5	10
A - 6, A - 7	7

If the material meets the above criteria, it shall be approved at the cement content determined in paragraph 3(b).

Normal testing time is 8 weeks.

Method of Test for DETERMINING THE MINIMUM LIME CONTENT FOR SOIL–LIME TREATMENT

DOTD DESIGNATION: TR 433-81

Scope

1. This method covers procedures for determining minimum lime content for soil-lime treatment, Type A and Type B.

Apparatus

2. As outlined in DOTD Designation: TR 416.

Procedure

3. Soils for treatment shall be tested in accordance with DOTD TR 416.

(a) For soils to be used as a base course, Type A, the liquid limit following lime treatment shall be a maximum of 40, and the plasticity index following lime treatment shall be a maximum of 10.

(b) For soils to be used as a subbase, Type A or Type B, the liquid limit following lime treatment shall be a maximum of 40, and the plasticity index following lime treatment shall be a maximum of 15.

Report

4. The minimum lime content meeting the above listed criteria shall be reported as the percentage by volume required.

Method of Test for

DETERMINING THE MINIMUM LIME CONTENT FOR SOIL-LIME TREATMENT

LDH DESIGNATION: TR 433-70

Scope

These methods cover procedures for determining minimum lime content for soil lime treatment, Type A and Type B as specified in 7/70 supplemental specifications Section 723 (New) Lime Treatment (Supplemental to Part VII, Standard Specifications.)

Apparatus

As outlined in LDH TR 410

Procedure

(a) Soils for treatment shall be tested in accordwith LDH TR-410

(1) For soils to be used as a base course, Types A or B, a minimum unconfined compressive strength of 100 psi as run under LDH TR 410 shall be reported as the minimum required percentage.

(2) For soils to be used as a subbase course, Types A or B, a minimum unconfined compressive strength of 50 psi as run under LDH TR 410 shall be reported as the minimum required percentage.

Report

The minimum lime content meeting the above listed criteria shall be reported as the percentage by volume required.

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