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16. Abstract

A review of current practice and a laboratory test program were conducted in an evaluation of protocol used for soils stabilized with Class C fly ash. The long-term performance of soils stabilized with Class C fly ash was also reviewed. The test program included an evaluation of material properties as well as mixed design methods with their corresponding testing requirements. X-ray diffraction and electron microscopy were performed in reviewing the long-term variations in strength and the effects of curing conditions on the cementitious products being formed.

The study reviews the unique physical properties that influence the development of strength. The relationship between gradation characteristics, density, compaction water, and strength are explored. A relationship of comparable strengths produced between portland cement and an equal amount of the CaO constituents in the Class C fly ash is proposed as a design guide. The curing requirements for the strengths produced in different mixtures varied. Curing time and conditions are recommended on the basis of strength development. A distinct long-term advantage was observed in using lime with the Class C fly ash in sands and coarse aggregate.

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LONG TERM EVALUATION AND IDENTIFICATION
OF THE
PROPER TESTING PROGRAM
FOR
ASTM CLASS C FLY ASH STABILIZED SOILS

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The authors also acknowledge the cooperation extended by Bayou Ash of Baton Rouge, Louisiana, in providing the fly ash and the test results of the material properties conducted by others on their product during this study.

ABSTRACT

A review of current practice and a laboratory test program were conducted in an evaluation of protocol used for soils stabilized with Class C fly ash. The long-term performance of soils stabilized with Class C fly ash was also reviewed. The test program included an evaluation of material properties as well as mix design methods with their corresponding testing requirements. X-ray diffraction and electron microscopy were performed in reviewing the long term variations in strength and the effects of curing conditions on the cementitious products being formed.

The study reviews the unique physical properties that influence the development of strength. The relationship between gradation characteristics, density, compaction water, and strength are explored. A relationship of comparable strengths produced between portland cement and an equal amount of the CaO constituents in the Class C fly ash is proposed as a design guide. The curing requirements for the strengths produced in different mixtures varied. Curing time and conditions are recommended on the basis strength development. A distinct long-term advantage was observed in using lime with the Class C fly ash in sands and coarse aggregate.

IMPLEMENTATION STATEMENT

It is the authors' understanding that current DOTD policy mandates the use of lime with Class C fly ash. As the result of this investigation, that policy is endorsed and recommended for continuation with all soils. A distinct long-term advantage was observed in using lime with the Class C fly ash in sands and coarse aggregates. Additionally, the pozzolanic action with the lime and fly ash can compensate for unforeseen material or construction problems.

The study also produced techniques suggested as guidelines for the identification of the required mix proportion. Initially, the full potential of a coarse-grained or sandy soil can be estimated as that amount of fly ash required to maximize density. Using the gradation characteristics of the sand and fly ash, the fly ash requirements for maximum strength are estimated.

The results of this and a previous investigation suggest that strength prediction for fly ash used as a lone stabilizing agent in sands can be estimated from the strength produced in the sand with portland cement. The estimate is based on the amount of calcium oxide present in the fly ash and the strength performance of an equal amount of portland cement. It is suggested that this hypothesis be tested further for validation. Thus, it is recommended that DOTD district laboratories compare the fly ash and cement strengths produced, and the gradation requirements for maximum density in analyzing their compaction tests results for mix design. The results should be documented with LTRC.

The unique properties of the materials and material combinations used in this study suggest different curing requirements. If the Class C fly ash is used alone with a coarse aggregate, a curing period of 7 days at normal temperatures, 23° C, is recommended. A 7-day accelerated cure is recommended for lime and fly ash used with sands. Continuation of a 1-day acceleration for fly ash used with lime in clays is recommended, but further evaluation is suggested.

The study demonstrated the fast set of the Class C fly ash. If the self-cementing characteristics are to be taken advantage of, mixing and compaction must be accomplished in a short time (within two hours or less), or the use of set retarder may be required. If the fast mixing and compaction requirements are unrealistic or uncertain, a testing program that simulates delayed compaction will more accurately measure the mixture's performance and is recommended.

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INTRODUCTION

The potential for stabilizing coarse and fine-grained soils in the construction of subgrades, subbases, and bases for pavements with ASTM Class C fly ash has been demonstrated (1). However, studies with locally produced ASTM Class C fly ash have emphasized the importance of the testing method and criteria used in evaluating their performance as stabilizing agents. These studies have demonstrated an incompatibility with respect to some of the current test procedures, field placement methods and the materials' properties. There has also been some indication of a reversal in stability of the fly ash/soil mixture over longer curing periods. Current Louisiana Department and Development test procedures and criteria for lime and cement treated soils are inadequate in analyzing fly ash as a stabilizing agent replacing lime or cement.

Much information concerning test procedures for fly ash has been developed using ASTM Class F fly ash. Also, the current test procedures and criteria used for lime and cement treated soils are inadequate for evaluating the performance of fly ash used as a full or partial replacement for portland cement or hydrated lime (2). Test criteria used for cement stabilized soils do not account for the long-term pozzolanic potential of a lime-fly ash mix nor the flash set reported to occur in some Class C fly ashes. Also, existing lime treatment criteria as used locally address only soil modification of plastic clays.

The incompatibility of some of these test procedures with the sequence of chemical events taking place with Class C fly ash demonstrates a need for further evaluation. There has been noted a deterioration in some fly ash mixtures that has not been previously reported (3). Routine tests currently utilized by the Louisiana Department of Transportation and Development do not fully evaluate the Class C fly ash as a stabilization agent.

Louisiana is faced with a number of major problems involving highway design, construction, and maintenance. Two of these include the lack of natural deposits of high-quality construction material and the existence of weak soils required to support the pavement structure. Methods for properly evaluating alternative construction materials are needed.

A testing program with documented results for properly assessing the potential of locally produced fly ash as a stabilizing agent will greatly enhance the use of this material. With knowledge of the material properties and confidence in the testing procedure, locally produced fly ash will become a more viable alternative. Significant savings could be provided with the increased utilization and disposal of this waste by-product.

OBJECTIVE

The objectives of this research were to further evaluate the characteristics of locally produced fly ash and to develop test procedures which would expedite the evaluation of fly ash stabilized soils. Because cement and lime stabilization techniques are well established, comparisons of fly ash stabilization methods with these materials were also necessary.

To accomplish this, information on other tests currently being performed were reviewed, tested and evaluated to determine their applicability. A test approach is proposed for evaluating the fly ash potential as an alternative to other conventional materials and methods.

Long term effects are also addressed to determine whether there is continued strength gain or possible deterioration with time.

SCOPE OF RESEARCH

The variables involved with fly ash stabilization are numerous and thus had to be limited to meet the objectives. The material variables include: soil types and fly ash source. The testing variables include: mix design criteria, mixing sequence, curing methods, and standard verses innovative testing.

In this research, an A-3 sand and bentonite were the soil types used. The stabilizers used with the sand were cement, fly ash, and mixtures of lime and fly ash. The bentonite mix design was limited to that used by Ferrel, et al (3), so that their results could be re-evaluated. One fly ash source, Bayou Ash, Baton Rouge, Louisiana, was chosen. The only testing variable not specifically addressed was the mixing sequence. In all cases mixing was performed in the same manner, all materials being mixed dry before any moisture was added. Mix design criteria from various sources, including state practices, were examined. Ratios of stabilizer to soil were limited by practicality, time, and cost. Normal curing methods were used along with the accelerated method proposed by Ferrel, et al (3). Standard tests were performed and evaluated. Innovative testing was limited by equipment and time.

1. SOIL STABILIZATION WITH CLASS C FLY ASH

The use of power plant ash with lime or portland cement in pozzolanic base course mixtures is not new. Those states which have made the most extensive use of pozzolanic base courses over the longest period of time in state highway projects are Illinois, Ohio, and Pennsylvania (4). Where available, ASTM Class C fly ash used alone or as a partial replacement for cement or lime has been found to be a very effective and economical technique. In a survey of projects conducted in 1984, 63 of 74 subgrade stabilization projects had used Class C fly ash. Most of these were located in the Midwest region of the United States (1). More recently, the use of recycled roadway materials with Class C fly ash has been demonstrated in Kansas (5) and in Louisiana (State Project 736-17-0000).

In most state specifications the method used for determining the actual mix design is rarely given. Strength requirements and/or limits of stabilizer percentages are often the only guides and trial and error procedures are common. Typically, specifications state that the mixture proportions "will be established by the engineer based on laboratory testing and field trial procedures necessary to determine proper soil modification" (6).

1.1 ASTM CLASS C 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 of amorphous components of siliceous or siliceous and aluminous material. Fly ash particles are generally spherical granules formed when the non-carbon materials are burned under high temperatures in the combustion chamber. The principal constituents of ash are silica, alumina, and iron oxide. 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.

Depending on the type of fly ash, there are smaller quantities of calcium oxide, magnesium oxide, potassium oxide, sodium oxide, sulfur oxide, and minute traces of other elements present. The composition and properties of the resulting fly ash are determined by the coal source burned, the incineration equipment used, and the methods in which the fly ash is handled. ASTM identifies two classes of fly ash based on coal source - Class F fly ash originating from bituminous coals and Class C from subbituminous and lignite origins.

Certain chemical and physical characteristics of the fly ash provide an indication of its pozzolanic potential. No single chemical or physical property gives a reliable prediction of the pozzolanic reactivity of fly ash. However, fineness or specific surface (ASTM 311), can be considered to be a good physical indicator of this reactivity. The finer the fly ash, the greater the rate of the pozzolanic reaction. One important chemical indicator is the carbon content of the fly ash measured as a loss on-ignition in accordance with ASTM C311. A high carbon content (>10%) tends to inhibit the pozzolanic reactivity of a fly ash.

Cementitious calcium silicate and calcium alumino-silicate hydrates are formed when the glassy components of the fly ash react with water and lime. Critical to the pozzolanic nature of the fly ash are conditions such as the amount of silica and alumina in the fly ash, the presence of moisture and lime, the fineness of the fly ash surface (surface area) and a low carbon content. The standard specifications which govern the quality of fly ash are ASTM C 311, "Sampling and Testing Fly Ash or Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete;" ASTM C 618, "Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete;" and ASTM C 593, "Fly Ash and Other Pozzolans for Use with Lime."

Lime and portland cement are commonly used in combination with fly ash for the treatment or stabilization of soils. The addition of fly ash provides a pozzolan and/or 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.

An important constituent in Class C fly ash is CaO, sometimes referred to as "free lime." Fly ashes with high CaO contents of 20% or more have been used to satisfactorily stabilize soils without the use of added lime (7). The high-lime fly ashes can possess both pozzolanic and cementitious characteristics. Power plants operating in Louisiana produce Class C fly ashes.

In addition to being pozzolanic, some Class C fly ashes possess sufficient amounts of calcium silicates/aluminates to exhibit cementitious properties similar to portland cement. A 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 physical properties of some soils and reacting with the siliceous and aluminous compounds in the fly ash to produce long-term pozzolanic cementation. 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 can be a wide range in the chemical composition of Class C fly ashes. It has been observed (8) that the calcium in these highly reactive fly ashes when mixed with lime yields very high compressive strengths. According to Ledbetter (9) and McKerdall (10), most of the CaO 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 similar to those found in portland cement. Thus, most of the calcium constituents in the Class C fly ash are not readily available for use in modifying the plastic behavior of fine-grained soils.

An investigation of several Iowa Class C fly ashes determined that they consisted of approximately 30 percent elemental calcium oxide, but only 1 to 2 percent of this existed as a free calcium oxide (11). The cementitious character of the Iowa fly ashes was attributed to some of the calcium (6 to 7.5 percent) being combined as tricalcium aluminate and tricalcium aluminate sulfate, producing a fast set and a high, early strength. The remainder of the calcium occurred as other crystalline components (5 to 10 percent) and in a glassy phase (20 to 25 percent). Some Class C fly ashes in the Iowa study were found to be high in calcium but not cementitious. In another investigation, the addition of ammonia phosphate has been reported to break down the calcium-rich glassy phase, increasing pozzolanic activity, as well as retarding the rapid hydration of the tricalcium aluminates present (12).

Once a source has been selected, the uniformity of ash characteristics from shipment to shipment is imperative in order to produce a consistently good end product. Variations in ash characteristics must be known ahead of time to allow adjustment of mix designs and field procedures as appropriate.

In Texas, Class C fly ash is used but must be prequalified in accordance with Departmental Materials Specification: D-9-8900, Fly Ash. Prequalification is required so that the properties of the fly ash are already known. Texas' chemical and physical requirements are a slight modification of ASTM C 618. Fly ash physical requirements are tested in accordance with ASTM C 311.

1.2 FACTORS AFFECTING PERFORMANCE

The strength and durability of a fly ash-stabilized soil are influenced by a number of factors. These include: soil type, fly ash properties, the presence and ratio of other activators used (cement/lime-to-fly ash ratio), percentage of stabilizing agents used, compaction, and the curing conditions.

1.2.1 INFLUENCE OF SOIL TYPES AND LIME

When the stabilizer is a lone additive, cement, fly ash or lime, designing a mix is much simpler then when two or more stabilizers are used. In the selection process, trial batches are mixed in the laboratory, compacted, cured, and tested, so that the best mix may be chosen. Due to the complexity of the reactions taking place, a number of different mixes for the same soil could produce similar results. The decision then involves selection of the most economical mix.

The quality of stabilized mixtures, as measured by their strength and durability, is closely related to the quality of the cementitious matrix in the mixture. Care must be taken to insure that the proportion of stabilizer in the matrix is sufficient to provide a good chemical reaction. Sand aggregates with single-sized particles and sands devoid of minus 200-sized particles may require high fly ash content to serve as a filler or void reducer as well as a pozzolan in the mixture (13).

For a given soil type, little definitive information is available on the most suitable stabilization techniques to use. Woods (14) provided the following guidelines:

> Clays Proper treatment of clays depends on the type of clay minerals. Lime-stabilization trial mixes recommended by the National Lime Association for clays are with 5, 7, and 10 percent lime. Best ratios are usually in the range 1:9 to 4:6 lime to fly ash, and total amount of admixture being governed by economics and usage. Clays susceptible to damage by freeze-thaw may have a total lime-pozzolan requirement on the order of 25 to 30 percent. With lime alone, the percentage needed will be 8 to 12 percent.
>
> Silty Soils Silty soils with less than 10 or 12 percent clay may be somewhat pozzolanic and can be stabilized with lime alone or a low lime-pozzolan ratio, 1:2.
>
> Sandy Soils These materials are too coarse to react with lime alone. The best ratios of lime to pozzolan are usually 1:5. Strengths show a somewhat linear relationship to percentage total admixture. Strength and durability are often improved by the better gradation.
>
> Coarse Granular Soils Less well graded mixes need the addition of a pozzolan. The best ratio is usually around 1:5.
>
> Pozzolanic Granular Materials Pozzolanic materials frequently develop very high strengths when stabilized with lime or lime-pozzolan. Typical mixes incorporate 4 to 7 percent lime and 10 to 15 percent fly ash.

In addition to strength requirements, lime to fly ash ratios are at times specified. Typical ratios chosen are 1:3 or 1:4, although ratios range from 1:2 to 1:9. The relative proportions of the materials range from 2 to 8 percent for lime and from 8 to 36 percent for fly ash (13).

The lime-fly ash ratio specified by Tennessee must be held between 1:3 and 1:4. They also set limits based on dry weight of total mix, lime ranges from 2 to 5 percent, fly ash from 6 to 20 percent, and aggregate from 75 to 92 percent. In one case, a 3.5 percent lime and 11 percent Class F fly ash was cited as being required to stabilize a "crusher run" limestone aggregate (15).

Proportions of lime and fly ash used in Texas include 2 percent lime with 4 percent fly ash or 3 percent lime with 6 percent fly ash. Texas also found ratios of 1:2 and 1:4 to work well for highly plastic clays (PI > 20) (6). These mixes exhibited strengths of 930 psi and 600 psi, respectively, after one year.

Arkansas used 6 percent lime and 30 percent Class C fly ash to stabilize an A-4-0 soil (6). Mississippi used a trial and error procedure to obtain a recommended mix design of 4 percent lime with 12 percent Class F fly ash. Their results after a 28-day cure at 100F are as follows: for an A-2 soil, 681 psi; an A-3 soil, 502 psi; and an A-4 soil, 476 psi (6).

1.2.2 DENSITY AND COMPACTION EFFORT

Density has a major effect on the strength and durability of a pozzolanic stabilized soil (16,17,18,19). The pozzolanic products that are produced are influenced by many factors, including density. Thus, the aggregate gradation has a very significant effect on the density, strength and durability of the mix (18).

ASTM C 593 requires the use of the modified Proctor compaction with an exception - three lifts are used instead of five. Many state agencies, however, substitute standard Proctor compaction for the higher compactive effort (4). Kentucky (20) and Arizona (21) are reported as using the standard Proctor compaction for ASTM C 593.

Alabama uses Harvard-size samples prepared at the optimum moisture content and unit weight (as determined by Proctor density tests) to determine mixture strengths (22).

Standard Proctor Compaction (ASTM D 698)

Procedure A (20 percent or less by weight of the material is retained on the No. 4, 4.75 millimeter, sieve) of this standard was used to produced a specimen that was 4 inches in diameter and 4.584 +/- 0.018 inches in height. The standard Proctor compaction uses 3 layers with a compaction effort of 12,400 ft-lbf/ft^3 (600 kN-m/m^3). A 5.5 pound rammer with a 12 inch drop is used.

Modified Proctor Compaction:(ASTM D 1557)

Modified Proctor compaction requires compacting a sample in 5 layers with 25 blows per layer. A 10 lb. rammer is used with an 18 inch drop to achieve the desired compactive effort. The size of the specimen is the same as the standard Proctor specimen, 4 inches in diameter by +/- 4.585 inches.

ASTM C 593 specifies the use of the modified Proctor compaction with the exception that three lifts are used instead of five. However, many state agencies substitute standard Proctor compaction for the higher compaction effort (4).

Harvard Miniature Compaction

The Harvard compaction test is not an ASTM standard. This type of compaction, however, probably best simulates field compaction techniques such as those obtained by a sheepsfoot roller. Compaction is essentially a rodding action, best described as kneading compaction. The tamping force, number of blows per layer, and number of layers can be easily adjusted to yield the desired compactive effort. The test has the advantage of using small samples that need not be reworked and that are suitable for unconfined compression testing without trimming. The specimens produced are 1.313 inches (3.334 cm) in diameter and 2.816 inches (7.153 cm) in height. The method is beneficial for research investigations where a large number of specimens need to be prepared in a short period of time and/or with only a small amount of soil. It has the disadvantage, however, of not having been widely correlated with standard tests and field compaction results (23).

Texas Compaction

The Texas Test Method, Tex-113-E, "Determination of Moisture-Density Relations of Soils and Base Materials," and Tex-127-E, "Fly Ash Compressive Strength Test," employs a compaction procedure similar to the modified Proctor, ASTM D 1557. This test requires the use of a mold of 4.0 +/- 0.25 inch diameter and 6.0 +/- 0.25 inch height. Four layers are to be compacted with the 100 lb. rammer and an 18 inch drop, providing a compactive effort of 13.26 ft-lb per cubic inch in fly ash applications. An advantage of using this test over the Proctor method is the larger specimen length to diameter (L/D) ratio.

1.2.3 CURING CONDITIONS

There are apparent similarities between soil-cement and lime-fly ash-soil mixtures which make it reasonable to assume that criteria similar to the established cement stabilization guidelines could be adopted for selecting suitable lime-fly ash-soil mixtures. However, one important difference is the rate of reaction. Cement-treated soils gain strength earlier than soils treated with lime.

The rate of strength developed from the pozzolanic products in a lime-fly ash mix is much slower than in soil-cement mixtures. There is a gain in strength over a longer period of time with the lime-fly ash mixture. The rate of strength for lime-fly ash mixtures has been cited as being approximately 10 percent of the ultimate strength at the end of 7 days under normal curing conditions (moist cure at 73° F or 21° C) and about 50 percent of the ultimate strength at the end of 28 days (24). The rate of strength gain can continue for a period of years.

Strength in many Class C fly ashes develops rapidly when compacted immediately after mixing. The initial set times are sometimes faster than with 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. Adequate mixing and rapid compaction are necessary to achieve maximum benefits of highly reactive fly ashes.

Pozzolanic 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 mixtures (11). 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 temperatures for all lime-fly ash soil mixtures.

Experimental test results obtained for different methods of curing indicate that the conditions of curing are optimum when the moisture content of the mixture is maintained at the level required to achieve the desired compaction (25).

The development of compressive strength is directly related to the chemical interaction between stabilizing agents as affected by the temperature of curing. The higher the curing temperature, the faster the rate of reaction. For soil cement mixtures, the 7-day compressive strength increases with increasing temperature by 2 to 2 1/2 percent per degree Centigrade when the temperature is near 25°C. Barenberg (17) noted that below 40°F the chemical reaction for lime-fly ash aggregate mix virtually stops. Above this temperature the rate of reaction increases with increasing temperature (26).

Curing time greatly affects the strength and durability of lime-fly ash mixtures. Under acceptable curing conditions, chemical reactions in LFA mixtures continue as long as sufficient lime and fly ash are available to react. Cores taken from pavements over a 10 year period indicate a continuing development in the strength of the mixture with time. This continuing reaction process can manifest itself in a phenomenon called autogenous healing, which is one of the properties of lime-fly ash mixtures. This can only occur, however, if there are sufficient quantities of unreacted lime and fly ash available to provide the necessary reaction components (13).

Because of the combined effects of time and temperature, it is difficult to specify combinations of curing conditions which simulate field conditions. The Illinois Department of Transportation specifications require a minimum compressive strength following 14 days of curing at 72°F (4).

The accelerated curing method established by ASTM C 593 (7 days at 100°F) is an approximation of the 28-day strength of a mixture under ambient conditions (7).

Ferrell et al. (3) concluded that 24 hours at 50°C parallelled 28 day curing at 23°C. The results of these conclusions were based on a comparison of strength (2099.43 kPa for a 1-day cure at 50°C as compared to a strength of 1800 kPa for a 28- day cure at 73°F) and the similarity of the cementitious materials formed at these temperatures. At 60°C other minerals were detected. The 50°C temperature was thought to be the upper limit for curing samples in laboratory studies that can be correlated directly with field results (3).

Brownie (27) suggests the use of an accelerated cure at 120°F in an oven for 30 hours as proposed by Dunlapp and Biswas, "Accelerated Laboratory Curing of Lime Stabilized Soils", Contract No. F29601-70-C-008, Air Force, 1974.

In Arkansas, two methods of specimen curing were selected -- the normal moist cure was used for soil cement specimens, and an air-cure method utilizing the same temperature requirements as the moist cure. Air-cured specimens consistently exhibited higher compressive strengths than those moist cured (6).

2. SPECIFICATIONS AND TEST CRITERIA

2.1 Fly Ash Quality Control

ASTM is currently developing specifications for characterizing fly ash for use in soil stabilization (ASTM D 5239). This characterization will be based on the compressive strength of the fly ash as determined by a cube analysis similar to that used on cement (ASTM C 189). Among the other tests being prescribed by this method will be set time by the vicat needle, fineness, and loss on ignition. Also of importance is the fly ash "quality assurance" program, which requires a producer to be prequalified. Prequalification means that a six-month history of ASTM C 311 test results is made available to determine if the fly ash characteristics are uniform. Testing during a project is also required. The proposed specifications for the characterization is only intended to assist in the evaluation of fly ash. It will not replace testing, design or specification requirements. A testing program is still required.

2.2 Mixture Performance

The unconfined compression test was not designed to evaluate stability of subgrade soils but is, nonetheless, commonly employed to compare them. It is often used to evaluate adequacy of soils stabilized with admixtures of such cementing agents as portland cement and lime (23). Terrel (13) summarizes various strength-based mixture design procedures for lime - stabilized soils for several states.

2.2.1 Test Specimen and Unconfined Compression Testing

In ASTM C 593-89, "Standard Specification for Fly Ash and Other Pozzolans for Use with Lime," a minimum compressive strength is specified with durability evaluated after the specimen has been subjected to a vacuum saturation. For nonplastic mixtures, these strength requirements stipulate 400 psi after 7 days of curing at 100° F and 4 hours of soaking in water. The compactive effort specified is Method C of ASTM D 1557 (Modified Proctor), except the 5 lift requirement is replaced by 3 lifts.

Coarse-grained soils should attain strengths of 400 psi; however, recommendations have been made for reducing the requirement to as low as 100 psi for subbase applications. It should be noted that high strength can produce pavement that is susceptible to reflective cracking (7).

Louisiana requires soils stabilized with cement to attain a minimum strength of 250 psi with 7 days of curing (McManis 1988, 15). Similar requirements for lime or lime-fly ash stabilization are impractical due to the late strength gain in lime and lime-fly ash mixtures. A former Louisiana Department of Highways designation, TR 433-70: "Determining the Minimum Lime Content for Lime-Soil Treatment," required a minimum lime content based on compressive strength. Seven day cure strengths of 100 psi were required for base courses and 50 psi for subbase courses. Current lime requirements address only the liquid limit and plasticity index (DOTD TR 433-81).

Texas test methods Tex-117-E, "Triaxial Compression Tests for Disturbed Soils and Base Materials," and Tex-127-E, "Fly Ash Compressive Strength Test Methods," describe the compression test methods for samples prepared by the test method Tex-113-E. The samples are compacted, cured 7 days at room temperature, subjected to capillary absorption and then tested in unconfined compression. Texas requires Tex 127-E to be performed if a minimum design strength or percent of stabilizer is to be specified.

When lime is to be used in soil stabilization, its reactivity with the soil must be determined. In Alabama, if the change in unconfined compression strengths of the raw soil and the lime treated soil after a curing period of 48 hrs at 120°F is greater than 50 psi, the soil is termed lime-reactive (22). The Navy determines optimum lime content based on unconfined compressive strength after 28 days of curing. (Naval Facilities Engineering Command 1979, 13).

Tennessee tests its mixes in accordance with ASTM C 593, except that cylinders are cured for 28 days at 100°F and a minimum average compressive strength of 950 psi is required with no individual test lower than 800 psi (15).

The Illinois Department of Transportation specifications require a minimum compressive strength of 750 psi following 14 days of curing at 72°F for mixes to be placed in the northern half of the state and 650 psi under the same curing conditions for mixes to be placed in the southern half of the state. Higher strengths are required in the event the base course mix is to be placed later in the construction season when the number of degree-days available for strength development are significantly reduced (4).

In Kentucky, strength requirements for pozzolanic mixtures used in subbases are not specified because they are not generally required to have strengths as great as those for bases (20).

2.2.2 Durability Testing

ASTM no longer requires the freeze-thaw brushing test, but has replaced it with an alternative test procedure which measures compressive strength after vacuum saturation (ASTM C 593). Current procedures require that samples be compacted in the same manner as those tested for compressive strength. However, before compression testing, the samples are subjected to a vacuum de-airing (24 inches of mercury) for 30 minutes followed by a soaking in water for an additional hour. The ASTM C 593 specifications require a minimum strength of 400 psi after vacuum saturation. The method has been found to be a good predictor for freeze-thaw durability.

The Louisiana DOTD TR 432-82 test method for determining minimum cement for soil cement stabilization requires that when the unconfined compressive strength meets the required 250 psi but the durability of the mixture is questionable, the soil cement shall be further tested by being subjected to twelve cycles of wetting and drying (AASHTO Designation: T 135)

3. DESIGN AND CONSTRUCTION CONSIDERATIONS

Test procedures and criteria used for evaluating lime and cement treated soils are inadequate for evaluating the performance of locally produced Class C fly ashes when used as a lone or partial replacement for portland cement or a hydrated lime (2). The methodology and criteria used in testing cement-stabilized soils does not account for the long-term pozzolanic potential of a lime-fly ash mix nor the flash set occurring in some Class C fly ashes. However, in addition to material characteristics, test criteria must also simulate or account for the construction methods utilized and the conditions of pavement service.

3.1 Design Considerations

Design and use of superior bases are often warranted for use in pavement structures. Pozzolanic materials in a pavement base offers many advantages including a continuous gain in strength and a corresponding increase in the stiffness of its elastic properties. These materials lend stiffness to the pavement and added resistance against fatigue failures (28).

Flexible pavements consist of a series of layers with the highest quality materials at or near the surface. The American Coal Ash Association's Flexible Pavement Manual (29) reviews three design methods for determining the thickness of a pozzolanic stabilized base for a flexible pavement system. These include the (1) AASHTO flexible pavement design procedure which uses structural layer coefficients; (2) a mechanistic design procedure, utilizing resilient modulus values for the pavement layers; and (3) a combination approach that uses mechanistic design concepts to determine pavement layer coefficients. All of the methods are based on resistance to fatigue from repeat traffic loads.

The AASHTO thickness design for flexible pavements uses the pavement structural number concept (30):

$$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3 \qquad (1)$$

where,

SN = design structural number for the pavement;

m_2 and m_3 = drainage coefficients for base and subbase;

D_1, D_2, and D_3 = thickness of pavement wearing course, base, and subbase layers, respectively (inches); and

a_1, a_2, and a_3 = structural coefficients for each layer;

The structural number, SN, is a function of the number and distribution of the wheel loads, the subgrade support, the terminal Present Serviceability Index (PSI), and a regional factor.

The structural layer coefficient for pozzolanic bases, a_2, is determined by their compressive strength and modulus of elasticity occurring at the end of 56 days of moist curing at 73° F. The AASHTO design guide recommends that each state agency develop its own relationship for specific materials and corresponding structural layer coefficients. A normal range of values is 0.2 to 0.28 with the smaller value corresponding to a minimum compressive strength of 400 psi.

Even though stiffer materials reduce the risk associated with a subgrade mode of distress, such as shear, the presence of this stiff layer brings about an increase in the tensile stress magnitude at the bottom of this layer as well as a marked increase in the horizontal shearing resistance. Thus, a subsequent design analysis is required to insure that both the shearing resistance and the flexural resistance of this stiff layer are great enough to sustain these higher stress conditions (28).

The second method discussed by ACAA (29) is a mechanistic approach that uses concepts and procedures developed in Illinois. In this method, the thickness-design criterion of a high-strength stabilized base is controlled by the flexural stress at the bottom of the layer. The flexural strength, elastic modulus, and the thickness of the base layer, along with the subgrade resilient modulus, influence the structural response and performance of the layer when subjected to specific wheel loads. The flexural strength of a pozzolanic base can be computed as being approximately 20 percent of its unconfined compressive strength, Q_u, and the elastic modulus estimated as:

$$E(\text{ksi}) = 500 + Q_u\ (\text{psi})$$

A third method proposed in the ACAA pavement manual uses the AASHTO method in which the structural layer coefficient, a_2, includes not only the compressive strength of the stabilized base but the thickness of the base layer. The layer coefficient varies from 0.2 to 0.4. A minimum compressive strength of 500 psi for an a_2 value of 0.2 is recommended.

Pavement structural design layer coefficients currently in use are known to vary from one state to another, depending on the nature of the materials used and the state's interpretation or adaptation of the design procedure. Arizona uses a structural coefficient of 0.2 for lime-fly ash-treated subgrades that meet the requirements of ASTM C593 (21). Research conducted at the Pennsylvania Transportation Research Facility concluded that the Pennsylvania Department of Transportation should establish the structural coefficient at 0.3 for an aggregate-lime-pozzolan base (31). The base material tested in the research was composed of 3 percent lime, 15 percent fly ash, and 82 percent limestone aggregate. For high-strength pozzolanic bases, in which laboratory test strengths exceed 1000 psi, a design coefficient of 0.4 can be used. For medium-strength pozzolanic bases, in which laboratory test strengths were less than 1000 psi, a design coefficient of 0.3 can be used (4).

3.2 Construction Requirements

One of the advantages in the construction of pozzolanic base course materials is that conventional paving, spreading and compaction equipment can be used. No special pieces of construction equipment are required. With mixed-in-place operations, the materials can be dumped on the ground and spread on the roadway by means of a motor grader (4).

Subgrade preparation includes excavation and replacement of any totally unsuitable soils, removal of root structure and oversize material or rubble to some prescribed depth, shaping to grade and crown, and preliminary compaction. The goal is a firm, well-drained subgrade not susceptible to volume change or damage by frost action (32).

After the subgrade has been prepared, the stabilizer(s) can be applied. Dry stabilizers can be blown from a tanker. To reduce dusting during application, a lower-discharge pressure should be used. Almost without exception, the fly ash is applied dry, mixed dry, then watered and remixed, and finally compacted. In some instances a pug mill, rather than mix in place, is used. Lime can be sprayed in a slurry form. In all cases, the binder is then worked into a prescribed depth of the subgrade (32).

Oklahoma (6) requires that the mixing of the fly ash with the subgrade soil (termed first mixing) follow application and spreading as a continuous construction operation. First mixing must be complete within 4 hours, and the addition of water is not to be permitted during this time. After the dry soil and fly ash have been satisfactorily mixed, additions of water can be made in the "final mixing" operations to initiate soil-fly ash reaction.

The number of compaction and finishing units should be sufficient to insure the initial compaction of the processed section of the stabilized base course within a prescribed time limit. The final finishing and compaction is determined by the engineer based on whether the material has reached an initial set. If for any reason construction operations are delayed or suspended, the engineer may order any loose or uncompacted material removed and disposed of by the contractor at his own expense. According to Tennessee specifications no lime-fly ash base course should be salvaged(15).

Compaction of the base course in place is the most critical step in the construction process. Achieving a high in-place density is the key to good strength development and durability of the mix. Final density should be achieved as quickly as possible to obtain maximum strength. An unsuccessful attempt in Texas (project FM 1604) at stabilizing a cohesionless, unconsolidated sand with 25 percent to 30 percent fly ash alone was attributed to low compaction. Only a 92 percent compaction was obtained on this project (6).

Static steel-drum or pneumatic rollers are the most common compaction techniques, but sheepsfoot and segmented wheel rollers are also effective in some cases. Vibratory rollers are not normally used but have also proven successful (32).

After final compaction, the completed sections should be immediately brought to final grade and cured until the next course is applied. Heavy construction traffic should not be permitted on the stabilized section for a minimum of 14 days (9).

Curing methods depend on the stabilizing agents used. Lime, cement, and fly ash-stabilized soils must be wet cured, usually for at least seven days. Occasional spraying from a water truck or covering the treated area with light plastic sheeting are the most common curing methods (32). Surface reworking after completion should not be permitted, as the cementitious advantages of the fly ash will be lost (9).

When using Class C fly ash in pozzolanic base mixtures, certain measures have been used to counteract self-hardening or flash setting. These measures include: 1) initial conditioning with 10 to 15 percent water and stockpiling for several months, then crushing to reduce agglomerations prior to use; or 2) use of a retarder (such as gypsum or a commercial retarding admixture), added in low percentages (1 to 2 percent) during mixing. An additional precaution concerning use of Class C fly ash in pozzolanic base mixtures is that, unless one of these measures is used, delays between placement and compaction of the material are usually accompanied by a significant decrease in the strength of the base mix (4).

Kentucky specified a 2 1/2 hour limit for an application using 100 percent ash. However, this time limit proved to be impractical because the base remained plastic for several days. In attempting to apply the cure coat, rutting occurred (20). Oklahoma also requires a limit of two hours after final mixing to complete compaction (6). Texas requires compaction to be completed within 6 hours of addition and mixing of the last stabilizing agent (9).

4. RESEARCH METHODOLOGY

The testing program attempts to examine the most common test methods used for fly ash-soil stabilization with respect to the unique material properties of locally produced Class C fly ash. The variables addressed in this investigation were soil and stabilizer types, mix design, and testing requirements. Two soil types and three stabilizers were chosen for the study. Mix designs were examined based on soil type as well as accepted state practices. Tests performed were a combination of ASTM requirements and state practices. Curing methods and a microanalysis were also included in the evaluation of test results.

4.1 Materials

Portland Cement: A type I portland cement meeting ASTM C 150 requirements was used in the soil cement mixtures. The pH was determined by ASTM C 977 to be 12.5. The specific gravity was determined in accordance with ASTM C 188 to be 3.12.

Hydrated Lime: The hydrated lime used in the lime-fly ash applications met the requirements ASTM C 207 for Type N. It had a high calcium content as indicated by its measured pH of 12.5 (ASTM C 977).

ASTM Class C Fly Ash: The fly ash used was an ASTM Class C fly ash obtained from Bayou Ash, Baton Rouge, Louisiana. A test analysis of the chemical and physical properties were provided by Bayou Ash and is presented with other test data. The CaO constituents of the fly ash were reported as 27.5 percent by Bayou Ash. As previously discussed, fly ashes with high CaO contents of 20 percent and more have been reported as satisfactorily stabilizing soils without the use of lime.

Fine Aggregate: Two sands were utilized in the tests conducted. Most of the test program made use of a common river sand that is readily available. A second sand was utilized in a few tests and consisted of a "clean" commercially available fine-sand.

Bentonite Clay: A bentonite clay was used in this study. It was purchased as a commercial product named "Pure Gold". It was identified as a sodium bentonite clay with a certified pH of 9 to 10 standard units.

4.2 Mix Proportions

The combination of material mixtures used in the analyses included:

(1) A-3 river sand plus cement;
(2) A-3 river sand plus fly ash;
(3) A-3 river sand plus lime plus fly ash; and
(4) bentonite clay plus lime and fly ash

(1) A-3 River Sand plus Cement

Moisture-density relationships using standard Proctor compaction were determined for the A-3 sand and 2, 4, 6, 8, 10 and 12 percent cement. Cylinders of each mix were compacted at the theoretical maximum dry density and optimum moisture content, cured for 7 days at 73°F, and tested on a UHTM so that comparisons could be made with fly ash admixture strengths.

(2) A-3 River Sand plus Fly Ash

For maximum stability, a base-course aggregate should have sufficient fines to fill the voids of the coarser aggregate particles. In addition to cementitious properties, a Class C fly ash also provides additional stability as a fine filler material. The following expression, referred to as "Talbot's relationship" by some and as "Fuller's maximum density curve" by others, was used as an approximation of the combined gradation requirements of the A-3 river sand and fly ash which would provide maximum density.

$$S = \left(\frac{d}{d_{max}}\right)^m$$

where:

S = weight percent of particles with diameter less than d
d_{max} = diameter of the largest particle in the mix
m = empirically determined exponent depending on d_{max}

Values of m typically range from 0.11 to 0.66, but the range 0.4 to 0.5 is more commonly used (33). Values for d and d_{max} were estimated based on the particle size analysis of the fly ash and sand. Values of 0.4 and 0.5 were used for m.

The actual gradation required for maximum density will also depend on other characteristics of the materials. Trial mixtures of various percentages of fly ash with the sand were also fabricated for comparison. Standard Proctor compactions were performed on the A-3 river sand with additions of fly ash ranging from 10 to 42 percent by total dry weight, to encompass all values predicted by the Talbot equation.

Standard Proctor specimens, prepared with fly ash percentages of 10, 20, 30 and 40, were cured at 23°C for 7 days then tested on the UHTM for unconfined compressive strength. A 25 percent fly ash plus A-3 river sand was chosen for more extensive testing.

(3) A-3 River Sand plus Lime plus Fly Ash

The selection process in choosing a lime to fly ash ratio was reviewed. Moisture-density relationships were determined for mixtures of A-3 sand with both fly ash and lime. Ratios of lime to fly ash tested include 1:2, 1:3, 1:4, and 1:5. These ratios were chosen because they are common ratios already used in the field with success.

Three percent is the minimum amount of lime that is suggested for use in the field due to mixing requirements, although lower rates have been used with success. Upper limits for lime addition are generally based on cost and fall within the range of 5 to 10 percent. Based on the pH test results and the minimum requirements, 3 percent lime was used for combination with fly ash in each of the chosen ratios. This resulted in lime-ash combinations of 3-6 percent; 3-9 percent; 3-12 percent and 3-15 percent.

Also tested were lime-ash combinations holding the fly ash percentage approximately constant at 15 percent. Thus, lime - fly ash combinations of 7.5-15 percent; 5-15 percent; and 4-16 percent were also tested.

Using the optimum moisture content as determined by the standard Proctor compactions, Harvard compaction samples were prepared for lime-fly ash ratios of 3-6 percent; 3-9 percent; 3-12 percent and 3-15 percent, cured for 1 day at 50°C, then tested for unconfined compressive strength. Additions of 3 percent lime and 15 percent fly ash were chosen for further testing.

Bentonite Mix Proportions

The mixture proportions used by Ferrell (3), 5 percent lime plus 20 percent fly ash plus 75 percent bentonite, were also used in this project. The method of determining the optimum moisture content differed from that study in that the Standard Proctor moisture-density test results for the lime-fly ash and bentonite were selected for this investigation. However, the test results produced similar results for the density and moisture content.

4.3 Testing Programs and Test Procedures

4.3.1 Compaction and Specimen Preparation

There are a variety of compaction methods and specimen types used in soil stabilization studies. The test specimens fabricated in this study included several different compaction techniques. These included the standard and modified Proctor compaction, the Harvard miniature compaction, and the Texas compaction method, TEX-113-E. Using different compaction techniques,this was done as part of a comparative study and, in the case of the Harvard samples, as a means of expediency in fabricating and storing a number of samples for the long term analyses.

In three of the compaction methods used (standard Proctor, modified Proctor and the Texas compaction series), a uniform, accurate degree of compaction was accomplished with a motorized soil compaction hammer. Duplicate specimens were fabricated for comparison in tests. An effort was also made to maintain the moisture contents to within plus or minus one percent and the dry densities within plus or minus three pounds of the theoretical dry weights for the duplicate specimens produced. The moisture density relationships established for standard and modified Proctor specimens included the following materials:

Standard Proctor

A-3 river sand, bentonite, and fly ash
10 to 42 percent fly ash + A-3 river sand
5 percent lime + 20 percent fly ash + bentonite
3 percent lime + 6, 9, 12, and 15 percent fly ash + A-3 river sand
4 percent lime + 16 percent fly ash + A-3 river sand
5 and 7.5 percent lime + 15 percent fly ash + A-3 river sand
2, 4, 6, 8, 10, and 12 percent cement + A-3 river sand

Modified Proctor

25 to 35 percent fly ash + A-3 river sand
2 percent lime + 4 percent fly ash + A-3 river sand

The Texas Method Tex-113-E requires a compactive effort of 13.26 ft-lbs/in^3. 16.67 blows per layer was calculated as being required to meet the compaction effort of this method. Thus, 17 blows per layer was selected for this project with a compactive effort of 13.5 ft-lbs/in^3. An extension collar was used with a Proctor mold to provide a compacted specimen height (6 in.) specified by the method. Specimens prepared by the Texas method included:

Texas Method Tex-113-E

25 percent fly ash + A-3 river sand
5 percent lime + 20 percent fly ash + bentonite
3 percent lime + 15 percent fly ash + A-3 river sand

The Harvard compaction specimens were fabricated in an effort to reproduce the compaction effort achieved in the standard Proctor method. Through trial and error, the number of tamps required to produce similar densities and moisture conditions was determined. Five layers with 25 tamps per layer met these conditions and were used for all Harvard preparations. The optimum moisture content obtained by the standard Proctor compaction was used to prepare the Harvard samples for testing. These specimens were prepared as part of the investigation concerning the effects of curing time and temperature and on long-term performance.

Curing of Compacted Specimen

After extrusion from the mold, samples were first wrapped in plastic wrap and then with aluminum foil for curing in a humidity room. Samples which were to be accelerated in an oven were prepared in a similar manner with the addition of a "double-bagged system." The inner bag was used to prevent the sample from saturation and the outer bag contained water to ensure that the sample did not dry out during oven curing.

Curing consisted of "normal" cures at 23° C (73° F) in a humidity room for 1, 7, 28, 90, 180 and 365 days and "accelerated" curing at 50°C (122°F) in an oven for 1, 7, 28 and 90 days. The ASTM C 593 curing method of 7 days at 38° C (100° F) was used in the durability tests.

4.3.2 Stability and Strength

Unconfined Compression Test

The unconfined compression test, ASTM C 39 or D 2166, is commonly used to compare and evaluate soils stabilized with cementing agents, portland cement, lime and fly ash. Compression tests, ASTM C 39, on the Proctor or Texas size specimen were accomplished with a Universal Hydraulic Testing Machine. The UHTM does not allow for control of axial strain other than through visual inspection. Only the peak or ultimate strength was measured on these samples. The compressive strength tests were conducted on sets of three specimens for the different mix combinations evaluated.

The combination of materials of the larger specimen types that were tested included:

Standard Proctor Specimen

7-day cure at 73°F
    10, 20, 30, and 40 percent fly ash + A-3 river sand
    2, 4, 6, 8,10, and 12 percent cement + A-3 river sand
1, 7, and 28-day at 50°C and at 73°F
    25 percent fly ash + A-3 river sand
    5 percent lime + 20 percent fly ash + bentonite
1-day at 50°C and 7-day at 73°F
    3 percent lime + 15 percent fly ash + A-3 river sand

Modified Proctor Specimen

1-day cure at 73°F
    30 percent fly ash + A-3 river sand
1 & 6-day cure at 73°F
    2 percent lime + 4 percent fly ash + A-3 river sand

Texas Method Tex-113-E

1-day at 50°C and 7-day at 73°F
    25 percent fly ash + A-3 river sand
    5 percent lime + 20 percent fly ash + bentonite
    3 percent lime + 15 percent fly ash + A-3 river sand
1, 7, and 28-day at 50°C and at 73°F
    25 percent fly ash + A-3 river sand

Measurements of stress and strain were recorded throughout the loading sequence in the unconfined compression tests, ASTM D 2166, conducted with the Harvard compaction specimen. Approximately 50 Harvard compaction specimens were prepared for each of the three fly ash mix combinations. These were used in tests evaluating long-term effects as well as a comparative analysis of specimen size effects. The mixtures and curing conditions used in the testing programs for these specimen included:

<u>Harvard Miniature Specimen</u>

- 1-day cure at 50°C
  - 3 percent lime + 6, 9, 12 & 15 percent fly ash + A-3 sand
- 1, 7, 28, and 90-day cure at 50°C and at 73°F
  - 25 percent fly ash + A-3 river sand (FAS & AFAS)
  - 5 percent lime + 20 percent fly ash + bentonite (FLB & AFLB)
  - 3 percent lime + 15 percent fly ash + A-3 river sand (FLS & AFLS)
- 180- and 365-day cure at 73°F
  - 25 percent fly ash + A-3 river sand
  - 5 percent lime + 20 percent fly ash + bentonite

Durability Tests:

Durability testing was done in accordance with ASTM C 593, except standard Proctor-specimens were used. After a 7-day cure at 38° C, the lime/fly ash-soil specimens were subjected to de-airing in a vacuum chamber for 30 minutes followed by soaking in water for an additional hour under vacuum saturation. The specimens were then subjected to unconfined compression testing. The specimens tested were compacted by the standard Proctor method and included:

- Vacuum Saturation (ASTM C 593); 7-day at 38°C
  - 25 percent fly ash + A-3 river sand
  - 5 percent lime + 20 percent fly ash + bentonite
  - 3 percent lime + 15 percent fly ash + A-3 river sand

Direct Shear:

The direct shear test (ASTM D 3080) is not commonly employed in an evaluation of stabilized soils. It was included in the testing program in an effort to better identify and understand the development of the engineering properties of the chemically stabilized soils.

Combinations of fly ash, lime and sand were tested. Specimens were compacted by hand into a direct shear mold as either dry materials or mixed with quantities at the estimated optimum moisture content followed by a 7 day cure (73°F or 21°C) prior to testing. Test specimens included:

A-3 river sand (dry), Fly Ash (dry)
A-3 river sand + 25 percent fly ash (7-day cure at 73°F)
A-3 river sand + 3 percent lime + 15 percent fly ash (7-day cure at 73°F)

4.3.3 Set Time and Delayed Compaction

There is no specification for determining the set time of fly ash mixes. However, at the time of this study, indications were that the proposed ASTM fly ash specification (D 5239) would include requirements similar to that of the established method for cement, ASTM C 191. This method requires the use of a vicat mold and needle apparatus. A plot of the depth of the penetration of the needle at various time intervals determines the set time. This test procedure was used in evaluating the time of set for the A-3 river sand and a coarse sand.

Because Class C fly ash is self cementing, the determination of set time is important for construction requirements. A series of tests were conducted to evaluate the loss in strength (if any) caused by delayed compaction.

Materials for standard Proctor specimens, 25 percent fly ash plus the A-3 river sand and including water, were mixed and allowed to sit for different periods of time prior to compaction. Specimens were fabricated at a 0, 1/2, 1, 1 1/2, 2, and 3 hours delayed compaction. The compressive strength of these specimens were tested after a 7-day curing period at a temperature of 23° C in a moisture control room.

4.3.4 Long-Term Effects

Sample Preparation:

Test specimens were prepared using a Harvard miniature compaction apparatus. Three test groups were fabricated; (1) 25 percent fly ash plus A-3 river sand, (2) 3 percent lime plus 15 percent fly ash plus river sand, and (3) 5 percent lime plus 20 percent fly ash plus bentonite clay. A set of three specimens consisting of these material proportions were made for different curing times and curing temperatures. Each specimen was compacted in five layers using 25 tamps per layer. The compaction moisture content used was the optimum moisture content determined from the standard Proctor compaction test.

After extrusion from the mold, samples were wrapped in plastic wrap followed by aluminum foil for curing. Two curing conditions were used. A "normal" cure at 23° C in a humidity room and an "accelerated" cure in an oven at 50° C. Samples which were subjected to an accelerated cure were prepared with the addition of a "double-bagged system." The outer bag contained water to ensure that the sample did not dry out during oven curing. The inner bag was to prevent the specimen from becoming inundated with the water. Normally cured specimens were tested after 1, 7, 28, 90, 180, and 365 days. Accelerated specimens were tested after 1, 7, 28, and 90 days. The mix proportions of materials used are given above.

Stress-Strain Properties and Microstructure:

The unconfined compression strength test, ASTM D 2166, was conducted on each specimen at the end of the curing periods. Variation in strength and elastic properties was measured in analyzing the effects of time.

The development of microstructure and cementitious crystals at the end of the various curing periods and curing conditions were established by scanning electron microscopy, energy dispersive spectral analysis, and x-ray diffractometry. Fractured specimens were used for investigation under the scanning electron microscope, SEM. Samples approximately 4 x 4 mm were mounted on sample stubs and coated with 250 Angstroms of carbon for energy dispersal spectral analysis, EDS. Samples to be photographed were additionally coated with gold to reduce charging effects. EDS analysis and photomicrography were performed on an AMRAY 1820 Digital Scanning Electron Microscope operated at 15 kV acceleration potential, utilizing a 300 micrometer final aperture for analysis and a 200 micrometer aperture for photography and generally large spot size for analysis and smaller spot size for photography. Working distance was kept at 18 mm; sample tilt for analysis was at 45 degrees. Samples studied under the backscatter electron detector were kept at 0 degrees tilt and a large spot size with 300 micrometer final aperture employed. Photography was performed using 100 ASA 4 x 5 inch sheet film (T-Max). Developing followed standard practice.

Approximately 100 mg of each sample were ground in an agate mortar to a particle size of less than 10 micrometers and submitted to X-Ray diffraction. Scans were performed on SCINTAG XDS 2000 X-Ray Diffractometer. A range of 2 to 65 degrees two theta and a scan rate of 1 degree two theta per minute were employed. Identification of materials with regular or repeating patterns of crystal structure is provided by the X-Ray diffraction patterns produced with the diffraction angle, theta. An automatic peak finder was utilized and peak files were attached to each scan. Resulting spectra were compared to potential mineral phases and matches were noted.

5. TEST RESULTS AND ANALYSES

5.1 Material Properties

Class C Fly Ash: The chemical and physical properties of the Bayou Ash fly ash used in this study are shown in Table 1. They exceed all ASTM C 311 and C 618 specifications in test reports provided by the supplier, Bayou Ash, and in those conducted during this study. The physical and chemical properties most often cited as having a significant influence on the pozzolanic character of ASTM Class C fly ash are the fineness of the particulates, the presence of carbon, and the amount and form of the calcium oxide constituents.

The fineness of the fly ash, i.e., percent retained on the No. 325 sieve, was measured in wet sieve tests (ASTM C 430). An average value of 8.6 percent retained on the No. 325 or 91.4 percent passing was determined. This was consistent with the test results provided by Bayou Ash.

The CaO component of the Bayou Ash fly ash was reported as 27.5 percent by Bayou Ash and measured in laboratory tests during the study to range between 28 and 32 percent. The percentage of CaO present was determined by wet chemistry methods.

The pH of the fly ash was determined by ASTM C 977 procedures to be 11.6. The pH reported in the Bayou Ash tests provided was 12.53. Bayou Ash reported the specific gravity as 2.61. The average results of specific gravity tests (ASTM C 188) conducted on the fly ash used in the study were 2.66.

A feature analysis of the fly ash particles was conducted with the scanning electron microscope (SEM). The results produced a statistical analysis in terms of the maximum chord length of the particles and the feature areas. The results of the feature analysis are provided in Table 2 and a plot of the gradation curve using the maximum chord length is included in Figure 1.

Sands: A river sand which is readily available locally was used in evaluating the performance of the fly ash. It consisted of a fine sand with some silt. Particle size analyses (sieve and hydrometer, ASTM D 422) were conducted to establish its gradation properties and soil classification, Table 3 and Figure 1. Its AASHTO Classification is A-3. Since the river sand included a small portion of fines (approximately 7 percent passing the No. 200 sieve), it has a dual classification under the Unified Classification system as a poorly-graded, silty-sand, SP-SM. The maximum dry density and corresponding optimum moisture content of the river sand were determined to be 95 pcf and 16.5 percent, respectively.

A second fine-sand was used in some tests and identified as Sand #2. It also had an AASHTO Classification as an A-3 sand. However, it contained little or no fines, i.e., sizes less than the No. 200 sieve size, Table 3 and Figure 1. Its Unified Classification was determined to be a poorly graded sand, SP. Both were fine sands with uniform particle sizes.

5.2 Density / Compaction

5.2.1 Compaction Effort and Materials Influence

The compaction methods compared in the laboratory included the standard and modified Proctor, the Harvard miniature, and the Texas Method Tex-113-E. Proctor and Texas samples were compacted using an automatic compaction device. The Harvard miniature compaction method was used only to produce smaller specimens with compacted characteristics similar to the maximum density and moisture conditions found in the standard Proctor method and for use in comparative strength tests.

Moisture-density relationships for various percentages of portland cement and river sand were established for use in comparing a cement-treated soil with the fly ash specimen in strength tests. The standard Proctor compaction method was used. These are summarized in Table 4.

The optimum moisture-maximum density relationships established for the fly ash and A-3 river sand with the standard and modified Proctor compaction methods are provided in Table 5. The moisture-density relationship for river sand and fly ash from the Texas method is given in Table 6. As would be expected, the maximum density determined for the 25 percent fly ash plus river sand reflects the compaction effort. The greatest density of 121 pcf achieved in the modified Proctor compaction compared to 116.5 pcf in the standard Proctor method. The results of the Texas method falls between with a maximum dry density of 118.86 pcf.

Optimum moisture contents did not vary greatly for the different compaction techniques. Values obtained by the standard and modified Proctor methods and the Texas method are 9.1, 8.5, and 9.4, respectively, for 25 percent fly ash added to A-3 river sand.

The particle sizes present, their frequency, and characteristic features also influence the ability to compact a mixture. Figure 2 provides a summary of the variation of the densities and moisture contents of the fly ash-sand mixtures with the variation of fly ash from 0 to 100 percent. The two curves of this figure demonstrate a unique relationship between the density, molding water and percentage of fly ash. For a given soil and fly ash combination, there is a quantity of fly ash that provides maximum density with the least amount of water used for compaction. The fly ash acts as filler for the sand's voids until this critical quantity of fly ash is achieved. Beyond that point, the fly ash begins to float the sand particles, forming a fly ash matrix for the mixture, followed by a decrease in density. This relationship has been noted in a previous study (2). Up to a point, the reduction in moisture requirements is attributed to the lubricating characteristics of the spherical fly ash particles.

The results of the compaction tests for the lime-fly ash plus the river sand and the bentonite clay are presented in Tables 7 and 8. Comments similar to those concerning the densities achieved and moisture requirements for the fly ash acting as a lone agent could be made for these other mixture proportions. There were increases in dry-weight densities with the addition of the fly ash and lime. In the river sand, this is again attributed to the fly ash and lime acting as a filler material and improving the gradation characteristics. In the case of the bentonite clay, the lime also alters the clay materials and their gradation characteristics.

5.3 Strength Characteristics

5.3.1 Portland Cement vs Fly Ash Stabilization

An extensive amount of experience using portland cement-treated soils exists within the Louisiana Department of Transportation and Development. The percent of cement to be used for stabilization is determined in accordance with DOTD Designation TR 432. In this standard, a minimum compressive strength of 250 psi at the end of a 7 day cure (23° C) is required. Cement-treated soil was used in comparing the stabilization performance of the Class C fly ash. The results of the compressive strength tests conducted on the portland cement-treated and fly ash stabilized river sand for varying percentages of admixture are presented in Table 9. The strength (311 psi for 7-day cure at 23° C) reported for the 25 percent fly ash specimen in Table 9 is inconsistent with the other tests conducted for the other fly ash percentages. This value seems high and is not in agreement with tests conducted on other specimens (discussed later) using the same percentage of fly ash.

In both series of tests for portland cement and fly ash, there was a corresponding increase in strength and density as the level of percentages was increased for each, Figures 3 and 4. With the exception of the 25 percent fly ash test, there appears to be a consistent relationship between the gain in strength or density and the increase of fly ash and cement over the range of additives used.

In Figure 5, the percentage of cement or fly used with the corresponding compressive strength developed is compared graphically. The least percentage of portland cement providing a compressive strength of 250 psi (LaDOTD TR 432) is approximately 9 percent compared with about 32 percent for the fly ash.

The "smoothness" and consistency of the curve for this relationship is similar to that occurring in a previous study (2) for a uniform sand without or with very little fines. The curves are somewhat more erratic with the existence of a higher silt/clay content. However, the general characteristics of the strength curves for cement and fly ash are typical for sands.

The curves in Figure 5 demonstrate a possibility of predicting the amount of fly ash required to equal the strength performance occurring with a specific amount of portland cement (19). Several studies (10, 9, 11) have shown that much (if not most) of the calcium constituents in Class C fly ash exists in the form of calcium silicates, C-S-H, similar to those of portland cement. Thus, the percent of CaO present in the fly ash could be a good indicator for estimating an equivalent percent of portland cement (in terms of strength delivered).

The percent CaO in the fly ash used was tested and reported by Bayou Ash as 27.5 and 26.24 percent. The analysis during this study measured the CaO content as 31.6 percent in wet chemistry tests and estimated as 28.12 percent in a quick test discussed later. Using the curves from Figure 5 and a CaO content of 28 percent, the "equivalent percentage" of portland cement (in terms of strength) found by multiplying by 0.28 and the percent of fly ash used gives very good agreement. For example, 30 percent fly ash equates to the performance of 8+ percent portland cement (0.30 x 0.28 = 0.084) producing a strength of approximately 203 psi. Other percentages also provide good agreement.

The ability to predict the amount of fly ash necessary to match the performance of cement-treated soils could eliminate or provide a short cut in comparing alternative stabilizing agents. The required quantities of fly ash could be estimated and evaluated on the basis of cost prior to testing. In seeking to identify the specific percentages of fly ash required, the range of mixture proportions could also be narrowed and the number of tests reduced.

There are, of course, other factors contributing to the gain in strength besides an "equivalent percentage of portland cement." In addition to the cementitious components of the fly ash, the mixture is also mechanically stabilized with the addition of filler material provided by the larger percentages of fly ash used. Also, the strength curves developed with test results on mixtures that utilize sands with a higher silt/clay content are more erratic. However, compensation for the silt/clay materials can also be addressed to some extent (19). The concept does seem to have potential as a guide for testing in mixture design.

5.3.2 Strength Performance of Specimen Types

The results of tests conducted using standard Proctor and the Texas method for different mixtures are provided in Tables 10 and 11. As stated above, the test results on the standard Proctor specimens for the fly ash-sand mixture appear to be inconsistent with tests conducted with other fly ash-sand mixtures. Specimens fabricated by the Texas method and Harvard miniature compactor tested at lower strengths, 215 psi and 140 psi, respectively.

A comparison for the effects of size and specimen geometry (L/D ratio) is made in Figure 6. The higher L/D ratios of the Texas (L/D = 1.5) and Harvard (L/D = 2.0) were evident by the specimens fracture surfaces which occurred on failure. The higher L/D ratios permitted the failure surface to develop within the specimens rather than at the end.

EPRI TR-100472 (1) recommends that the length be twice the diameter, an L/D of 2. If not, the resultant strength should be adjusted in accordance with ASTM C 42. In this case, the correction factor for the Harvard, Texas, and Proctor specimens would be 1.00, 0.96, and 0.90 +/-, respectively. However, ASTM C 593 uses 4.0 x 4.6 Proctor specimens and states that no L/D correction will be considered in the computation of the compressive strength.

Considering all the possible testing anomalies, Figure 6 is inconclusive with respect to L/D requirements. Figures 7 through 12 present the variation in strength with respect to specimen size for each of the material mixtures. Generally, the test results of the largers specimens, Proctor and Texas, yielded higher compressive strengths. With the limitations of compacted laboratory test specimens to simulate field conditions, soil particle sizes, compaction effort, etc., the larger samples are more desirable. In considering the L/D ratio, the larger ratio does eliminate the problems of the failure surface passing through the specimen's end. However, as long as the loaded surface is smooth (preferably capped) and normal to the specimen's axis, the test is performed in a consistent manner, and the strength criteria meets design requirements, either the Proctor or the Texas specimen should be acceptable.

5.3.3 Direct Shear Tests

The results of direct shear tests on the dry sand and fly ash yielded friction angles of 28° and 25°, respectively, Table 12. Direct shear tests were also conducted on 7-day cured (23° C) mixtures of 25 percent plus A-3 river sand and 3 percent lime plus 15 percent fly ash plus A-3 river sand, Table 12. The cohesive strength reflecting the state of the cementitious products of the fly ash-sand was 4.5 times that developed in the lime-fly ash-sand specimen for the 7-day curing period. However, the friction angle was greater in the lime-fly ash-sand specimen (33° vs. 21°). The fly ash-sand appears to be more brittle with less shear resistance available after the strength of the fly ash cement products have been exceeded. It may be possible that the magnitude of the load required to overcome the cement products (cohesion) is significantly greater and overshadows the mechanical, frictional resistance (phi angle).

5.4 Durability

Table 10 provides a comparison of the strength performance of the three mixtures at the end of a 7-day cure with and without vacuum saturation. Both mixtures with the A-3 river sand experienced a reduction in compressive strength after being subjected to the vacuum saturation procedures. This did not occur with the bentonite clay. Figure 13 positions the performance of each with respect to a line of equal performance. Both sands fall below the line and the bentonite mixture is positioned above. The nonpozzolanic silt materials of this river sand, approximately 7 percent, may play some role in the strength loss under vacuum saturation conditions and/or the 7-day cure may be inadequate.

The nature of this test is probably more applicable to the sand than the clay. The extent of test time to which the clay sample is subjected to the vacuum and then saturation condition may be too brief for the water to thoroughly penetrate the clay specimen. Also, in this procedure, the vacuum saturation specimens are cured at a higher temperature (100° F) than that of the "normal" cured specimen (73° F) used in this study. The enriched pozzolanic materials of the bentonite clay specimens in combination with the higher percentages of lime and fly-ash used may have also produced an additional strength gain.

5.5 Cure Conditions: Time and Temperature

As discussed in previous sections, the time and temperature requirements of the various curing specifications are an attempt to establish the potential of the pozzolan-soil mixture. The strength criteria after a specified curing period and under controlled conditions are not the ultimate strength of the stabilized soil but an indication of what can be achieved.

A curing period of 28 days has been cited as being required to assess the strength development of a lime-fly ash-soil. The ASTM C 593 accelerated cure of 38° C (100° F) is an attempt to simulate the 28-day cure at 23° C (73° F) required for lime-pozzolan mixtures (1). Ferrel (3) suggests a 1-day cure accelerated with a curing temperature of 50° C for lime-fly ash mixtures. Cement-treated soils are commonly cured at 7 days under 23° C. The properties of a Class C fly ash and the cementitious products produced include both the fast-acting calcium silicates and with lime, the slower pozzolanic reactions. Differences in required curing conditions make it difficult to compare the results of different test methods. As a result, mixture criteria for a particular test method may not apply to different materials and analyses.

In reviewing long-term effects, test specimens were cured at normal temperatures (23° C) for time periods of 1, 7, 28, 90, 180 and 365 days. An accelerated cure at 50° C (3) was made with duplicate specimens. The three mixtures reviewed included: (1) 25 percent fly ash plus the A-3 river sand, (2) 3 percent lime plus 15 percent fly ash plus A-3 river sand, and (3) 5 percent lime plus 20 percent fly ash plus bentonite clay.

Tables 13, 14, and 15 include the strengths developed at 7, 28 and 365 days under normal temperatures and the strength measured on specimens subjected to an elevated temperature of 50° C for a curing period of 7 days. The developed strength for other curing periods was also determined. The curing times and conditions were selected because they are used or have been proposed; i.e.; 1-day accelerated cure (3); a 7-day cure for portland cement and an accelerated cure for the same time period for lime-fly ash (ASTM C 593); 28-days normal curing for lime-pozzolanic mixtures (1). The ratio of the developed strength occurring under normal conditions at the 7- and 28-day cures was compared to that at one year. The strength of the specimens cured under the higher temperature in 1 and 7 days was compared with the 28- and 365-day normal temperature cure.

The rate and magnitude at which these three material groups gained strength for the first 7 to 28 days were similar. Note that strengths achieved for the fly ash-sand, the lime-fly ash-sand, and the lime-fly ash-bentonite at the end of 28 days were 150 psi, 191 psi, and 189.3 psi respectively. However, the long-term increase in strength after that was much greater with the lime-fly ash-sand specimens (430 percent increase or 5.3 times greater). The rate of strength gain in the other two groups was much slower, i.e., only 30 percent increase in the bentonite and a gain of 139 percent in the fly ash-sand strengths. The continuous, large gains in strength that occur in the lime-fly ash specimens are credited to the lack of fines and the enriched pozzolanic environment of the sand's voids created by the lime and fly ash.

In comparing the 7-day normal cure strength with the 365-day cure, the fly ash-sand and bentonite specimens indicate a development of strength that ranges from 39 to 48 percent of the ultimate or 365-day strength. The lime-fly ash-sand in 7-days is only at 5 percent of its 365-day strength. Similar observations can be made with the 28-day normal curing periods where 42 and 77 percent of the 365-day strength are achieved for the fly ash-sand and bentonite specimens compared to 20 percent for the lime-fly ash-sand specimens. In reviewing the rate of strength gain for the three material groups, the fly ash-sand and lime-fly ash-bentonite appear to be approaching their ultimate strength at 365 days. However, the rate of strength gain by the lime-fly ash-sand specimen appears to be continuing. Thus, while the 7-day or 28-day normal cure may predict or give some indication of the strength potential for the fly ash-sand and bentonite mixtures, neither of these cure periods would seem to adequately measure the strength potential of the lime-fly ash-sand mixture.

An examination of the accelerated cure strengths as a predictor of stabilized mixtures potential leads to similar observations, Tables 13, 14, and 15. The 1-day accelerated cure at 50° C proposed by Ferrel (3) did a good job of predicting the 28-day normal (23° C) strength for the fly ash-sand and the bentonite mixture. However, this test procedure predicted only 24 percent of the 28-day strength developed in the lime-fly ash-sand specimens. Thus, it may not be a good comparison of the 28-day strength for lime-fly ash used with a "clean" sand, i.e., without fines. An examination of the 7-day accelerated cure for the 50° C temperature leads to similar conclusions.

5.6 Long Term Effects

5.6.1 Strength / Elastic Properties

A summary of the long-term testing results for the three fly ash-soil mixtures is given in Tables 16, 17, and 18. All test specimen were molded with the Harvard miniature compaction device. The average density and moisture content for each curing set corresponding to a specific cure period are included. Samples were either cured under a "normal" temperature of 23° C or at an "accelerated" temperature of 50° C as noted. The unconfined compressive strength and elastic properties (strain, elastic modulus at failure, and E_{70} at 70 percent of failure stress) for each are also shown in the tables.

Figures 14 through 19 graphically depict the performance of each fly ash-soil combination at various curing periods in the course of one year. Each figure documents and compares the strength or elastic modulus record for the normal cured specimens and those cured under the accelerated temperature. General comments concerning the compressive strength and elastic modulus for each fly ash-soil is as follows:

Unconfined Compressive Strength:

The gain in strength for the fly ash plus A-3 river sand specimens appears to have occurred in two stages or plateaus, Figure 14. The major gain in strength is almost immediate, i.e., within the first 1- to 7-day cure periods. Almost fifty percent of the strength acquired over the period of one year was acquired within the first week. The next major increase took place between 90 and 180 days. In comparing the 1-day accelerated strength with the 28-day strength (3), they are the same at 150 psi.

Strength development in the lime-fly ash-sand specimen is slow initially, Figure 15. The strength test results on specimens cured within 7 days are unreliable in predicting this mixture's potential, including the accelerated 1-day cured specimens. However, after the first week, large gains in strength occurred with an indication that they will continue to make significant increases in strength beyond the one year test period. The 1-day accelerated strength is not indicative of the 28-day normal cure strength for this mixture.

The strength development in the lime-fly ash-bentonite specimens appear to begin to peak at 28 days, Figure 16. Significant gains in strength for this mixture are not projected by the strength curve of this figure. The 1-day accelerated cure gave an acceptable prediction of the 28-day strength, i.e, approximate 86 percent of the 28-day strength.

Modulus of Elasticity:

Similar to its strength development, the elastic modulus or stress-strain properties of the fly ash-sand specimens are acquired within the first seven days, Figure 17. The lime-fly ash-sand specimens had large gains in stiffness through the first 90 days, Figure 18. Seventy-five percent of the lime-fly ash-sand stiffness was acquired during this time. After 90 days the elastic modulus is increasing but at a slower rate. The bentonite specimen, Figure 19, increased in stiffness through the first 90 days, similar to the lime-fly ash-sand specimen, but experienced a decrease or a reduction in its modulus after 90 days. Almost a 50 percent reduction in the elastic modulus occurred after the 90-day cure. The elastic modulus measured in the accelerated tests for the bentonite also gave an erratic up and down performance.

To compare the performance of the different material combinations, Figures 20 and 21 show each mixture's one year record for strength and elastic modulus, respectively. In summarizing the long term performance of the three mixtures, the following observations are noted. Where Class C fly ash is used as a lone stabilizing agent, a seven day normal cure would seem adequate. There is a distinct long-term advantage in using lime with fly ash in a sand or coarse grained soil. The 1-day accelerated cure is not reliable in predicting the 28-day normal cured strength for lime-fly ash and sand. A decrease in the modulus of elasticity of the lime-fly ash-bentonite clay makes its long-term performance questionable.

5.6.2 Microanalysis

A microanalysis was conducted on samples taken from test specimens used in the long term study. The cementitious products and morphology of the different material groups were identified. The microanalysis included the following sample sets:

- FAS - fly ash and sand cured at 23° C in the humidity room, sample no. 004 to 018,
- AFAS - fly ash and sand cured at 50° C under humid conditions, samples no. 101 to 119,
- FLS - lime, fly ash, and sand cured at 23° C in the humidity room, sample no. 030 to 047,
- AFLS - lime, fly ash, and sand cured at 50° C under humid conditions, sample no. 301 to 320,
- FLB - lime, fly ash, and bentonite cured at 23° C in the humidity room, sample no. 064 to 079
- AFLB - lime, fly ash, and bentonite cured at 50° C under humid conditions, sample no. 601 to 615.

The moisture content and density of each sample were also determined after compression strength tests.

Approximately 100 mg of each sample were ground in an agate mortar to a particle size of less than 10 micrometers and submitted to X-Ray diffraction. An automated peak finder was utilized and peak files were attached to each scan. Resulting spectra were compared to potential mineral phases and matches were noted.

Fractured samples from the failed compression specimens were used for investigation under the scanning electron microscope, SEM. Energy dispersive spectral analyses (EDS) and photomicrography were performed.

Group FAS:

The moisture content measurements indicated a general trend to a drier material in longer curing periods. No trend in the variation of the densities was detected. General observations made with x-ray diffraction showed the reflections of secondary phases, particular Ca-silicate phases and Ca-Al-silicates become more pronounced with extended curing times. In comparing the 7, 28, 90, 180, and 365 day cured samples, Figure 22, portlandite is seen to decrease after 7 days. Calcite is most abundant from 28 to 90 days.

Rustumite is most common in the 365 day samples. Plombierite is most common from 180 days on. Calcium silicate hydrates are becoming abundant in the 365-day group. Ettringite increases from the 90-day cure and shows a maximum at 365 days. Quartz reflections remain strong throughout the curing sequence. Gypsum and calcite reflections tend to decrease in intensity with time. Evidently, the Ca in these compounds is easier mobilized and utilized in the formation of Ca-bearing silicates. Cebollite appeared more distinctly in the 90 day samples and appears less distinctly in the 180 day group. However, a local heterogeneity in the sample material may be responsible. Thaumasite seems to decrease in abundance as curing time increases.

Crystalline crusts investigated by EDS in the 7-day cure group indicated dominant Ca and lesser Si, S, and Al. Thus, the crust may consist of more Ca-sulfates and -carbonates than of silicates. In the 180-day sample, dominant Ca and Si are seen, with lesser amounts of Al and S. A potential pitfall may be introduced here, however. Only a flat and polished sample will yield consistent results when investigated by EDS. A rough surface sample like the ones investigated may cause absorption of lighter elements if the travel path of the characteristic X-rays is increased due to sample geometry. Sample areas were selected so this problem would be as insignificant as possible but it cannot be completely avoided. A similar comparison was made with a fly ash sphere in a 7-day sample. The result indicates that Si is dominant, with lesser amounts of Ca, K, and Al. A sphere in the 180-day sample yielded high Ca concentrations, lesser amounts of Si, Al, and K. This may indicate some replacement of the fly ash spheres in process.

Samples cured for 7 days already exhibit crystal growth on sand grains and fly ash sphere surfaces in the photomicrography. The early samples do not yet possess a dense, felty network of prismatic to fibrous secondary phases. This increasing interlocking becomes more and more pronounced in the samples cured for more extensive periods. Figures 23 and 24 are typical photomicrographs of the FAS microstructure.

Group AFAS:

No discernable changes or trends in the moisture contents or densities between the curing periods of the AFAS samples were identified. Figure 25 provides a comparison of the 1-, 7-, 28-, and 90-day cured AFAS samples. Ettringite becomes common from day 7 on and remains about equally abundant in the time span from 7 to 90 days. Calcite is maximum at 90 days. Afwillite does not change much in its abundance but may decrease a little at 90 days. Gypsum increases in the 7- and 28-day samples and shows a maximum at 90 days. This may represent a sample with a localized gypsum-enrichment. Thaumasite is present throughout but more so in the 1- to 7-day samples. Rustumite seems to increase a little with time.

The crystalline secondary phases investigated by EDS during the early cure times (1 day) indicate a composition with dominant Ca and lesser amounts of Si, Al, S, and Fe, while samples cured for 90 days show an increase in Si with high Ca. Some prismatic crystals only showed Ca as the dominant element. Thus, carbonate is also forming. The composition of the fly ash spheres in some early samples (1 day) is predominantly Si, and those cured for 7 to 28 days are Si and Ca, and AL, Si, and Ca, respectively. A 90-day sample showed major Si, lesser Al and Ca. However, some of the fly ash may be different in composition from the start.

In the photomicrography study, samples cured from 1 to 7 days already show crystals of secondary phases. A more intense intergrowth of crystals results in a dense network that occurs as time of curing increases, Figures 26 and 27.

Group FLS:

The moisture contents measured indicated a decrease in the retention of water with curing time. The densities remained constant. A comparison of the x-ray diffraction for the 1-, 7-, 90, 180-, and 365-day cures is shown in Figure 28. Ettringite reaches a maximum abundance in the 90 day group. It is still high at 180 days but decreases at 365 days. Rustumite, plombierite, and afwilite are most abundant at 90 days but decrease thereafter.

Gypsum is most plentiful at 7 days, as is calcite. However, a decrease in the abundance of gypsum and cebolite takes place as curing time increased. Quartz remained unchanged with time. Portlandite is still present in the 7-day samples but disappears by day 90. Most likely it is reacting to form other phases. Thaumasite is most common at 365 days. There may be periclase detectable in the day 1 samples. Calcite is present in all groups but seems to decrease a little in the 90 day group.

EDS analysis of early crystalline crusts indicates dominate Ca (either portlandite or calcite). Later crusts become more enriched in Si, with evidence of Ca-bearing silicates beginning to form.

In the early samples, a distinct crust of secondary phases can be seen with the SEM in microphotography. This crust is only loosely attached to sand grains. Later samples show a more tightly, interwoven crust of secondary phases and better adherence to grain surfaces. Figures 29 and 30 provide two different magnifications of the microstructure for the 7-day cure FLS.

Group AFLS:

No changes in either the moisture content or densities for the different curing times were evident. A comparison of the x-ray diffractions for the 1-, 7-, and 90 day accelerated cure samples from the AFLS group is shown in Figure 31. Ettringite does not show much variation in its abundance. Calcium silicate hydrates become more abundant with time. Plombierite reaches a maximum at 7 days and then appears to decrease a little in abundance. Portlandite decreases after day 1. Rustumite is most abundant at 90 days, after a slight decrease at 7 days compared to a little higher abundance at day 1. Afwillite follows the same trend. Thaumasite seems to be most common at day 7 and decreases slightly at 90 days. Calcite seems to increase with time, while gypsum decreases.

The crystalline crusts of secondary phases investigated with EDS do not differ with regard to curing time. In both cases Ca is dominant, with lesser Si, Al, and Fe. Fly ash spheres investigated show a change in composition. Samples cured in one day consist of dominant Si, Al, lesser K and Ca. Spheres of the 90-day samples show dominant Si and Ca and lesser Al.

In the photomicrographs, early samples already show crystalline products, but the intergrowth of the crystalline crust increases as curing time increases. The outer layer of the crust of secondary phases in the 90 day group has a smooth, almost amorphous appearance but the crystalline structure is evident when fractured crust is inspected, Figures 32 and 33.

Group FLB:

No trend could be identified in the variation of moisture content between samples cured at different times. However, a slightly decreasing density with longer curing times seems to exist. X-ray diffraction shows the clay minerals, quartz, and calcite remaining largely unchanged and in abundance as curing progresses, Figure 34. Portlandite disappears and ettringite decreases in abundance as curing increases. The portlandite disappeared by day 7. Calcite and plombierite appear unchanged throughout the time span. Rustumite seems to be most abundant at 7 days and decreases thereafter. Gypsum is most common from 7 to 28 days and decreases after that. Thaumasite increases up to the 180 days and then drops off a bit. Afwillite is most common in the range from 90 to 180 days and then drops off. Ettringite is most abundant from 7 to 28 days and then decreases in abundance. The calcium silicate hydrates become identifiable at 365 days.

The crystalline phases of secondary origin in the early samples detected by EDS indicate dominant Ca with a little less Si and Al. Samples of 90 to 180 days curing time are largely composed of Si and Ca with less amounts of Al. The composition of the fly ash spheres appears to be more variable. Early samples are rich in Al and Si, with less amount of Ca and K. Later samples show considerable variation in the abundances of Ca, Si, and Al.

Photomicrographs, Figures 35 and 36 show a poorly indurated matrix of fly ash spheres. With increasing curing time, the crust of secondary phases forms a more tightly adhering structure. Some secondary carbonate cement forms sporadic patches of tightly adhering grains and spheres.

Group AFLB:

Moisture content and densities decreased with the longer cure time. The quartz and clay minerals remained largely unchanged over time in the x-ray diffraction analyses, Figure 37. Portlandite disappears after day one. Ettringite decreases in its abundance at 90 days. Afwillite and rustumite decrease a little at 90 days. Plombierite, thaumasite, and gypsum remain unchanged. Calcite increases in its abundance at 90 days.

The EDS analysis showed the crystalline phases of secondary origin to differ in their Ca content as curing proceeds. Ca increases with time, while Al and Si remain about equal. Some of the thinner crusts are richer in Al than that found in the thicker crusts.

Photomicrographs show that the solidification of the matrix by crystalline materials increases with time. Fly ash spheres tend to become replaced as curing time proceeds, Figure 38.

Comparison of Sample Groups and Curing

Comparisons were made of the secondary phases occurring among and between the different sample groups at the same and at different curing times. Five different combinations were reviewed. These included a comparisons of all mixture groups (FAS, FLS, and FLB) for the 90-day 23° C cure, all mixture groups (AFAS, AFLS, and AFLB) for the 1-day accelerated 50° C cure, the AFAS 1-day cure with the FAS 28-day cure, the AFLB 1-day cure with the FLB 28-day cure, and the 1-day AFLS with the FLS 7- and 90-day cures.

The 90-day samples of the FAS, FLS, and FLB, cured at 23° C, are compared in Figure 39. Ettringite appears most abundant in the FLS, second most in the FLB, and least abundant in the FAS. Rustumite is lower in the FLB than in the FLS and FAS. Thaumasite seems more abundant in the FLB and the FLS than in FAS. Gypsum seems to be more abundant in the FLB and FLS than in the FAS.

Calcite seems to be approximately equally present, though the FLB may have a little more. Plombierite appears to be more common in the FLB and FLS.

In the comparison of the 1-day accelerated cure of the AFAS, AFLS, and the AFLB, portlandite is still identifiable in all three groups, Figure 40. Gypsum seems to be a little more common in the AFLS and less in the AFLB and AFAS. Ettringite appears to be the most common in the AFAS, while it is in less and approximately comparable amounts in the AFLS and AFLB. Gypsum is most common in the AFLS and less in the AFLB and AFAS (both similar amounts). Calcite is more abundant in the AFLB and AFLS than in the AFAS. Rustumite and thaumasite are about equally abundant in all.

The 1-day AFAS sample contains more ettringite and gypsum in a comparison with the 28-day cured FAS, Figure 41. The FAS sample contains more rustumite and calcium silicate hydrates than the AFAS. FAS may also have a little more calcite than the AFAS. The afwillite and plombierite present appear to be the same.

The 1-day AFLS is compared with the 7- and 90-day FLS, Figure 42. Portlandite appears in the AFLS but is used up in the 7- and 90-day FLS samples. Calcite seems to be more common in the 1-day AFLS and the 7-day FLS groups than in the 90-day FLS sample. Gypsum is about equal in abundance in the 7- and 90-day FLS samples, but is less common in the 1-day AFLS. Plombierite and rustumite are most prevalent in the 90-day FLS sample. Rustumite is about equally abundant in the 1-day AFLS and the 7-day FLS samples, but increases in the 90-day FLS sample. Plombierite is most abundant at 90 days (FLS), less abundant at 7 days (FLS), and least at 1-day (AFLS). Afwillite appears throughout all samples in about equal amounts.

The secondary products of the 7-day AFLS are similar to those of the 90-day-cure FLS. The 90 day normal cure demonstrates the strength potential of the mixture with approximately one-third of the strength gain achieved in one year. Ettringite remains about the same throughout the 90 days in AFLS but reaches a peak in the 90-day FLS group. Thaumasite in both the 7-day AFLS and the 90-day FLS cures are about equal. Gypsum also seems to be about the same for the 90-day FLS and 7-day AFLS cures, as is calcite. In both cases, the scans of the 7-day AFLS and the 90-day FLS show comparable similarities with respect to the presence of secondary phases. The Plombierite in the 7-day AFLS reaches a maximum and then decreases. Plombierite is highest in the 90 day FLS group. Also, rustumite is low in 7-day AFLS but is most common in the 90 day FLS group. Afwillite seems to be more abundant in the 90-day FLS than in the 7-day AFLS. However, in none of the samples is there a total absence of the secondary phases of these curing periods for these two groups (AFLS and FLS). It is only the relative proportions that fluctuate in some cases.

In the comparison of the 1-day accelerated cure of the AFLB and the 28-day cure of the FLB, both scans reveal basically the same content of chemical species, Figure 43. The exception is portlandite which appears to be present only in the 1-day accelerated cure sample, AFLB. This is not surprising since $Ca(OH)_2$ absorbs CO_2 readily from air and is converted to $CaCO_3$. The content of calcite, gypsum, plombierite, and thaumasite in both samples is about the same. Ettringite appears to be more abundant in the 28 day cure sample (FLB). Afwillite may be a little more abundant in the 1-day accelerated sample (AFLB). Rustumite may also be a little less common in the accelerated sample (AFLB).

5.7 Set Time/Delayed Compaction

Class C fly ashes are often described as having a fast set time. This can be very important in attempting to interpret and extend laboratory test results to field conditions. In reviewing the set characteristics of the Bayou Ash fly ash, ASTM C 191 test procedures were used to determine the time of setting of a fly ash-sand paste using the A-3 river sand and a uniform, coarse sand whose sizes varied between the No. 16 sieve (1.18 mm, 99.8 percent finer) and the No. 30 sieve (0.60 mm, 1.2 percent finer).

Table 19 provide the results of the vicat needle penetration with time for the fly ash pastes using the fine river sand and the coarse sand. Table 20 gives the penetration rates for a lime-fly ash paste made with the river sand. Gypsum has been cited as being a retarding agent that has been used with some degree of success. Its effectiveness was investigated by adding gypsum to each paste.

Figures 44, 45, and 46 plot the penetration distance of the vicat needle with respect to time. Figure 44 compares the set of the fly ash in the two different sizes of sands. Both have relatively short setting times. However, the set in the coarse sand occurs within a matter of minutes. The final set (no penetration) of the coarse sand is complete in 15 minutes. Its initial set, defined by ASTM C 191 as the time corresponding to a 25 mm penetration, is estimated at 5 minutes or less.

The final set in the paste made with the river sand is estimated as occurring at 80 to 85 minutes. The initial set takes place at approximately 60 minutes. The set of the river sand mix is not as rapid as the coarse sand mix but nevertheless fast. The larger voids between particles and the greater enrichment of the matrix paste (fly ash alone) is identified as being the reason for the almost immediate set of the coarse sand paste.

Gypsum did appear to have some effect on the setting characteristics. In Figure 45, there appears to be an initial delay with the addition of 5 percent gypsum in the river sand. By adding 20 percent gypsum to the coarse sand paste, the setting rate was slightly slowed the first 5 minutes, then stalled for approximately 10 minutes before beginning again, Figure 46. The final set was complete within 30 minutes. The resistance to penetration of the river sand paste with and without the gypsum was also measured using a pocket penetrometer, Figure 47.

This locally produced fly ash definitely has a flash set. Its set will vary with soil types and other site conditions. However, it is definitely a characteristic that must be addressed in the laboratory testing used to design the pozzolan mixture as well as in construction control.

To further evaluate the effect of a fast set on strength development, a series of Proctor specimens were fabricated at different periods of delay time. The materials including the molding water were thoroughly mixed and allowed to set before compaction. The results are portrayed in Figure 48 with the curve representing the general trend. A one hour delay in these tests represents anywhere from a 7 to 30 percent loss in strength.

As little as 2 to 6 hours to complete compaction after final mixing has been specified by some states in attempting to address the fast set. Others have cited the use of retarders. It may be more realistic to account for these characteristics in the testing program, i.e., delayed compaction or mix adjustments. Even though Class C fly ash has significant cementitious characteristics, many states seem reluctant to use it as a lone stabilizing agent. The addition of lime, even with a very reactive Class C fly ash, provides compensation (through autogenous healing) for destruction of cementitious products formed before final compaction and provides higher strength over longer periods of time.

5.8 Mix Design and Quality Control

5.8.1 Fly Ash QA/QC

The material variation that occurs between and within sources of fly ash are emphasized by many. Steps taken in the past and current trends seem to emphasize the need for material prequalification (i.e., the proposed new standard, ASTM D 5239). The emphasis here is to ensure quality and uniformity of the fly ash. A six month history documenting the fly ash physical and chemical characteristics is proposed. Spot checking during the project is also necessary.

Problems unique to the use of Class C fly ash are concerned with the nature and quantity of the CaO present. Their cementing qualities and availability of "free lime" must be fully understood if they are to be used properly. Also, these self-cementing fly ashes have a tendency to harden or set very quickly. The ability to mix and complete compaction of the stabilized soil must be addressed in terms of construction operation and control, the need for set retarding agents, and/or in the testing program for the mix design.

5.8.2 Gradation Concepts

The initial step required in developing the proper proportions of fly ash mixture is to estimate the fly ash required to fill the voids in the aggregate. Sand aggregates with single-size particles and sands without fines (< 0.075 mm) may require a high fly ash content to serve as a filler or void reducer, as well as a pozzolan in the mixture.

The percentage of fly ash required to meet the theoretical requirements for maximum density (Talbot/Fuller maximum density curve or similar) can be estimated for each particle size using sand and fly ash gradation measurements. While the percentage of fly ash required for a specific grain size can be computed, that percentage required can vary with each particle size. However, by assuming different percentages of fly ash to be added to the sand, the resulting fly ash-sand gradation curve can be established and compared with the theoretical particle distribution for maximum density.

In reviewing the particle distribution for the A-3 river sand, the number 40 sieve size (0.425 mm) was assumed to be the upper limit or maximum particle size. Using this d_{max} value in the Talbot relationship (with an exponent m value of 0.5) a theoretical grain size distribution curve for maximum density was generated, Table 21 and Figure 49. The gradation curves corresponding to 10, 20 and 30 percent fly ash additions to the A-3 river sand are also shown in the figure. Note that with increasing amounts of fly ash, the resulting gradation curve approaches the "theoretical maximum density" gradation curve. The shape and fit of the fly ash-sand curves are not exact due to variation between the particle sizes of both the fly ash and the sand.

An estimate of the percentage of fly ash additive that would provide the maximum density can also be computed with the Talbot relationship. Using the particle size of the No. 325 sieve (0.045 mm), which is also a measurement of the fineness of the fly ash, the percentage for maximum density corresponding to this size is 33 percent.

The percentages finer of the fly ash and sand particles determined for this grain size were 80.4 and 4.85 percent, respectively, Tables 2 and 3. The percentage of fly ash (percent F) that must be added to the fly ash-sand mixture to achieve a 33 percent particle size finer than 0.045 mm is determined as follows:

$$0.804\ (\%F) + 0.0485\ (\%S) = 0.33$$

but Percent Sand, $\%S = 1.0 - \%F$, thus

$$0.804\ (\%F) + 0.0485\ (1.0 - \%F) = 0.33$$

$$\%F = 0.37$$

In order to achieve a 33 percent population of the 0.045 mm size for this fly ash-sand combination, we must add 37 percent fly ash with 63 percent A-3 river sand.

Values between 0.4 to 0.5 for the exponent "m" are most commonly used. If a value of 0.4 is used, the percentage of particles finer than the 0.045 mm size required for maximum density is 41 percent for the mixture and the corresponding percentage of fly ash that must be added to the sand to achieve this level is 47.8 percent. The percentage of fly ash predicted by this approach is in good agreement (especially with an $m = 0.4$) with the density curve produced by trial mixtures for the fly ash and river sand in Figure 49.

5.8.3 Calcium Oxide Component in ASTM Class C Fly Ash

A simple method for estimating the amount of CaO constituents present in fly mixtures would be a valuable tool for quality control in construction operations and, possibly, mixture design. The application of a procedure described in (10) was reviewed in this study. The method involves the measurement of the changes in temperature caused by the exothermic reactions that occur when 20 gm of fly ash/CaO is mixed with 75 ml of 15 percent HCl. The relationship between the percentage of CaO present and the change in temperature is linear. The relationship developed between the CaO content and temperature change produced the following:

$$CaO = 0.395\ (\Delta T) + 3.234$$

where

CaO = percent weight of total CaO

T = increase in temperature, °C

Using the heat evolution test, the CaO content of the fly ash used in this study was determined. The relationship, established by McKerall (10), between temperature rise (63° C) and CaO content was used to identify the CaO content of the Bayou Ash fly ash. The results showed a 28.12 percent CaO content, Figure 48. This agrees very well with the tests provided by Bayou Ash (27.5 and 26.24 percent) and those determined in this study (31.6 percent).

The heat evolution test was also applied to other mixtures, Table 22. A pure CaO was added to an Ottawa sand in quantities varying from 5 to 50 percent. Different percentages of the fly ash were also added to Ottawa sand. When graphed, there does appear to be a linear relationship between the percent additive (fly ash and CaO) and the heat generated in the test, though not the same curve, Figure 51.

The test may have potential in field control. It is a very simple test to perform and requires very little equipment, i.e., a Thermos jug, thermometer, stopper, and scales. It could be used in field quality control for checking the CaO levels in the Class C fly ash received and/or the quantity and mixing performance of the fly ash in the field.

5.8.4 Lime: Fly Ash Ratios

In lime stabilization of clays, the high alkalinity of the lime attacks the silicates of the soil, freeing the silica and alumina to react with the calcium to form cementitious, calcium silicate products. This occurs under conditions of high alkalinity and occurs within one hour (34). Because fly ash is a highly siliceous material and usually used in combination with lime, the amount of lime required to increase the pH of the lime-fly ash mixture from 11.6 to 12.5 and maintain that value for one hour was determined. The method proposed by Eades and Grim (34) was used, except fly ash was substituted for soil.

Additions of 1, 2, 3, 4, 5 and 6 percent (dry weight) of lime were added to the fly ash, Table 23. For the 1 percent and 2 percent lime additions, the pH readings were 12.3. For all the higher percentages, a pH of 12.4, was measured. Based on these results, a 3 percent lime addition is adequate to maintain a pH level similar to the lime alone. This is a very small lime to fly ash ratio (L:FA) of approximately 1:33. This concept, obviously, does not lend itself to selecting a lime to fly ash ratio. A better approach would be to include the soil in question, especially clays, with different ratios and quantities of lime-fly ash mixtures.

There are many lime-fly ash ratios in combination with various amounts of total lime-fly ash that when added to a soil can achieve the stabilization objectives of a particular project. The best selection will be that which satisfies the performance requirements and is the most economical. By varying the lime to fly ash ratio (L:FA) and holding the total percentage of lime-fly ash mixture constant, the effects of the L:FA ratio in compressive strength tests were observed. The lime to fly ash ratios of 1:2, 1:3, 1:4, and 1:5 were selected. These ratios were chosen because they are common ratios used in the field with success. The percentage of fly ash and/or the percentage of lime-fly ash added in fabricating the specimens was kept constant (approximately), Table 24. With the ratios selected and by holding the fly ash percentage to approximately 15 percent, the total lime-fly ash added ranged only between 18 to 22.5 percent, i.e., similar for all specimens. A graph of the resulting strengths occurring after a 1-day accelerated cured (50° C) and 7-day normal cure (23° F) is presented in Figures 52 and 53. Although there may appear to be some inconsistencies between the test results of different mixtures, there is an indication of greater strength corresponding to lime enriched ratios.

Lower rates have been used with success; however, three percent is the minimum amount of lime that has been suggested for use in the field due to mixing requirements. Upper limits for lime addition are generally based on cost and fall within the range of 5 to 10 percent.

6. CONCLUSIONS AND RECOMMENDATIONS

Class C fly ash has excellent properties that makes it a versatile construction material. It should be considered for general use in soil stabilization. It can contribute greatly in those situations where natural, high quality aggregates are unavailable.

If Class C is to be used effectively, its unique characteristics must be understood. Some of the attributes attributed to Class C fly ash can be misleading or may be detrimental if used improperly. A hit or miss end product will result by using it blindly.

6.1 Conclusions

6.1.1 Materials/Test Performance

A unique relationship exists between maximum dry density, optimum moisture, and the percentage of fly ash used with sands. For a given compaction effort, the largest dry density possible corresponds to the least amount of water, i.e., smallest optimum moisture content. Also, a relationship between strength development with an increase in max dry density has been demonstrated. Thus,in addition to greater mechanical stability, there is a corresponding reduction in the water required for compaction which reduces the water to cement/pozzolan ratio.

Strength curves produced for the different percentages of portland cement or Class C fly ash showed promise as a guide for predicting the percentages of fly ash required to produce comparable strengths with soil-cement. Such a tool could assist in the design of mixtures by reducing the volume of tests required and expediting the comparative cost analysis.

In reviewing the compaction method and specimen types, either the Proctor method or Texas method specimen sizes are acceptable. The failure fractures occurring in the larger (L/D) Texas specimens tested in compression were confined to the central portions of the specimens instead of the specimens' ends. Intuitively, this would seem to be an advantage. However, the strengths measured in the Texas specimen for this portion of the study, in general, were similar to those of the Proctor specimens. Ultimately, if the test is conducted properly and consistently, it is the authors' view that the critical concern here should be the strength criteria selected for the test that meets the design requirements.

The vacuum saturation procedure for the sand specimens in evaluating durability seems to be appropriate. This may not be the case for clay soils. Other tests and criteria such as the AASHTO Designation T 135 should be investigated for application to Louisiana's conditions.

A one-day cure at a temperature of 50° C was successful in predicting the strength potential for the fly ash-sand and lime-fly ash-bentonite clay mixtures. However, neither of these cure periods provided an adequate measure of the strength potential of the lime-fly ash-sand mixture. Where Class C fly ash is used as a lone stabilizing agent, a seven day cure is adequate.

The one-day accelerated cure (50° C) strength provides an acceptable prediction of the 28-day strength under normal cure conditions for the lime-fly ash-bentonite mixture. The similarity between the secondary phases produced at the end of these two curing periods and conditions was found to be in agreement with that of Ferrel (3). The one-day accelerated cure is not indicative of the strength potential for any curing period used with the lime-fly ash-sand mixture. The strength measured after a seven-day accelerated cure was approximately that occurring in normal temperature cures after 90 days. In the microanalysis, the secondary products produced after a seven-day accelerated cure for the lime-fly ash-sand were similar to those observed in the 90-day normal cure specimens.

The fly ash in this study definitely has a flash set. The set occurring in laboratory tests varied with different sand sizes. The final set time occurring with the fly ash in a coarse sand took place in 15 minutes. The fly ash set time in a fine, somewhat silty, sand was approximately 85 minutes. Results of strength tests after delayed compaction were significantly reduced with as little as a one hour delay.

6.1.2 Long-Term Tests

When fly ash is used alone with a sand, much of the strength is developed immediately. Almost 50 percent of the ultimate strength development occurs within seven days.

There is a distinct long-term advantage in using lime with the Class C fly ash in a sand or coarse aggregate. The strength development is slow for the first seven days. Thereafter, the gain in strength is rapid. After one year, the strength measured with the specimens of this mixture exceeded 1,000 psi and appears to continue to increase.

A decrease in the modulus of elasticity of the lime-fly ash-bentonite clay makes its long term performance questionable. The dissolution of the fly ash grains with longer cure periods similar to that observed by Ferrel (3) was also noted in this study.

6.1.3 Mix Design/Quality Control

In order to insure uniformity and quality, pre-qualifications are emphasized by many states and specifications. Spot checking at the construction site is also generally recommended.

Class C fly ash has several beneficial attributes for soil stabilization. These include its role as a fine filler material, its cementitious characteristics, and its pozzolanic properties. Its performance as a stabilizing agent is improved with increased density. The percentage of fly ash required for maximum density in sands can be estimated using the Talbot/Fuller maximum density relationship ($m = 0.4$ or 0.5).

The high percentage of the calcium oxide constituents is responsible for its cementicious properties. In heat evolution tests the CaO content present in fly ash (or fly ash in the mixed soil) is linear with the rise in temperature. The procedure has potential in providing assistance in construction quality control.

6.2 Recommendations

Suggestions for further research developed as a result of this study include:

1. The strength potential of different soil types stabilized with Class C fly ash and other pozzolan materials should be identified and documented. This should include the historical development of the strength and elastic properties with the corresponding cure conditions.

2. Strength and elastic criteria for pozzolanic base materials and other highway uses utilizing Class C fly ash and other pozzolan materials should be established. This should be done along with the identification of structural layer coefficients, a_2, for use in pavement design. A study to evaluate the performance and establish the structural layer coefficient of different aggregate soils stabilized as a Class C fly ash-pozzolan base is recommended as part of the Accelerated Load Facility (ALF) program.

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APPENDIX A

TABLES

Table 1
Chemical And Physical Properties Of Bayou Fly Ash

Parameter	**Bayou Ash Test**		**UNO Test Results**	**ASTM C618 Spec.**
Chemical Analyses	4/ /91	5/ /91	6/ /93	
Silicon Dioxide, SiO_2, %	32.3	36.81	42.0	---
Alum. Dioxide, Al_2O_3, %	21.7	20.00	6.6	---
Iron Oxide, Fe_2O_3, %	6.2	7.02	7.5	---
Sum: $SiO_2+Al_2O_3+Fe_2O_3$, %	60.2	63.83	56.1	50 min
Calcium Oxide, CaO, %	27.5	26.24	31.6	---
Magnesium Oxide, MgO, %	6.8	6.06	2.1	---
Alkali, Na_2O, K_2O, %	2.06	1.82	1.8	---
Sulfur Trioxide, SO_3, %	1.9	2.04	3.3	5.0 max
Moisture Content, %	0.1	0.03		3.0 max
Loss on Ignition, %	0.28	0.02	2.9	6.0 max
Physical Analyses				
Retained on No.325 Sieve	6.9	11.60	8.6	34 max
Pozzolanic Act. Index:				
Portland Cem., % Cont. (@ 7 days)	120	---	---	75 min
Portland Cem., % Cont. (@ 28 days)	126	---	---	75 min
Lime @ 7 days, % Cont.	---	---	---	NA
Water Req., % Control	90	---	---	105 max
Autoclav Expansion	+0.04	---	---	0.8 max
Specific Gravity	2.76	2.75	2.66	---

Table 2
Grain Size Feature Analysis

BAYOU FLY ASH GRAIN SIZE DISTRIBUTION

Maximum Chord, d, mm	**% d**	**% Finer**
0.150 (sieve #100)		
0.130	0.7	99.3
0.120	0.7	98.6
0.110	0.7	97.9
0.106 (sieve #140)		
0.090	2.2	95.7
0.080	1.4	94.3
0.075 (sieve #200)		
0.070	4.3	90.0
0.050	3.6	86.4
0.045 (sieve #325)		
0.040	12.2	74.2
0.030	23.7	50.5
0.010	50.4	0.1

Feature Area $(mm)^2$	**Equiv. Sphere d, mm**	**% Size**	**% Finer**
0.020	0.160	0.7	99.3
0.012	0.124	0.7	98.6
0.006	0.087	0.7	97.9
No. 200 sieve	0.075		
0.004	0.071	7.2	90.7
0.002	0.050	90.6	0.1
No. 325 sieve	0.045		

Table 3
Sand Properties

Parameter	River Sand	Sand #2
% Coarse Sand (Ret. #4)	1.4	26.2
% Fine Sand (Ret. #200)	93.0	73.1
% Silt	6.6	0.7
% Clay & Colloids	4.9	
Liquid Limit	NP	NP
Plasticity Index	NP	NP
Max. Dry Wt. Density (pcf)	94.4	
Optimum Moist. Cont., %	17.5	
Specific Gravity		
pH	10.5	
Soil Classification:		
AASHTO	A-3	A-3
Unified	SP-SM	SP

Table 4
Standard Proctor Dry Density And Optimum Moisture Content For Portland Cement And A-3 River Sand

MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT
STANDARD PROCTOR

% PORTLAND CEMENT ADDED TO A-3 SAND	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT, %
2	98.35	15.69
4	100.33	15.13
6	103.23	14.96
8	104.02	15.47
10	104.70	12.27
12	106.26	12.17

Table 5
Standard and Modified Proctor Dry Density and Optimum Moisture Content for A-3 River Sand with Fly Ash

THEORETICAL MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT STANDARD PROCTOR		
PERCENT FLY ASH Added to A-3 Sand	Max. Dry Density (pcf)	Optimum Moisture Content, %
0	94.4	17.5
10	105.0	13.8
12	106.6	12.3
14	108.1	11.8
16	109.1	10.1
18	110.3	10.7
20	112.8	10.0
22	113.9	8.6
24	115.2	9.2
25	116.5	9.1
26	117.8	9.1
30	120.9	8.4
35	122.6	8.0
40	124.4	7.9
42	124.7	8.4
100	112.17	10.3
MODIFIED PROCTOR		
25	121.09	8.5
30	124.80	7.4
35	123.54	7.0

Table 6
Optimum Moisture - Max Density Values for Texas Compaction Method

MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT
TEXAS METHOD TEX-113-E

Mixture Proportions	Max Dry Density (pcf)	Optimum Moisture Content, %
25% FA + A-3 Sand	118.2	9.4
3% L + 15% FA + A-3 Sand	111.0	11.8
5% L + 20% FA + Bentonite	67.5	52.7

L, Lime; FA, Fly Ash

Table 7
Lime, Fly Ash and Sand Compaction

MAXIMUM DENSITY AND OPTIMUM MOISTURE CONTENT
STANDARD PROCTOR TEST

% Lime + % Fly Ash + A-3 River Sand	Maximum Dry Density (pcf)	Optimum Moisture Content, %
3% L + 6% FA	101.80	12.81
3% L + 9% FA	104.31	12.75
3% L + 12% FA	106.85	12.48
3% L + 15% FA	109.02	12.12
4% L + 16% FA	111.25	11.25
5% L + 15% FA	111.25	11.25
7.5% L + 15% FA	112.70	10.63
MODIFIED PROCTOR TEST		
2% L + 4% FA	105.74	13.6

Table 8
Standard Proctor Density and Optimum Moisture Content for Bentonite Clay

MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT BENTONITE CLAY and STANDARD PROCTOR COMPACTION		
Materials	Density (pcf)	Moisture Content, %
Bentonite Clay	72.8	31.27
5% Lime + 20% Fly Ash + Bentonite	74.5	41.00

Table 9
Strength / Density Variation With Percentage of Stabilizing Agent: Portland Cement & Fly Ash

COMPRESSIVE STRENGTH OF STANDARD PROCTOR SAMPLES
AFTER 7 DAYS OF CURING AT 73°F (23°C)
A-3 SAND + CEMENT

percent CEMENT	2	4	6	8	10	12
maximum density (pcf)	98.35	100.83	103.23	104.02	104.70	106.26
optimum moisture content (%)	15.69	15.13	14.96	15.47	12.27	12.17
U.C. strength (psi)	12.0	36.3	97.6	174.0	371.0	548.0

COMPRESSIVE STRENGTH OF STANDARD PROCTOR SAMPLES
AFTER 7 DAYS OF CURING AT 73°F (23°C)
A-3 SAND + FLYASH

percent FLYASH	10	20	25	30	40
maximum density (pcf)	105.0	112.80	116.5	120.90	124.40
optimum moisture content (%)	13.80	10.0	9.10	8.40	7.90
U.C. strength (psi)	14.6	58.0	311.0	203.0	452.0

Table 10
Unconfined Compressive Strength of Standard Proctor Specimen

STANDARD PROCTOR SPECIMEN			
Materials Mixture Proportions	UC Strength 7-Day Cure Normal* psi	UC Strength 1-Day Cure Accelerated** psi	UC Strength Vac. Sat. ASTM C519 psi
Portland Cement:			
2%	12.0		
4%	36.3		
6%	97.6		
8%	174.0		
10%	371.0		
12%	548.0		
+ River Sand			
25% Fly Ash + River Sand	311.0	231.0	279.0
3% Lime + 15% Fly Ash + River Sand	129.0	66.2	99.2
5% Lime + 20% Fly Ash + Bentonite	110.3	97.0	147.0

* Normal Cure: 23° C (73° F)

** Accelerated Cure: 50° C (122° F)

Table 11
Unconfined Compressive Strength of TEX Method 113-E Specimen with Different Cure Conditions

TEXAS TEX METHOD 113-E SPECIMEN				
Materials Mixture	Curing* Conditions	UC Strength 1-Day Cure	UC Strength 7-Day Cure	Strength 28-Day Cure
25% FA + A-3 River Sand	Normal	109.1	215.3	114.0
	Accel.	168.1	96.5	560.0
3% Lime + 15% FA + A-3 River Sand	Normal		122.6	
	Accel.	66.5		
5% Lime + 20% FA + A-3 River Sand	Normal		125.2	
	Accel.	143.1		

* Normal Cure: 23° C (73° F)
Accelerated Cure: 50° C (122° F)

Table 12
Direct Shear Tests

SUMMARY OF DIRECT SHEAR TESTS			
MATERIAL	State/ curing	Cohesion (psi)	Phi (degree)
A-3 Sand	dry	0	28.0
Flyash	dry	0	25.0
A-3 S + 25 FA	7 day; normal	18.0	21.0
A-3 S + 3 % L + 15 % FA	7 day; normal	4.0	33.0
S: sand; L: lime; FA: flyash			

Table 13
Time and Temperature Effects on Strength Gain for Fly Ash and A-3 River Sand

CURING CONDITIONS: TIME AND TEMPERATURE					
25 % FLY ASH + A-3 RIVER SAND					
Cure Time Factor at 23° C (73° F)					
Cure Time (days)	7	28	365	7/365	28/365
Tested Strength (psi)	140	150	358	0.39	0.42
Cure Temperature Factor: 50° C (122° F)					
Cure Time (days)	accel 1A	accel. 7A	1A/28	1A/365	7A/365
Tested Strength (psi)	150.2	506	1.00	0.42	1.41

Table 14
Time and Temperature Effects on Strength Gain for Lime-Fly Ash and A-3 River Sand

CURING CONDITIONS: TIME AND TEMPERATURE					
3 % LIME + 15 % FLY ASH + A-3 RIVER SAND					
Cure Factor at 23° C (73° F)					
Cure Time (days)	7	28	365	7/365	28/365
Tested Strength (psi)	48.7	191	1,013	0.05	0.19
Cure Temperature Factor: 50° C (122° F)					
Cure Time (days)	accel. 1A	accel. 7A	1A/28	1A/365	7A/365
Tested Strength (psi)	45	457	0.24	0.04	0.45

Table 15
Time and Temperature Effects on Lime-Fly Ash and Bentonite Clay

CURING CONDITIONS: TIME AND TEMPERATURE					
5 % LIME + 20 % FLY ASH + BENTONITE CLAY					
Cure Time Factor at 23° C (73° F)					
Cure Time (days)	7	28	365	7/365	28/365
Tested Strength (psi)	117	189.3	245	0.48	0.77
Cure Temperature Factor: 50° C (122° F)					
Cure Time (days)	**accel. 1A**	**accel. 7A**	**1A/28**	**1A/365**	**7A/365**
Tested Strength (psi)	**163**	**124**	**0.86**	**0.67**	**0.51**

Table 16
The Variation of Strength and Elasticity With Time, Temperature, and Cure Conditions

UNCONFINED COMPRESSION STRENGTH TEST

Material: A3 River Sand + 25% Fly Ash
Sample: Harvard Miniature
Cure: Normal 23° C (73° F)

Cure Time (days)	1	7	28	90	180	365
Max. Dens. (pcf)	111.1	110.0	109.6	109.6	--	--
Moist. Cont. (%)	8.2	7.8	7.7	7.7	--	--
Failure Str.(psi)	85	140	150	156	320	358
Failure Strain(%)	1.8	1.02	1.21	1.4	1.56	1.9
Elast. Mod. (psi)	6,252	15,543	12,786	12,046	20,517	19,093
70% Str. (psi)	60	97.6	105	109	223.8	251
Strain (%)	1	0.6	0.75	0.63	1.1	1.35
E_{70} (psi)	11,953	20,200	15,044	18,343	20,492	18,745

Cure: Accelerated 50° C (122° F)

Cure Time (days)	1	7	28	90		
Max. Dens. (pcf)	112	111	112	109		
Mois. Cont. (%)	7.6	6.0	7.32	11.0	12.32	16
Failure Str.(psi)	150.2	506	642	927		
Failure Strain(%)	1.52	2.32	3	2		
Elas. Mod. (psi)	10,189	22,520	22,204	49,707		
70% Str. (psi)	105.1	354	450	649		
Strain (%)	0.75	1.72	2.2	1.32		
E_{70} (psi)	15,651	21,913	21,023	52,959		

Table 17

Variation of Strength and Elasticity With Time And Temperature
Lime + Fly Ash + Sand

UNCONFINED COMPRESSION STRENGTH TEST

Material: 3% Lime + 15% Fly Ash + A3 Sand
Sample: Harvard Miniature

CURING: NORMAL 23°C (73°F)

Cure Time (days)	1	7	28	90	180	365
Max. Dens. (pcf)	107	104	109	107	--	--
Moist. Cont. (%)	11	10.5	11	10	--	--
Failure Str.(psi)	65	48.7	191	346	649	1,013
Failure Strain(%)	1.52	1.05	1.37	1.55	2.32	3.0
Elast. Mod. (psi)	4,965	4,630	13,957	2,297	27,849	34,116
70% Str. (psi)	45.5	34.1	133.5	242	454.3	709
Strain (%)	0.68	0.66	0.8	0.95	1.7	2.2
E_{70} (psi)	8,252	5,265	16,735	25,556	26,603	32,321

CURING: ACCELERATED 50°C (122°F)

Cure Time (days)	1	7	28	90		
Max. Dens. (pcf)	108	109	108	109		
Mois. Cont. (%)	11	9.7	9.62	--		
Failure Str.(psi)	45	457	745	866		
Failure Strain(%)	0.85	2.1	2.44	2.32		
Elas. Mod. (psi)	5,548	21,333	30,597	38,472		
70% Str. (psi)	31.5	320	521.15	606		
Strain (%)	0.43	1.47	1.6	1.51		
E_{70} (psi)	11,168	21,794	32,729	40,495		

Table 18
The Variation of Strength and Elasticity With
Time, Temperature, and Cure Conditions

UNCONFINED COMPRESSION STRENGTH TEST

Material: 5% Lime + 20% Fly Ash + Bentonite Clay
Sample: Harvard Miniature

Curing: Normal 23°C (73°F)

Cure Time (days)	1	7	28	90	180	365
Max. Dens. (pcf)	68	67.4	67	65.8	--	--
Moist. Cont. (%)	46	45	44.3	43.5	--	--
Failure Str. (psi)	95.3	117	189.3	192	181	245
Failure Strain (%)	1.93	1.64	2.05	1.93	1.5	2.2
Elast. Mod. (psi)	4,983	7,222	11,089	10,597	15,707	11,374
70% Str. (psi)	66.73	82	132.5	134.4	126.5	172
Strain (%)	0.93	0.77	0.82	0.67	1.35	1.63
E_{70} (psi)	7,725	11,265	16,328	22,929	11,190	10,605

Curing: Accelerated 50°C (122°F)

Cure Time (days)	1	7	28	90		
Max. Dens. (pcf)	66	65.5	67.4	65.2		
Mois. Cont. (%)	43.4	46.1	43.2	49		
Failure Str. (psi)	163	124	205.4	171		
Failure Strain (%)	1.45	1.53	1.65	1.21		
Elas. Mod. (psi)	11,647	8,475	12,776	13,928		
70% Str. (psi)	114	87	143.5	119.4		
Strain (%)	0.74	0.9	0.66	0.71		
E_{70} (psi)	16,276	10,052	23,020	17,224		

Table 19
Fly Ash Time of Setting In Fine and Coarse Sands With and Without Gypsum

TIME OF SET WITH VICAT NEEDLE APPARATUS - ASTM C 807

Materials	25% FA + A-3 River Sand		25% FA/w 5% Gyp. + A-3 River Sand			25% FA + Coarse Sand	25% FA 5% Gyp. + CRS Sand	25% FA/w 20% Gyp. + CRS Sand
Moist Cont.	9%	12%	15%	12%	12%	7%	7%	7%
Time: min.	Pen.: (mm)							
0		43.0	43.0	43.0	43.0	38.0	40	40.5
5						5.0		17.0
10						2.0		
15						0		17.0
20								4.0
30	40.5	43.0	43.0	43.0	43.0		0	1.0
45	30.0	10.0	43.0	43.0	42.0			0
60		7.0	43.0	20.0	10.0			
65	35.0							
75		3.5	13.0	13.0	4.0			
80	6.0							
90		2.0	2.0	9.0	4.0			
95	6.0							
105			1.0	5.0	2.0			
110	3.0	1.0						
120		1.5	0.	4.0	2.0			
125	5.0							
135		1.0		4.0	2.0			
140	4.0							
150		1.0		4.0	2.0			
155	3.5							
165		0.5		3.0	1.0			
170	3.0							
180		0.5		3.0	2.0			
195		0.5		2.0	2.0			

Table 20
Time of Setting Of Fly Ash Used With Lime, With and Without Gypsum

TIME OF SET WITH VICAT NEEDLE APPARATUS - ASTM C 807			
Materials	3% L + 15% FA + A-3 River Sand	3% L + 15% FA + A-3 River Sand	3% L + 15%FA/w (5% Gyp. + FA) + A-3 River Sand
Moist Cont.,%	12%	12%	12%
Time (min.):			
0		43.0	43.0
30	37.5	3.0	14.0
45	22.0	1.0	10.0
60	13.5	1.0	13.0
75	16.0	1.0	8.0
90	0.5		8.0
105	0.5		7.0
120			7.0
135			6.0
150			5.0
165			4.0
180			3.0
195			4.0

Table 21
Talbot/Fuller Theoretical Maximum Density Curves

MAXIMUM DENSITY CURVE S(%) = $(D/D_{max})^m$				
	Percent Finer			
	d_{max} = 0.425 mm		d_{max} = 0.85 mm	
d, mm	(m = 0.4)	m = 0.5	m = 0.4	m = 0.5
0.425			76	71
0.250	81	77	61	54
0.180	71	65	54	46
0.125	61	54	46	38
0.075	50	42	38	30
0.045	41	33	31	23
0.005	16	11	13	8

Table 22
Calcium Oxide Heat Evolution Test

Material Mixtures + Sand	CaO %	Fly Ash %	Portland Cement, %	ΔT °C
CaO	5			7
	10			13
	15			18
	20			26
	25			31
	35			46
	50			64
Fly Ash		15		15
		30		25
		50		34
		100		63
Portland Cement			5	7.5
			8	14
			10	18
			15	26
			30	32
			50	48
CaO + FA	5	5		11
	10	10		23
	15	15		32
	20	20		46
	25	25		53
CaO + PC	5		5	13
	10		5	22
	15		5	32
	20		5	37
	25		5	47

Table 23
pH Values for Lime, Fly Ash, Soil Combinations

LIME L, %	Fly Ash FA, %	L:FA Ratio	LFA Total, %	Sand %	pH* @ 1 hr
100	-	-	-	-	12.5
-	100	-	-	-	11.6
1	99		100	-	12.3
2	98		100	-	12.3
3	97		100	-	12.4
-	-		-	100	10.5
3	15	1:5	18	82	12.4
4	16	1:4	20	80	12.4
5	15	1:3	20	80	12.4
7.5	15	1:2	22.5	77.5	12.4

*Eades and Grim procedures, 1963

Table 24
Lime:Fly Ash Ratios and Total Activator Present

<table>
<tr><th colspan="5">L:FA RATIO</th></tr>
<tr><td colspan="5">1 DAY ACCELERATED CURE, 50°C</td></tr>
<tr><th>L:FA Ratio</th><th>Lime %</th><th>Fly Ash %</th><th>Total LFA %</th><th>Strength psi</th></tr>
<tr><td>1:2</td><td>7.5</td><td>15</td><td>22.5</td><td>221.4</td></tr>
<tr><td>1:3</td><td>5</td><td>15</td><td>20</td><td>62.1</td></tr>
<tr><td>1:4</td><td>4</td><td>16</td><td>20</td><td>93.4</td></tr>
<tr><td>1:5</td><td>3</td><td>15</td><td>18</td><td>66.2</td></tr>
<tr><td colspan="5">7 DAY NORMAL CURE, 23° C</td></tr>
<tr><td>1:2</td><td>7.5</td><td>15</td><td>22.5</td><td>224.3</td></tr>
<tr><td>1:3</td><td>5</td><td>15</td><td>20</td><td>127.6</td></tr>
<tr><td>1:4</td><td>4</td><td>16</td><td>20</td><td>111.8</td></tr>
<tr><td>1:5</td><td>3</td><td>15</td><td>18</td><td>129.0</td></tr>
</table>

APPENDIX B

FIGURES

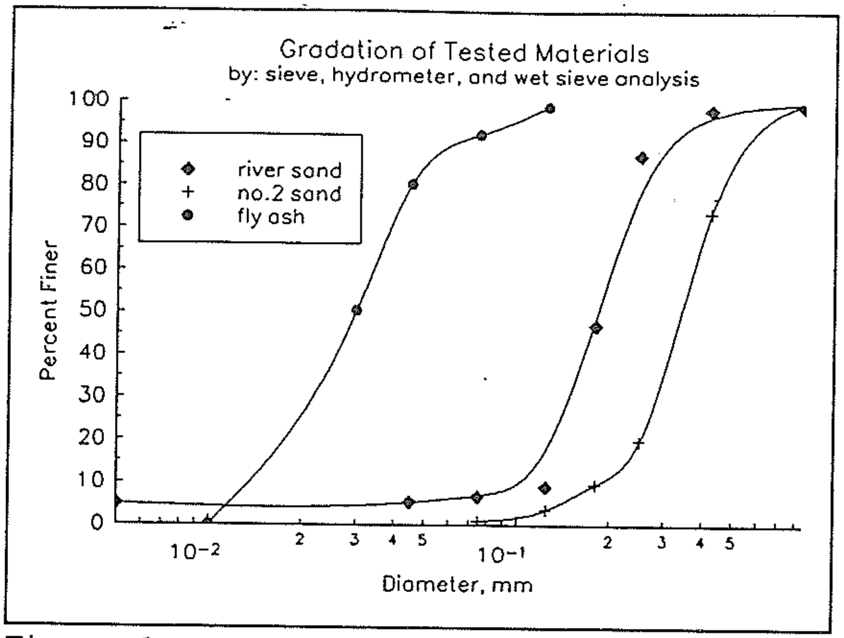


Figure 1 Grain Size Analyses for Class C Fly Ash and Sands

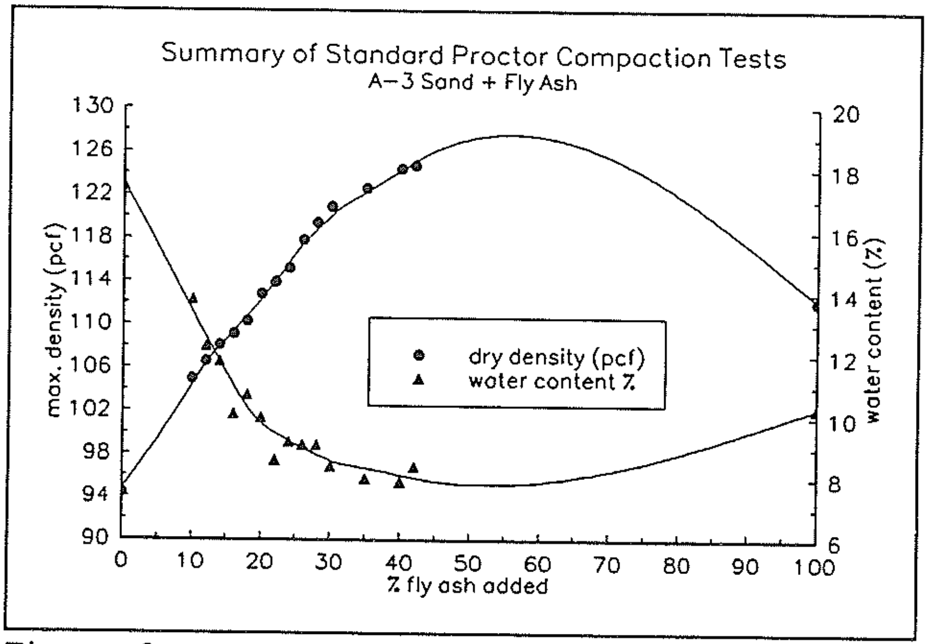


Figure 2 Percent Fly Ash vs Density and Moisture Content

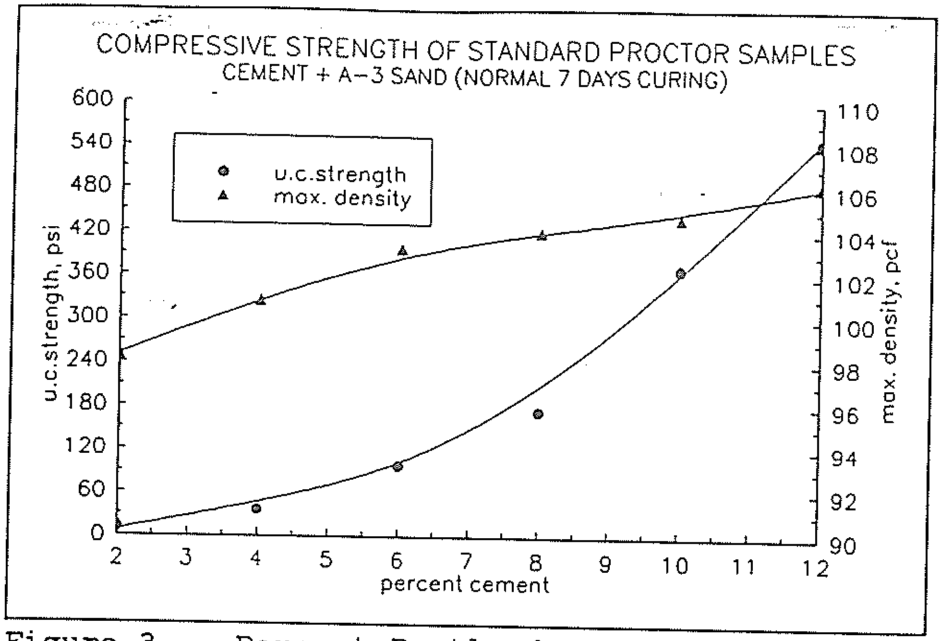


Figure 3 Percent Portland Cement vs Strength and Density

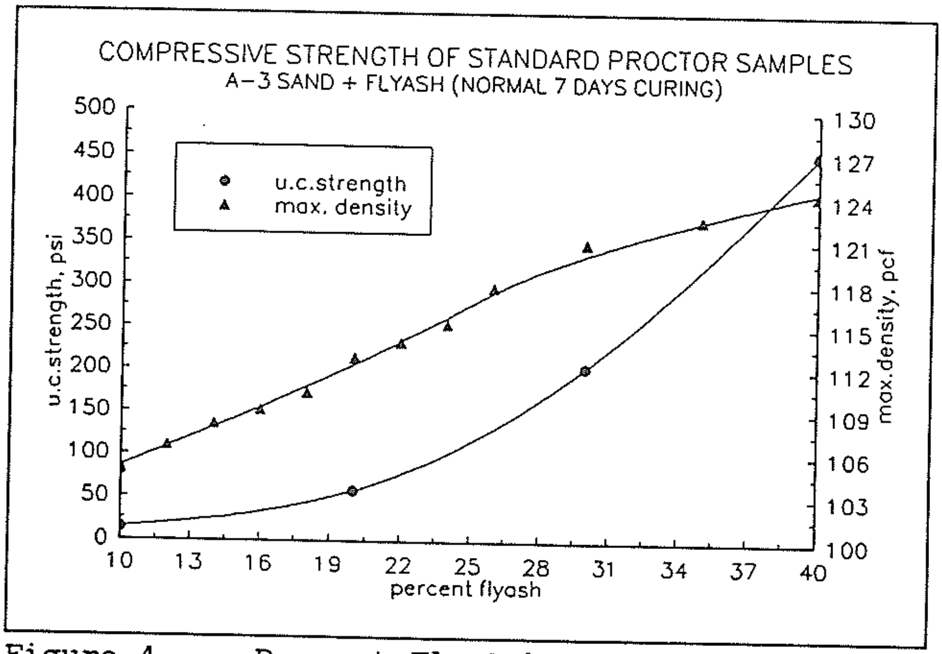


Figure 4 Percent Fly Ash vs Strength and Density

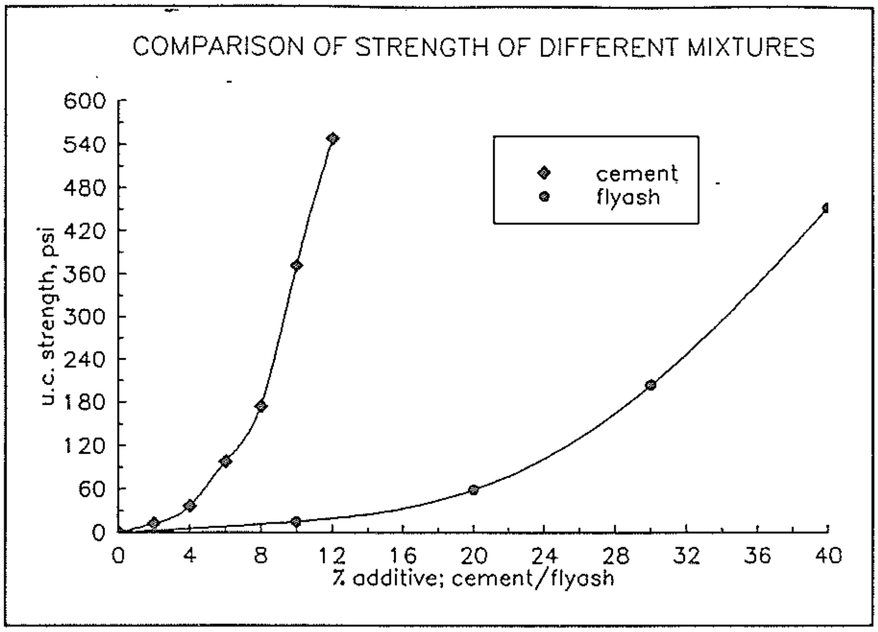


Figure 5 Percent Cement/class C Fly Ash vs Strength

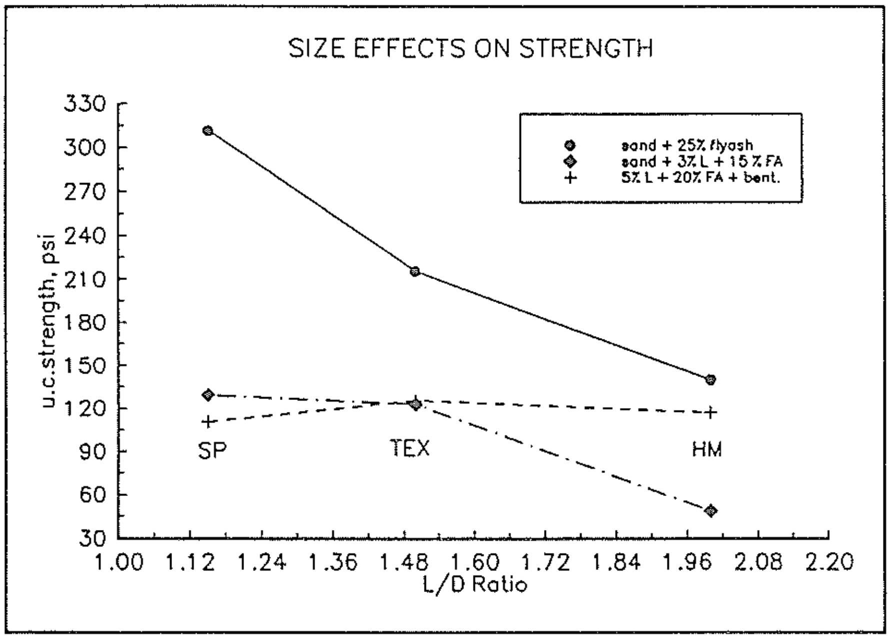


Figure 6 Effect of Specimen Geometry, L/D

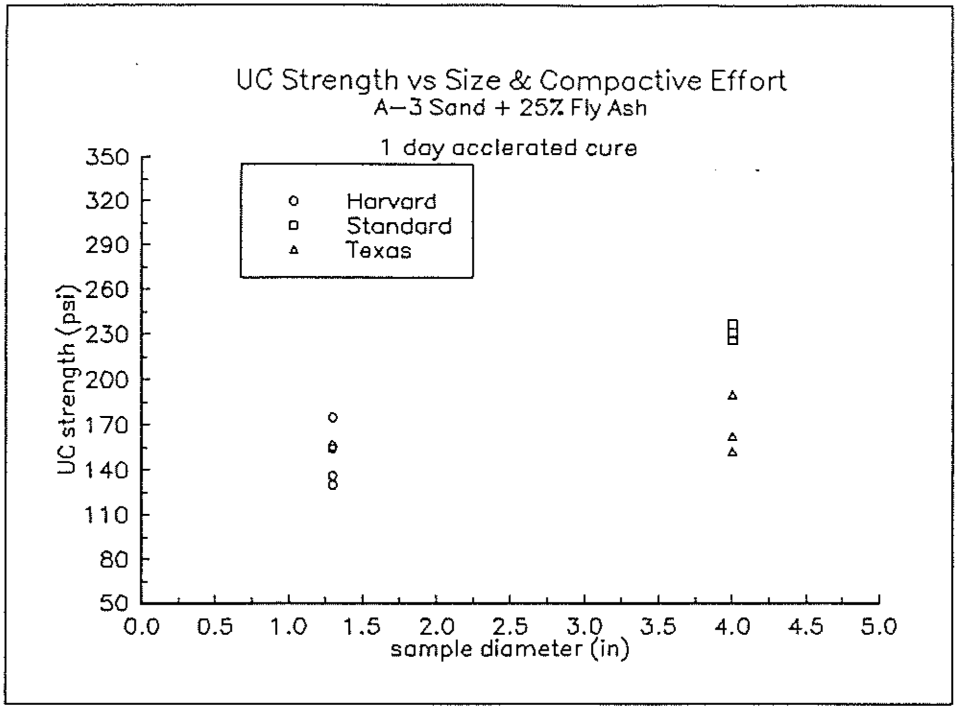


Figure 7 Comparative Strength Performance of Test Specimen in Unconfined Compression: 1-Day Accelerated Cure

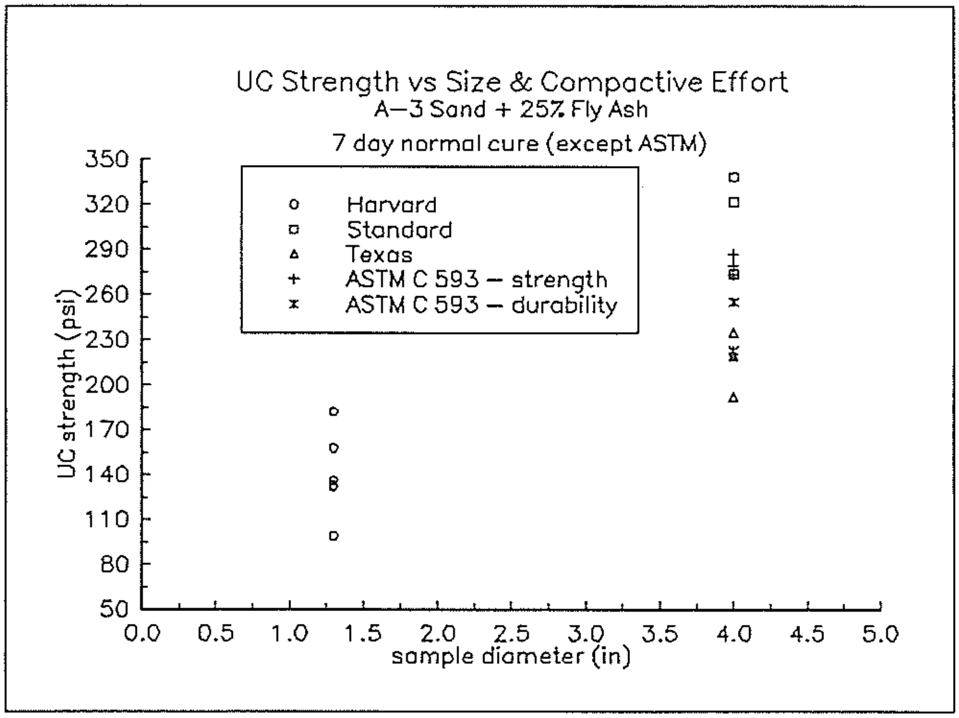


Figure 8 Comparative Strength Performance of Test Specimen in Compressive Strength 7-Day Normal Cure

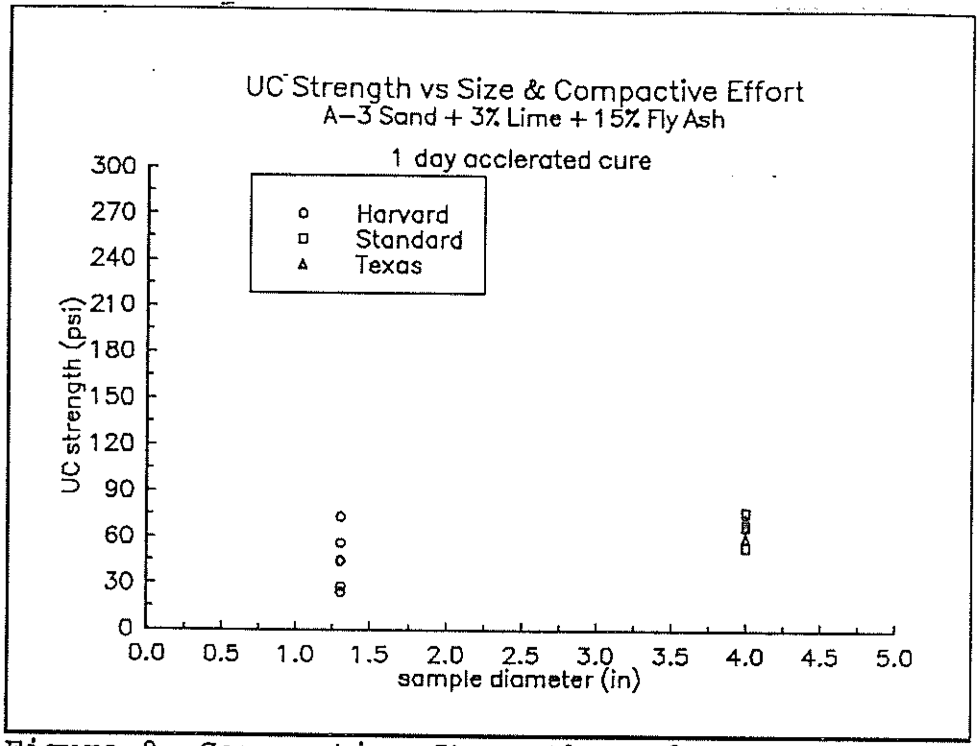


Figure 9 Comparative Strength Performance of Lime-Fly Ash Specimen: 1-Day Accelerated Cure

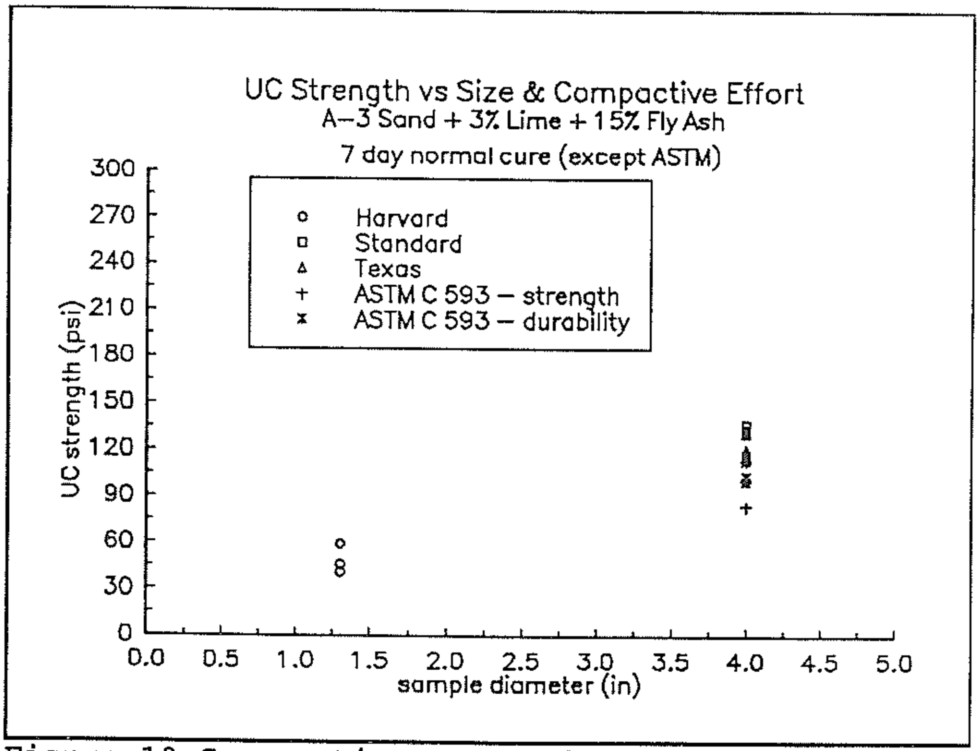


Figure 10 Comparative Strength Performance of Lime-Fly Ash Specimen: 7-Day Normal Cure

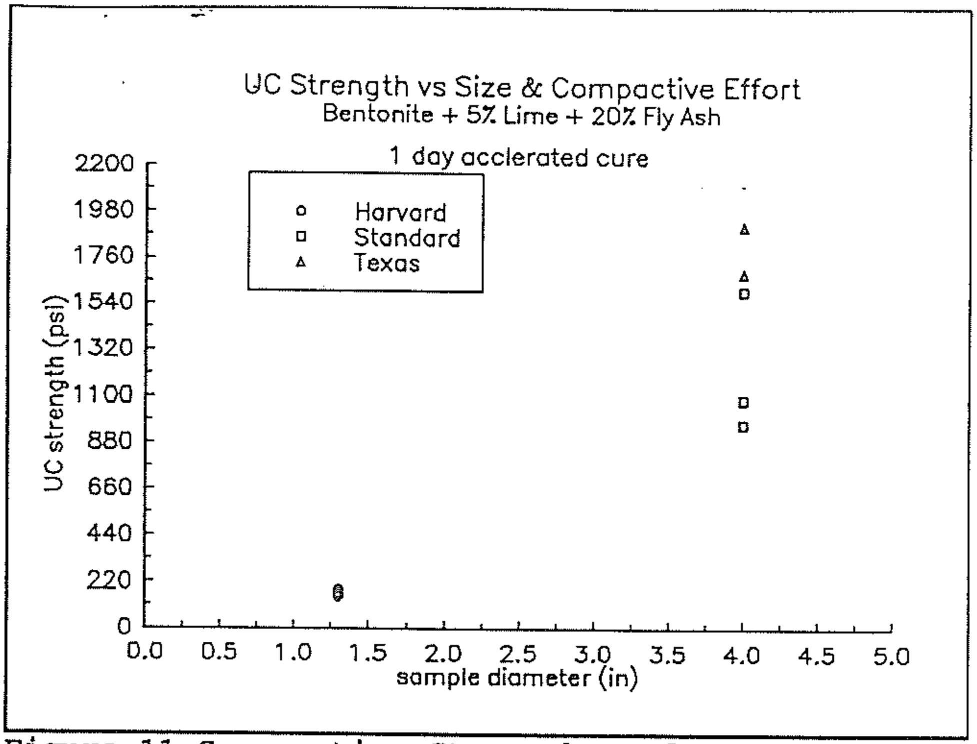


Figure 11 Comparative Strength Performance of Lime-Fly Ash-Bentonite Specimen Types: 1-Day Accelerated Cure

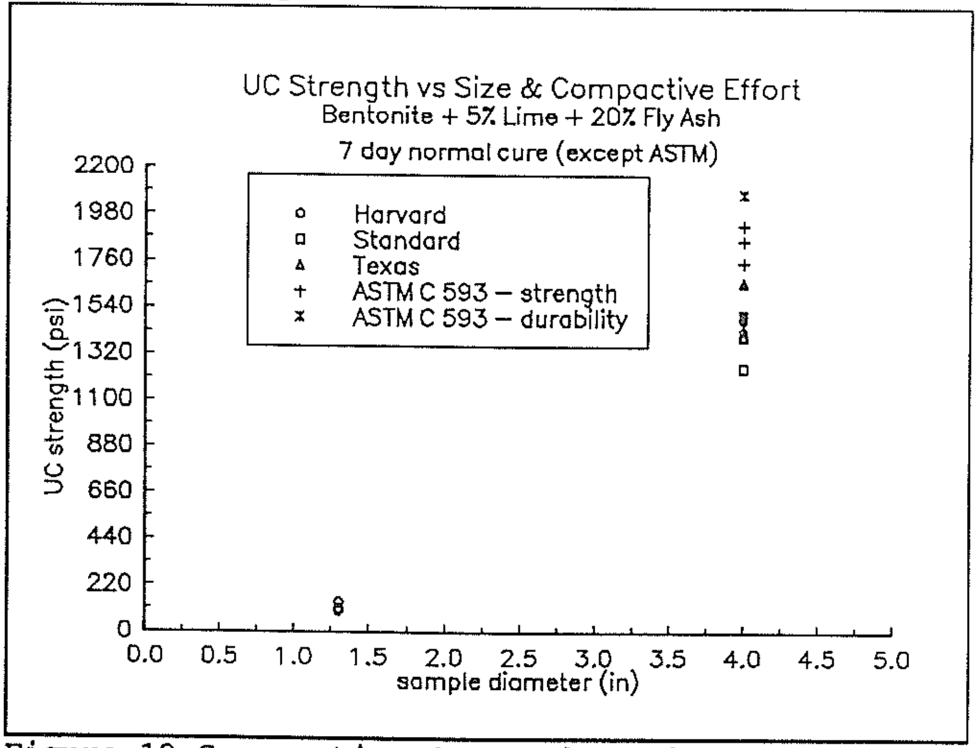


Figure 12 Comparative Strength Performance of Lime-Fly Ash-Bentonite Specimen Types: 7-Day Normal Cure

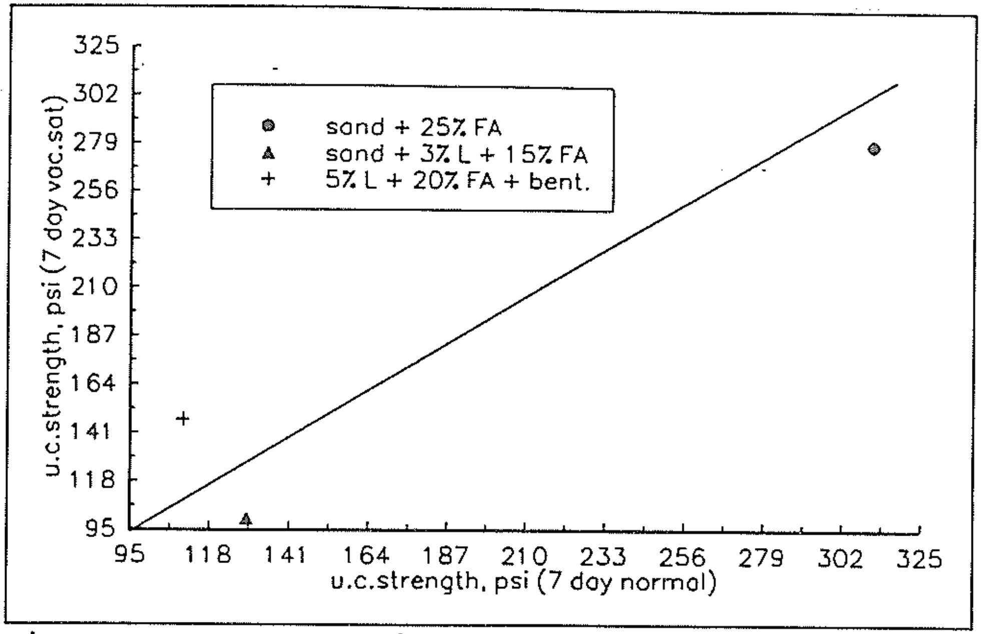


Figure 13 Comparison of Strength After Vacuum Saturation

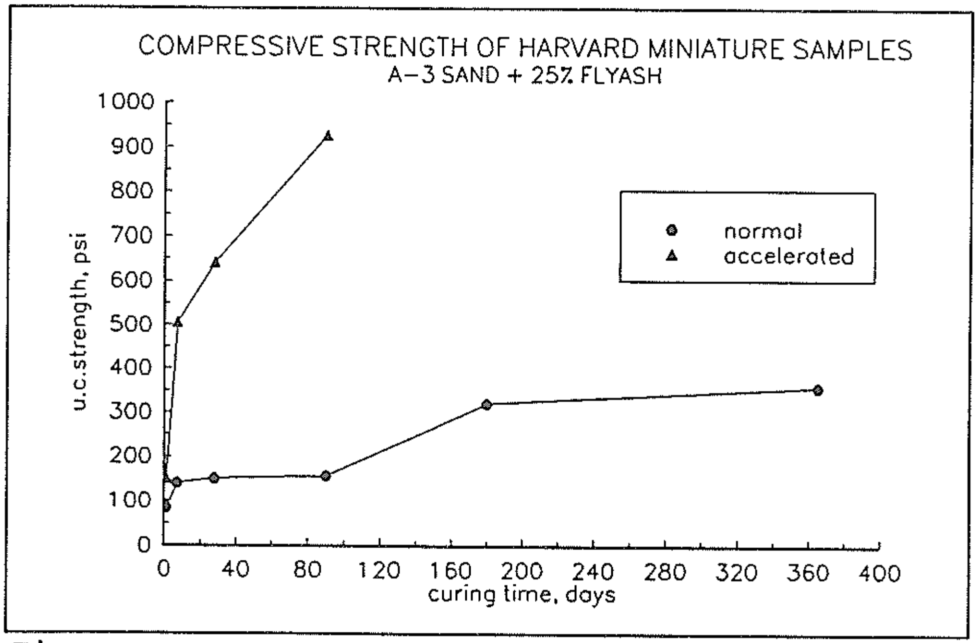


Figure 14 Long-Term Strength - Fly Ash + Sand

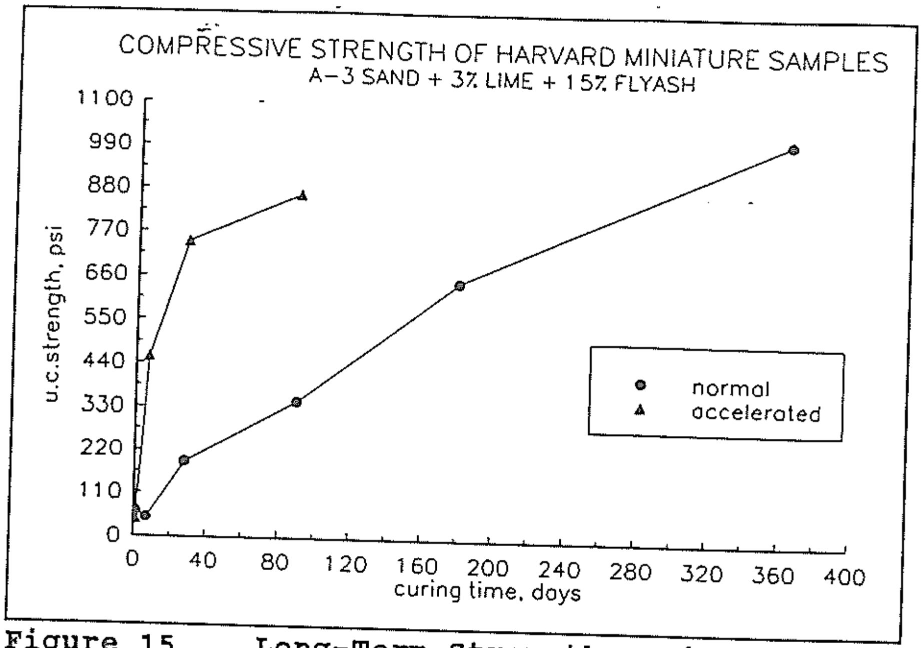


Figure 15 Long-Term Strength - Lime + Fly Ash + Sand

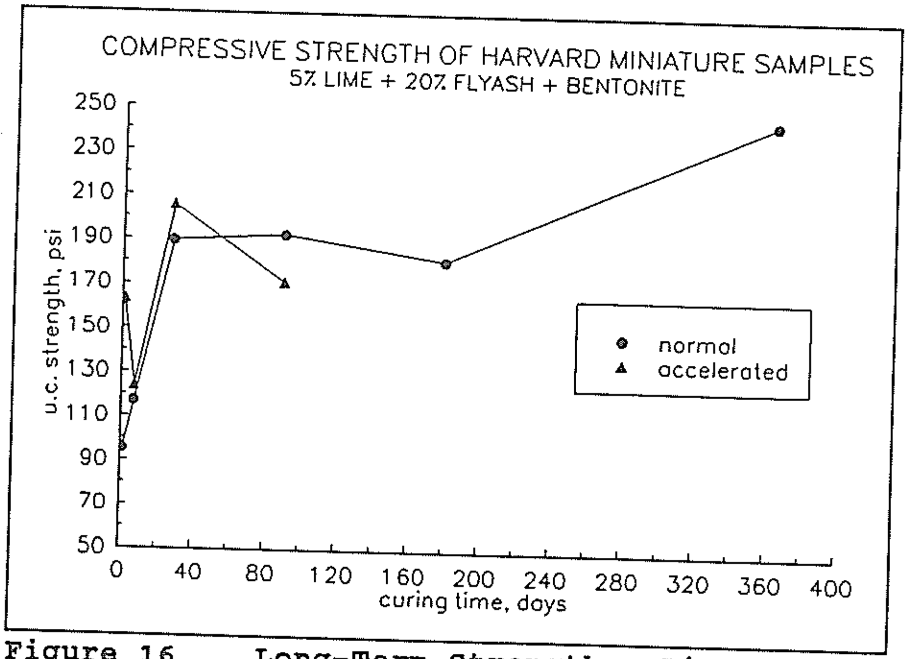


Figure 16 Long-Term Strength - Lime + Fly Ash + Bentonite

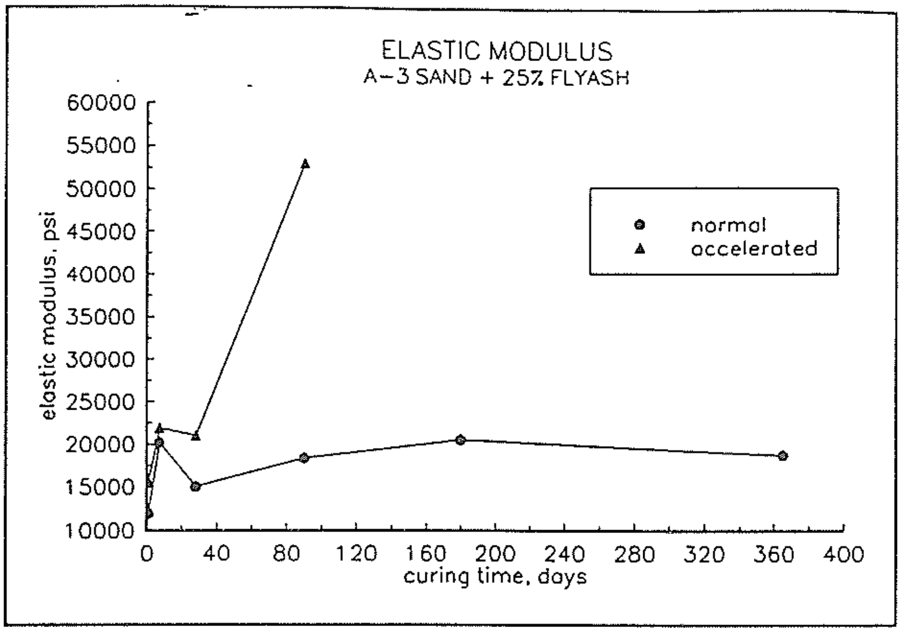


FIGURE 17 Long Term Development of Elastic Modulus - Fly Ash + Sand

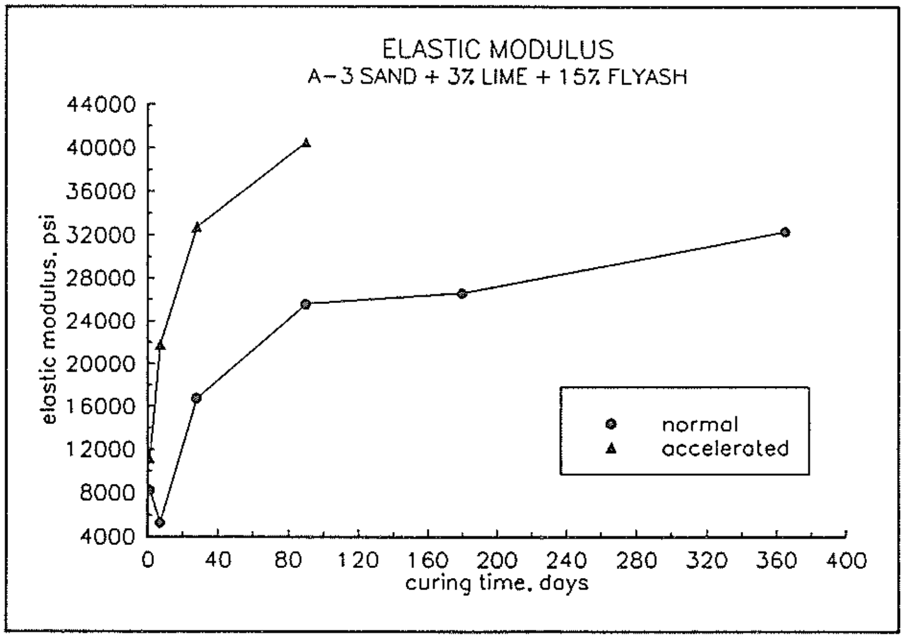


FIGURE 18 Long Term Development of Elastic Modulus -Lime + Fly Ash + Sand

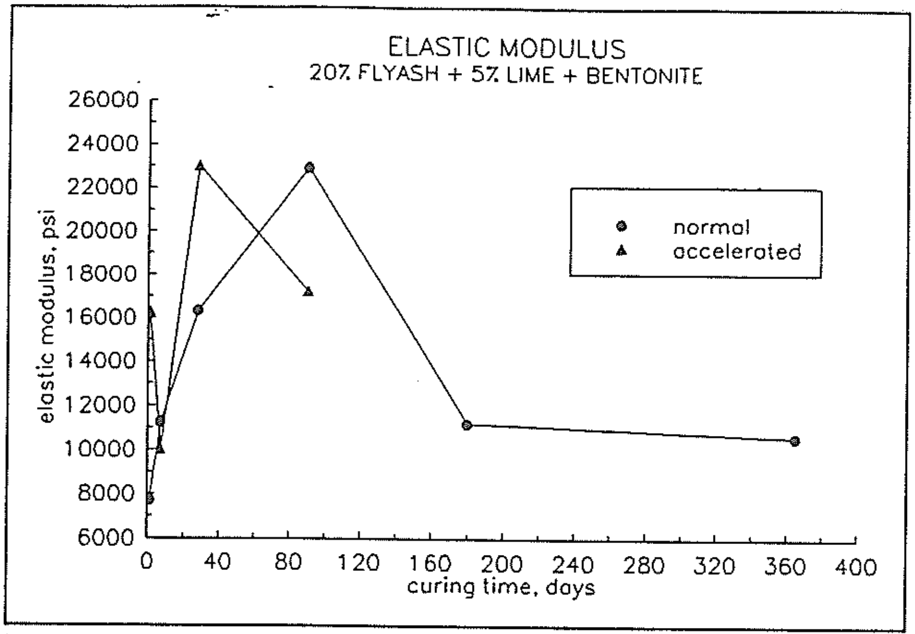


FIGURE 19 Long Term Development of Elastic Modulus - Lime + Fly Ash + Bentonite Clay

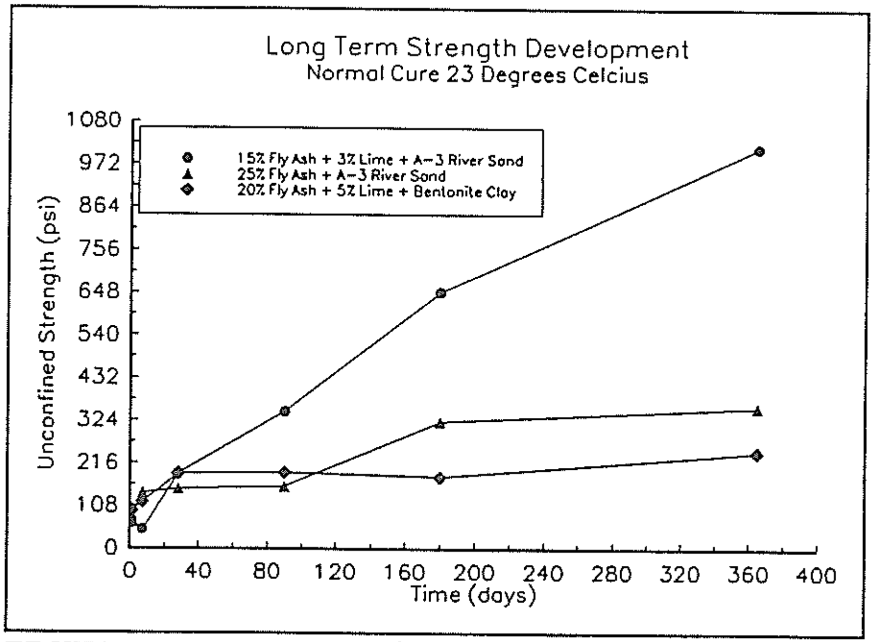


FIGURE 20 Comparison of Long Term Strength Development in All Specimen

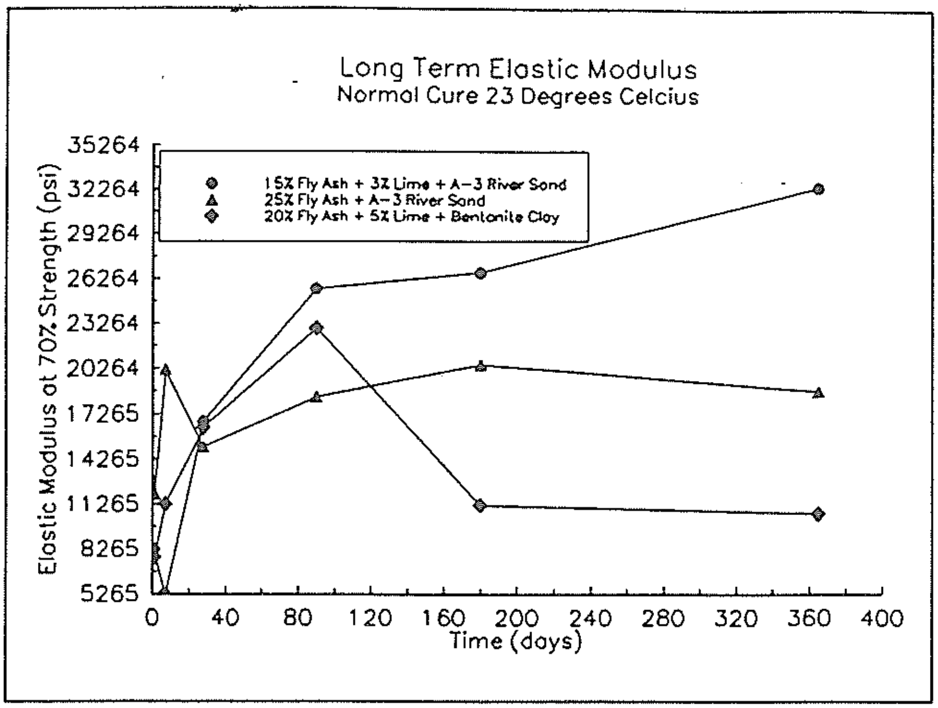
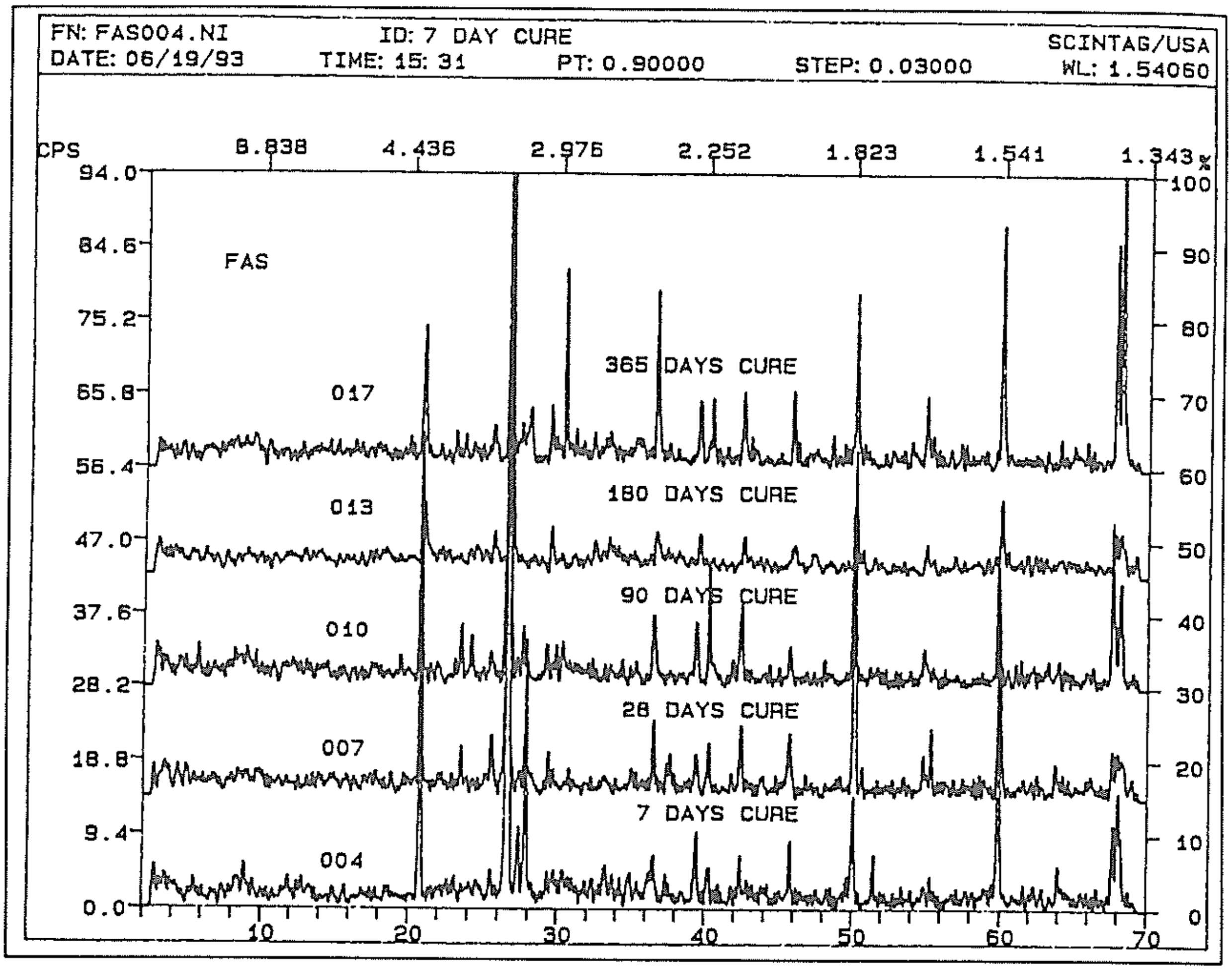


FIGURE 21 Comparison of Long Term Development of Elastic Modulus of All Specimen

FIGURE 22 X-ray Diffractographs of the Fly Ash A-3 River Sand Specimen at Different Times and with Normal Cure Conditions.



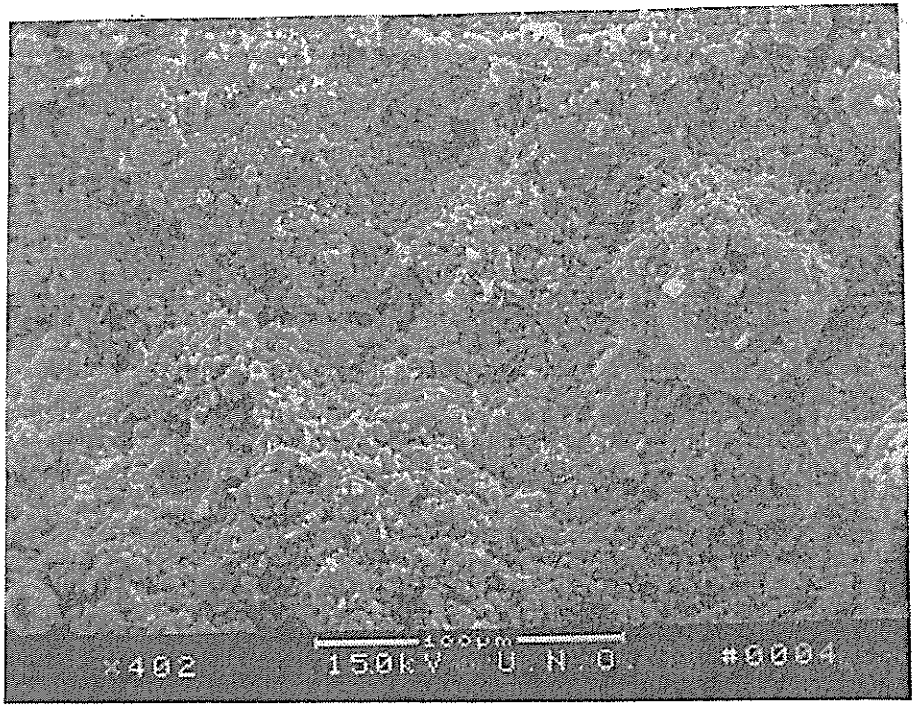


FIGURE 23 Sample FAS 004 (7-day cure) - Loosely cemented crust of fly ash splines and secondary phases

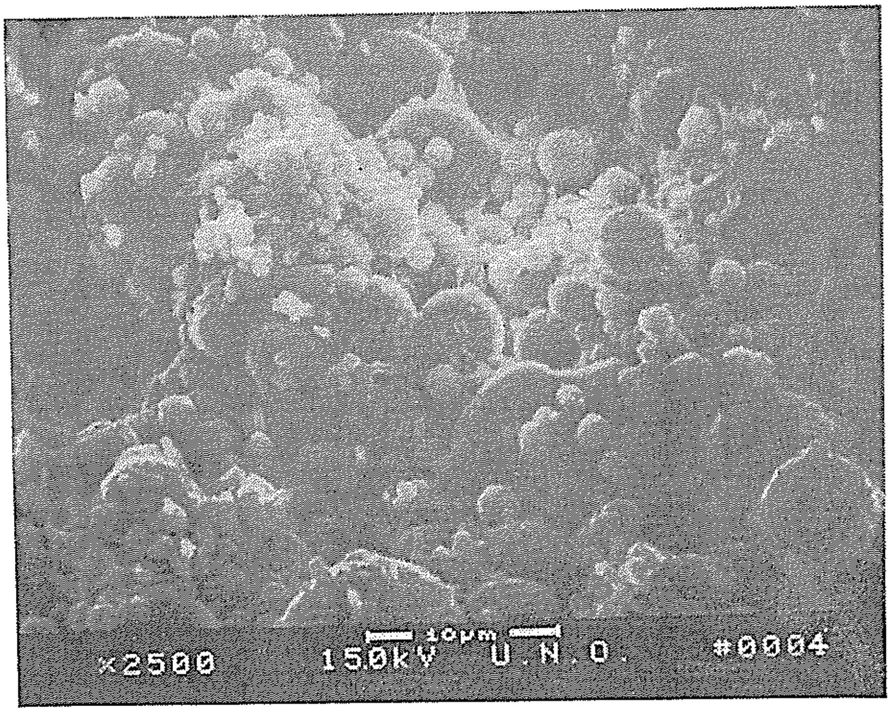
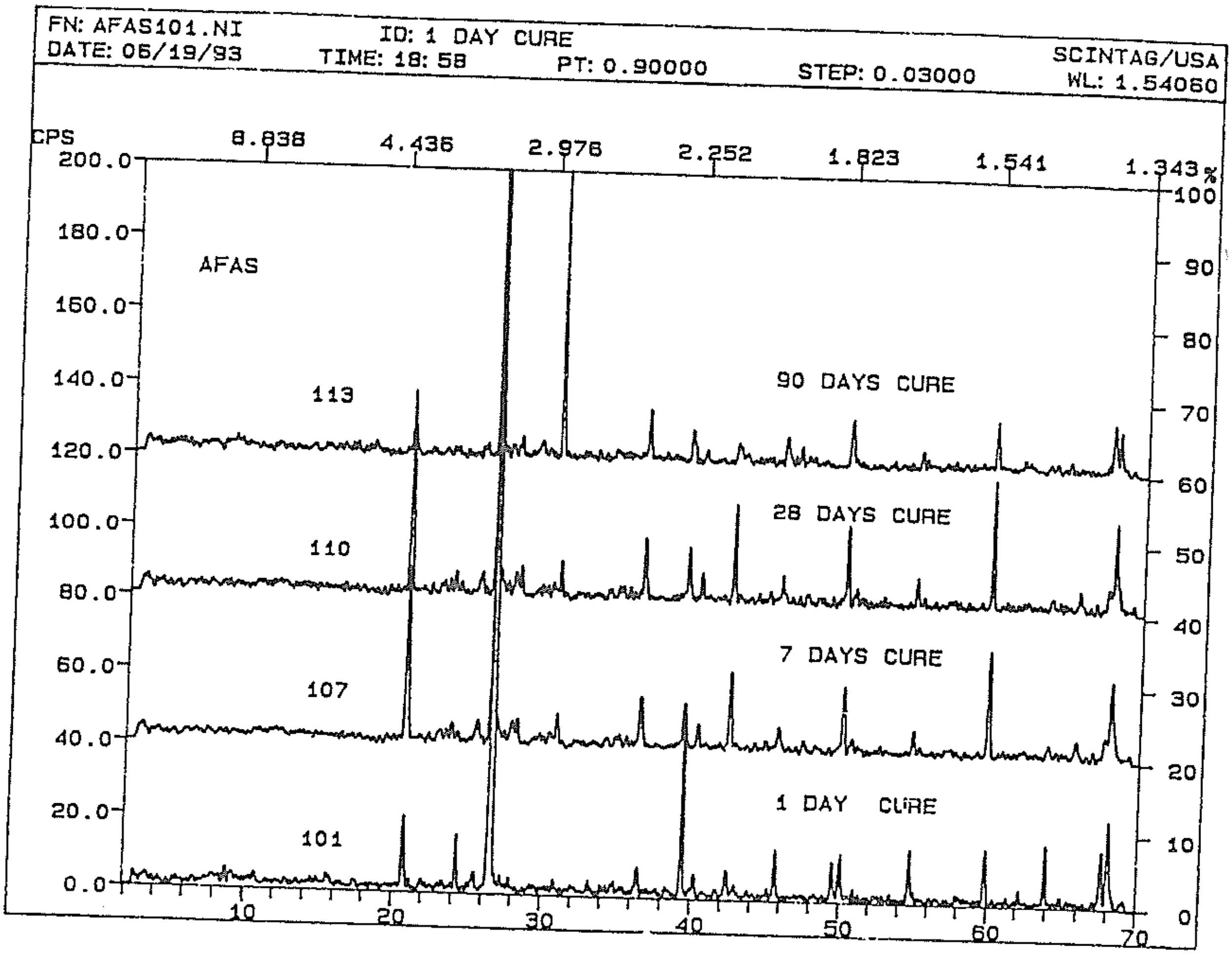


FIGURE 24 Sample FAS 004 - Greater magnification showing fly ash splines loosely cemented by a few long prismati secondary phases.

FIGURE 25 X-Ray Diffractographs of the Fly Ash - A-3 River Sand Specimen at Different Times with Accelerated Curing Conditions.



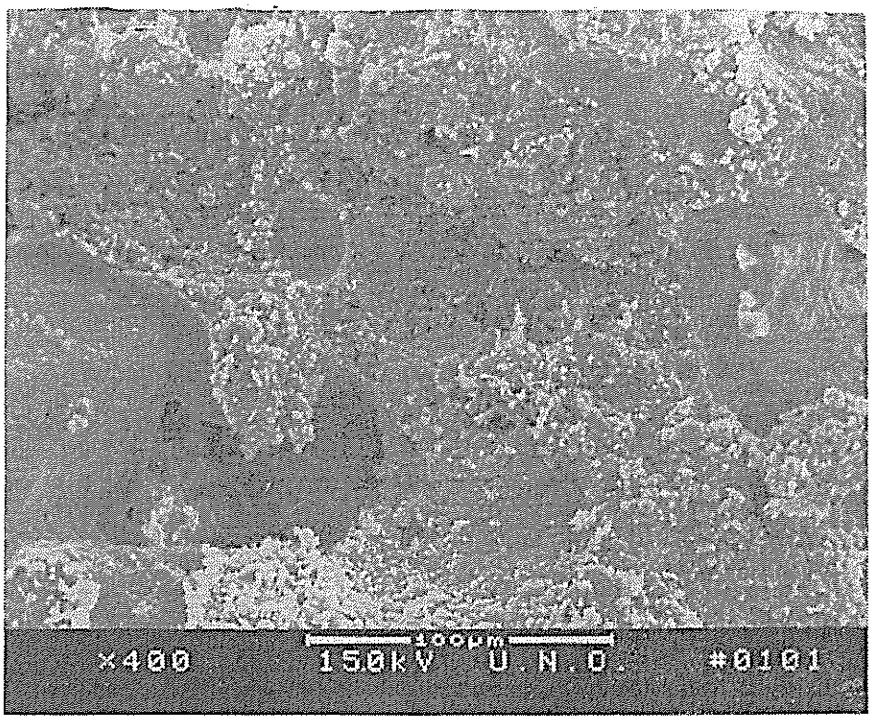


FIGURE 26 Sample AFAS 101 (1-day accelerated cure) - Dense mat of felty secondary phases.

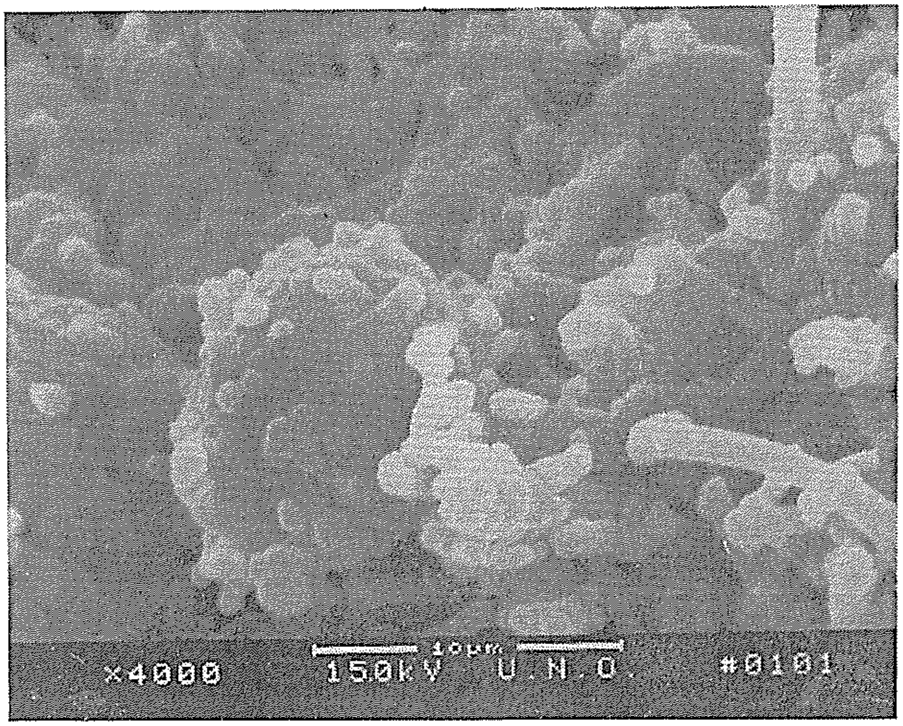
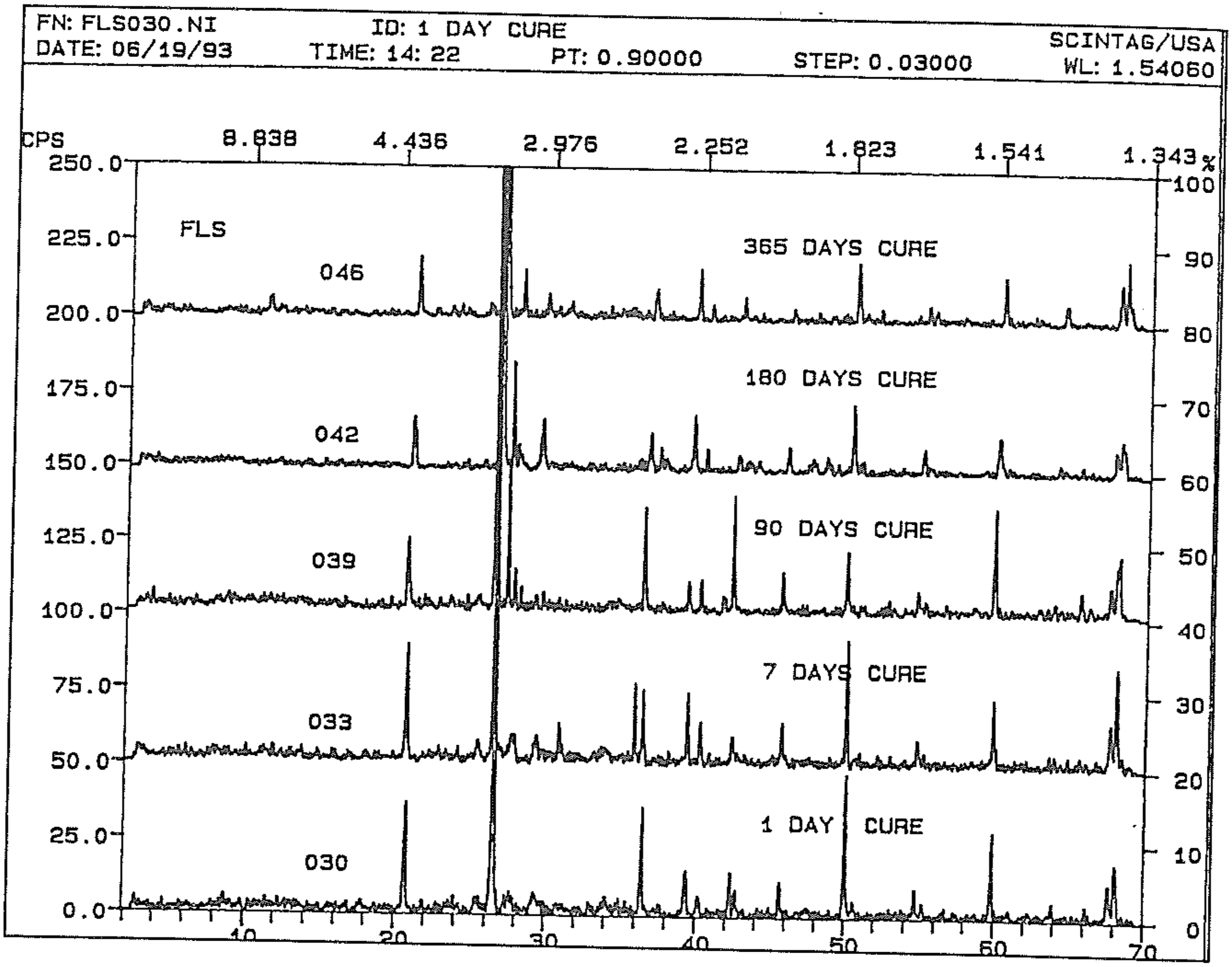


FIGURE 27 Sample AFAS 101 - Magnified rod shaped and rounded secondary phases with fly ash splines.

FIGURE 28 X-ray Diffractographs of the Lime-Fly Ash and A-3 River Sand Specimen at Different Times and with Normal Cure Conditions.



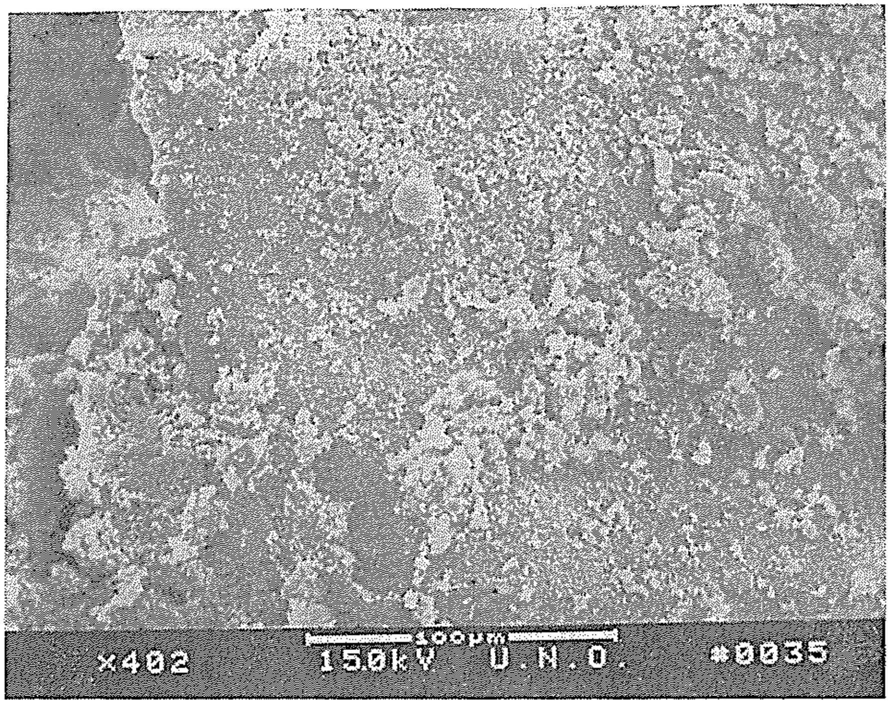


FIGURE 29 Sample FLS 035 (7-day cure) - Loosely adhering fly ash splines and crust of long prismatic secondary phases on quartz grains.

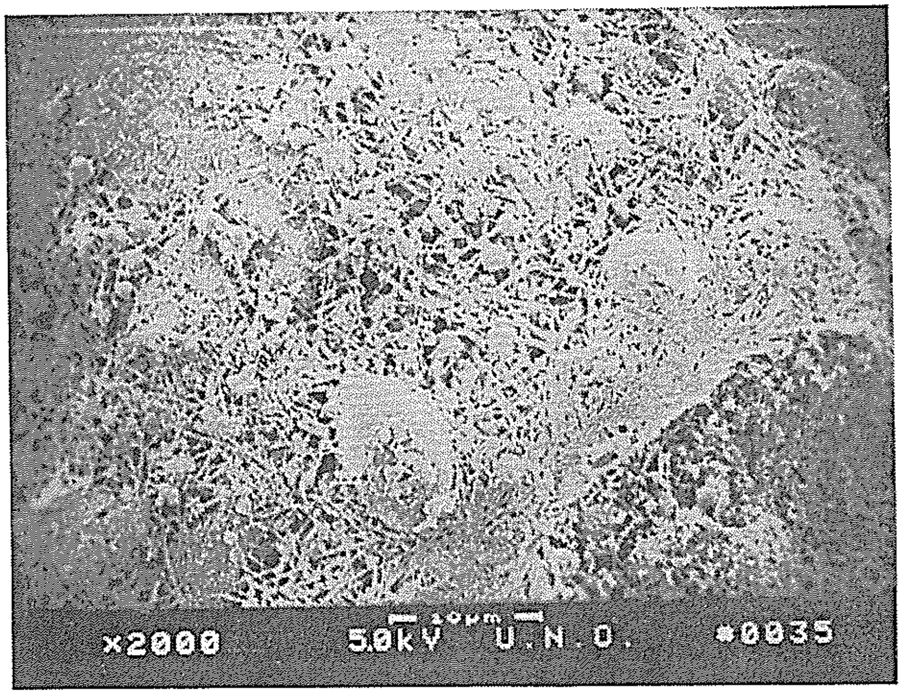
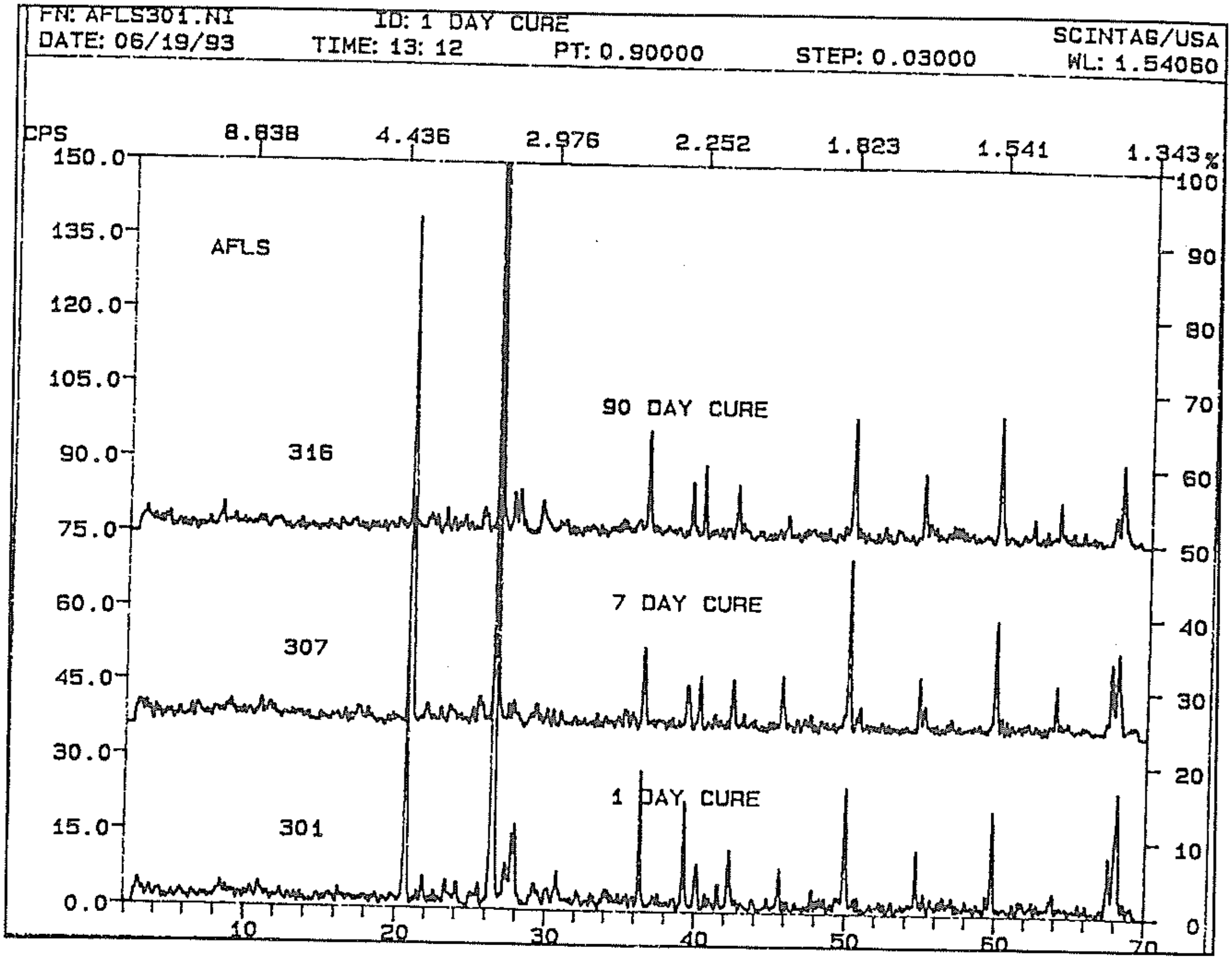


FIGURE 30 Sample FLS 035 - Reticulated network of long prismatic crystals enveloping fly ash spheres on surface of quartz grain.

FIGURE 31 X-ray Diffractographs of the Lime-Fly Ash and A-3 River Sand Specimen at Different Times with Accelerated Curing Conditions.



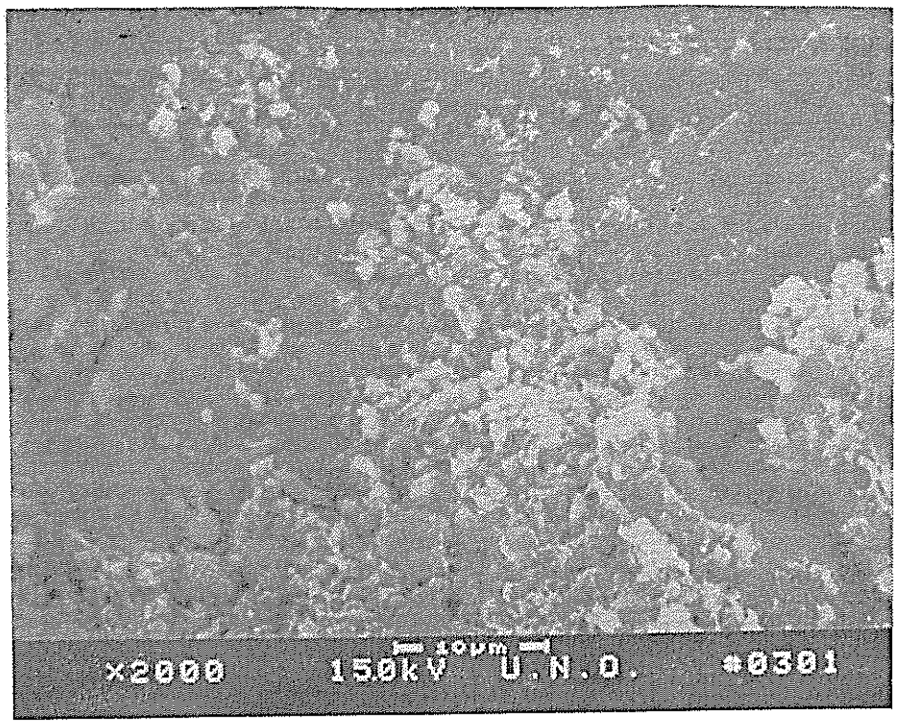


FIGURE 32 Sample AFLS 301 (1-day accelerated cure) - Several fly ash caked together with quartz grain.

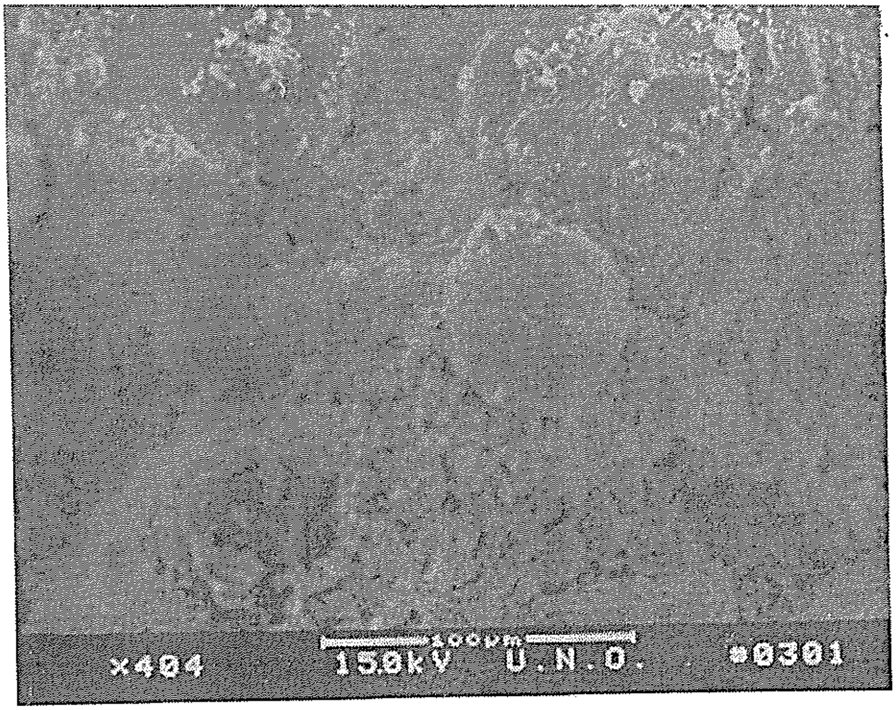
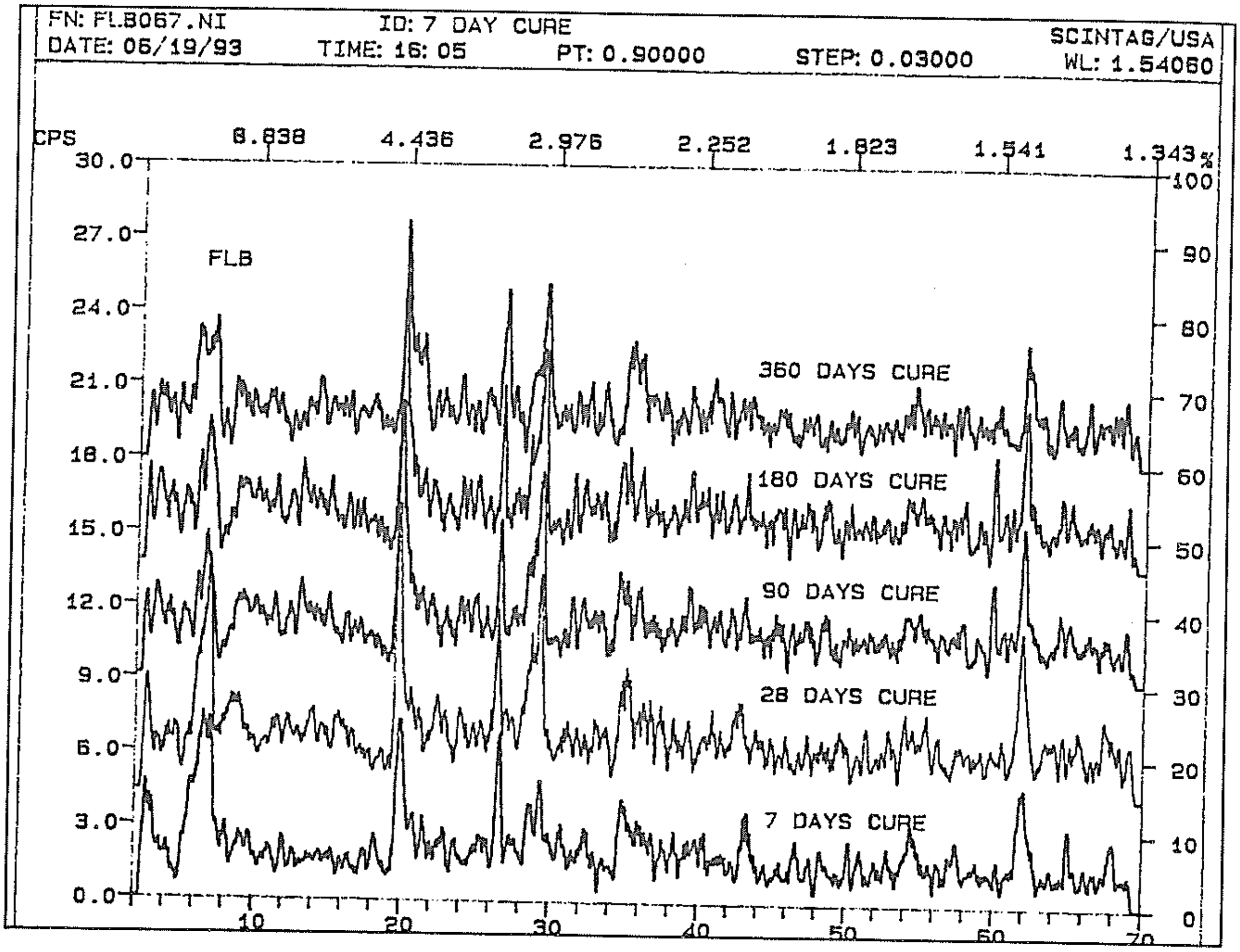


Figure 33 Sample AFLS 301 - Crustose of secondary phases on quartz grain with fly ash splines.

FIGURE 34 X-ray Diffractographs of the Lime-Fly Ash and Bentonite Clay at Different Times



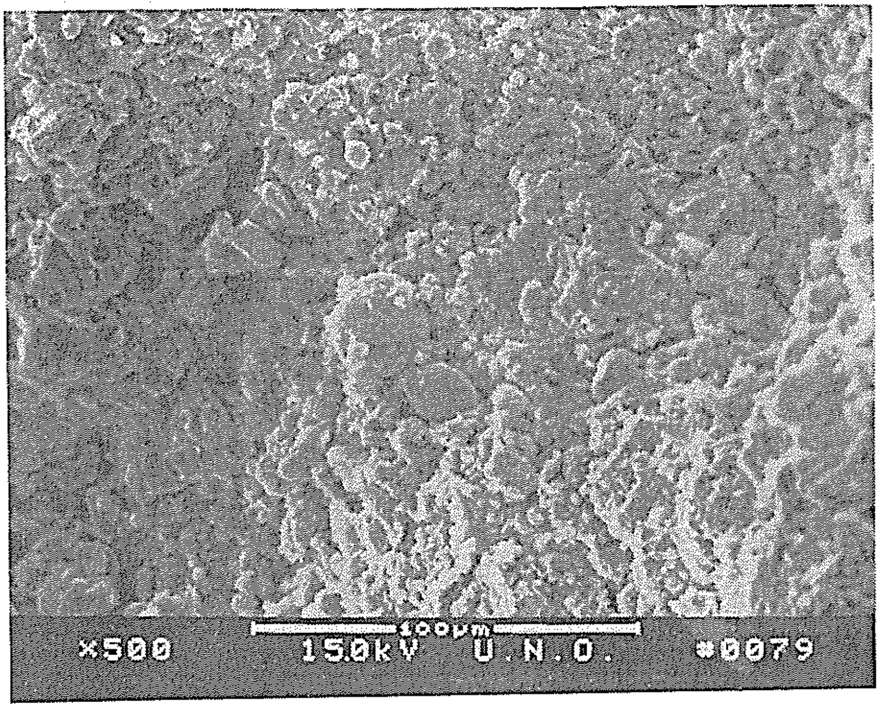


FIGURE 35 Sample FLB 079 (365-day) Fly ash splines in clay and secondary matrix.

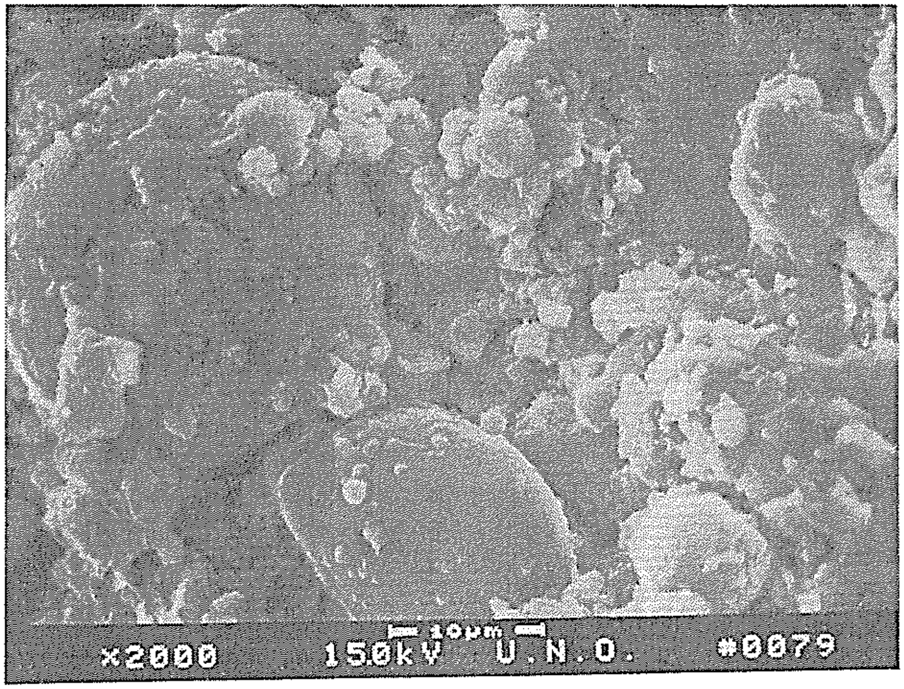
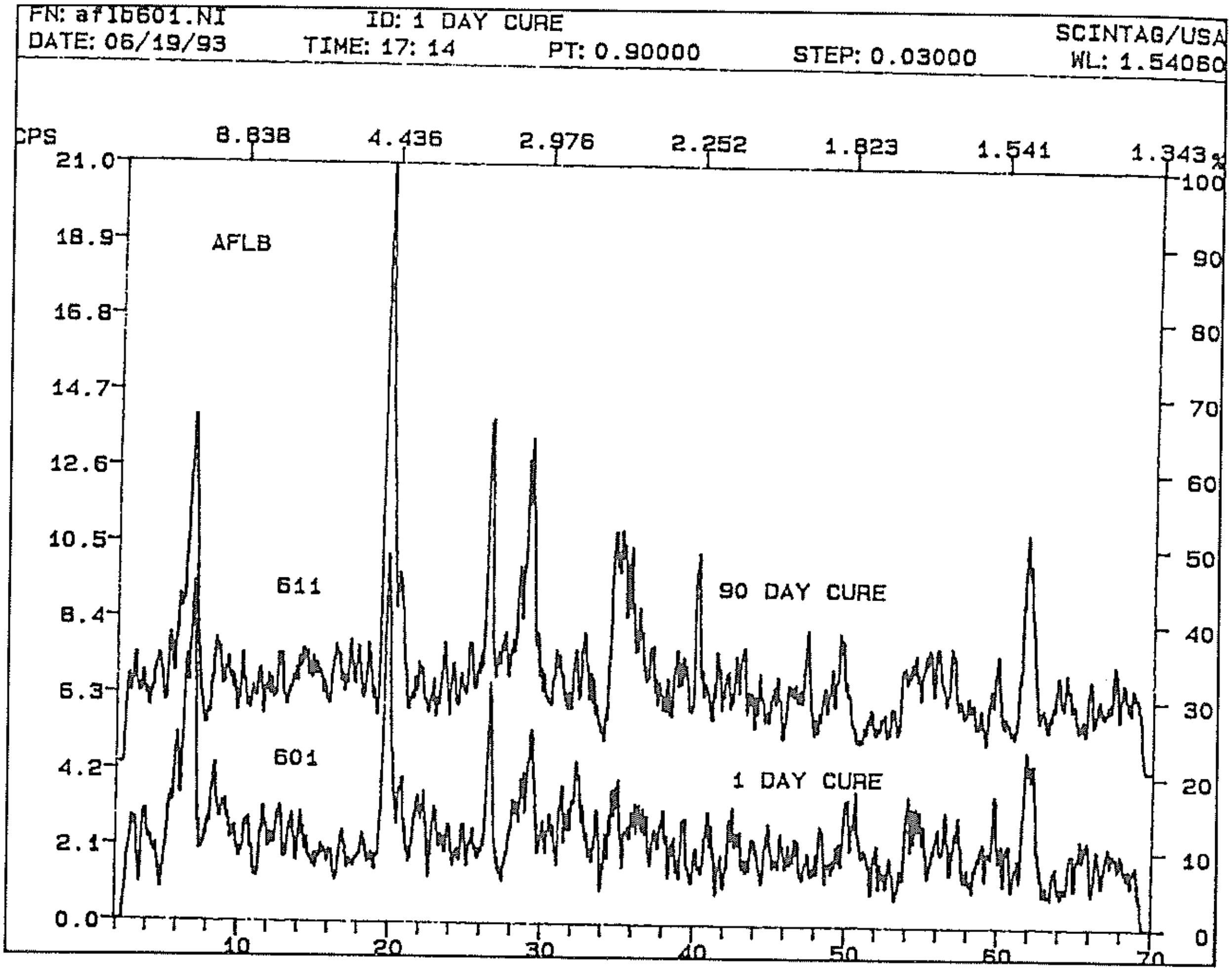


FIGURE 36 Sample FLB 079 - Fly ash splines with crusty overgrowth. Splines appear to be at least partially void.

FIGURE 37 X-ray Diffractographs of the Lime Fly Ash and Bentonite Clay at Different Times and with Accelerated Curing Conditions.



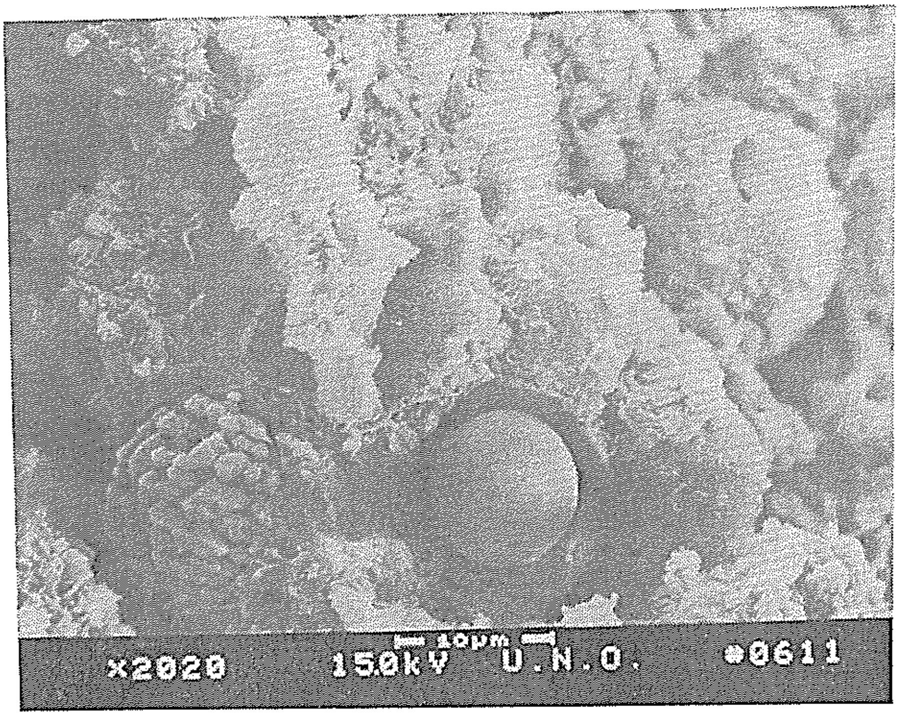


FIGURE 38 Sample AFLB 611 (90-day accelerated cure) Clay rich matrix with embedded partially dissolved fly ash splines and secondary phases. Three fly ash spheres in various stages of dissolution and recrystalization

FIGURE 39 X-ray Diffractographs Comparing All Specimen Mixtures (FAS, FLS and FLB) for 90-day Normal Cure.

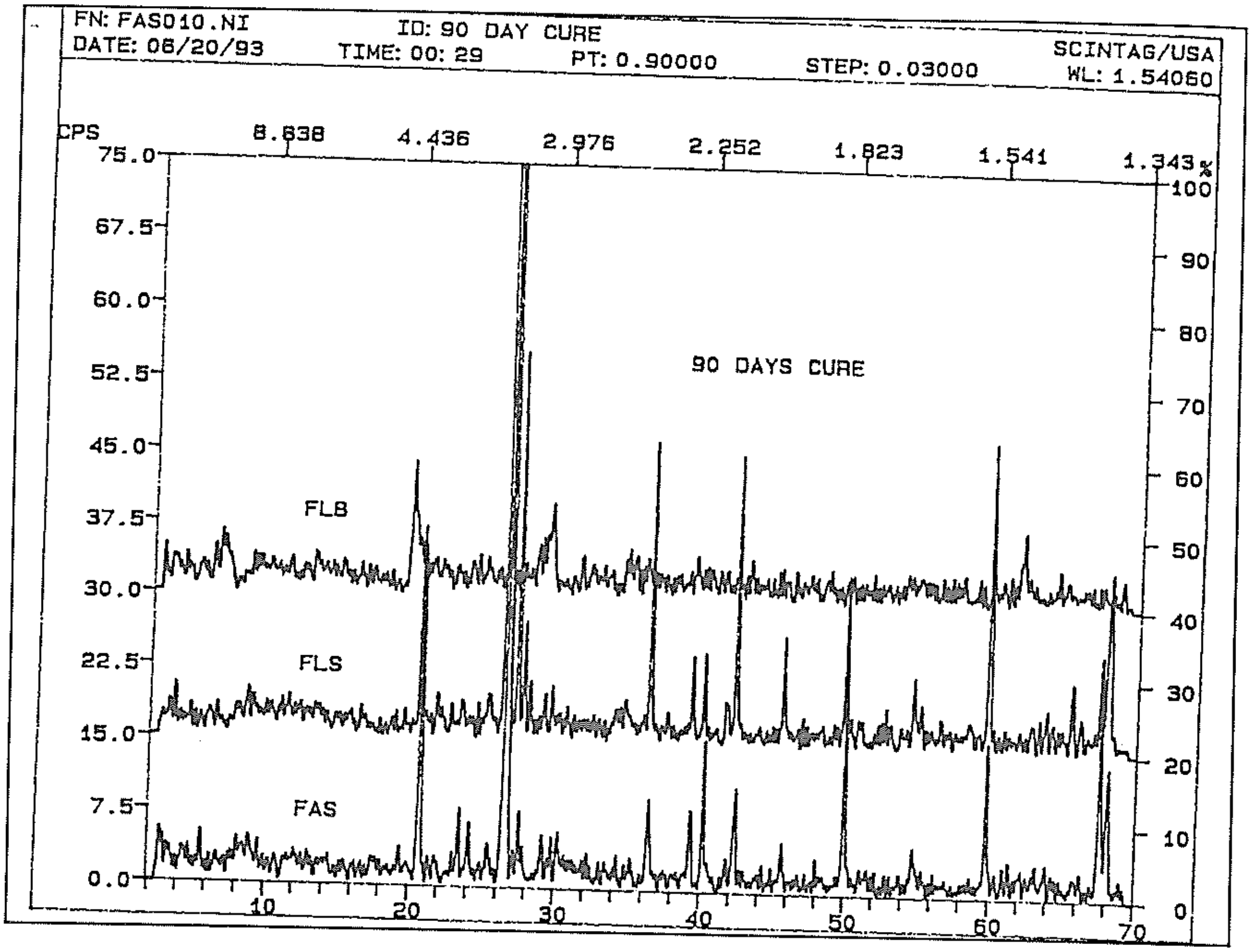


FIGURE 40 X-Ray Diffractograph Comparison of the 1-Day Cure Accelerated Cure for All Samples

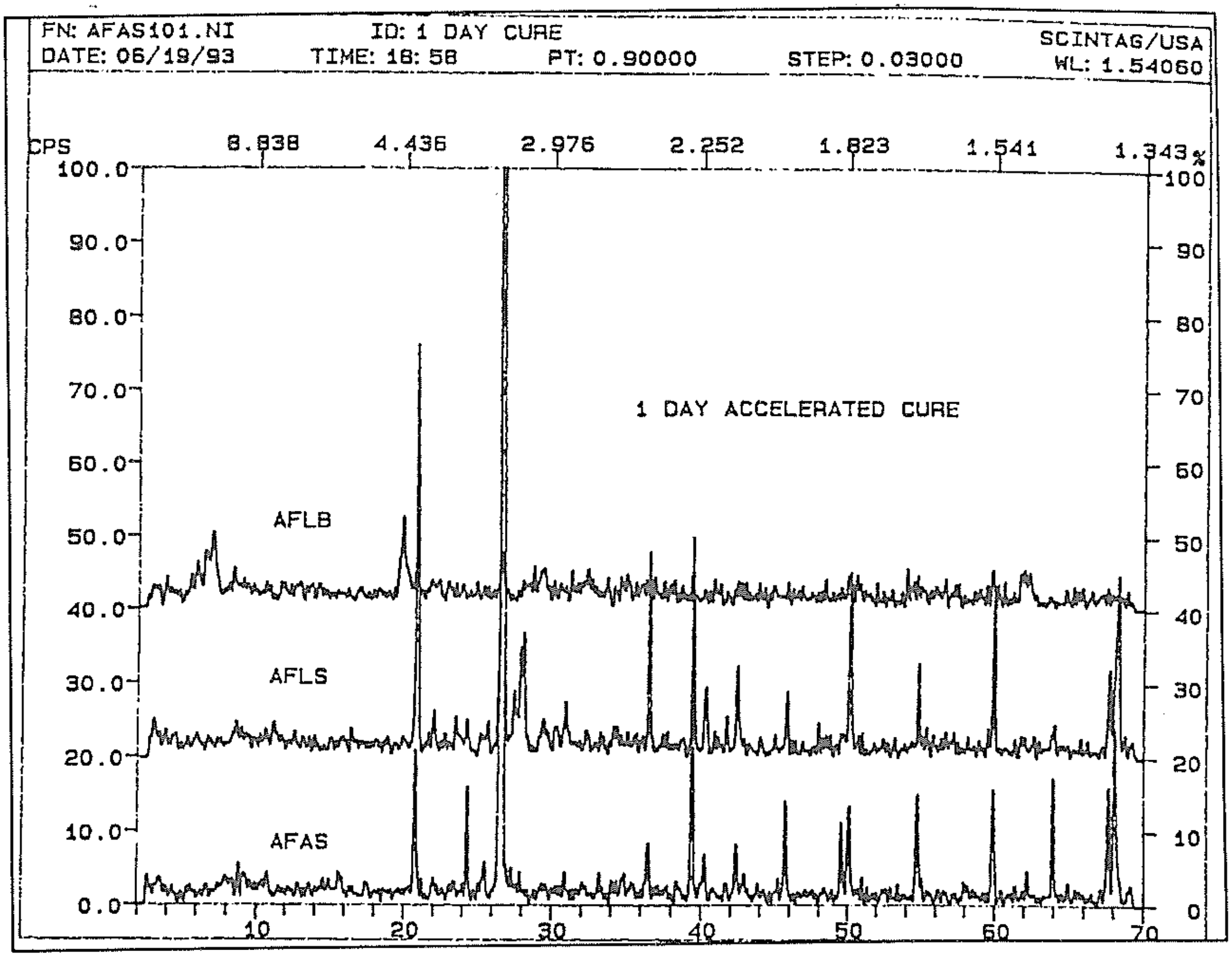


FIGURE 41 X-ray Diffractographs Comparing 1-day Accelerated Cure with 28-day Normal Cure for the Fly Ash and A-3 River Sand Specimen.

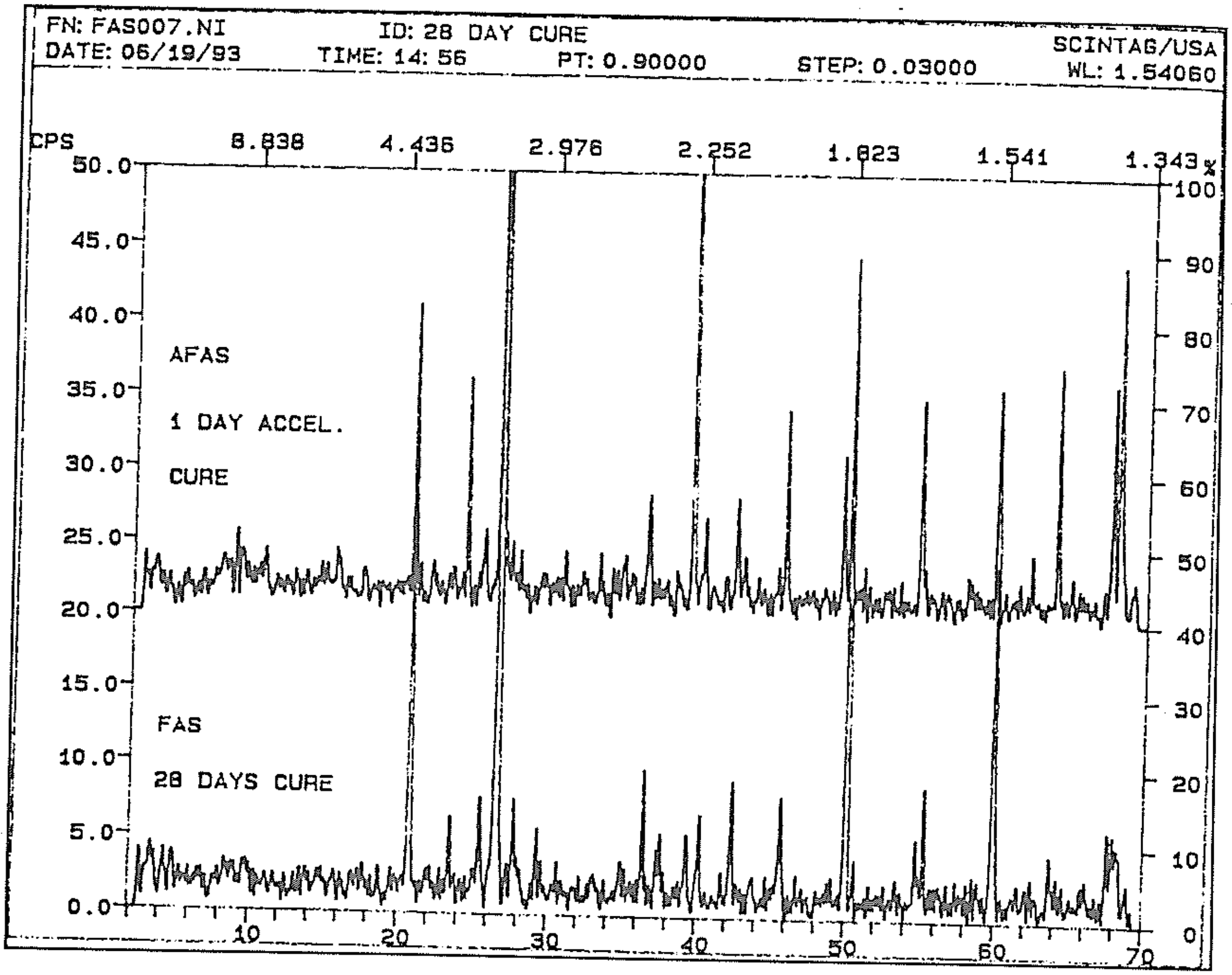


FIGURE 42 X-Ray Diffractograph Comparing the 1-Day Accelerated Cure with the 7-Day and 90-Day Normal Cure for the Lime-Fly Ash and Sand

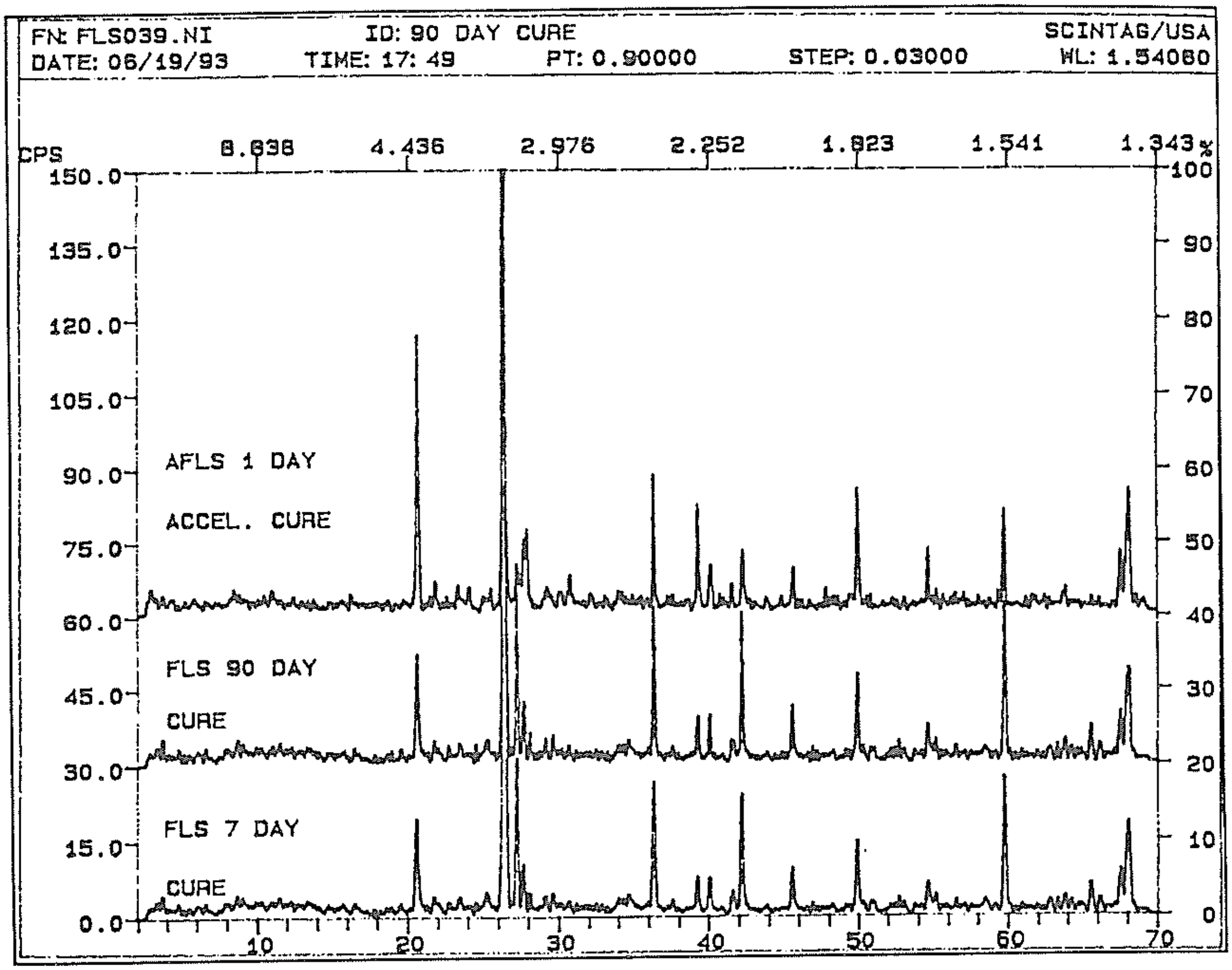
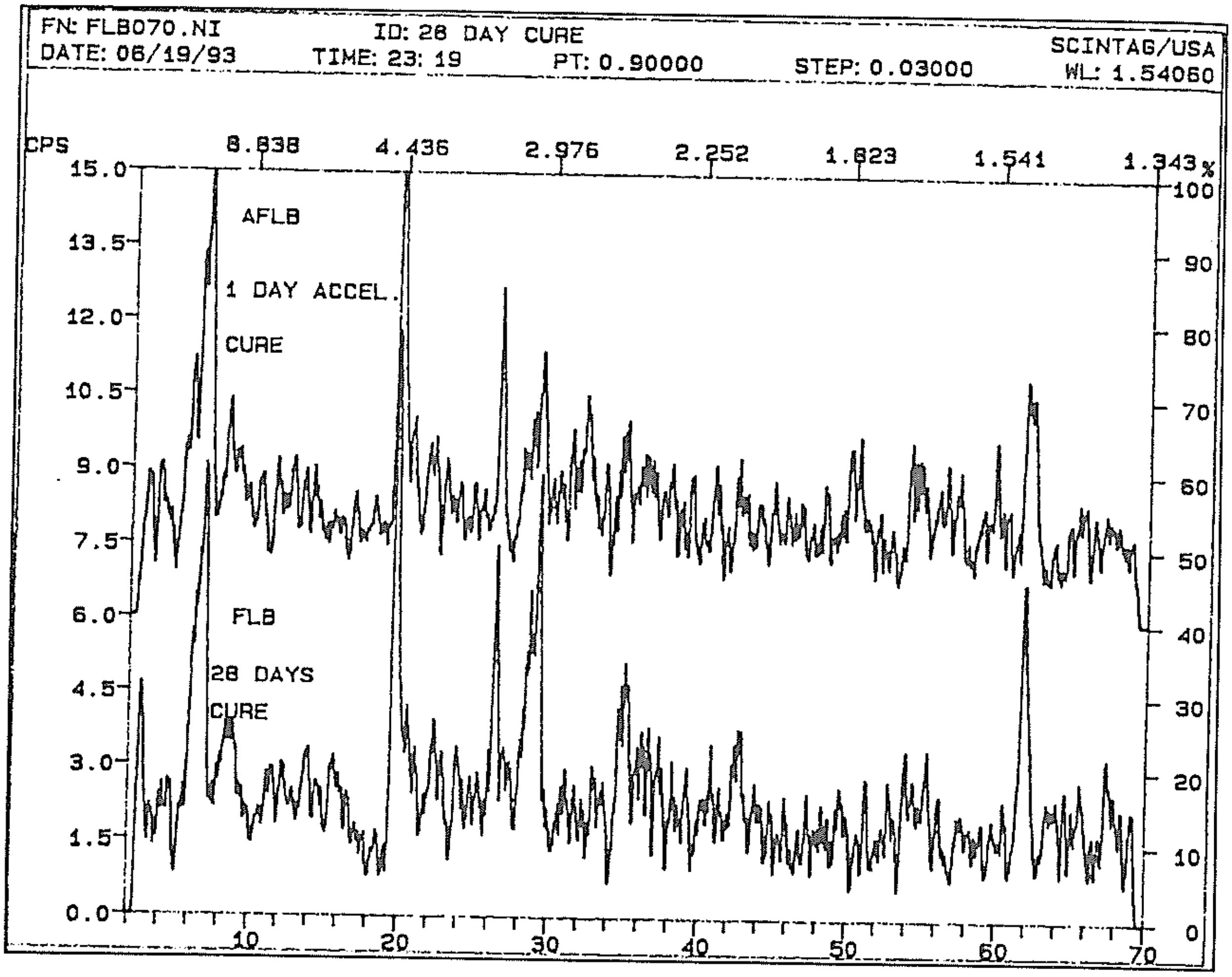


FIGURE 43 X-Ray Diffractographs Comparing 1-day Accererated Cure with 28-Day Normal Cure for the Lime-fly Ash and Bentonite Clay Specimen.



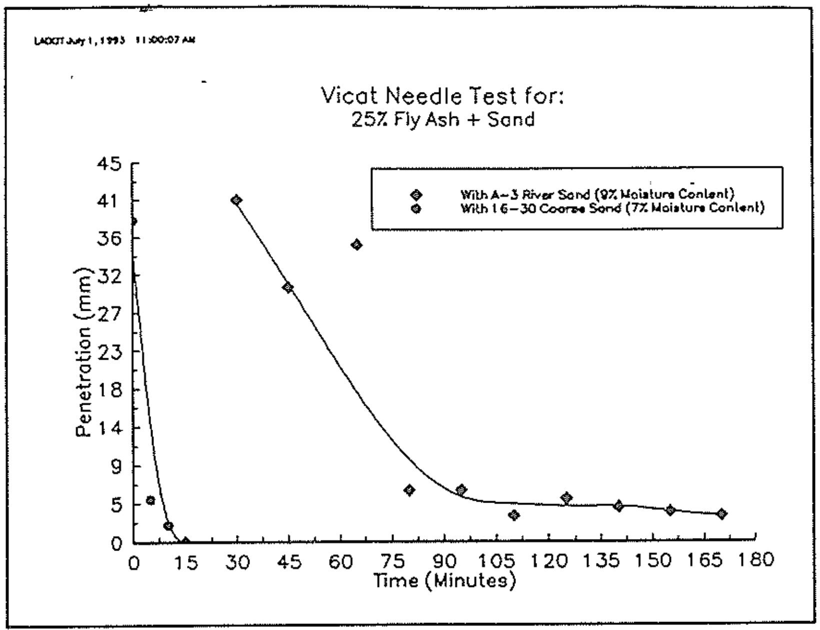


Figure 44 Vicat Needle Penetration vs Time: ASTM Class C Fly Ash with Coarse and Fine Sands

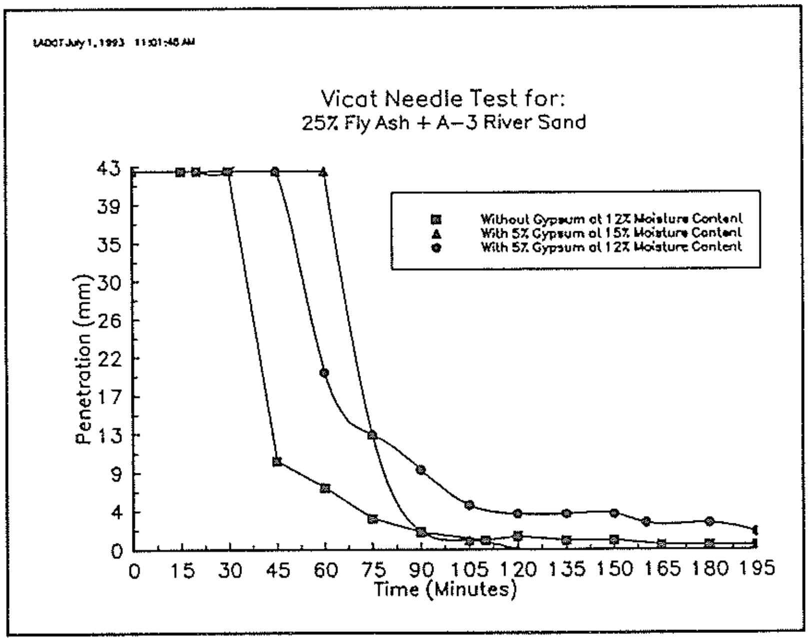


Figure 45 Vicat Needle Penetration vs Time: Fine Sand w/Gypsum

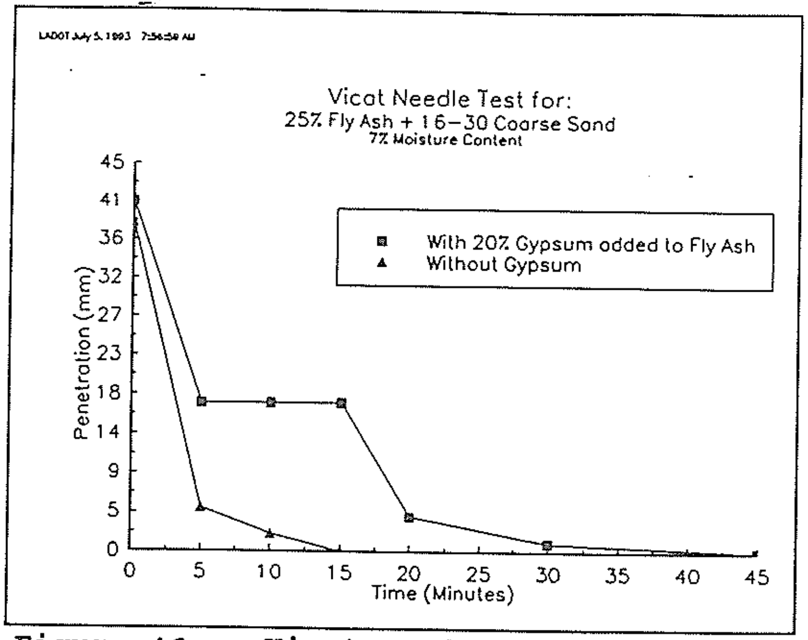


Figure 46 Vicat Needle Penetration vs Time: Coarse w/Gypsum

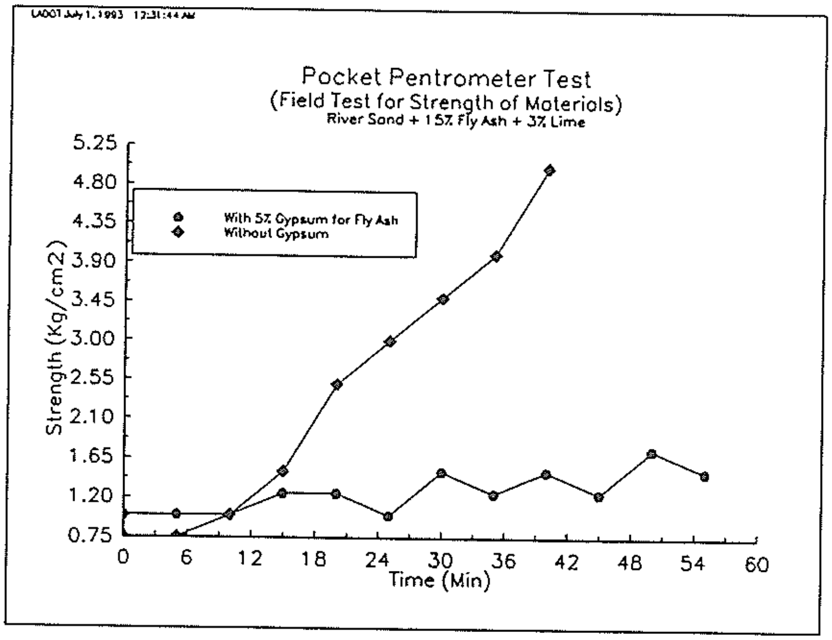


Figure 47 Pocket Penetrometer Resistance vs. Time

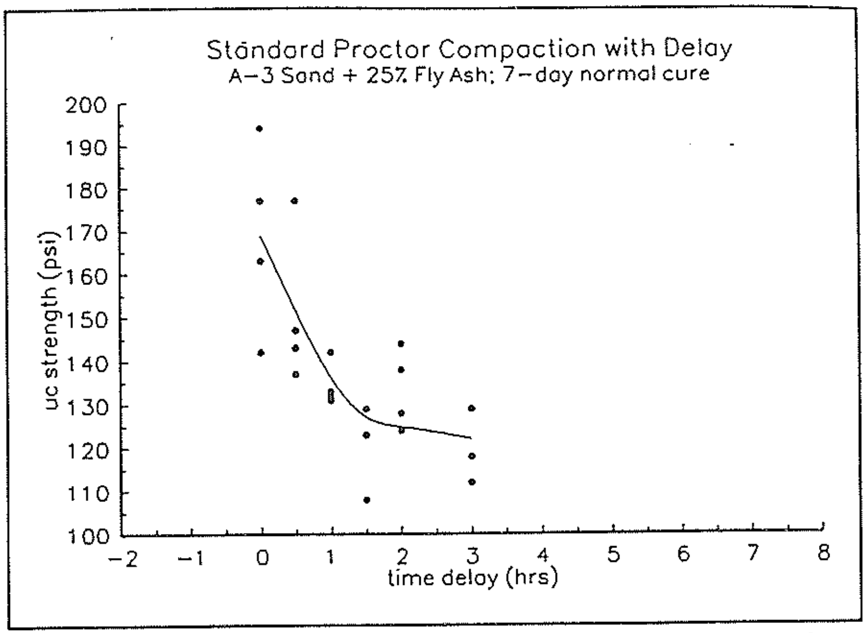


Figure 48 The Effects of Delayed Compaction on Strength

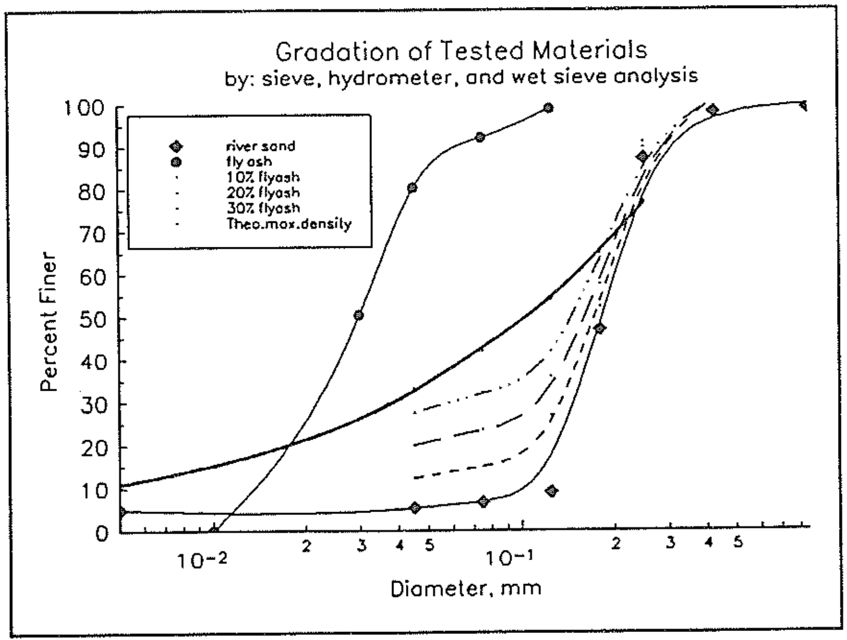


Figure 49 Optimum Fines Content: Fly Ash Required to Produce Maximum Density

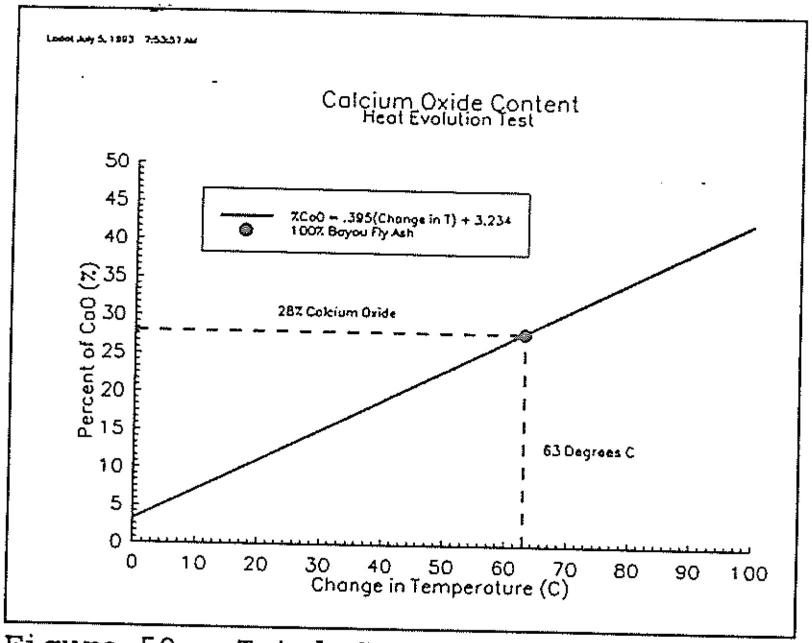


Figure 50 Total CaO Content in AST Class C Fly Ash (McKerall et al, 1981)

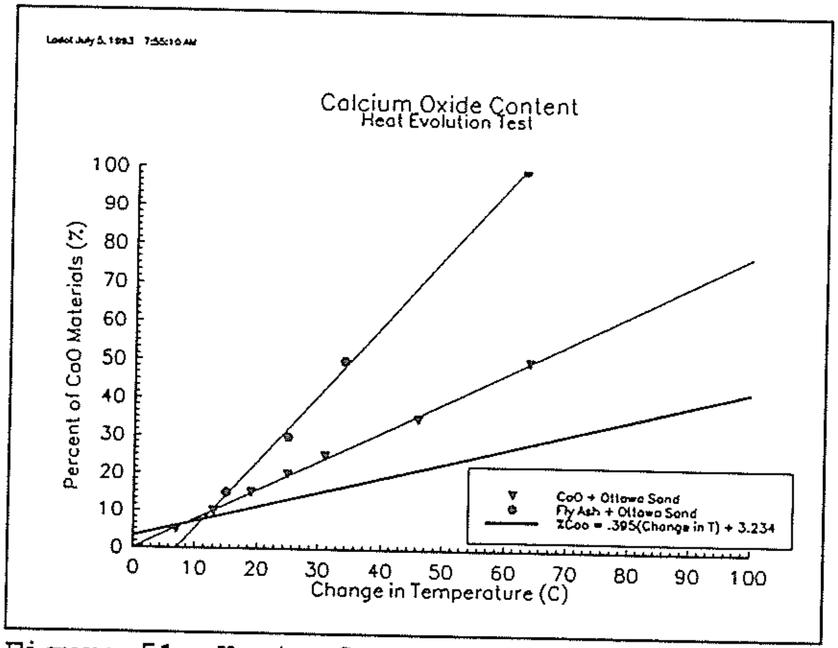


Figure 51 Heat of Evolution Relationships with CaO Materials

The following test method (McKerall et al, 1980) has potential in providing assistance in construction quality control and other field tests. The method only requires a thermos, stopper and thermometer.

(1) Allow the separated fly ash, acid and thermos bottle each to reach an equal and constant temperature and record this as the initial temperature.

(2) Weigh 20 grams +/- 0.2 grams of fly ash and place it in the thermos bottle.

(3) Add 75 ml of 15 percent HCL to the fly ash within the thermos bottle and stir to insure mixing. [Fifteen percent HCl is made by mixing 6 parts of distilled water to 4 parts of 12 molar HCL (35.5% pure).]

(4) Quickly cover the thermos bottle with the stopper and insert the thermometer, being sure the tip of the thermometer is touching the bottom of the bottle.

(5) Observe the highest thermometer readings until a drop in temperature is seen.

(6) Subtract the highest temperature observed from the original temperature. This will give the change in temperature, ›T, in °C.

(7) Calculate the total content by use of the following expression:

CaO = 0.395 (›T) + 3.234
Where ›T = change in temperature, °C
CaO = total CaO content