Engineering properties and construction guidelines for soil stabilized with self-cementing fly ash

by

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

INTRODUCTION

A majority of the electricity in the United States is produced at coal fired electric utilities. The burning of coal results in over 117 million tons of coal combustion by-products, most of which is in the form of fly ash (2). Although utilization of fly ash is continuing to grow, less than 32% of coal combustion by-products produced are recycled (2). The remainder of fly ash and other coal combustion by-products are wasted in sluice ponds or landfills, taking up valuable space. The most widely used application of fly ash is as a partial replacement to cement in Portland cement concrete. States such as Iowa allow up to 15% replacement of cement with fly ash, which improves various concrete mix properties and strength gain (33).

Another use of fly ash is in soil stabilization. Soils can be treated with self cementing fly ash to modify engineering properties as well as produce rapid strength gain of unstable soils. The volume of fly ash used for soil stabilization is less than that used for cement replacement in concrete, but as knowledge is gained about the mechanisms of stabilization with self-cementing fly ash volume used in soil stabilization will increase.

Primary benefits of using self cementing fly ash for soil stabilization are: (1) Environmental Incentives - Material that is used does not have to be wasted; (2) Cost Savings - Fly ash is typically cheaper than cement and lime; and (3) Availability - Fly ash sources are distributed geographically across the state. When volumes of fly ash produced exceed demand in the construction industry, the material is typically hydrated or conditioned and stored on site. The hydrated and conditioned materials can then be reclaimed at later times and used as soil stabilizers or as select fill under pavement structures. This is beneficial as storage on site also keeps the material out of sluice ponds and landfills.

Iowa soils generally rate as fair to poor as subgrade material. The majority of the soils classify as AASHTO A-4 to A-7-6, meaning they are predominately fine-grained silt and clay soils. These soils exhibit poor strength, high volumetric instability, and freeze/thaw durability problems. Interest has been increasing in the use of stabilization with self cementing fly ash to improve soil properties and strength, as well as uniformity under pavement sections. There are 12 power plants in the state of Iowa that produce self-cementing fly ash. Half of these power plants have sources of hydrated or conditioned fly ash.

OBJECTIVE

The primary objective of this research study is to evaluate the effects of self-cementing fly ash addition on the engineering properties of several Iowa soils. Tests include strength tests (unconfined compressive strength and California Bearing Ratio), influence of curing temperature on strength gain, and effects of compaction delay time on density and strength gain. Soil modification involving changes in plasticity, reduction in swelling potential, and increasing wet/dry and freeze/thaw durability are also evaluated. Research results are also provided for engineering properties of hydrated and conditioned fly ash from six power plants in Iowa. The secondary objective of this research was to develop construction guidelines and specifications for use of self-cementing fly ash to stabilize soils, for use of hydrated or conditioned fly ash to stabilize soils, and for use of hydrated or conditioned fly ash as select fill under pavement structures.

LITERATURE REVIEW

Introduction

A literature review was initiated to examine the construction operations for us of self-cementing fly ash as a soil stabilizer. Procedures for mixing, moisture control, compaction, and curing were investigated. Several methods of quality control testing were also reviewed, which include field density and moisture, stability, and in-service performance-based tests. Case histories describe the use of fly ash as a stabilizer or as a fill material itself. A review of chemical properties and reaction mechanisms of fly ash is also provided. Lastly, suggested fly ash construction specifications are described.

Construction Operations

Mixing

One of the main concerns when using self-cementing fly ash as a soil stabilizer is achieving thorough and uniform mixing with the material to be stabilized. There are two approaches generally used in construction: (1) Off-site mixing using continuous or batch type mixing and (2) On-site mixing. Off-site mixing operations have the advantage of achieving more uniform mixtures because the amount of materials batched can be controlled to a greater extent than on-site mixing. A disadvantage when using self-cementing fly ash is that it exhibits a relatively rapid set which can lead to a decrease in strength with delayed compaction (1). Most off-site mixing operations have been used in the case of combined

Class F (non self-cementing) fly ash and lime stabilization, as Class F material does not exhibit self-cementing characteristics. Off-site continuous mixing plants have all materials to be mixed brought in on a conveyor system to a mixer where they are combined with water and then loaded directly into a truck, at a constant interval. Batch type operations are similar to a batch plant used for Portland cement concrete. Enough material for a single truck is mixed and then loaded into the truck. The Federal Highway Administration (44) has suggested that for large scale projects a continuous mixing plant is preferred because material can be generated at a higher rate compared to a batch type mixing operation.

As use of self-cementing fly ash has expanded for stabilization practices, the American Coal Ash Association (ACAA) has stated the preferred mixing method has become on-site mixing (1). This approach does not require establishment of a mixing plant and better takes advantage of the rapid set time of self-cementing fly ash. In this case the fly ash is trucked to the site by belly dump or tanker trucks, and then spread on the subgrade. The mixing operation is then completed using a pulvamixer or disc (46). A typical pulvamixer is shown in Figure 1. Pulvamixers are either single or multiple shaft mixers, or pavement recyclers. One or two passes of the pulvamixer equipment is usually required to obtain a thoroughly mixed material, generally with 100% passing the 1-inch sieve and a minimum of 50% passing the #4 sieve (1). In some cases, as when fly ash is used as a drying agent, discing with a construction disk has been effective in fine grained soils. Incorporation of ash by discing can also be used to bridge unstable subgrades and reduce the effects of water pumping upward due to construction traffic (26).

Figure 1. Typical Pulvamixer Used for On-Site Mixing

Application of Water

The process of adding and monitoring the mixing water during the stabilization operations is one of the most important steps in the construction process. When using a mixing plant setup, general suggestions for water are that it should be between 80 and 110 percent of the optimum moisture content based on the moisture-density relationship of the stabilized mixture in order to obtain proper density at time of compaction (1). As mentioned before, this process is giving way to on-site mixing of self-cementing fly ash, subgrade soils, and water.

Before addition of fly ash, water can be added to the subgrade soils (46), however a disadvantage of this approach is that the subgrade may become unstable, complicating the rest of the construction process (26). Alternatively, water can be added after the fly ash has been incorporated into the soil, but more passes of the mixing equipment are generally required and strength loss can occur due to hydration of the fly ash prior to final compaction (26). The ACAA reports that the most effective method for controlling mixing water has been to add the water directly into the mixing drum of the pulvamixer. This procedure produces the most uniform mixing and the least amount of delay in the construction process (26).

Moisture control also includes the properties of the water to be used on the project. The ACAA (26) suggests the water be potable or meet the requirements of AASHTO T 26 [Method of Test for Quality of Water to be Used in Concrete], which is similar to ASTM C94 [Standard Specification for Ready-Mixed Concrete] (33). This is to assure the water is free of sewage, organic matter, oil, acid, and alkali, which can have detrimental effects on the performance of self-cementing fly ash stabilized material.

Compaction of Fly Ash Stabilized Soil

A variety of compaction equipment can be used to increase the relative compaction of fly ash soil mixtures, and is dependant on soil type. Due to the self-cementing properties of fly ash, it can be an effective stabilizer for granular and fine grained materials. FHWA (44) classifies granular materials as AASHTO A-1, A-3, A-2-4, and A-2-5 soils, while fine grained materials are AASHTO A-4, A-5, A-6, A-7, and some A-2-6 and A-2-7 soils. For stabilizing granular materials, steel wheeled, vibratory, or pneumatic rollers are recommended (1). FHWA (44) suggests initial compaction with a sheepsfoot or padfoot roller and finish rolling with a pneumatic roller for fine-grained materials. Sheepsfoot or padfoot rollers are preferred because good compaction of the lift from the bottom up is achieved while the kneading action helps to further mix the fly ash, soil, and water (26).

Compaction delay time should also be taken into consideration because the stabilized material can lose strength gain capacity as the fly ash hydrates while in an uncompacted state. For Class F fly ash stabilization work a maximum compaction delay time of up to 4 hours has been specified (44, 46). With the increased reactivity of self-cementing fly ash, however, a much shorter compaction delay time is typically specified. For self-cementing fly ash stabilized sections, the ACAA recommends that compaction commence as soon as possible after final mixing and be completed within two hours so the stabilized material will show less strength and density decrease (44, 26). In most cases, the initial compaction begins with a padfoot type roller directly behind the pulvamixer and can be finished within 15 minutes after final mixing (26).

Curing of Completed Fly Ash Stabilized Sections

Curing self-cementing fly ash stabilized sections involves sealing the completed sections before overlying pavement sections are placed to allow the fly ash to hydrate and gain required strength (44, 46). Availability of moisture, temperature during curing, and length of cure time all have an effect on strength gain of fly ash stabilized soils (46, 1, 26). Typically, mixtures are cured by sprinkling with water or by coating with a thin layer of emulsion or cutback asphalt (26, 30, 46, 44, 1). The Armed Forces and FHWA recommend that the sealer be applied within one day of completion of the section and that multiple coats may be required (44, 30, 46). Completed sections can also be cured with water for a short time and then sealed with thin coats of asphalt products (32).

According to FHWA (44) and Johnson and Vandenbossche (46), before heavy traffic or pavement sections are placed, the completed sections should be cured for 3 to 7 days. From Armed Forces (30) observation, paving can begin within a day or two after completion of the stabilized section as long as the subgrade can carry paving traffic. In contrast, a cure time of 28 days was specified for one project in eastern Iowa (32). The FHWA (44) also recommends that a protective layer of crushed stone be applied on areas where traffic will be present before paving is completed; however the protective layer delays the release of the volatiles in the asphalt seal coat. The volatiles negatively react with the stabilized base and inhibit strength gain during curing.

According to the ACAA (1, 26) and Johnson and Vandenbossche (46), fly ash stabilization operations should not proceed when the air temperature is below 40° F. As with most chemical reactions, hydration of fly ash needed for the mixture to gain strength will be slowed at lower temperatures and the required strength will take longer to achieve (26). An example from the Portland Cement Association (33) shows that concrete cured at 25° F has compressive strength of only 28% of the strength attained by samples from the same batch cured at 73° F. Johnson and Vandenbossche (46) further recommend that frozen soils not be used in stabilization processes and that completed sections should be allowed to cure in temperatures above 40° F for at least one week before freezing temperatures occur. Another concern arrives from the fact that highly plastic soils will need more passes with the mixing equipment to pulverize the material to sizes smaller than one inch at temperatures below 50°

F, and more passes of the compaction equipment are recommended to meet required density standards (26).

Quality Control Testing

Field Testing

Many methods have been used to measure the quality of completed sections of self-cementing fly ash stabilized soils. These procedures are used to measure the in-place density, stability and moisture content of the compacted sections.

In-Place Density Determination

Compacted density is one of the main quality control parameters, as the stabilized material cannot gain the required strength if not compacted to a dense state, typically 90% to 95% of standard Proctor maximum density (1, 26, 44, 30). Tests used in practice to measure in-situ density are the sand cone, rubber balloon, nuclear gauge, and drive cylinder (1, 44, 42). The advantage of using the sand cone, rubber balloon, and drive cylinder is that the material removed can be used directly to measure moisture content. Disadvantages to these tests are that they take longer to get the moisture results and performing the tests can be time consuming. The nuclear gauge provides reasonable values for the total compacted density, but the dry density calculated by the gauge is variable at best when compared to measurements determined by the rubber balloon method (35). This is believed to be a result of erroneous nuclear gauge moisture readings (35).

Moisture Content Determination

The most difficult, yet one of the most important parameters to measure accurately in the field is the moisture content of the stabilized soil. Without proper moisture, typically +/- 4% of optimum based on maximum density (42), the fly ash stabilized material cannot reach the specified limits of relative compaction, generally 90% to 95% of standard Proctor maximum. The ACAA (26) states that if the relative compaction is not reached, the self-cementing fly ash will not reach the required strength. The most accurate method to determine the moisture content of the compacted material is the direct heating method. This involves taking samples back to a laboratory and drying the samples in an oven or by directly heating them (1, 44, 42). Compared to direct heating, oven drying takes a longer time, which could cause construction delays.

A "Speedy Moisture Tester", ASTM D4944 [Standard Test Method for Field Determination of Water (Moisture) Content of Soil by the Calcium Carbide Gas Pressure Tester Method] can be used to quickly determine moisture content in the field (42, 1). Seals (42) reported that results for the "Speedy Moisture Tester" have shown to be somewhat unreliable and variable, but his studies in Virginia and West Virginia have produced very acceptable results. Seals (42) also states that the "Speedy Tester" needs to be calibrated with oven or direct heating tests.

The quickest and easiest way to determine moisture content in the field is by the use of a nuclear moisture gauge. A disadvantage of using the nuclear moisture gauge is that moisture measurements are usually subject to errors (26, 42, 1, 35). Mahrt (35) states that the difference between actual and nuclear moisture measurements may stem from the elemental and compound structure of the fly ash and how the neutrons from the gauge interact with these parameters. Seals (42) recommends not using the nuclear gauge at all; alternatively, the ACAA (26) suggests that the nuclear gauge can be used as long as calibrations and correction factors are determined and figured into the results. These calibrations are mixture specific. The relative quickness of this test method has proven useful when used to measure the moisture content of uncompacted material directly behind the mixing equipment, thereby monitoring the water addition rate in the drum.

Stability of Compacted Material

Quality control testing has also involved measuring the stability of soil stabilized with self-cementing fly ash. Two main test methods have been used in the past, the Clegg Impact Test, ASTM D5874 [Standard Test Method for Determination of the Impact Value (IV) of a Soil] and the Dynamic Cone Penetrometer (DCP). The Clegg Impact value is correlated to California Bearing Ratio (CBR) of in-place materials. The measured values are taken at the surface of the stabilized layer, and this test can be completed in less than one minute (35). Correlations between Clegg Impact Value (CIV) and CBR of a material exist, but ASTM suggests calibrating CIV and CBR for specific materials to be used on each project. Mahrt (35) determined a correlation between CIV and CBR for hydrated fly ash used as select fill; $CBR = 12.241e^{0.0572(CIV)}$.

The DCP test provides a plot of material stability versus depth. The results of the DCP can be correlated to CBR (50). A common correlation between DCP depth in mm/blow and CBR is $CBR = (292)/(DCP^{1.12})$.

In-Service Performance Testing

Non Destructive Testing

Iowa State University (ISU) researchers have used two different forms of non destructive tests to determine the in-service characteristics of a project in Wapello County, Iowa.

The first test is the Road Rater Test which measures the structural rating of the subgrade by dynamically loading the pavement surface and measuring the deflection of the pavement (50). The deflections are converted to a structural rating which in turn is used to calculate the Modulus of Subgrade Reaction (k-value) of the subgrade. Road rater testing has been used on numerous projects by the Iowa Department of Transportation (IDOT) (32).

The other test that has been used on the pavement surface is roughness testing, which provides the International Roughness Index (IRI) of the pavement. Testing is conducted by pulling a trailer with a vehicle tire along the pavement. Surface defects are measured by the bounce of the wheel and are reported as IRI values, which are in units of m/km. This test can also be used to measure subgrade deterioration over the lifetime of the pavement (50).

Destructive Testing

ISU researchers have also been monitoring compressive strength of subgrade materials on two projects in which hydrated fly ash (HFA) and conditioned fly ash (CFA) were stabilized and used as fill materials. In-service testing involved coring the pavement and subgrade in order to recover cores to test in unconfined compression (51). Sample recovery has shown to be a problem. Coring was also used successfully to monitor the strength gain of a cement-fly ash stabilized base in Des Moines County, Iowa (32). This testing procedure allows visual observation of the subgrade material in addition to the strength testing data.

The DCP test is another destructive method that can be used to monitor subgrade performance after construction. ISU (50) personnel have used to the DCP to measure the stability of the shoulders and select fill underlying the pavement for a project in Wapello County, Iowa. Cores of the pavement are taken in order to expose the subgrade and allow DCP testing to be completed. Testing can be completed relatively quickly.

Construction Method Specifications

Several agencies (44, 1, 26) have developed specifications that provide suggestions for the process of stabilizing soil with fly ash. These specifications cover construction of the stabilized section through quality control testing. Most specifications are broken up into some or all of the following sections that address properties of materials that can be used: (1) laboratory testing procedures prior to construction; (2) construction requirements and operations; (3) quality control and assurance: (4) measurement of materials; and (5) payment for services and materials.

Engineering Properties of Coal Combustion Products

Self-Cementing Fly Ash and Soil Mixtures

Fly ash has been used since around 1950 as a soil stabilizer, but most frequently non self-cementing Class F fly ash was mixed with lime and soil (45). Since the onset of lime-fly ash stabilization, burning of Western United States coal has resulted in production of self-cementing Class C fly ash. Self-cementing fly ash has become the preferred stabilizer in the Midwest and Western parts of the United States due to its availability. Self-cementing fly ash has been used to modify engineering properties such as swell potential, plasticity characteristics, and strength of poor soils (48, 32, 26, 23, 25, 31, 36, 37, 38, 41, 43, 52).

Modification of Plasticity Characteristics and Swell Potential

Self-cementing fly ash has been shown to decrease the plasticity of heavy clay soils, which decreases the swell potential (25). Çoçka (23) noted that plasticity and swell potential exhibit greater decrease due to larger addition rates of fly ash, and he observed that ash addition rates greater than 20% are comparable to lime addition rates of 8% for reducing plasticity and ultimately swell potential for a soil consisting of 85% kaolinite and 15% bentonite. Ferguson (25) notes that the decrease in plasticity and swell potential is generally less than that of lime due to the fact that fly ash does not provide as many calcium ions to modify the surface charge of the clay particles. Also, according to Ferguson (25), the application of self-cementing fly ash to expansive soils decreases the swell potential in three ways: (1) Fly ash contains some calcium ions that reduce the surface charge of the clay particles, (2) Fly ash acts as a mechanical stabilizer by replacing some of the volume held by clay particles, and (3) Fly ash cements the soil particles together. The ACAA (26) recommends that careful laboratory evaluation of different fly ash contents for a given soil is necessary in order to find the optimum ash addition rate. Parsons (38) reported that a disadvantage to using self-cementing fly ash to modify heavy clay soils (PI≥30) is that the swell potential may still be significant (>2.5%) after incorporation of self-cementing fly ash. Nalbantoglu and Gucbilmez (37) reported on the swell potential and compressibility of Degirmenlik soil (LL=67.8, PI=45.6) stabilized with Soma fly ash. Sample mixtures were cured 24 hours before compaction. Swell potential decreased as cure time increased. After curing 7 days, swell values of 4.8 and 3.7% were observed for 15 and 20% fly addition, respectively. Thirty days of curing reduced the swell potential to near 0 for both addition rates. They also noted that compression (C_c) and rebound (C_r) indices decreased as curing time and fly ash content increased. Zia and Fox (52) evaluated the swell potential of low plasticity (PI=0) Indiana loess-fly ash mixtures. Swell was measured during soaking of CBR samples. Ten-percent fly ash addition caused a swell decrease of 55% compared to loess alone. It is interesting to note that swell magnitude for the 10% samples increased with greater amounts of relative compaction. Samples containing 15% fly ash actually exhibited a 255% increase in swell potential compared to the loess soil. Zia and Fox (52) attribute this behavior to formation of ettringite, although the fly ash contained 3.6% SO_3.

Strength Gain Due to Addition of Self-Cementing Fly Ash

The most widely used application for self-cementing fly is to increase the strength of unsuitable or unstable subgrade materials. The strength of soils stabilized with self-cementing fly is usually determined from unconfined compression tests and CBR tests (25, 26, 38, 48). Generally clay soils have soaked CBR values from 1.5 to 5% (40), which results in very little support to the pavement structure. Ferguson (25) has shown that addition of 16% self-cementing fly ash increases the soaked CBR values of heavy clay soils into the mid 30s, which is comparable to gravelly sands (40). Zia and Fox (52) also found the CBR of loess was increased five times with 10% fly ash addition, but an ash addition rate of 15% showed lower CBR than the 10% mixtures. They theorized this was also due to the formation of ettringite. Unconfined compressive strengths of soils stabilized with self-cementing fly ash are typically on the order of 100 psi, but can be as high as 500 psi at seven days, depending on ash content and ash properties (25, 26). White (48) compacted an oxidized glacial till soil and a non-oxidized glacial till soil with 10% self-cementing fly ash at approximately -2%, +/- 0%, and +2% of optimum moisture content based on maximum density and allowed them to cure for 28 days. The data show that at 2% dry of optimum the compressive strength was 140 psi for fly ash-oxidized till and 160 psi for fly ash-non-oxidized till. When the mixtures were compacted near optimum the strength decreased to 85 psi and 135 psi for the oxidized till and non-oxidized till, respectively. The last set of samples was non-oxidized till and fly ash at 2% wet of optimum. The strength of these samples was approximately 100 psi. These samples were not soaked and show a trend of decreasing strength with increasing moisture content. A report from the ACAA (26) states that the optimum moisture content needed for maximum strength is typically 0% to 8% lower than optimum moisture content for maximum density.

Misra (36) states long-term strength gain is expected for Class C fly ash stabilized soils. Shrinkage cracks may occur over time and may be detrimental to strength development. His studies involving kaolinite mixed with 0, 2, 4, and 6% bentonite showed most strength gain occurred within 24 hours, and smaller increases were noted up to 7 and 14 days, but after 7 and 14 days, strength gain was retarded and strength actually began to decreases. Zia and Fox (52) also reported the majority of strength development occurs within 7 days of compaction for Indiana loess-fly ash mixtures, and between 14 and 28 days strength of stabilized loess decreased to the strength of the loess alone. Misra (36) theorized that the strength degradation was due to fly ash content and the amount of smectite material in the soil, and he observed that higher ash and smectite contents slowed the strength loss.

Zia and Fox (52) attributed the strength loss to shrinkage cracks that developed in the stabilized samples, which they observed to be more prominent at higher ash contents.

Khoury and Zaman (31) reported on the effect of wet-dry cycles on resilient modulus (M_r), elastic modulus (E), and UCS for Class C fly ash stabilized soft limestone aggregate. M_r was increased 55% for specimens cured 3 days prior to testing and subjected to 30 wet-dry cycles. Twenty-eight cured specimens exhibited an increase in M_r up to 12 cycles, at which time M_r began to decrease. They also observed that E and UCS values increase as the number of wet-dry cycles increase.

Strength gain, as well as compacted density, of self-cementing fly ash and soil is more sensitive to compaction delays than is the soil modification application (25). Ferguson (25) found that compaction delay can cause a pronounced decrease in the compacted unit weight and strength gain. As the ash hydrates, the fly ash soil mixture flocculates and agglomerates. While uncompacted, the mixture tends to become quite aggregated, therefore requiring more compactive effort to break down the cemented particles (26). Ferguson (25) has observed decreases in density of 10 pcf or more resulting from compaction delay. The loss of strength is probably due to the loss of cementitious reaction products used up during hydration and loss of interparticle contact points that result from a lower compacted density (26). Materials compacted immediately after mixing exhibit 6 to 12 times the strength of non-stabilized soils. Mixtures compacted at times exceeding one hour only show a 3 to 5 times increase in strength over non stabilized soils. This strength loss can be as much as 50% reduction in strength from the no compaction delay material, as reported by the ACAA (26). In addition Şenol et al. (43) report UCS of low plasticity clay and 20% fly ash decreased after 2-hour compaction delay, and CBR was reduced by 18% for 2-hour compaction delay.

Ferguson (25) and the ACAA (26) suggest that compaction of self-cementing fly ash stabilized materials be completed within two hours of initial mixing, and they have observed delay times of less than one hour during well planned construction operations. Şenol et al. (43) suggests strength can be maximized by stabilization at a mixture specific moisture content and minimizing compaction delays.

Hydrated Fly Ash and Conditioned Fly Ash

Due to the excessive volume of fly ash produced that is not utilized in other industries, some power plants store this material on site rather than waste it to landfills. This material can be stored one of two ways. One way to store the ash is to spread it in thin lifts, water it, and compact it, thereby producing hydrated fly ash (HFA). When this material is needed it can be reclaimed using pavement reclamation techniques. Another way to store the fly ash is to apply water to the fly ash in a pug mill and then stockpile it on site; this is termed conditioned fly ash (CFA). CFA is considered to be more reactive that HFA because only some of the raw ash has been hydrated, not all as in the case of HFA. CFA materials are typically excavated from the stockpiles using a front end loader.

The advantage of HFA and CFA materials is that they can be used as structural fill, pavement subgrade, and soil stabilizers. The major disadvantage from a stabilization standpoint is that these materials gain strength at a much slower rate than freshly hydrated self-cementing fly ash. When reclaimed, the HFA and CFA act similar to a lightweight aggregate (18). ISU has conducted extensive research on the engineering properties and uses of HFA and CFA. The engineering properties evaluated are moisture-density relationships, compressive strength, CBR, freeze/thaw durability, hydraulic conductivity, and volumetric stability.

Barnes (18) and later Mahrt (35) reported that the moisture-density relationships of HFA and CFA are typically flat curves without a pronounced peak as shown in Proctor curves for cohesive materials. Four sources of HFA materials and one source of CFA from Iowa show a range of maximum dry unit weight between 74 and 94 pcf, and optimum moisture contents from 22% to 37%.

The majority of strength testing by ISU personnel for these materials has been through the use of CBR tests, both soaked and unsoaked tests, and in some cases the HFA and CFA were activated by CKD, lime, or raw self-cementing fly ash. Testing of CKD as an activator was discontinued because it became commercially unavailable due to environmental concerns. Mahrt (35) reported molded moisture content does not appear to have an effect on CBR, but the level of compactive effort does. Sub standard compactive effort (~95% of standard Proctor) produces unsoaked and soaked CBR values around 40% while modified compactive effort yields values between 80 and 90%. Barnes (18) conducted research on the influence of an activator on CBR and unconfined compressive strength of HFA and CFA. An activator of 10% raw fly ash was shown to increase CBR values that were as low as 60% up to 110%, and 2.5% lime addition showed values over 200%.

Test results show unconfined compressive strengths for HFA and CFA materials are generally between 50 and 100 psi after 7 days, but shows gradual strength increase at 56 days due to pozzolanic reactions (18). HFA and CFA mixed with 10% raw fly ash have shown compressive strength increases as much as 300% and 2.5% lime addition has produced strengths 700% greater than those of untreated material as reported by Barnes (18).

The addition of an activator (CKD, lime, or raw fly ash), or the material being left untreated has no bearing on the amount of shrink and swell HFA and CFA exhibit. Berg (19) observed no problems related to volumetric instability for air cured or 100% saturated samples.

HFA and CFA have low freeze/thaw durability when tested in accordance with ASTM C593 [Standard Specification for Fly Ash and Other Pozzolans for Use With Lime], but the addition of lime and raw self-cementing fly ash as activators tends to greatly increase the freeze/thaw durability (19). Berg reports the activators are necessary to facilitate strength gain of the HFA and CFA (19).

Hydraulic conductivity of untreated HFA and CFA is in the range of 1×10^{-3} to 5×10^{-6} cm/s, which is comparable to silty clay soils (35). Mahrt (35) has shown that differences in hydraulic conductivity are dependant on compactive effort, molded moisture content of the material, and the source of the HFA and CFA. More compaction energy and higher molded moisture content tend to decrease hydraulic conductivity.

Hydrated Fly Ash/Conditioned Fly Ash and Soil

White (48, 47) has experimented on the feasibility of HFA and CFA as soil stabilizers. Addition of 20% Prairie Creek Generating Station CFA to oxidized glacial till increased crushing strength to 80 psi, while 20% Prairie Creek CFA addition to non-oxidized glacial till increased strength to 70 psi, compared to 60 psi for the non-oxidized till alone. Another study initiated by White (47) has shown that regardless of addition rate of HFA to both lean and fat clays, the strength increase was 2 to 3 times that of the soil, when compacted dry of optimum moisture content. The strength gain of HFA stabilized soils can be HFA material dependant as Ottumwa HFA showed strength increases of 8 to 12 times when compacted wet of optimum with fat clay, whereas Neal 3 HFA did not produce nearly the strength gain of the Ottumwa HFA (47). White's (47) results also showed that lean clay stabilized with HFA fines and prepared wet of optimum moisture content did not appear to have much of strength gain over about 2 times that of the natural soil. A small compaction delay study was completed with non-oxidized glacial till and 20% Prairie Creek CFA; this study showed it was actually beneficial for a long compaction delay (~8 hours) to facilitate maximum strength gain, although the reasons for this are not understood (48).

Expansive clays that have a high plasticity index can show a dramatic decrease in plasticity index and therefore swell potential with the addition of HFA fines due to mechanical stabilization as well as pozzolanic reactions. The expansive clay used in White's study originally had a very high swell potential (PI=47) which was reduced to medium swelling potential (PI=22) after HFA addition and a cure time of 28 days (47).

Chemistry of Self-Cementing Fly Ash

Chemical Composition and Reaction Mechanisms of Self-Cementing Fly Ash

Fly ash is the fine residue produced from burning ground or powdered coal (6). Fly ash is collected from the flue gas of coal fired boilers. Most self-cementing fly ashes are Class C as designated by American Society for Testing and Materials (ASTM) C618 [Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Concrete] and are in fine powder form, usually dark or light tan in color (18). Self-cementing fly ash is produced from the burning of low sulfur, subbituminous and lignite coals. The greatest percentage of self-cementing fly ash composition is from silica, alumina, ferric oxide, and calcium oxide. Class C fly ash typically has the composition shown in Table 1 and the chemical requirements of ASTM C618 are also shown in Table 1 (18, 6).

Table 1. Typical Chemical Compositions of Class C Fly Ash and ASTM C 618 Chemical Requirements for Class C Fly Ash

Oxide	Self Cementing Fly Ash (% of Total Weight)	ASTM C618
SiO_2	20-40	Summation between 50% and 70%
Al_2O_3	10-30	
Fe_2O_3	3-10	
CaO	10-32	
MgO	0.5-8	
Na_2O	0.5-6	
K_2O	0.5-4	
TiO_2	0.5-2	
SO_3	1-8	Maximum of 5%
LOI	0-3	Maximum of 5%

Fly ash particles are typically glassy spheres that contain some crystalline and carbonaceous matter (18). Fly ash is a pozzolanic material that ASTM (6) defines as materials rich in silica and alumina which in themselves have little or non self-cementing properties, but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties. Barnes (18) reports that the pozzolinity of fly ash is mainly dependant on the amounts of silica and alumina, presence of moisture and free lime, and fineness of the fly ash. The calcium in self-cementing fly ashes is mostly in the form of crystalline compounds of aluminates and silicates, which account for hydration characteristics that are more like portland cement rather than lime. Initial formation of cementitious reaction products is due to the hydration of tricalcium aluminate, which the ACAA (26) reports is the cause of problems during long compaction delay times. Strength gain at periods over 28 days is mostly attributed to the pozzolanic reactions between calcium oxide and the aluminous and siliceous materials in the fly ash.

Negative Reactions Resulting from Pozzolans and Sulfur

Due to environmental concerns, some power plants have converted over to fluidized bed combustion (FBC) or flue gas desulferization (FGD) to help remove SO_2 from the boiler exhaust streams. The effectiveness of these two procedures can be seen in the fact that the resulting fly ashes have more than 15% sulfate content (26). The ACAA (26) reports that FBC ashes can contain up to 35% SO_2 while FGD ashes contain SO_2 contents greater than 35%.

Problems are encountered when growth of crystals composed of sulfate compounds occur after incorporation of these high sulfate ashes into the material to be stabilized. As the calcium sulfate reactions proceed in the stabilized material, gypsum, ettringite, and thaumasite form and continue to form which produce long term expansion. Ettringite and thaumasite are formed by reactions of calcium, sulfates, alumina, silica, and water. Ettringite forms initially and these crystals occupy a volume over 200% of the volume the constituents once did. The secondary formation of expansive crystals is the conversion of thaumasite from ettringite, which takes a longer time and results in an additional 200% volume expansion (26). Thaumasite is formed at a lower temperature than ettringite, and by way of isomorphous substitution of the alumina in ettringite for silica. The amount of clay present and the pH of the soil are major factors involved in the formation of expansive materials. Addition of these high sulfur ashes typically raises the pH of the soil to around 12. The ACAA (26) stated that as the pH of a soil reaches about 10.5, the alumina and silica in clay particles becomes soluble, which provides a source of extra ions needed to form the expansive crystals.

The suitability of high sulfur, about 30%, FBC fly ash was evaluated in a limited study at ISU. White (48) observed that 56 days after samples were molded, delineation and expansion were observed, and at 90 days the FBC stabilized specimens had shown a volumetric expansion of 35% compared to original molded volume. It has been noted that fly ashes meeting the requirements of ASTM C 618 for sulfur content (<5%) show no evidence of potential expansion problems, while fly ashes with sulfur contents of 5% to 10% may be beneficial to construction, as the sulfates tend to retard the initial set of the ash due to tricalcium aluminate (26). The ACAA provides the following guidelines for stabilization with high sulfur ashes (26):

- Ashes with sulfur content in the range of 5% to 10% should be considered expansive until laboratory testing proves otherwise.
- High sulfur ashes with sulfur contents greater than 10% should not be used for stabilization operations.
- In addition to sulfur content of the fly ash, soluble sulfates in both the soil and groundwater used for the project must be considered. These can also influence the swell potential of the stabilized mixture.

- Saturated conditions make the ions needed for crystal growth more mobile, whereas non saturated conditions tend to slow crystal growth.
- The potential for problems related to swell increases with increasing clay and colloids content.

Reports are available concerning projects that had catastrophic results when calcium based stabilizers, usually lime or portland cement, were used: (1) To stabilize sulfate bearing clay soils, (2) In areas where there is a supply of fresh groundwater that continues to provide available sulfates, or (3) When the mixing water for the project contained high amounts of soluble sulfates (29, 34, 39). Hunter (29) reported on problems that arose in Las Vegas when sulfate bearing clay soils were stabilized with lime. The sulfate content of the clay was high enough that lime should not have been used to begin with, but the sulfate content of the soil alone was not large enough to account for the all of the measured expansion. The extra sulfates needed for the expansion were provided through a granular backfilled utility trench that ran along the length of the project, which provided excess dissolved sulfates to the stabilized areas through groundwater.

The Texas Department of Transportation has investigated sulfate bearing soils and calcium induced heave. The soils in Texas have a high concentration of gypsum ($CaSO_4 \cdot 2H_2O$), which is a precursor to the formation of ettringite and thaumasite. One project involved ettringite formation as rain water infiltrated the subgrade that was stabilized with lime and Type II portland cement. Areas on the same project that were not stabilized did not exhibit swelling related problems (34).

The other major project dealt with a double application of lime. Heaving of the subgrade stabilized with the first lime treatment was observed within 6 months of project completion. Kota et al. (34) hypothesized that gypsum laden water was entering the subgrade, therefore causing the formation of ettringite. After laboratory testing, it was recommended that the pavement be removed and an additional treatment of lime be applied. As with the first treatment, heave was again observed within 6 months, destroying the pavement structure. Kota et al.(34) provides some ideas to prevent the destruction caused by sulfates and calcium based stabilizers:

- Double application of lime.
- Low calcium stabilizers such as cement and fly ash.
- Non calcium stabilizers
- Geotextile or Geogrid soil reinforcement.
- Stabilizing the top with non sulfate select fill.
- Pretreatment with barium compounds.
- Asphalt stabilization of the sulfate bearing soils.
- Compacting to lower densities.

Rollings et al.(39) examined a project in Georgia that involved a cement stabilized sand base course material that was mixed off-site at the sand borrow pit. As in the Texas examples, sulfate induced heave was evident within 6 months after construction. A preliminary investigation provided no definite answers as to why the base course heaved. Sulfur was not present in the cement used or in the sand. Closer inspection showed the mixing water used at the off-site mixing plant contained over 10% sulfur, and the water was also a major contributor of calcium. When the cement was added, the pH increased to about 12 and the alumina and silica in the soil became soluble, leading to the formation of ettringite.

Case Histories Involving Self-Cementing Fly Ash Stabilization

Self-Cementing Fly Ash Stabilization for an Industrial Road, Missouri

The project, completed in 1973, was located in Kansas City, Missouri and involved an industrial road underlain by clay soils with a liquid limit of 65, plasticity index of 43, and low CBR value of 3.5. Initial design of the pavement was 12 inches of full depth asphalt. However, it was desired to reduce the pavement thickness by improving the subgrade.

Laboratory testing using a fly ash content of 15% from Hawthorne Power Station showed decreases in liquid limit and plasticity index to 45 and 18, respectively. The 28 day unconfined compressive strength of the stabilized clay soil was 7 times that of the native soil. Mixing took place on grade, in two 4.5-inch layers. Field CBR values increased to 9% unsoaked and 12.5% soaked. The increase in CBR allowed pavement thickness to be reduced to 9 inches, and as of 1975 the pavement was holding up to light traffic loads (27).

Low Cost Fly Ash Stabilized Sand, Des Moines County, Iowa

The main objective of this Iowa Department of Transportation project was to develop a low cost fly ash stabilized roadway using locally available unprocessed sand. The project was county road H-40 in Des Moines County. The roadway is located adjacent to the Mississippi River Levee and traffic was estimated at 27,000 ESALs. The mixture for the project consisted of 5.1% Type I cement, 13.7% Ottumwa Class C fly ash (23% CaO), sand, and water. The grade was prepared in July 1984. Construction of the base course began on August 1, 1984 and was completed on August 4, 1984.

The mixture was mixed off-site in a central plant mixer, then transported to the site in dump trucks and placed in front of a subgrade trimmer. Compaction of the mix was difficult at times due to the material shoving under the roller. Average density was 97.6% of standard Proctor and strength testing was conducted by coring the base at 14, 28, 91, and 313 days. Strengths were greater than typical lime-fly ash mixtures. The main objective, low cost, was not met. After the road had been through two years of heavy tractor-trailer traffic, it was noted that an overlay would be necessary (32).

Recycled Pavement, Shawnee County, Kansas

This 1.5 mile section of 93rd Street is considered a rural road but carries a high volume of truck traffic. The existing thickness of the road material varied from 1 to 6 inches for the asphalt surface and 1 to 8 inches for the granular base overly a clay subgrade. The mix design was for 18% class C fly ash to be added to the pulverized pavement and base materials at a moisture content of 10%.

Starting in June 1987, the pavement and base was pulverized to a 6 inch depth and lightly compacted. The fly ash was spread on the surface and mixed in with a Bomag MPH 100; water was added through nozzles in the mixing drum. Initial mixing was completed with a vibratory padfoot roller and final compaction was completed with a smooth drum or pneumatic rubber tired roller. The stabilized section was kept moist for 5 days before a layer of cold mix asphalt was placed. Two months after the asphalt was placed a chip seal surface was applied. The road was in excellent shape after 4 years of service (27).

Construction of the Heartland Park Race Track, Topeka, Kansas

Soils on the site of the proposed race track were classified as lean clay, weathered shale, and fat clay. Stabilization of the on-site soils was needed to reduce volume change potential, increase shear strength, and reduce pavement thickness. The self-cementing fly ash was from Kansas Power and Light Jeffrey Energy Center and had calcium oxide content between 28% and 33%. Fly ash contents of 14%, 16%, and 18% were evaluated, and moisture-density and moisture-strength testing was completed at compaction delay times of 0 and 2 hours. A fly ash content of 16% was chosen for the project, with a moisture range of 0% to 4% above optimum for maximum compressive strength. The completed stabilized section was 12 inches thick. Construction of the subgrade started in October 1988 and finished in December of that year. The soil temperature was closely monitored during construction. Field monitoring included nuclear density testing and sample molding of the mixture in the field to monitor strength gain of the production mix. The areas that were stabilized when the temperature was below 4.4°C required more passes of the compaction equipment but these areas have remained stable. As of 1992, the race track pavement was in excellent condition and performing at the expected level (25).

Northwest Highway Fly Ash Stabilization, Oklahoma

The material on the site was sandy clay that required stabilization. The design engineers specified an ash addition rate of 15%, and this was to be initially mixed to a depth of 8 inches with the subbase. After preliminary mixing, water was sprinkled on the mixture and immediately following was a second pass of the mixing equipment. The compaction window on the project was 4 to 6 hours. Compaction was completed the next day, and this compacted stabilized layer was trimmed to meet project requirements. The stabilized subgrade was finished of with a 10-inch hot mix asphalt layer (22).

Power Plant Access Road, Marshalltown, Iowa

Construction began in June 1994 on a 1700-foot long by 22-foot wide access road to the Sutherland Generating Station in Marshalltown, Iowa. The road was constructed on a 10-inch thick base of CFA from the Prairie Creek Generation Station in Cedar Rapids, Iowa. Since the base material had been previously conditioned, the project called for an activator to be used to promote the pozzolanic reactions. Cement kiln dust (CKD) and atmospheric fluidized bed combustion (AFBC) residue were both used as activators on the project. The activators were mixed at 15% by dry weight of CFA. The CKD was used on 1000 feet of the access road. For this portion of the project, the CFA was placed on-site and then the CKD was spread over it. Next a reclaimer mixed the CFA, CKD, and water together to a loose depth of 12 inches. This mixing process was repeated until the proper moisture content for compaction was reached, at which time the mixture was compacted using a padfoot roller first and then a smooth steel drum roller for final compaction. The compacted section was kept in a moist condition until paving. The 700-foot long AFBC section was completed in much the same manner except that the CFA was prewetted prior to application of the AFBC, and water was again applied after the first pass of the reclaimer. Compaction of the AFBC was the same as the CKD section and the compacted AFBC section was also kept in a moist condition. Final surfacing was a 2-inch chip seal. Beginning in November 1994, ISU personnel have extracted cores of the base material annually through July 2002. The AFBC became unrecoverable several years ago and recently the CKD cores have shown horizontal delamination near the top and vertical cracks that propogate down through the samples. The cause of the cracks is believed to stem from high vehicle loads and freeze/thaw damage. The freeze/thaw damage is probably due to a decrease in permeability of the stabilized material. Now the materials are behaving similar to a Macadam base. The cores recovered in 2002 still had compressive strengths of 970 psi. Overall the pavement is performing well with some areas along the turning radii of the road having to be resurfaced with hot mix asphalt in early 2002 (51).

Landfill Access Road, Ottumwa, Iowa

The Ottumwa-Midland Landfill is located 5 miles north of Ottumwa, Iowa. Construction of the road base took place from May 30 to June 1, 1995. The road is 2500 feet long, and had 1800 feet of CKD (10% by dry weight) stabilized HFA base and 700 feet of AFBC (15% by dry weight) stabilized HFA base constructed. The stabilized HFA was placed on a 4-inch aggregate subbase, which was on top of a 12-inch fly ash stabilized subgrade. The project began in April 1995 with clearing and grubbing, cut and fill operations, stabilization of the subgrade, and placement of the aggregate base. The HFA and activators were mixed at the Ottumwa Generating Station. The activators were spread on the compacted HFA and a reclaimer mixed the materials to a depth of 8 inches. A loader was then used to stockpile the reclaimed material. The mixtures were then hauled to the site and spread on the aggregate subbase, at which time water was applied and final mixing was completed. A 50 ton double drum roller was used for initial compaction and final compaction was achieved by use of a smooth drum roller. The compacted material was kept moist by the use of an asphalt prime coat. A 1.5-inch asphalt concrete surface was applied after the base had been allowed to cure for one week. Coring of the base has been completed annually since August 1995 by ISU personnel. A maximum compressive strength of 2235 psi was reached in 1997. The strength has decreased since 1997, but the 2002 cores still had an average compressive strength of 2055 psi. The asphalt surface is showing longitudinal cracking in both the AFBC and CKD sections. Breakdown of the activated HFA base is causing the material to behave as a Macadam base. Overall the road is still performing well (51).

Hydrated Fly Ash as Select Fill, Chillicothe, Iowa

This project is a 4.43-mile road that starts at the Ottumwa Generating Station, just outside of Chillicothe, Iowa, and runs west to the Monroe-Wapello County line. No select soils were available for construction of the road; as an alternative HFA from the Ottumwa Generating Station was used as select fill. The original plan called for the use of Class 10 subgrade, which is not preferred if other options can be utilized. Two sections of the road that totaled 3.1 miles were constructed on HFA, while the remainder was constructed on Class 10 soils, which served as a control section. The HFA was compacted to a depth 12 inches and extended the full width of the roadway, including the shoulders. A sheepsfoot roller was used for initial compaction and a steel or pneumatic rubber tired roller completed the compaction process. Overlying the HFA was 9.5 inches of PCC. Construction was completed in the fall of 1999. Annual monitoring of the project includes visual observations, DCP testing, Road Rater testing, and Roughness testing. In April 2000 it was observed that the pavement has been milled at several locations along its length to improve the surface smoothness. The HFA shoulder sections are in good shape with no, vegetation, erosion, or settlement observed. The shoulders in the control section have shown excessive erosion, settlement, and vegetative growth. The DCP testing program has been carried out on both shoulders and under the mainline pavement structure. The CBR of the Class 10 subgrade under all areas of the project was shown to average 12%. The HFA on the shoulders and under the mainline pavement had a range of CBR values between 48 and 98%, which is in close agreement to the value of 55% used in the pavement thickness design. The Iowa DOT has performed Road Rater testing annually on this project, with the exception of the year 2002. Based on the Road Rater testing, the HFA sections are behaving like there is between a 10.4 to 12.2 inch thick PCC slab, while the equivalent thickness for the control section is in the range of 9.1 to 10.5 inches. The HFA sections are providing very good structural support to the slab. Roughness testing results are in the range of 1.47 to 1.58 m/km for the pavement. This is in the high range for a new PCC pavement. It is expected that results comparing the HFA and control sections will be of better use in the future. Overall the pavement is performing very well (50).

Fly Ash Stabilization of RAP, Waukesha County, Wisconsin

Highway JK in Waukesha County, Wisconsin is a ¾-mile long county road that lies in a low area with very silty subgrade soils underneath. The silty nature of the underlying soil and large amounts of available water has led to problems with frost heave in the past. Construction of the new road base began in October 2001. The idea for fly ash RAP stabilization was thought of because most of the time the poor soils were mixed with breaker run, which is expensive. Fly ash RAP stabilization was much more cost effective for this project. The existing asphalt was pulverized to a depth of 6 inches, at which time a water truck was used to water the milled material and a second pass of the mixing equipment pulverized the material to a 12-inch depth. The water content for the project was 6%. Fly ash was added to the RAP at 8%, and the final pass of the mixer was made. Initial compaction was completed with a vibratory sheepsfoot roller in less than half an hour. Final compaction was made with a smooth drum roller. The compacted stabilized RAP was allowed to cure for 24 hours before 5 inches of E-3 Superpave mix was laid down. The following winter, no problems from frost heave were observed (28).

Fly Ash Stabilization of City Streets, Overland Park, Kansas

The city of Overland Park, Kansas required fly ash stabilization for soils with a liquid limit greater than 40 and a plasticity index over 25. Stabilization work began in 1993 and became mandatory in 1996. During the late spring of 2002 field testing using the DCP was completed on 12 existing stabilized subgrades in Overland Park, with the oldest being 9 years. Overall the test results on the stabilized subgrades show final CBR strength values of 140% to 350% of the original strength, as measured by the unstabilized underlying soils. No correlation existed between CBR and the age of the subgrade. Observations of the streets were made during testing, and it was noted that the streets are in good condition (38).

Fly Ash RAP Stabilization of Parking Lots, Ames, IA

The parking lots surrounding Iowa State University's Jack Trice Stadium were in poor shape in 2002. The stadium is located in the floodplain of the South Skunk River. The soil under the parking lots was classified as clayey sand, and had high moisture content, making construction operations nearly impossible. The consulting engineer specified fly ash

RAP stabilization for the reconstruction of the parking lots around the stadium. Ash addition rate was 10%, and optimum moisture content was between 10% and 11%. Construction of the project began and was completed in the summer of 2002. The existing pavement was milled in place and then leveled with a motor grader. Water was then placed on the pulverized material and fly ash was then added on top of the mixture. The reclaimer made a final pass, and immediate compaction was completed with a vibratory padfoot roller. Compaction was finished with a smooth drum steel roller to seal the surface. The stabilized material provided a very stable paving platform. Field testing was completed with DCP tests and unconfined compression testing of samples molded in the field (41).

SOURCES AND MATERIAL PROPERTIES OF SOIL, RAW FLY ASH, AND HYDRATED AND CONDITIONED FLY ASH

Material Sources

Soils

Turin, Iowa Loess

Silty Western Iowa loess was sampled from containers located in Town Engineering Building on the Iowa State University campus. Two 20-gallon bins of material were transported to Spangler Geotechnical Laboratory in February 2002. The loess was originally collected from a site in Monona County, near Turin, Iowa. This material is from the loess hills.

Neola, Iowa Alluvium

Three 18-gallon containers of alluvium from a creek bed were collected in late August 2001 from a research project site outside of Neola, IA in Pottawattamie County. The creek is located in the loess hills of Western Iowa, and the alluvium was derived from loess that covers the region.

Le Grand, Iowa Loess

Central Iowa loess had been sampled by previous ISU personnel and was stored at Spangler Geotechnical Laboratory. Eight 18-gallon containers of the material were available for use. The samples were collected near Le Grand, Iowa and have higher clay content than the loess collected near Turin, Iowa.

Cedar Rapids, Iowa Glacial Till

Iowa State University research associates sampled the glacial till material during a research project on Highway 151 in the summer of 2000. Approximately 20 gallons of the material were available for use. The location of the research project was northeast of Cedar Rapids, Iowa. This location is located on the Iowa Erosional Surface.

Argyle, Iowa Paleosol

During October of 2001, an Iowa State University graduate student collected roughly 50 gallons of the Southeast Iowa paleosol, with the aid of Iowa DOT personnel. The material came from an area about 2 miles north of Argyle, Iowa, in Lee County, and was part of Highway 218 expansion. This area is located in the Southern Iowa Drift Plain.

Self-Cementing Fly Ash

Ottumwa Fly Ash

Four five-gallon buckets of Ottumwa Generating Station (OGS) Class C fly ash were delivered to Spangler Geotechnical Laboratory on February 23, 2001. This power plant is located near Chillicothe, Iowa. Ottumwa Generating Station burns subbituminous coal from the Powder River basin in Wyoming. An additional four five-gallon containers was delivered to ISU on August 21, 2001. This material was used to stabilize the soils listed above.

Council Bluffs Fly Ash

Class C fly ash from Council Bluffs Generating Station (CB), near Council Bluffs, Iowa was delivered in four five-gallon containers to Spangler Geotechnical Laboratory on February 23, 2001. The fly ash came from boiler #3 at the plant. This material is formed by the burning of Wyoming subbituminous coal. Four additional five-gallon containers were delivered to ISU on August 21, 2001. This fly ash was used to stabilize the soils collected throughout the state.

Louisa Fly Ash

Louisa Generation Station (LGS), located near Muscatine, Iowa, burns Wyoming subbituminous coal, which produces Class C fly ash. Twenty gallons of this fly ash were delivered to Spangler Geotechnical Laboratory on February 23, 2001. This material was used as a soil stabilizer during testing. An additional twenty gallons was delivered to ISU on August 21, 2001.

Ames Fly Ash

One five-gallon container of Ames power plant self cementing fly ash was delivered to ISU in the early spring of 2002 by ISG personnel. The Ames power plant burns Wyoming subbituminous coal but also has 10% of the fuel stream consisting of refuse derived fuel (RDF) from the Ames Resource Recovery plant. Ames ash cannot be classified as Class C due to the additional material in the fuel stream, but it does have self cementing properties allowing it to be used as a soil stabilizer.

Prairie Creek Fly Ash

The Prairie Creek Generating Station is located near Cedar Rapids, Iowa. Prairie Creek unit #3 is marginal Class C fly ash due to the coal burned. Unit #4 at the station burns subbituminous coal from Wyoming. Prairie Creek Station combines the ash produced from units #3 and #4 into one silo, thereby producing a marginal Class C fly ash. A five-gallon sample of the combined ash from units #3 and #4 (PC3+4) was delivered to ISU on May 10, 2002. Prairie Creek fly ash was used as a soil stabilizer during stabilization testing.

Port Neal Fly Ash

The Port Neal Generating Station produces Class C fly ash by burning subbituminous coal from Wyoming. Port Neal is located near Sioux City, Iowa. Two units (#3 and #4) are used at the plant. Four five-gallon containers of both Port Neal #3 (PN3) and Port Neal #4 (PN4) were delivered to Spangler Geotechnical Laboratory on February 23, 2001. An additional 10 gallons of each fly ash was delivered to ISU on September 10, 2001. Both of these fly ashes were used to stabilize the soils used in the testing program.

Sutherland Fly Ash

Sutherland Generating Station (SGS) is located in Marshalltown, Iowa. Sutherland Station produces marginal Class C fly ash due to the stoker fired boilers, which cause the fly ash to have sulfur contents a little greater than 5%. A five-gallon sample was delivered to ISU on December 13, 2001. SGS fly ash does exhibit self cementing properties, allowing to be used as a potential soil stabilizer.

Hydrated and Conditioned Fly Ash

Ottumwa Hydrated Fly Ash

Iowa State personnel had previously sampled OGS HFA on November 6, 1998 from a previously milled stockpile located at the power plant. Four thirty-gallon containers of the material were collected in 1998, and two of the cans were available for use in this study. An additional 18 gallons of OGS HFA was collected during the summer of 2002 while testing placement of HFA on US Highway 63 near Eddyville, Iowa.

Port Neal Hydrated Fly Ash

Three thirty-gallon cans of hydrated Port Neal #3 fly ash were collected from a stockpile at the Port Neal Generating Station on March 23, 1999 by ISU personnel. Sixty gallons of PN3 HFA were left over from a previous study and available for testing.

Council Bluffs Hydrated Fly Ash

Personnel from ISU collected CB HFA on March 23, 1999 from a stockpile located at the Council Bluffs Generating Station. Two thirty-gallon cans of this material were available for this study.

Louisa Hydrated Fly Ash

Louisa HFA material was collected on May 20, 1999 by ISU personnel. The material was sampled from a previously milled stockpile located at the power plant and sixty gallons of material were left for this study.

Prairie Creek Conditioned Fly Ash

Twenty gallons of PC CFA were delivered to ISU in early June, 2001. The PC CFA was derived from a combination of fly ash from Units #3 and #4 and was collected from stockpiles at the Prairie Creek Generating Station.

Sutherland Conditioned Fly Ash

Two five-gallon buckets of CFA from the Sutherland Generating Station were delivered to Spangler Geotechnical Laboratory in late July, 2001. The material was sampled from stockpiles located on the Sutherland Station property.

Material Properties of Soils

Grain Size Distribution

Grain size analysis of the test soils was conducted according to ASTM D422 [Standard Test Method for Particle-Size Analysis of Soils] (7). Results are shown in Figure 2. The percentages of gravel, sand, silt, and clay for each material are summarized in Table 2.

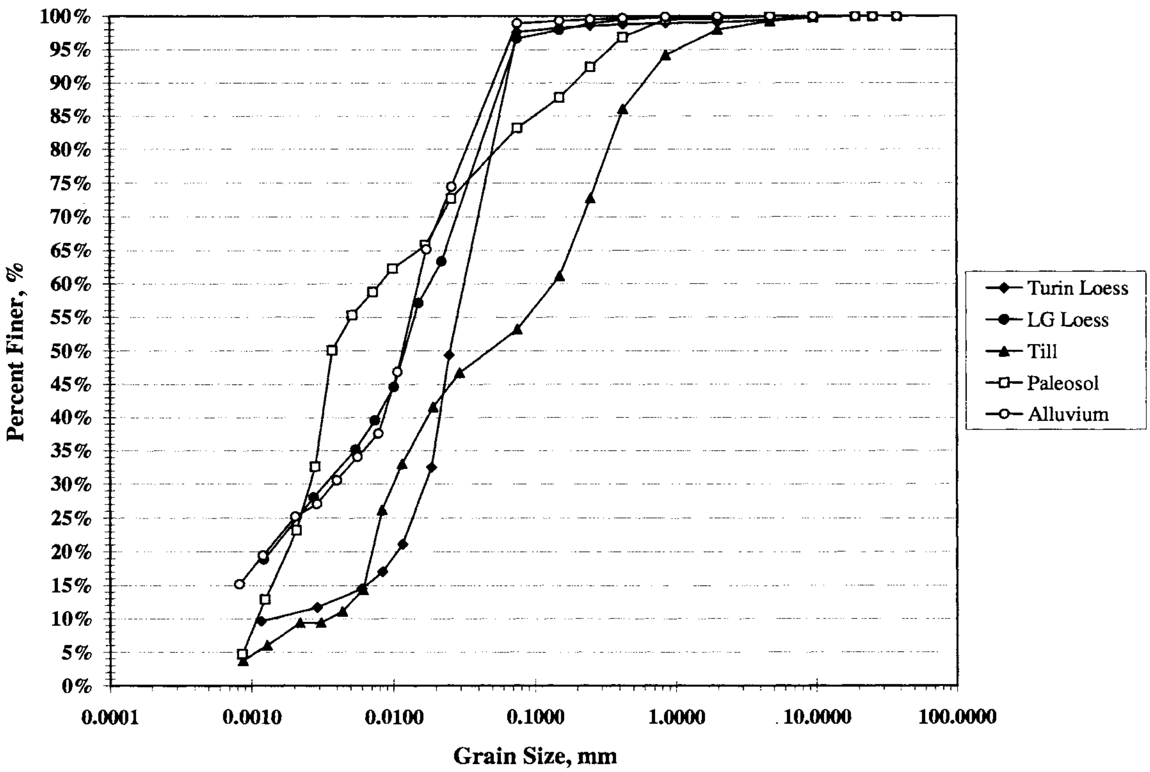


Figure 2. Particle Size Distribution of Soils Used in Study

Table 2. Summary of Soil Properties

Property	Turin Loess	Le Grand Loess	Cedar Rapids Till	Argyle Paleosol	Neola Alluvium
LL	33	40	40	48	47
PL	29	21	17	17	22
PI	4	19	23	31	25
G_s	2.74	2.68	2.70	2.74	2.80
% Gravel	1%	0%	2%	1%	0%
% Sand	1%	3%	45%	16%	1%
% Silt	87%	70%	44%	60%	74%
% Clay	11%	27%	9%	23%	25%
USCS	ML	CL	CL	CL-CH	CL-CH
Group	Low Plasticity Silt	Lean Clay	Lean Clay	Lean to Heavy Clay	Lean to Heavy Clay
AASHTO	A-4 (5)	A-6 (20)	A-6 (9)	A-7-5 (26)	A-7-6 (28)

Plasticity Characteristics and Engineering Classification

High plasticity is characteristic of many Iowa soils. Atterberg limits were determined in general accordance with ASTM D4318 [Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils] (15). Results are shown in Table 2. The Turin loess has little plasticity while the alluvium and paleosol soils have the highest plasticity. The paleosol is the remnant of an ancient B horizon, while alluvium is typically comprised of silts and clays. The Le Grand loess sample has more clay than the Turin Iowa loess sample, most likely due to its origin from the smaller Iowa River floodplain. The soils were classified in general accordance with ASTM D2487 [Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System)] (14) and AASHTO [Classification of Materials for Subgrades and Granular Type Roads] (24). The USCS and AASHTO classification symbols as well as the USCS group names are shown in Table 2.

Specific Gravity

Soils were tested in accordance with ASTM D854 [Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer] (10) to determine specific gravities. The specific gravity values shown in Table 2 ranged from 2.68 to 2.80. Specific gravity of 2.80 for the alluvium is high but not uncommon for fine-grained soils.

Clay Mineralogy

An extensive study was undertaken to determine the types of clay minerals present in the soils. Testing consisted of x-ray diffraction (XRD), thermogravimetric analysis (TGA), differential thermal analysis (DTA), cation exchange capacity (CEC), scanning electron microscope (SEM), and energy dispersive spectrometry (EDS). All of these tests were conducted on material smaller than 2 μm. Clay sized material was collected by decanting off the clay-water slurry that was left after silt and fine sands had settled out in a hydrometer cylinder. Water in the slurry was removed through the use of a filter candle, and then the remaining clay sludge was allowed to air dry.

The diffractograms from the XRD analyses are located in Appendix A. All clay fractions showed quartz having the largest intensity, which was expected due to its resistance to weathering. Montmorillonite was the dominant clay mineral in all samples. All four samples also contained lesser amounts of kaolinite, which is clay in an advanced state of weathering. Kaolinite has had very little time to form due to past glaciations and unstable landforms of this part of the continent. The alluvium also contained some illite, which is a clay mineral derived from micas. The mica was likely derived from the northern US and Canada and was transported to the area by the Missouri River.

The CEC data is shown in Table 3. It is assumed that all exchange sites were filled before testing was conducted and that the exchangeable cations only consisted of potassium, magnesium, calcium, and sodium. Duplicate tests were completed for each soil and the results were in good agreement. The CEC for the various clay fractions seem relatively low for the clay minerals present. Smectite minerals, such as montmorillonite typically have CEC values of atleast 80 meq/100g. The values reported in Table 3 are typical of kaolinite and illite, which have low CEC due to little isomorphous substitution and fixed basal spacing. The reasons that the CEC data does not reflect the XRD findings are not completely understood.

SEM images provide some indication of the type of clay minerals present in a sample. Figures 3, 4, 5, and 6 show the Le Grand loess, alluvium, till, and paleosol clay fractions, respectively. These images were taken at 20,000x magnification. All samples show some thin, wavy particles, typical of montmorillonite, as well as thicker, blockier type particles, which are typical of kaolinite.

Table 3. Summary of Cation Exchange Capacity (CEC) for Soil Clay Fractions (< 2 microns)

Soil	K, ppm	Ca, ppm	Mg, ppm	Na, ppm	CEC, meq/100g
Glacial Till	306	14	2.3	8	9
Glacial Till	308	14	2.3	8	9
Alluvium	688	15	3.4	12	19
Alluvium	702	16	3.5	13	20
Paleosol	451	13	4.8	28	14
Paleosol	435	13	4.7	28	13
Le Grand Loess	354	12	3.1	9	10
Le Grand Loess	330	12	3.0	7	10

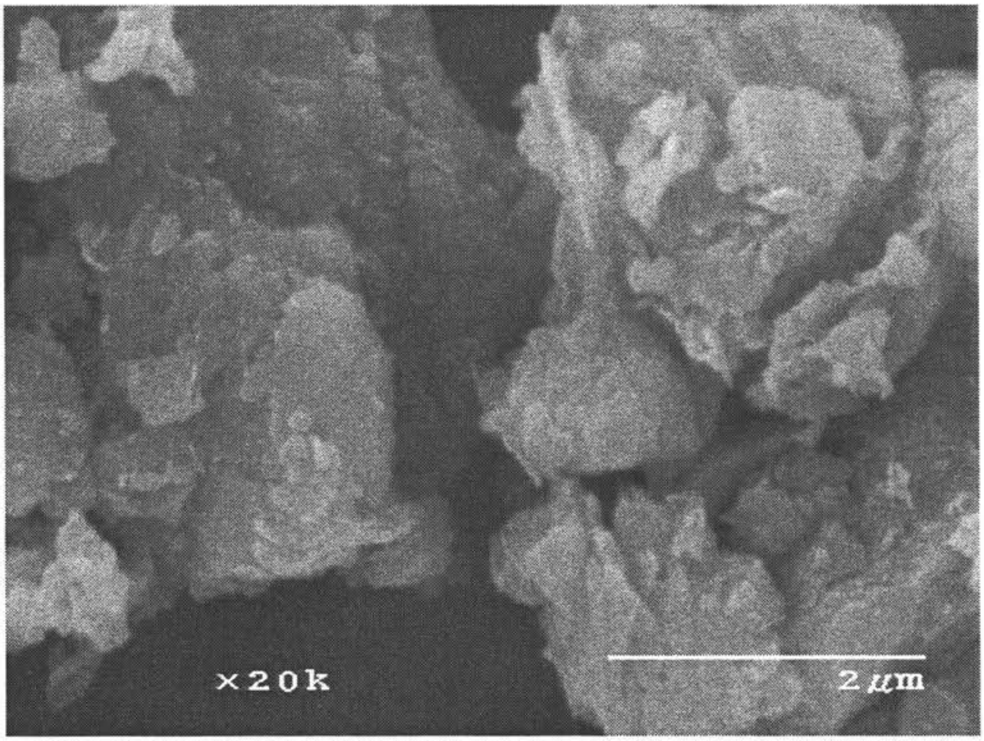


Figure 3. SEM Image at 20,000x of Le Grand Loess Clay Fraction (< 2 microns)

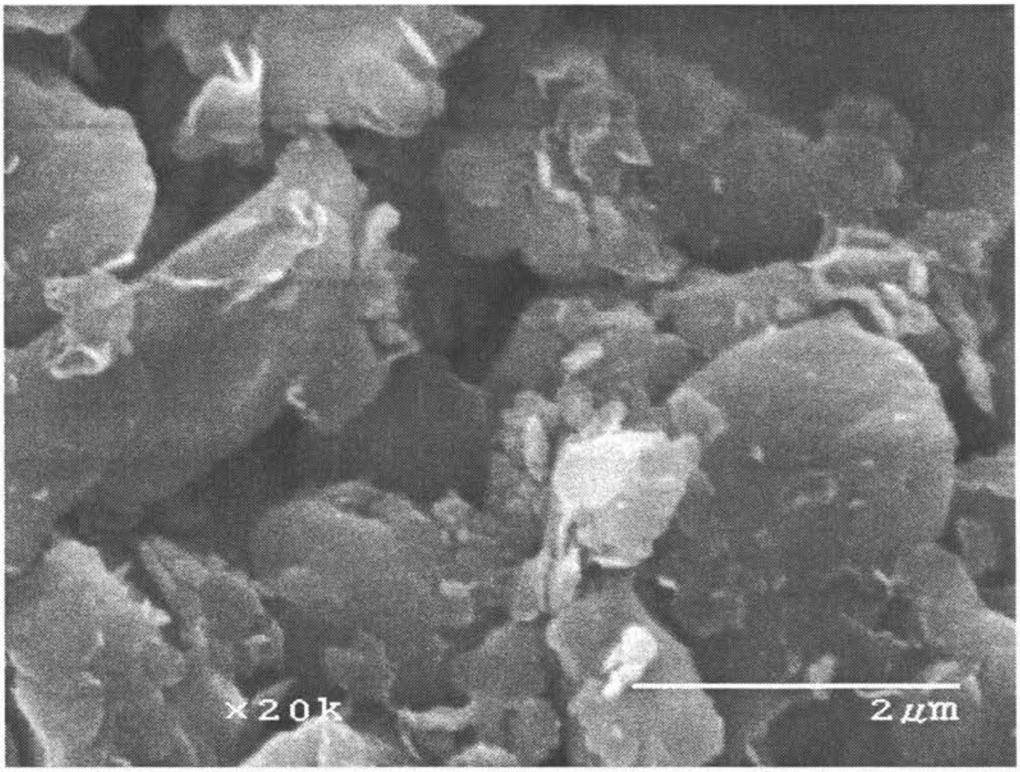


Figure 4. SEM Image at 20,000x of Alluvium Clay Fraction (< 2 microns)

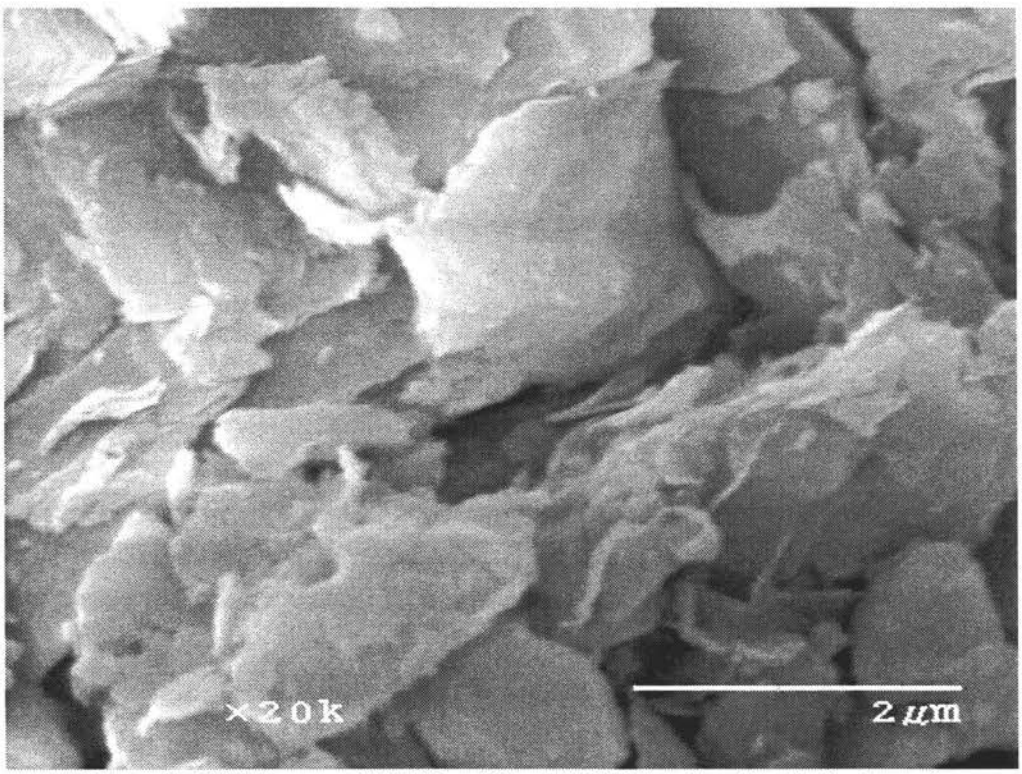


Figure 5. SEM Image at 20,000x of Glacial Till Clay Fraction (< 2 microns)

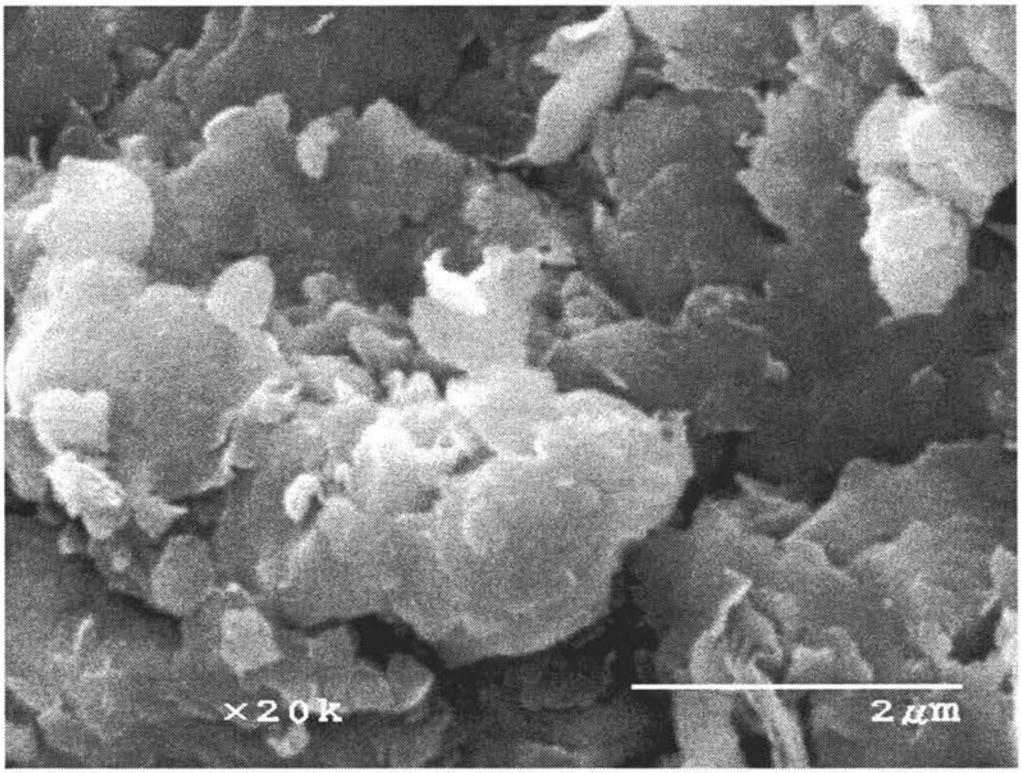


Figure 6. SEM Image at 20,000x of Paleosol Clay Fraction (< 2 microns)

EDS testing is conducted by use of SEM apparatus. Characteristic radiation is produced when the sample is bombarded with high speed electrons, and the results produce elemental maps. The maps are shown in Figures 7, 8, 9, and 10. Each map was produced at 3000x power of the picture in the upper left corner of each figure. Clay minerals are comprised of silicon, aluminum, magnesium, and oxygen. Each map shows that these are the elements most common to each soil, which was expected due to the clay fraction being tested. Calcium, sodium, and potassium are also present, most likely located on exchange sites of clay particles. Some carbon is present due to organic matter. Titanium and iron oxides showed up in the samples as well.

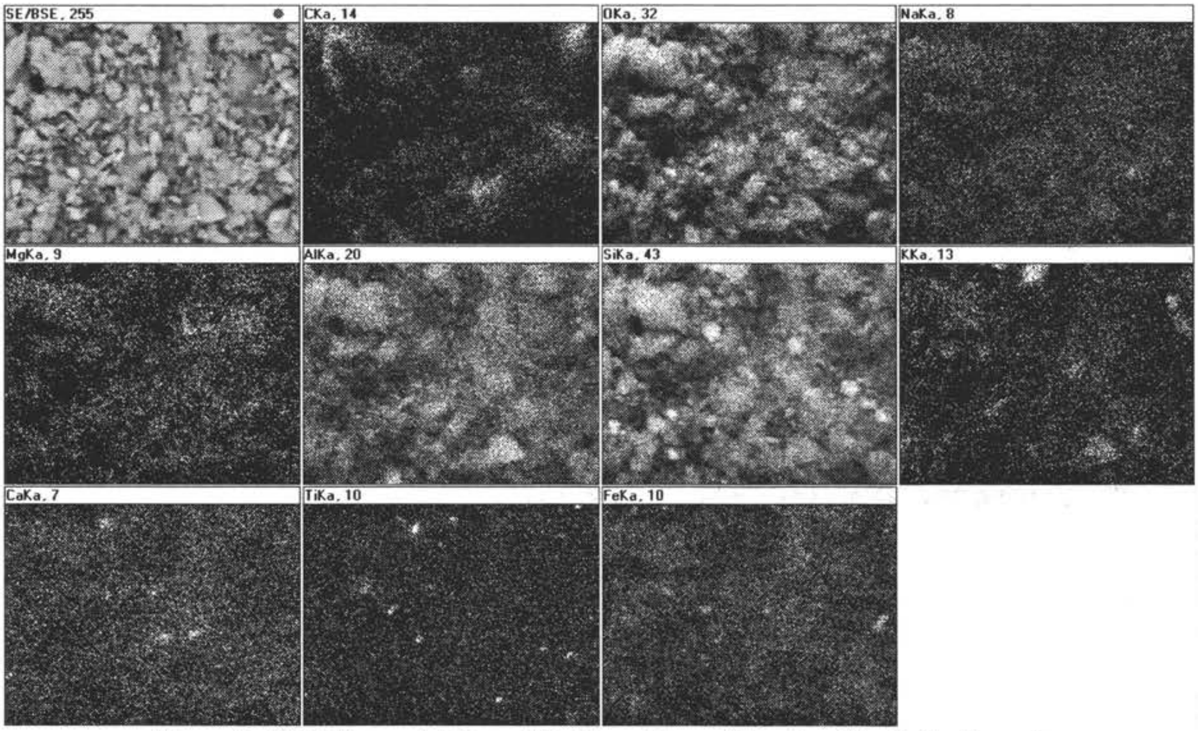


Figure 7. EDS Elemental Map of Le Grand Loess Clay Fraction (< 2 microns)

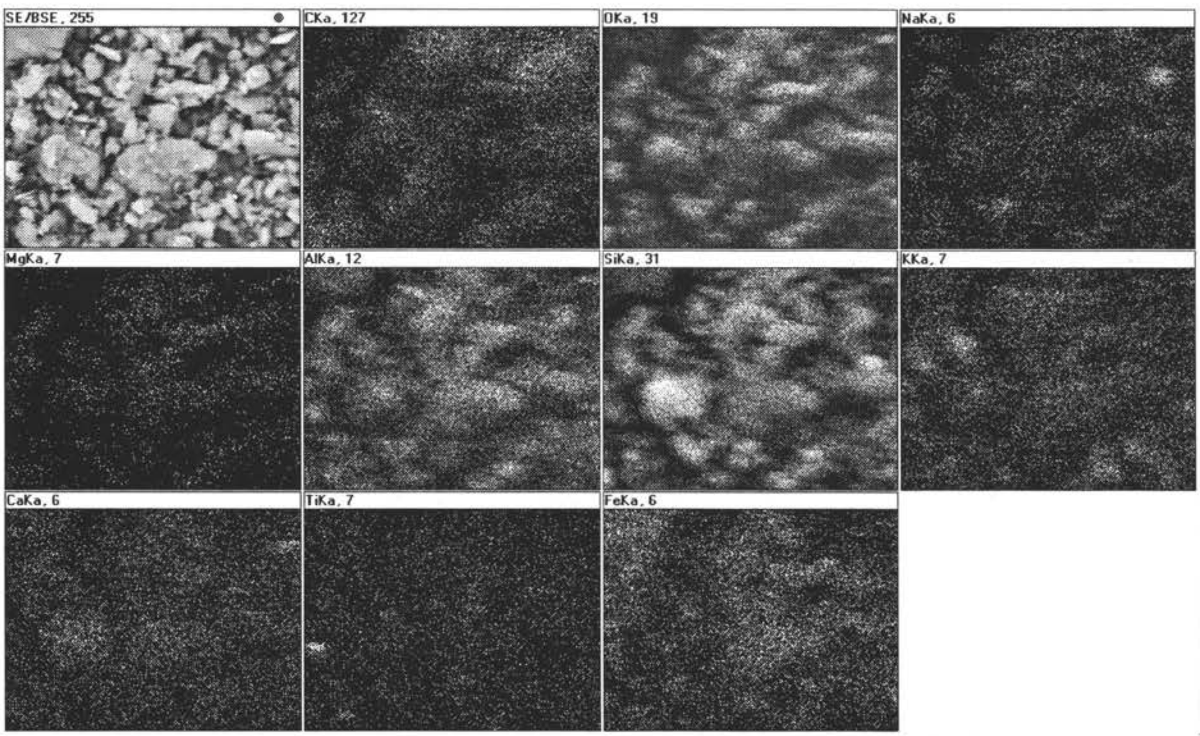


Figure 8. EDS Elemental Map of Alluvium Clay Fraction (< 2 microns)

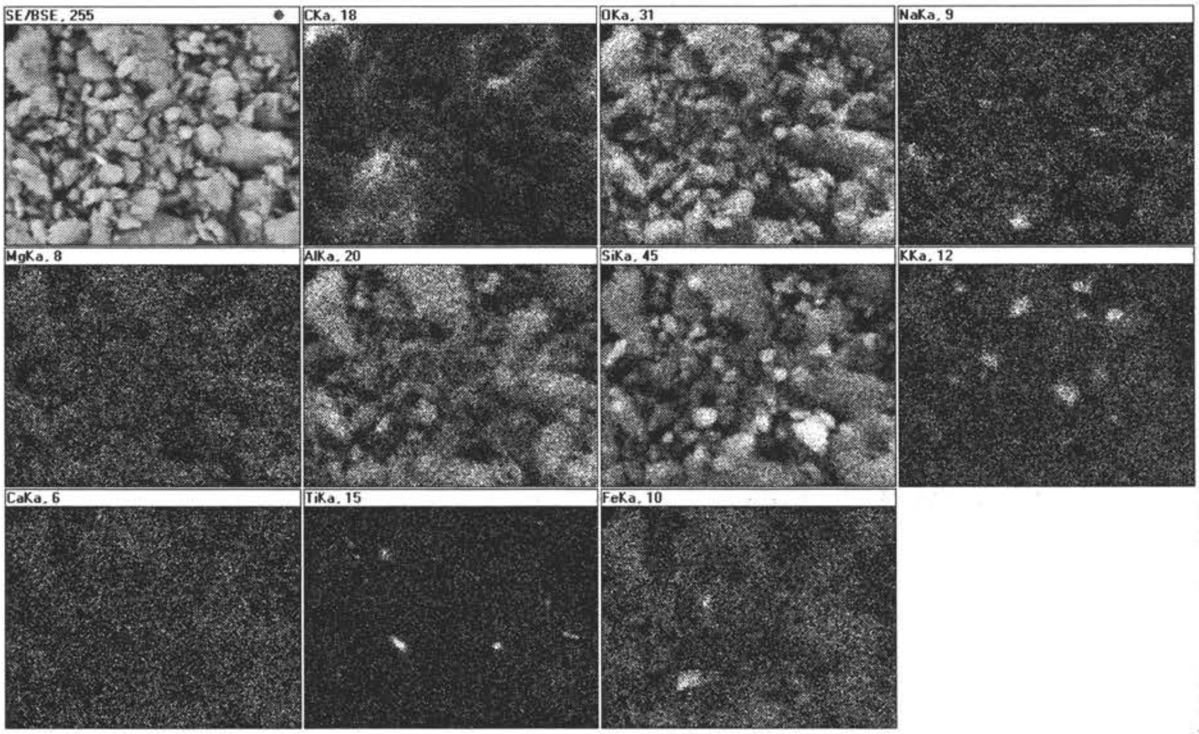


Figure 9. EDS Elemental Map of Glacial Till Clay Fraction (< 2 microns)

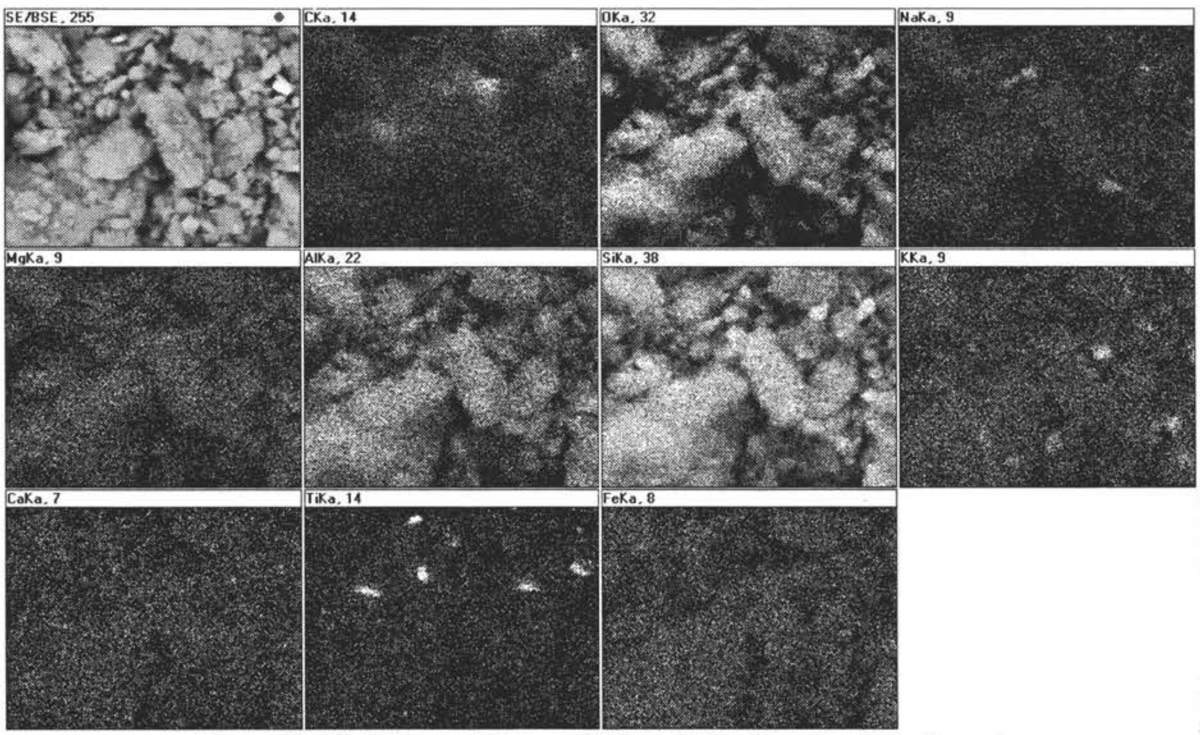


Figure 10. EDS Elemental Map of Paleosol Clay Fraction (< 2 microns)

Two different thermal analyses were also conducted on the clay fractions. The tests were carried out in an inert nitrogen atmosphere and heating occurred at 10° C/min. TGA testing measures the mass loss of the sample during heating, while DTA compares the temperature difference between an inert sample and the unknown sample. Figure 11 shows the TGA results, while Figure 12 shows the DTA data. Duplicate tests were conducted for each sample and the results were almost the same for each test. The TGA plot shows the samples began to lose water around 100° C and then lost mass again between 375 and 500° C. The DTA results are hard to comprehend. Endothermic peaks are evident at 90 and 180° C for all samples. Other than these two peaks, no other large peaks are evident. An exothermic peak at approximately 920° C is shown for all samples, although this peak is very small. The till shows an endothermic peak near 880° C. The exothermic peak near 900°C and endothermic peak between 100 and 200° C is typical of montmorillonite while the endothermic peak close to 105° C is generally shown in kaolinite. The thermal analysis results show there is a mixture of clay minerals present in the soils.

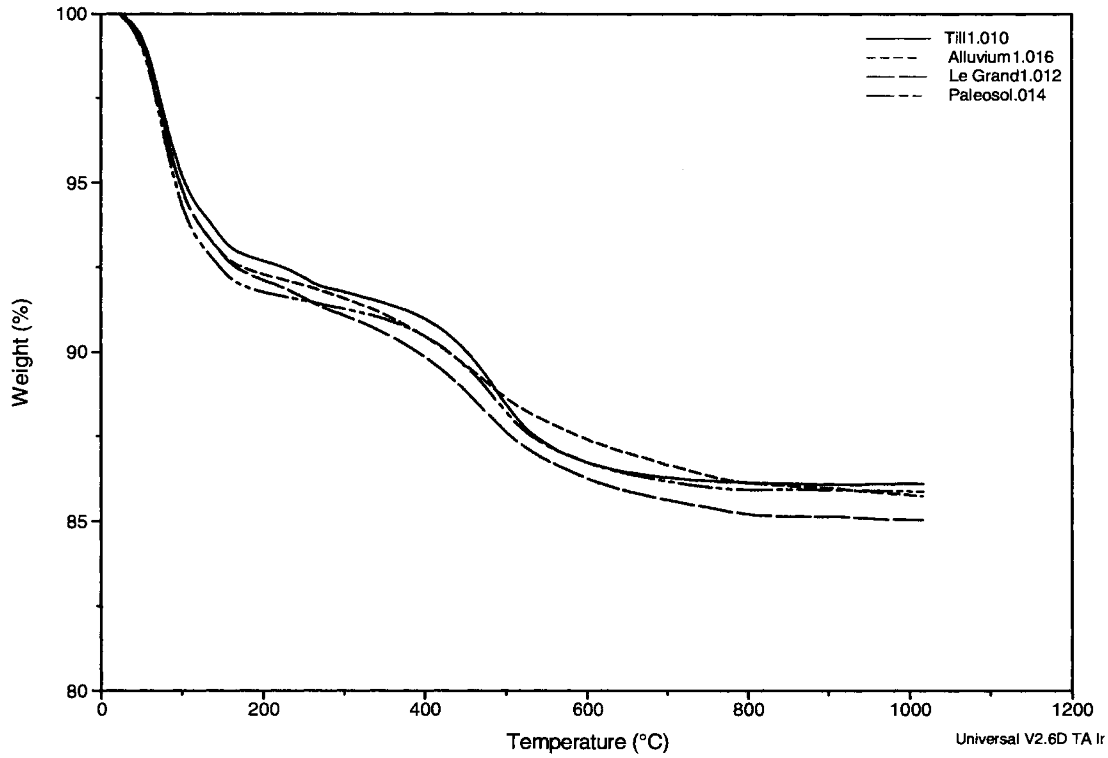


Figure 11. TGA Results for Soil Clay Fractions (< 2 microns)

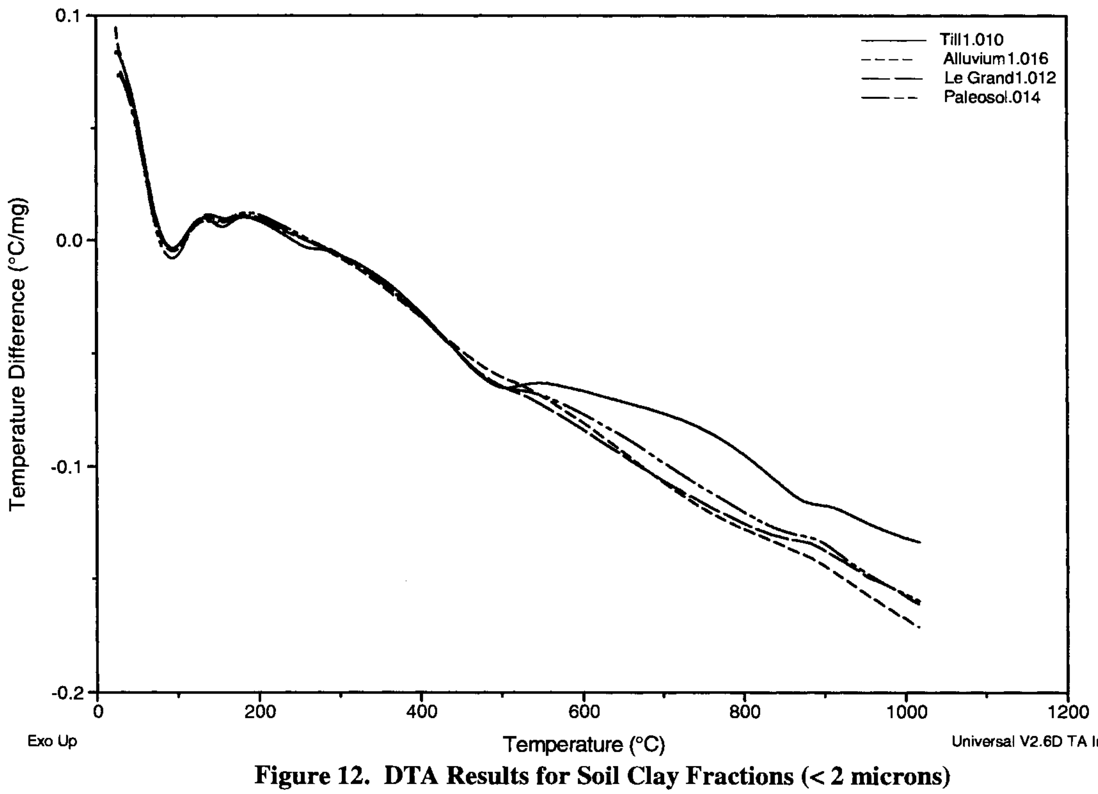


Figure 12. DTA Results for Soil Clay Fractions (< 2 microns)

Moisture-Density Relationships and Unconfined Compressive Strength

Moisture-density relationships for each soil were determined according to ASTM D698 Method A [Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lb_f/ft^3)] (9). A range of maximum densities and optimum moisture contents was found for the soils. Results are plotted in Figure 13. Table 4 summarizes the optimum moisture content and maximum dry density for the soils. Due to the largest percentage of sand, the glacial till has the highest maximum density and the lowest optimum moisture content. The paleosol, Le Grand loess, and Turin loess have similar moisture-density relationships with optimum moisture content between 16.6% and 17.2 % and maximum dry densities ranging from 105.2 pcf to 106.7 pcf. The alluvium exhibits the lowest maximum dry density (102.6) and highest optimum moisture content (19.8%).

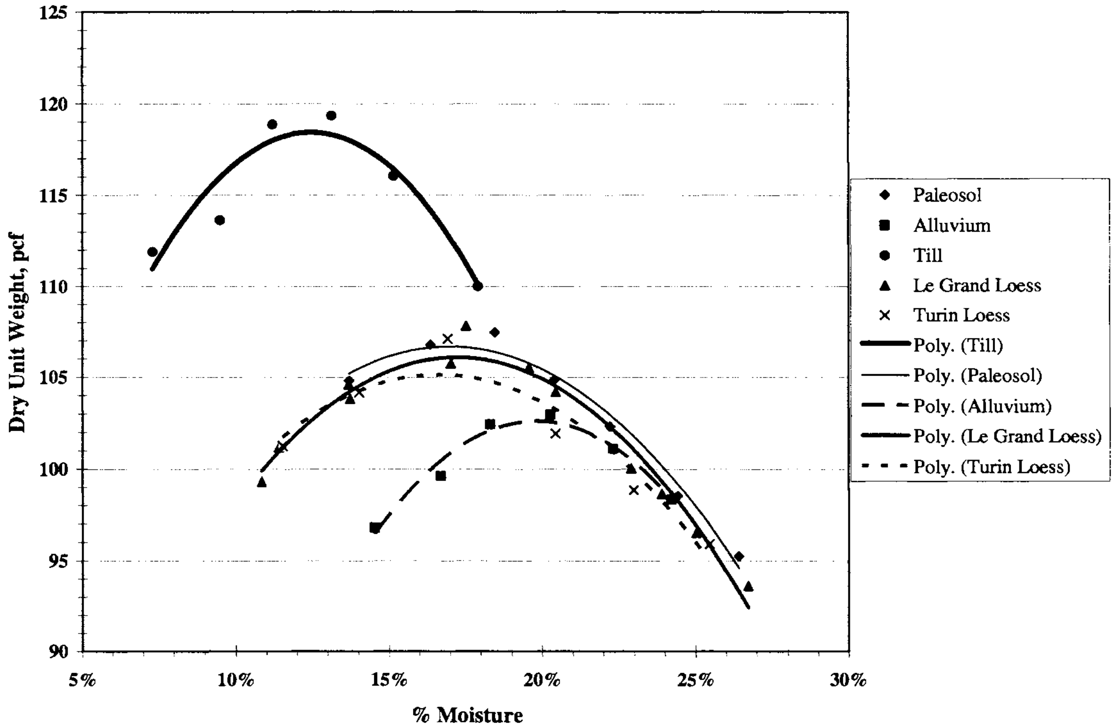


Figure 13. Moisture-Density Relationships of Soils Used in the Study

Table 4. Summary of Optimum Moisture Content and Maximum Dry Density of Soils Used in the Study

Soil	Optimum Moisture Content, %	Maximum γ_d, pcf
Paleosol	17.0%	106.7
Alluvium	19.8%	102.6
Glacial Till	12.5%	118.4
Le Grand Loess	17.2%	106.1
Turin Loess	16.6%	105.2

The soil samples that were molded to determine moisture-density relationships were extruded and subjected to unconfined compression tests at a loading rate of 0.05 in/min. Proctor size samples have a length to diameter ratio (L/D) of 1.15. The results are plotted as strength versus molded moisture content and shown in Figure 14.The samples were not soaked prior to testing. It is anticipated that soaking the samples would have resulted in lower strength and possibly deterioration of the samples compacted dry of optimum to the point that they would not have been able to be tested. The till has the highest strength due to the higher sand content and being compacted at lower moisture contents. The alluvium exhibited the next highest strength, with 62 psi at 14.5 % moisture. The paleosol and Le Grand loess have similar strengths with the paleosol being approximately 5 psi larger throughout the moisture content range. Turin loess is predominately composed of silt-sized particles, and has the lowest overall strength. Table 5 presents the compressive strength at optimum moisture content along with the relative strength decrease in psi per percent moisture. The Turin loess had 33 psi compressive strength at optimum moisture content, while the paleosol had strength of 48 psi. The other soils have approximately the same strength of 44 psi at optimum moisture content. The glacial till decreases approximately 7 psi/%M while the other soils decrease between 3 and 5 psi/%M.

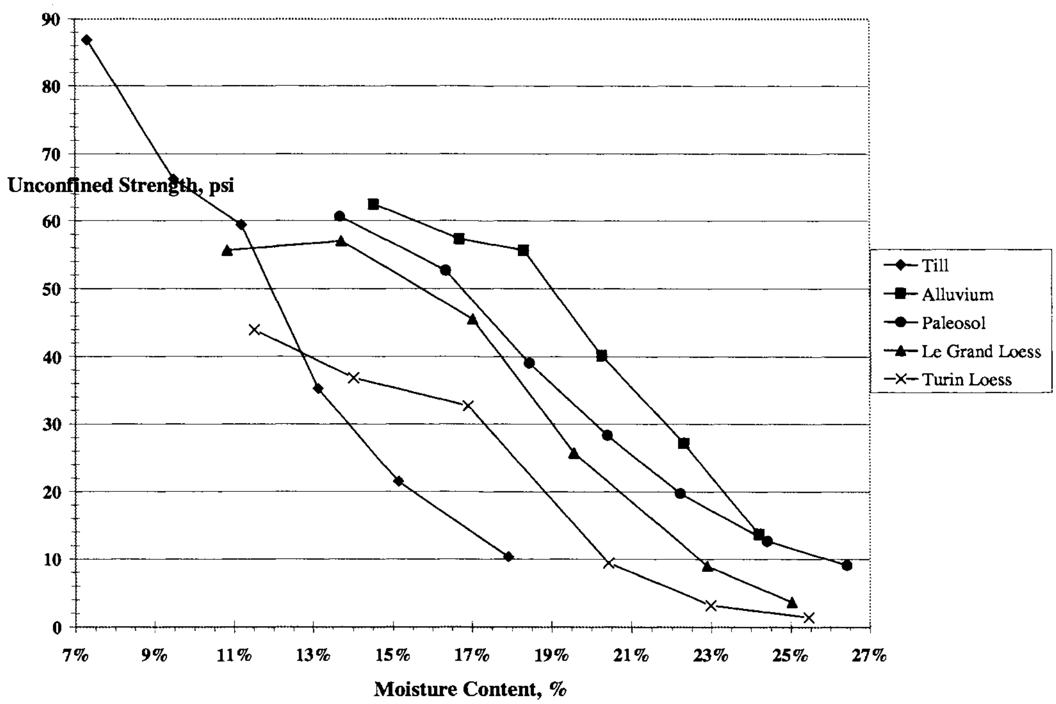


Figure 14. Moisture-Strength Relationships of Soils Used in the Study

Table 5. Compressive Strength at Optimum Moisture Content and Strength Decrease of Soils Used in the Study

Soil	Optimum Moisture Content, %	Strength @ Optimum Moisture Content, psi	Average Strength Decrease, psi/%M
Paleosol	17.0%	48	4
Alluvium	19.8%	44	5
Glacial Till	12.5%	44	7
Le Grand Loess	17.2%	44	4
Turin Loess	16.6%	33	3

Engineering Properties of Self-Cementing Fly Ash

X-Ray Analysis

X-ray analyses consisting of x-ray diffraction (XRD) and x-ray fluorescence (XRF) were conducted to determine the minerals in the fly ashes and the elemental composition of the fly ashes, respectively. The fly ash was sampled according to ASTM C311 [Standard Test Methods for Sampling and Testing Fly Ash or Natural Pozzolans for Use as a Mineral Admixture in Portland-Cement Concrete] (4), and testing was conducted by personnel at the Materials Analysis and Research Laboratory (MARL) at ISU during the spring of 2003. XRD is the process of bombarding a sample with x-rays over a range of angles. The intensity of the reflected x-rays is measured and this data is converted to planar spacing of molecular layers, which is then used to determine the different minerals present in the sample. Table 6 summarizes the XRF results. The major minerals determined by XRD are summarized in Table 7. The complete XRD diffractograms are located in Appendix A. Most of the ashes meet the requirements of ASTM C618 [Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Concrete] (6)

Class C, while the ones that do not meet ASTM C618 Class C can still be used as soil stabilizers because they exhibit self cementing properties.

The important parameters to observe from XRF testing are the loss on ignition (LOI), sulfur (SO_3) content, calcium (CaO) content, and silica (SiO_2), alumina (Al_2O_3), and iron oxide (Fe_2O_3) contents. The sum of silica, alumina, and iron oxide must total between 50% and 70% to be considered Class C by ASTM C618. The sum of oxides for the SGS ash is the lowest at 50.98% while the largest sum of oxides, 63.52%, was found in the older OGS fly ash. The literature states that sulfur can have detrimental effects in soil stabilization when greater than 5%. All of the fly ashes tested have sulfur contents less than 5%. Moisture content was also determined prior to testing. The measured values are typically structural water that was absorbed from the environment. LOI is a measure of the unburned carbon content of fly ash. SGS and PC3+4 materials have the largest LOI, due to the boilers used at these generating stations. The CaO contents of the materials ranges from 22.23% to 28.47%. CaO, along with silica and alumina are the materials that form the cementitious reaction products. Larger amounts of calcium should mean that the fly ash will have the capacity to provide more strength when water is added.

The minerals in Table 7 are listed in decreasing intensity as reported in the diffractograms. The materials have large amounts of quartz and tricalcium aluminate. The other common minerals are lime (CaO), anhydrite ($CaSO_4$), and periclase (MgO). Tricalcium aluminate causes initial hardening when water is added to fly ash. As mentioned previously, lime is needed to form cementitious reaction products. Anhydrite is a precursor to ettringite formation, although this will be minimal due to the low sulfur content determined in XRF testing. Periclase is also evident in the fly ash samples, which is composed of MgO.

Fly Ash Set Time

Self-cementing fly ashes form cementitious reaction products when mixed with water. Set time of fly ash can have a significant bearing on the phenomenon of decrease in strength gain and density due to compaction delay, as was discussed earlier. To determine set time of fly ash, ash is mixed into a paste with water at a water/ash ratio of .275 then the paste is placed in a shallow dish. A pocket penetrometer is used to take readings at a convenient time interval depending on the rate at which the fly ash sets. When the reading on the pocket penetrometer reaches 4.5 tsf, the fly ash is determined to be set. Initial set time is the time at which the fly ash begins to rapidly gain strength. Lapke and Bergeson (20) believed that set time of the fly ash was directly related to the CaO content of the ash. The initial cementitious reactions in Portland cement are due to tricalcium aluminate, and this may also be the case in fly ash, in addition to the CaO. Figure 15 and Figure 16 show the complete set time tests for the ashes used in this study and Table 8 summarizes the initial and final set times for each ash. Comparison of the figures and table shows fly ash is an extremely variable material and in one case the CB fly ash did not set up in four hours. This could be related to the precipitator aid burned with the ash.

Table 7. XRF Summary of Major Minerals in Fly Ash

Ash Source	Date Sampled	Noteable Minerals
CB3	8/21/01	Quartz, Tricalcium Aluminate, Anhydrite, Lime, Periclase
LGS	8/21/01	Quartz, Tricalcium Aluminate, Anhydrite, Periclase, Lime
PN3	2/23/01	Quartz, Tricalcium Aluminate, Anhydrite, Periclase, Lime
PN3	9/10/01	Tricalcium Aluminate, Quartz, Anhydrite, Periclase, Lime
PN4	9/10/01	Tricalcium Aluminate, Quartz, Anhydrite, Lime, Periclase
OGS	8/21/01	Quartz, Tricalcium Aluminate, Anhydrite, Periclase, Lime
SGS	12/13/01	Quartz, Tricalcium Aluminate, Brownmillerite, Periclase, Lime, Anhydrite
PC3+4	5/10/02	Quartz, Tricalcium Aluminate, Anhydrite, Lime, Periclase
CB3	2/23/01	Quartz, Tricalcium Aluminate, Anhydrite, Periclase, Lime
LGS	2/23/01	Quartz, Tricalcium Aluminate, Anhydrite, Periclase, Lime
PN4	2/23/01	Tricalcium Aluminate, Quartz, Feldspar, Anhydrite, Periclase, Lime
OGS	2/23/01	Quartz, Anhydrite, Tricalcium Aluminate, Periclase, Lime
AMES	2/15/02	Quartz, Tricalcium Aluminate, Periclase, Anhydrite, Lime

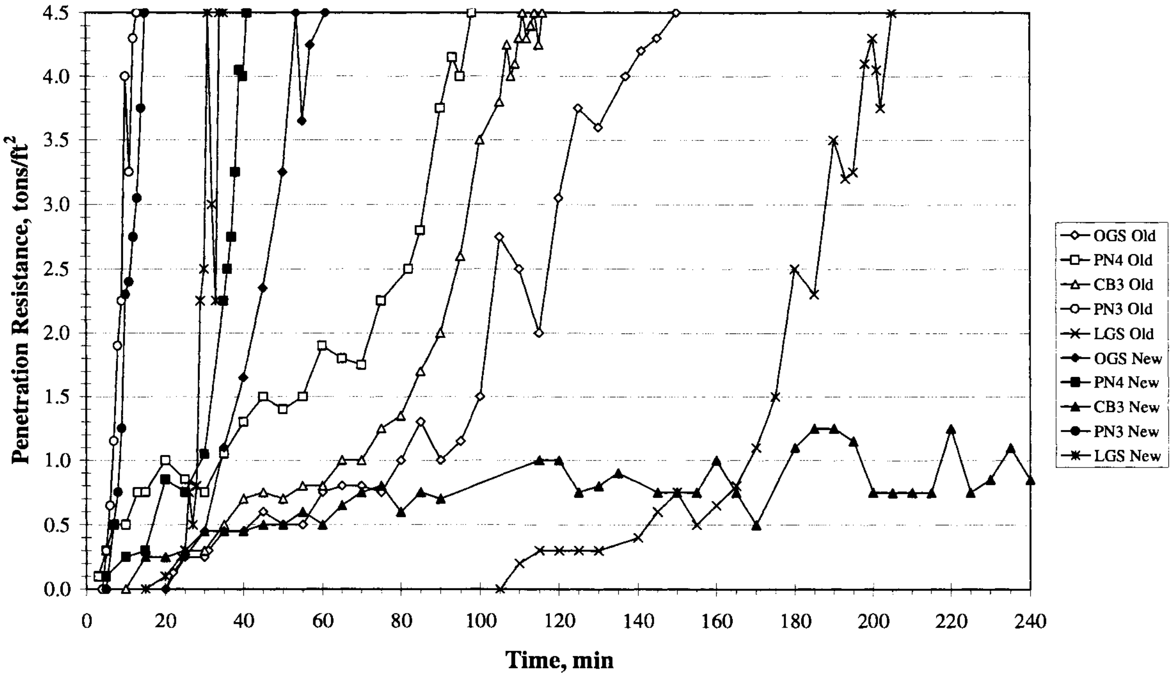


Figure 15. Set Time of Class C Fly Ashes Used in the Study

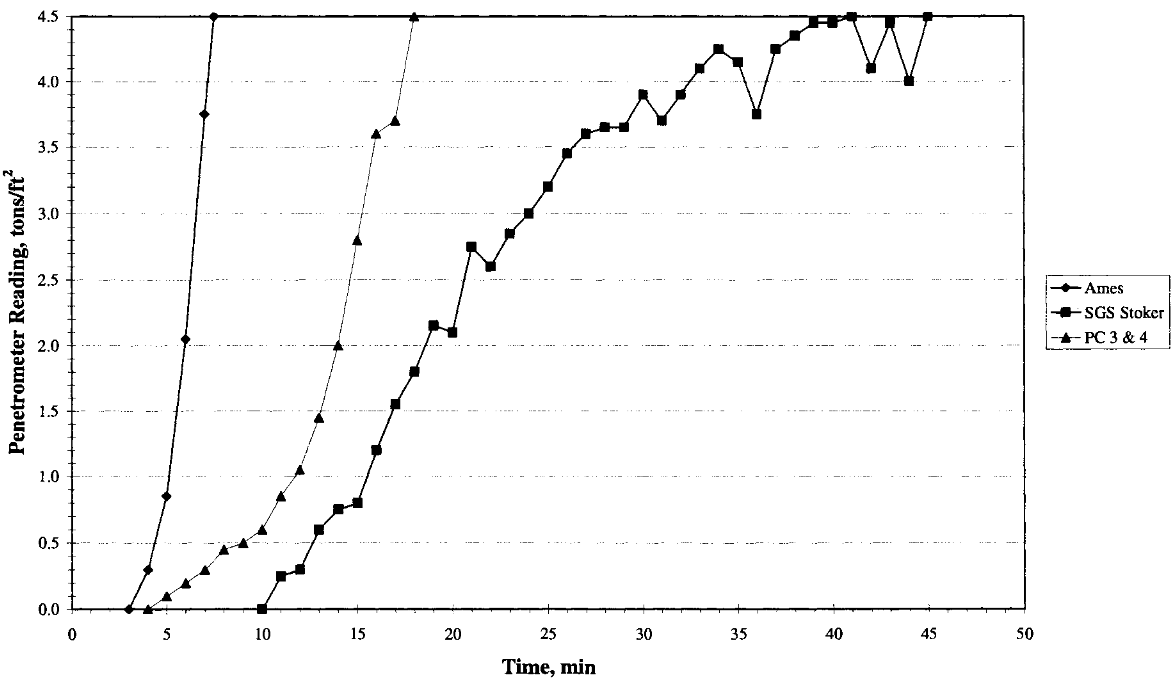


Figure 16. Set Time of Non-Class C Fly Ashes Used in the Study

Table 8. Initial and Final Set Times of Fly Ashes Used in the Study

Fly Ash	Initial Set, min	Final Set, min
SGS	10	45
Ames	3	7.5
PC3+4	4	18
CB Old	15	116
CB New	15	> 240
PN4 Old	3	98
PN4 New	5	41
PN3 Old	5	13
PN3 New	7	15
OGS Old	22	150
OGS New	25	61
LGS Old	110	205
LGS New	20	34

Fly Ash Paste Strength

ASTM D5239 [Standard Practice for Characterizing Fly Ash for Use in Soil Stabilization] (17) is said to provide a measure of fly ash effectiveness as a soil stabilizer. In this test the fly ash is mixed with water at a water/ash ratio of 0.35. The paste is then formed into 2-inch x 2-inch cubes and allowed to cure for seven days, at which time they are tested in compression. Non self-cementing fly ashes have strength less than 100 psi, moderately-cementing ashes have strengths between 100 and 500 psi, and extremely-cementing fly ashes have strength over 500 psi. This study varied from ASTM D5239 in that the fly ashes were mixed into pastes over a range of water/ash ratios and allowed to cure for 28 days in a humidity room at 70° F. The strength results are shown in Figure 17 and Figure 18. Portland cement shows an exponential decay as the water/cement ratio increases (33), and the same should be true of fly ash, but this is not the case. The strengths are highly variable. Also at 28 days some of the ashes do not have strengths greater than 100 psi for higher water/ash ratios.

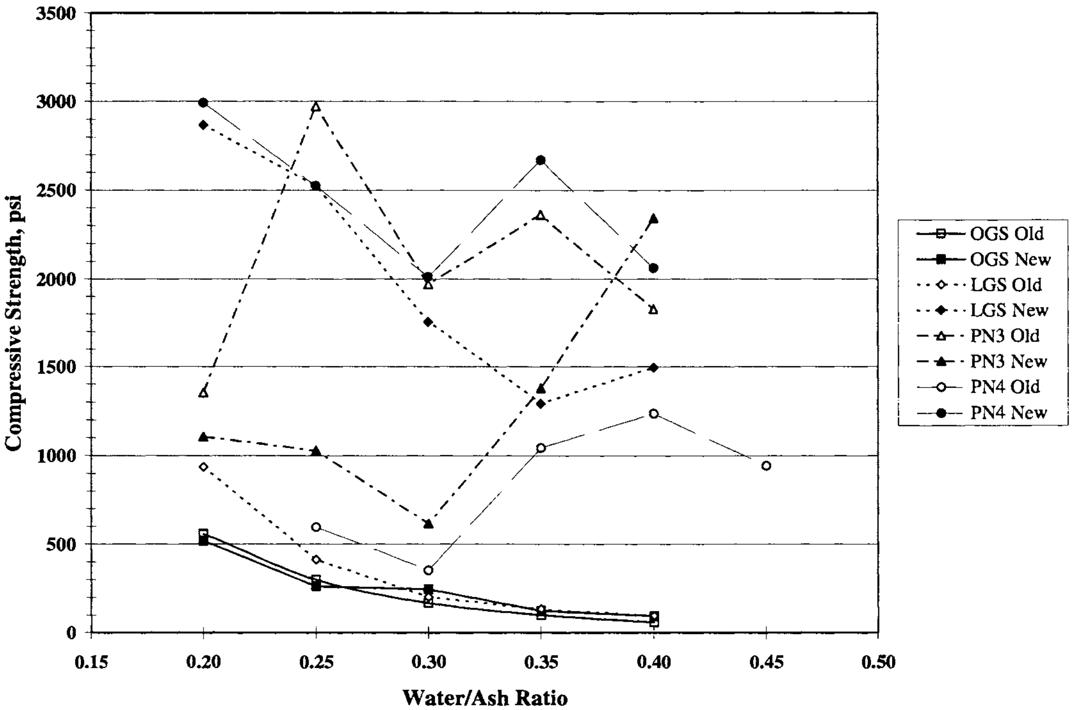


Figure 17. Class C Fly Ash Paste Strength

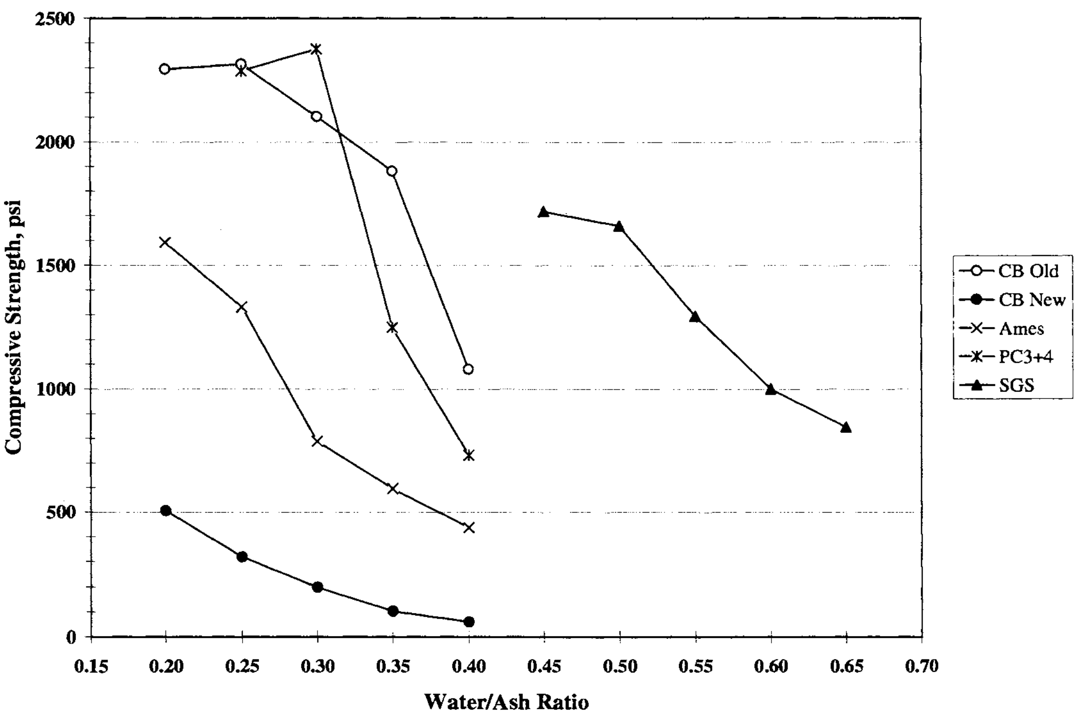


Figure 18. Class C and Non-Class C Fly Ash Paste Strength

Engineering Properties of Hydrated and Conditioned Fly Ash

Grain Size Distribution

The grain size distributions of the CFA and HFA were determined in accordance with ASTM C136 [Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates] (3). Each sample was oven dried to a constant weight and then agitated through a nest of sieves by a mechanical shaker. The results are shown in Figure 19. The data show that the CFA materials have between 20% and 40% material finer than the #200 sieve. This is due to the process of wetting in a pug mill and then stockpiling without compaction. The compaction and reclamation techniques used to produce HFA materials leads to a coarser distribution when compared to CFA materials. All of the materials are well graded.

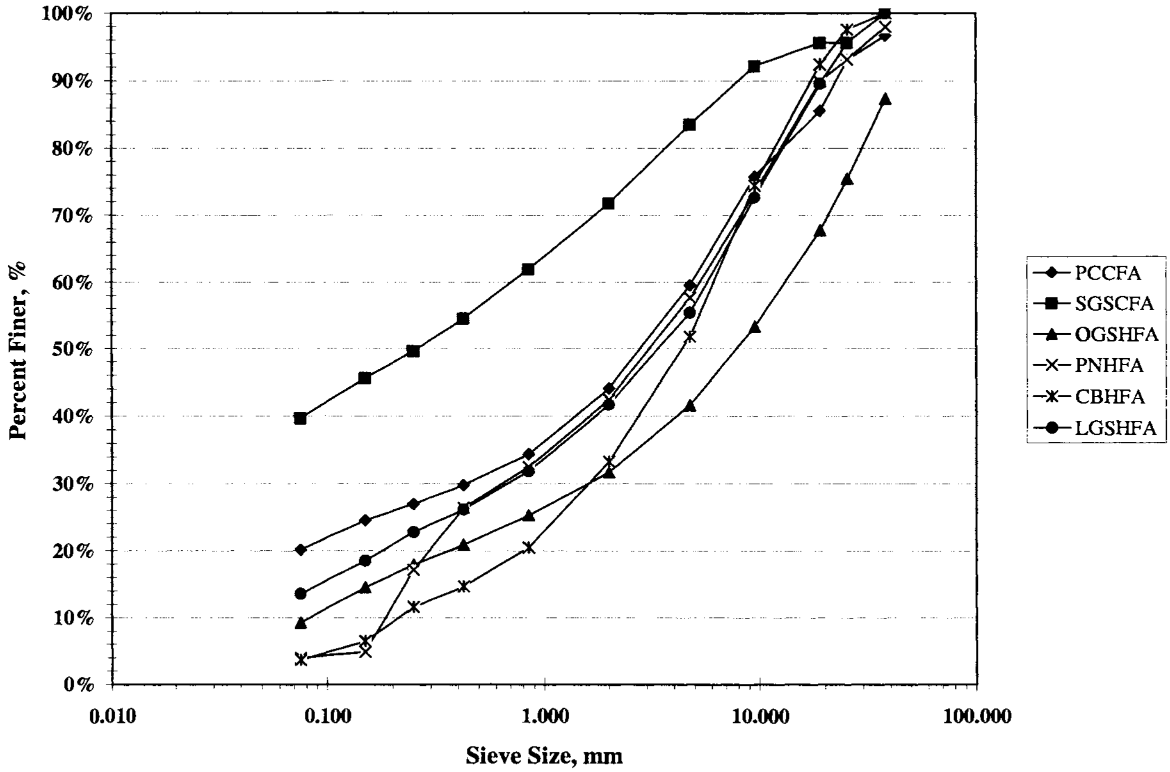


Figure 19. Particle Size Distribution of Hydrated and Conditioned Fly Ash

X-Ray Analysis

X-ray analysis consisting of x-ray diffraction (XRD) and x-ray fluorescence (XRF) was conducted on the hydrated and conditioned fly ashes used for this project. The testing was conducted by MARL personnel during the summer of 2001. Most of the HFA and CFA materials have moderate self-cementing properties due to the fact that the HFA and CFA were formed from Class C or self-cementing fly ashes. Table 9 shows the total XRF results while Table 10 shows the XRF results corrected for LOI and moisture content. Common minerals in the HFA and CFA materials are shown in Table 11.

LOI for HFA and CFA is quite large due to the loss of free water as well as structural water and unburned carbon. Sulfur content remains relatively constant after hydration, and the sum of oxides remains in the range of 50 to 70%. Some calcium has been utilized to form reaction products and is no longer in the form of free lime. For the most part the constituents of the cementitious reaction products decrease after fly ash has been hydrated; however these materials are still available for long-term pozzolanic reactions.

Table 11 shows some CaO has been converted to calcite. Anhydrite has reacted with alumina and water to form ettringite. Quartz is still present in the HFA and CFA samples. The tricalcium aluminate is residual and for the most part has been converted to other products. Periclase is still present in both the CFA and HFA, showing very little reactivity with water and other minerals.

Table 11. XRD Summary of Major Minerals in Hydrated and Conditioned Fly Ash

Source	Date Sampled	Noteable Minerals
CB3 HFA	3/23/99	Calcite, Tricalcium Aluminate, Quartz, Ettringinte
LGS HFA	5/20/99	Ettringite, Quartz, Calcite, Periclase, Tricalcium Aluminate
PN HFA	3/23/99	Quartz, Tricalcium Aluminate, Ettringite, Calcite
OGS HFA	11/6/98	Ettringite, Quartz, Tricalcium Aluminate, Calcite, Periclase
SGS CFA	7/5/01	Tricalcium Aluminate, Quartz, Periclase, Calcite, Ettringite
PC CFA	6/15/01	Ettringite, Quartz, Stratlingite, Tricalcium Aluminate, Calcite

Moisture-Density Relationships

A variation of ASTM D698 Method A (9) was used to determine the moisture-density relationships of HFA and CFA products. These materials have a large percentage of aggregate greater than three-quarters of an inch; for this reason the material was passed through a reciprocating jaw type crusher until all of the material to be compacted passed the three-quarter inch sieve. The results of the moisture-density testing are shown in Figure 20, and the optimum moisture content and maximum dry density are shown in Table 12. Only one material, the PC CFA shows a relationship similar to typical Proctor curves. It is noticed that the moisture-density relationships of the HFA and CFA are fairly flat with only about 4 pcf difference of dry density over a wide range of moisture contents. Densities of the materials are between 72 pcf and 93 pcf. The OGS and PN HFA and the PC and SGS CFA materials show a small change in density that can be related to optimum moisture content. The CB and LGS HFA do not show optimum moisture content.

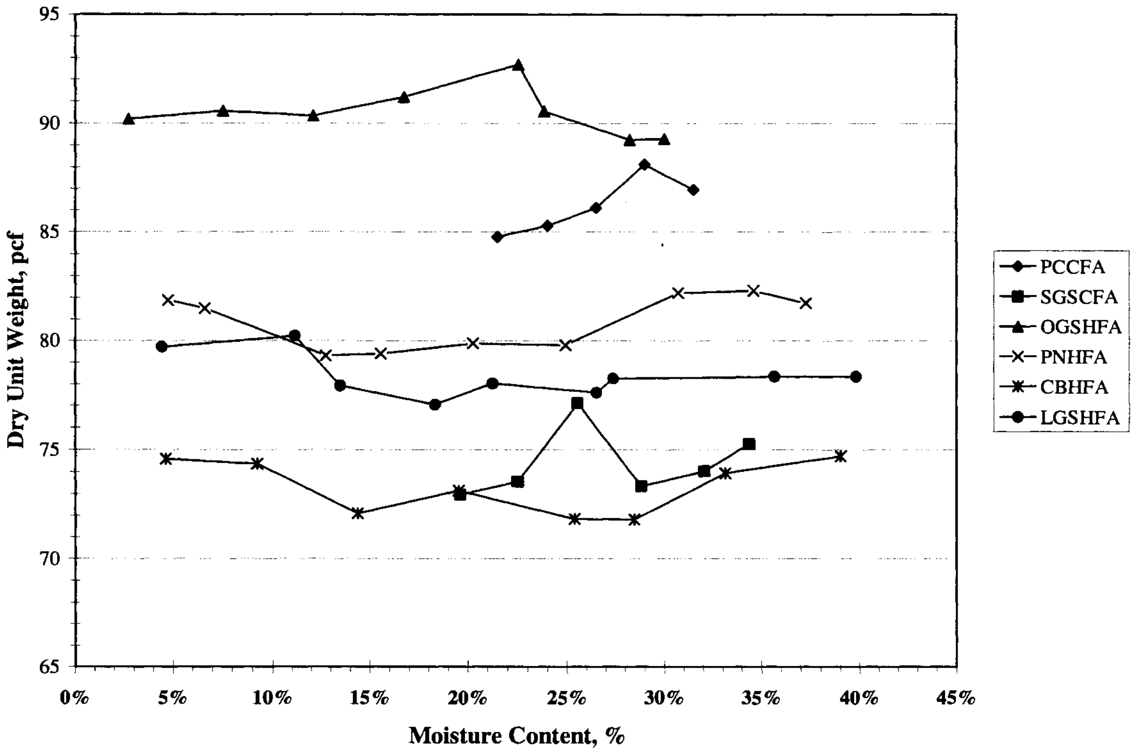


Figure 20. Moisture-Density Relationships of Hydrated and Conditioned Fly Ash Used in the Study

Table 12. Optimum Moisture Content and Maximum Dry Density of Hydrated and Conditioned Fly Ash

Material	Optimum Moisture Content, %	Maximum γ_d, pcf
OGS HFA	22.6%	92.7
PN HFA	34.6%	82.3
LGS HFA	27.0% to 40.0%	78.3
CB HFA	39.0%	79.0
PC CFA	31.5%	87.0
SGS CFA	25.5%	77.1

COMPACTION CHARACTERISTICS OF SOIL STABILZED WITH SELF-CEMENTING FLY ASH

Effect of Fly Ash Addition on Moisture-Density Relationships

The addition of fly ash alters the maximum dry density and optimum moisture content of a soil. To study this effect, tests involved compacting samples of varying amounts of fly ash blended with Turin loess and Le Grand loess. Moisture-density relationships were determined in general accordance with ASTM D698 Method A (9) and the ISU 2-inch x 2-inch methods (21). The ISU 2-in x 2-in method correlates well with the standard compactive effort of ASTM D698 (48). Samples were initially moistened to appropriate moisture contents and allowed to cure for 24 hours before the fly ash was added. Fly ash was added at rates of 5%, 10%, 15%, and 20% based on dry weight of soil and samples were immediately compacted, i.e., no compaction delay.

Figure 21 shows the results of test performed on Turin loess. It was observed that the ISU 2-in x 2-in and standard Proctor methods correlate well. The maximum dry density for the Turin loess alone was 104.5 pcf and optimum moisture content was 18.5%. For 5 and 10% fly ash, the ISU 2-in x 2-in method produced maximum dry density of 105.5 pcf and optimum moisture content of approximately 19.0%. The Proctor results show maximum dry density increased from 105.5 pcf for 5% fly ash to slightly greater than 109 pcf for 20% fly ash. The Proctor optimum moisture contents were in the range of 15.5 to 16.0%. Dry density and optimum moisture content for 15 and 20% fly ash determined from the ISU 2-in x 2-in method were approximately the same as the standard Proctor results for those two addition rates. In this case it is believed the fly ash spheres acted mechanically to fill the voids that were present and produce a much denser sample. This behavior of fly ash spheres was also reported by Zia and Fox (52) for stabilized Indiana loess.

The moisture-density for the Le Grand loess and fly ash contrast those observed for the Turin loess, as shown in Figure 22. The soil itself had a maximum dry density near 108 pcf at a moisture content of 17.6%. The Le Grand loess-fly ash mixtures did not exhibit the same correlation between ISU 2-in x 2-in and standard Proctor methods. The ISU 2-in x 2-in method produced greater densities for all fly ash contents, but optimum moisture content was between 16.2 and 17.0% for all tests. The 10% fly ash samples had the largest dry density for both test methods, while the 5 and 20% mixtures had approximately the same dry density. Fly ash addition increased the dry density and reduced the optimum moisture content of Le Grand loess, however the relationship of increased density with increased fly ash addition was not observed as in the case of the Turin loess.

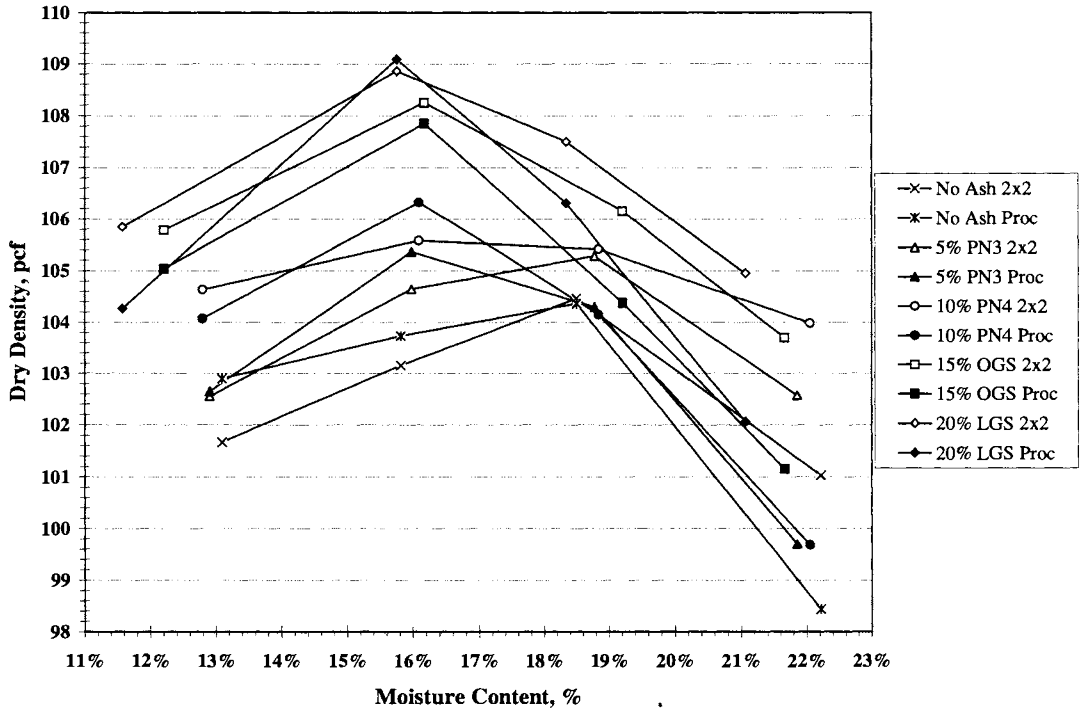


Figure 21. Influence of Fly Ash Additioiı on the in Moisture-Density Relationship of Turin Loess Determined by Standard Proctor and ISU 2-in x 2-in Methods

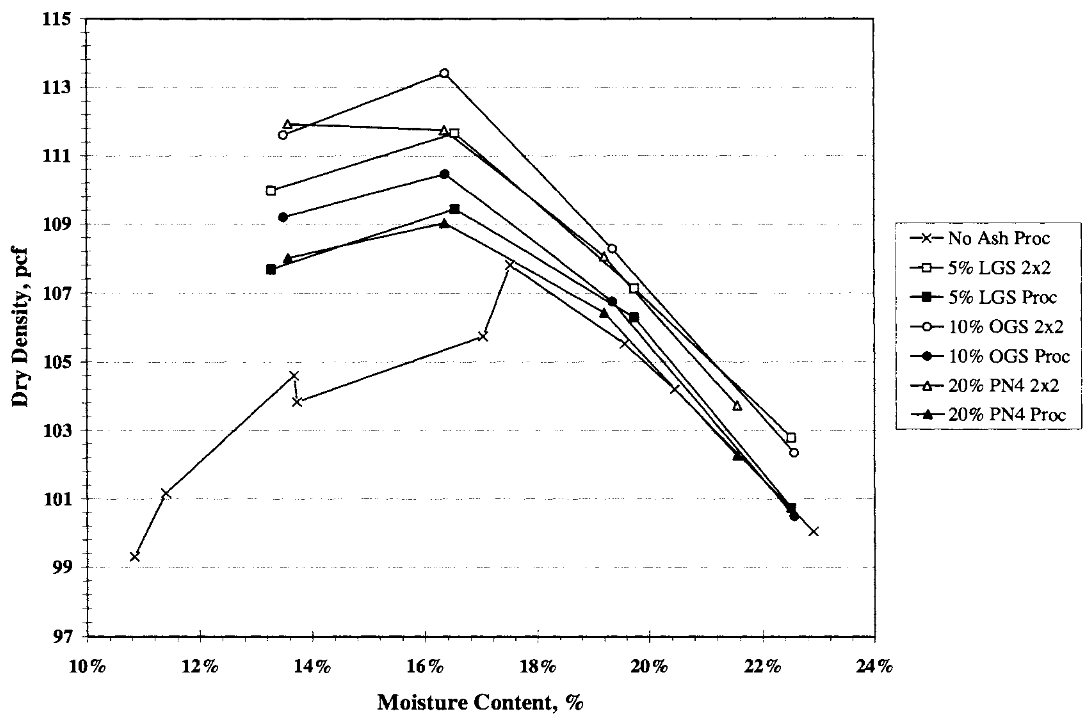


Figure 22. Influence of Fly Ash Addition on the Moisture-Density Relationship of Le Grand Loess Determined by Standard Proctor and ISU 2-in x 2-in Methods

Influence of Compactive Effort on Moisture-Density Relationships

Fine-grained soils typically show higher maximum densities and lower optimum moisture contents when subjected to higher levels of compactive energy. It is theorized that increased energy realigns the soil particles closer together and decreases the need for more water as lubrication between particles during compaction, but this theory has been partly discredited. Samples were prepared in accordance with ASTM D698 Method A (9), ASTM D1557 Method A [Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lb_f/ft^3)] (11), and standard and modified ISU 2-in x 2-in methods. Twelve percent (by dry weight) OGS fly ash was added after the soils were allowed to cure for 24 hours, and the samples were compacted immediately after mixing. Alluvium and glacial till soils were used for this series of tests. Figures 23 and 24 show the till and alluvium test results, respectively.

The glacial till test has good correlation between the two modified levels of compaction energy while the standard energy results show some variation. It should be noted that at moisture contents larger than optimum for either compaction energy, the curves start to converge at the same value of dry unit weight. Results of tests on the alluvium show a similar correlation between the two modified tests that the till showed and a slightly stronger correlation for the standard tests. A possible reason for this is the alluvium soil contained little sand, and the soil matrix having more uniform particle size. Sand could have contributed to the variation in the standard till tests. The alluvium-OGS mixture exhibited convergence of dry density at high moisture content, similar to the behavior of the till.

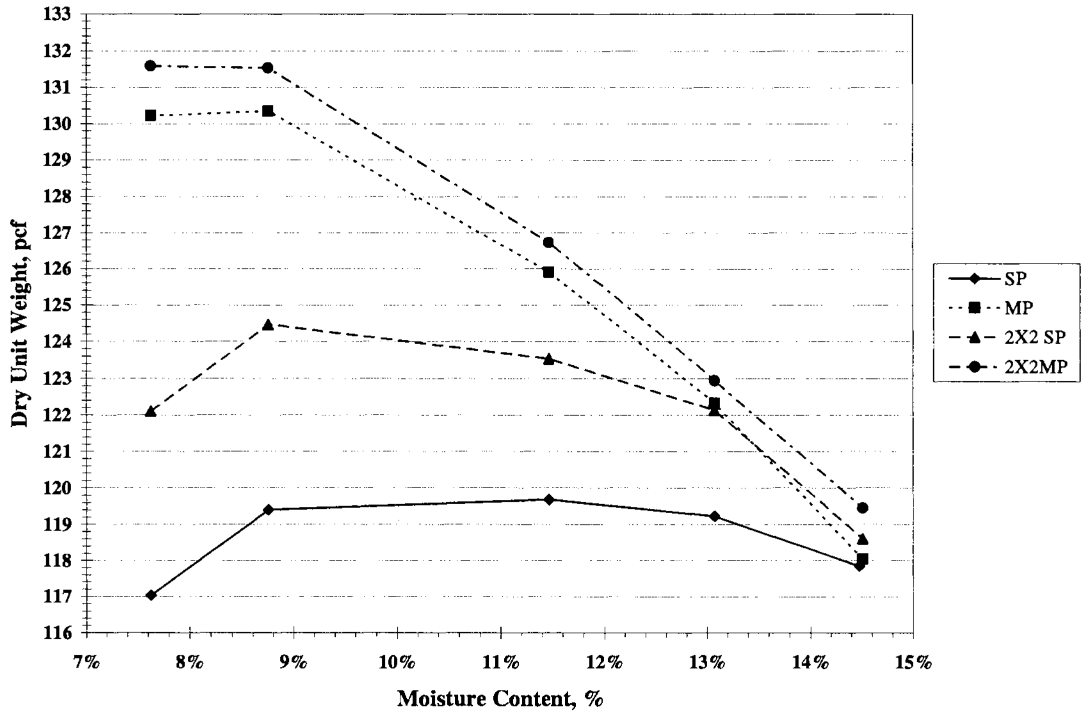


Figure 23. Influence of Compaction Energy on Moisture-Density-Relationship of Till and 12% OGS Ash

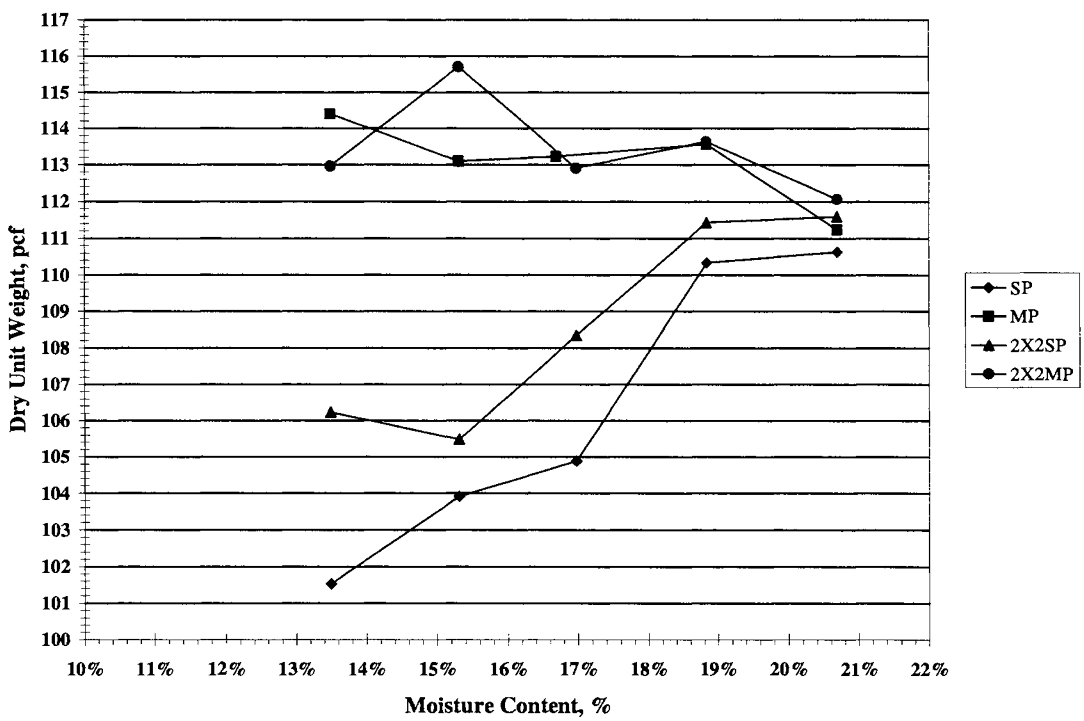


Figure 24. Influence of Compaction Energy on Moisture-Density Relationship of Alluvium and 12% OGS Ash

Influence of Compaction Delay on Unit Weight and Moisture-Density Relationships

The literature search revealed strong trends of density decreasing with increasing compaction delay times. Two sets of tests were performed for this part of the study. The first test used Turin loess stabilized with 20% of each fly ash used in the study. The loess was prepared at an initial moisture content of 20%, so the final mixture would be at approximately optimum moisture content for density. Three 2-in x 2-in samples were molded at delay times of 0, 0.5, 1, 2, and 4 hours to evaluate the short-term effect of compaction delay on unit weight for the various fly ashes. Moisture contents were not taken at each delay time, so the results are reported on a total unit weight basis. Figure 25 shows the Turin loess and fly ash mixtures that had the greatest loss of compacted density.

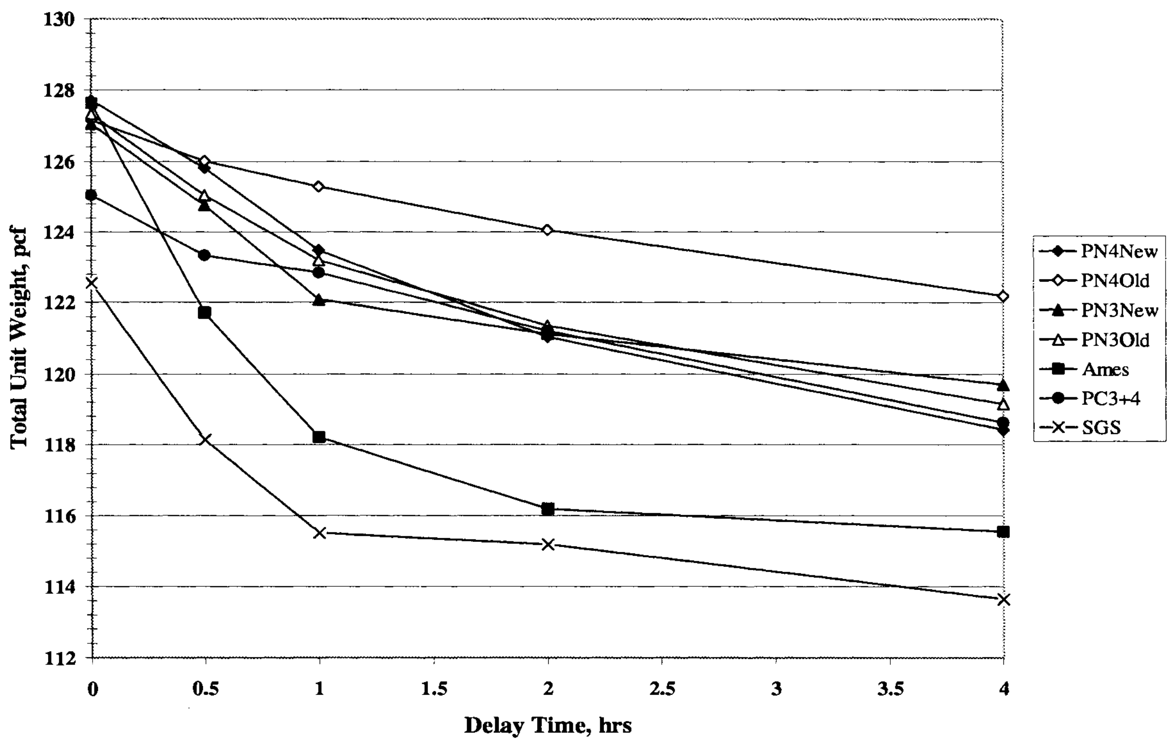


Figure 25. Influence of Compaction Delay on Total Unit Weight of Turin Loess and 20% Ash

When comparing the set time data and Figure 25, it can be seen that the fly ashes are more reactive. Figure 25 shows that density decreases from 7 to 13 pcf. Figure 26 shows results for fly ashes with lower reactivity. This figure was plotted to the same scale as Figure 25 to show the lesser decrease in density, typically on the order of 4 pcf or less after a four hour delay time. The reactivity of the fly ash used for stabilization can have a profound influence on density loss of the stabilized mixture in as little as one-half an hour time. The Turin loess contains a relatively low amount of clay, and effects of the more reactive ashes could be more pronounced for a soil with higher clay content.

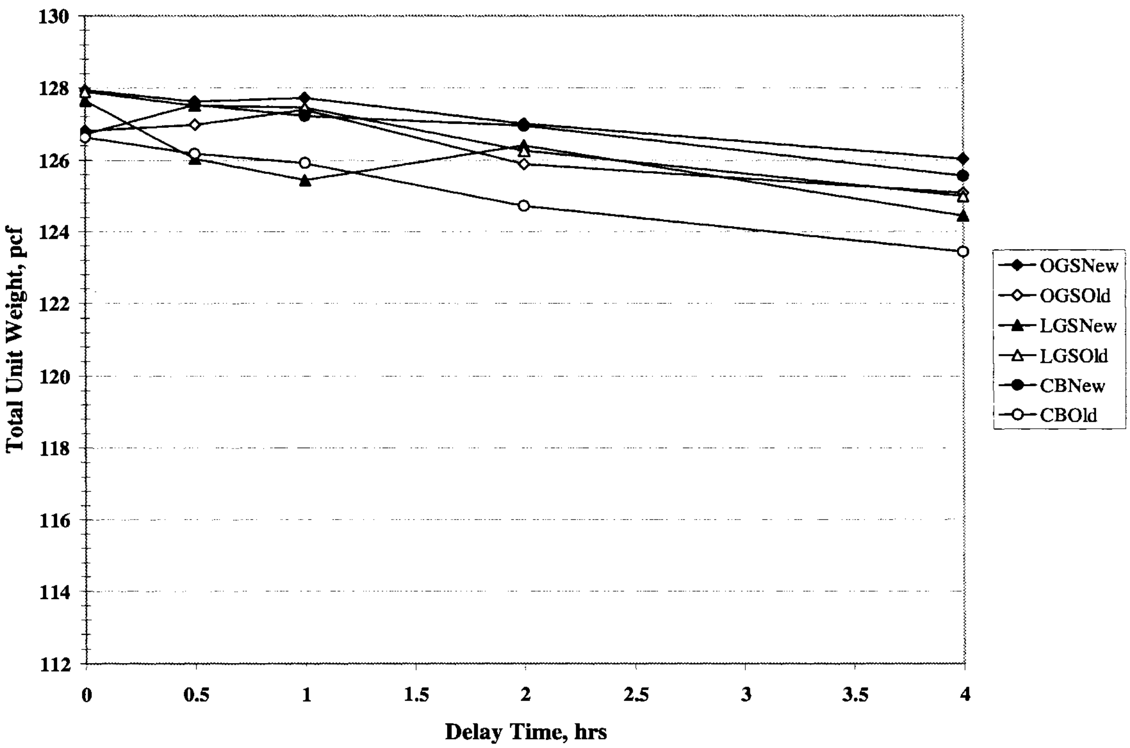


Figure 26. Influence of Compaction Delay on Total Unit Weight of Turin Loess and 20% Fly Ash

Another series of tests was performed to evaluate density loss due to long-term compaction delay. The Argyle paleosol blended with 20% PC 3+4 fly ash (by dry weight) was used for this test. Five batches of soil were prepared at various moisture contents to produce a complete moisture-density relationship. Fly ash was added to soil samples after 24 hours and compaction proceeded at delay times of 0, 4, and 24 hours. Three 2-in x 2-in samples were prepared for each moisture content at each delay time. The results are presented in Figure 27.

The PC 3+4 fly ash is very reactive and the paleosol contains a large amount of clay-sized particles. As the mixture was allowed to rest uncompacted, flocculation and agglomeration effects occurred. The zero-hour delay samples resulted in a typical Proctor curve. However, as delay time increased, the curves began to flatten. It is theorized that the reduction in dry unit weight would have been greater than the three pcf shown because of the higher clay content of the soil. The alumina and silica in clay particles react with calcium in fly ash to produce cementitious reaction products. Higher availability of the reaction product constituents should have accelerated the flocculation and agglomeration of the soil, thereby reducing the compacted density. It should also be noted that free water was continuously turned into structural water during the 24 hour period. Structural water is not available for ash hydration, therefore slowing the chemical reactions.

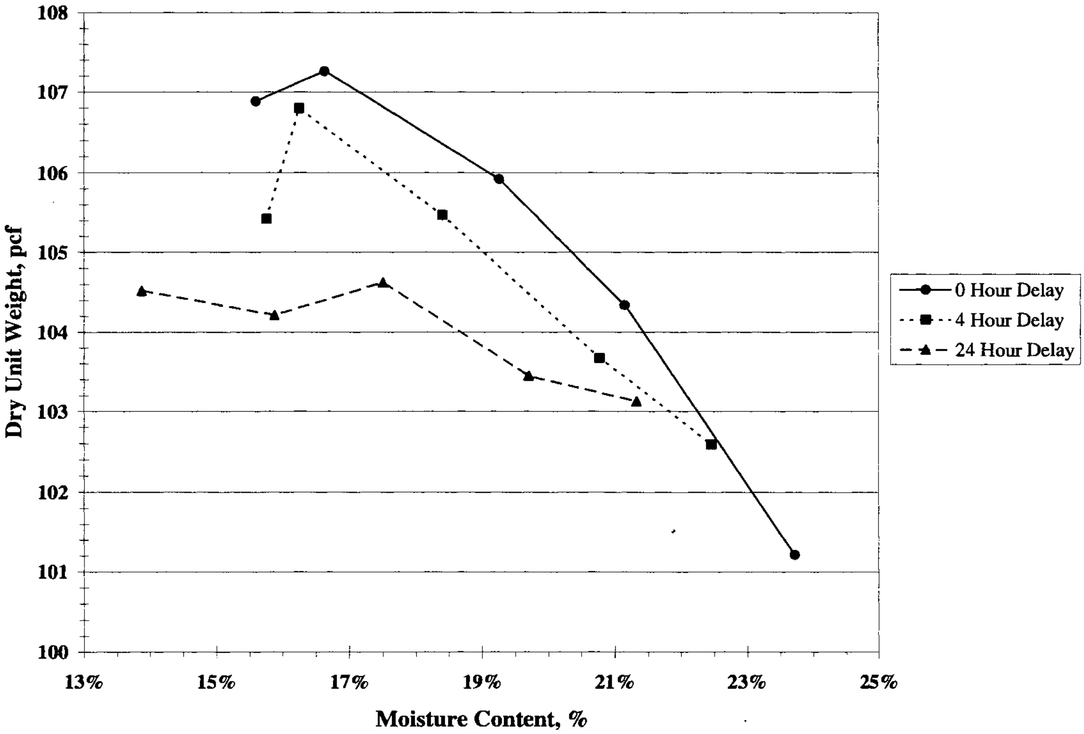


Figure 27. Influence of Long-Term Compaction Delay on Paleosol and 20% PC3+4 Ash

STRENGTH OF SELF-CEMENTING FLY ASH AND SOIL

Self-cementing fly ash is primarily used in soil stabilization to increase strength of soil. As mentioned before, soils stabilized with self-cementing fly ash typically exhibit strengths between 100 and 500 psi (25). The strength of stabilized mixtures can depend upon a number of factors, including compactive effort, compaction delay, and percentage of fly ash. Testing was completed to measure the effects of these three factors, as well as to measure the CBR, true (L/D=2), 2-in x 2-in (L/D=1.0) and Proctor (L/D=1.15) sized unconfined compressive strength, and long-term strength gain of self-cementing fly ash stabilized soils.

Influence of Compactive Effort on Strength Gain

Samples that were compacted to measure the influence of compactive effort on moisture-density relationships were extruded from their molds, sealed, and cured in an oven at 100° F for 7 days. At the end of 7 days, samples were tested in compression at a loading rate of 0.05 in/min. The 4-inch diameter Proctor samples had a L/D ratio of 1.15 while the L/D ratio of the 2-in x 2-in samples was approximately 1.0. Samples were not soaked prior to testing. Both the alluvium and glacial till samples were prepared with 12% OGS ash by dry weight. Figure 28 presents results for the alluvium samples and Figure 29 for the glacial till samples. It was believed that more compactive energy would produce higher density which in turn would lead to increased strengths. Higher compacted density decreases the air voids in materials, which increases the particle to particle contacts. Greater amounts of particle to particle contacts increases the internal angle of friction of the material which increases strength.

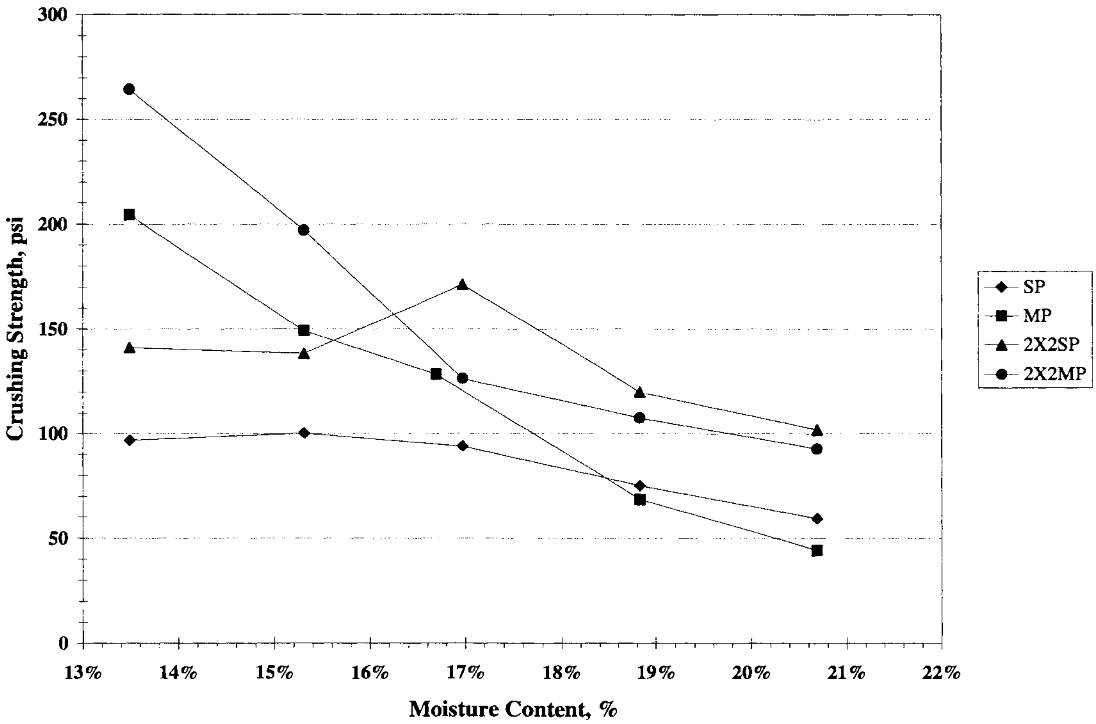


Figure 28. Influence of Compaction Energy on Alluvium and 12% OGS Ash

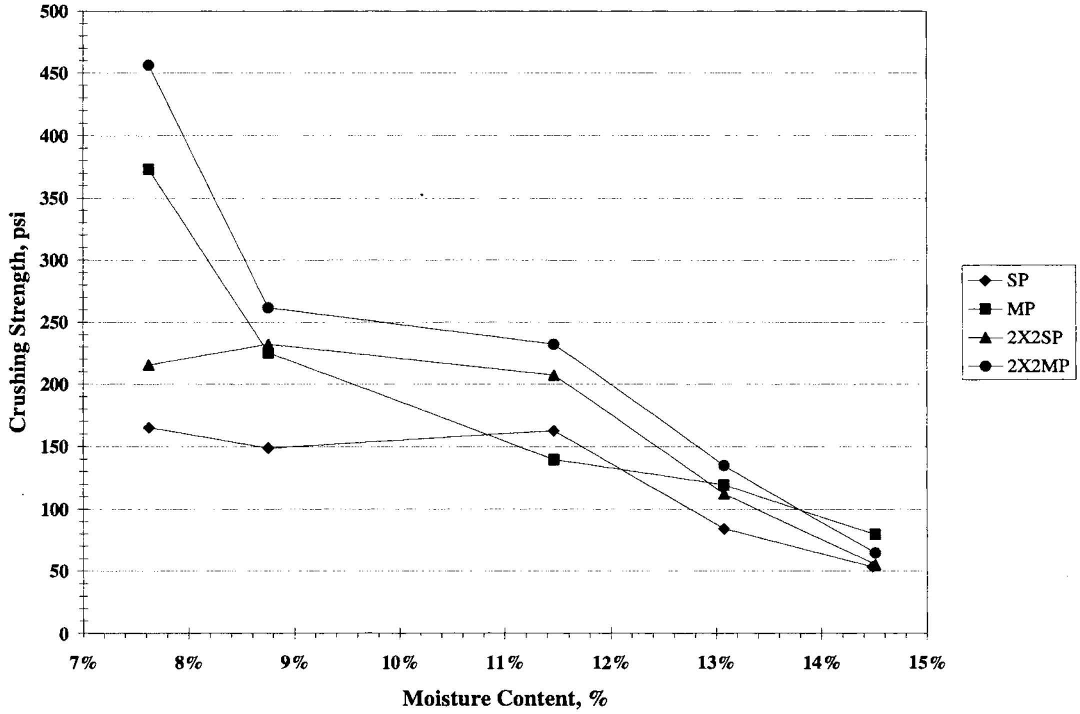


Figure 29. Influence of Compaction Energy on Glacial Till and 12% OGS Ash

All four sets of alluvium samples show the general trend of decreasing strength with increasing moisture content. Figure 24 showed that densities actually increased or stayed relatively constant, indicating the strength of the alluvium mixture was more dependant on molded moisture content. Samples compacted with standard energy have flatter moisture-strength relationships than the modified energy samples. It should be noted that near 17% moisture, strengths tend to be more related to sample diameter rather than compaction energy. Near 17% moisture the 2-in x 2-in samples showed strengths greater than the Proctor sized samples, regardless of compaction energy. At the final molded moisture content of 20.7%, the 2-in x 2-in samples have strengths of approximately 100 psi while the Proctor sized samples exhibited strengths near 50 psi.

The OGS ash stabilized glacial till samples compacted with modified energy showed an initial pronounced decrease in strength with increased moisture content. The 4-inch diameter samples dropped from 375 psi to 75 psi while the 2-in x 2-in samples decreased from 460 psi to 70 psi. The two sets of samples compacted with standard energy have relatively flat relationships up to 11.5% ± moisture, after which a decrease is evident. As with the moisture-density relationship of all these till samples, strengths began to converge, regardless of sample size or compaction energy.

Influence of Compaction Delay on Strength Gain

Testing was completed to measure strength versus compaction delay. In the first study, 2-in x 2-in (L/D=1.0) samples of Turin loess containing 20% fly ash (by dry weight) were molded at approximately 17.5% moisture. The delay times were 0, 0.5, 1, 2, and 4 hours. A set of 10 samples was molded for each fly ash/soil mixture. After molding, samples were extruded, sealed, and allowed to cure in a 100% humid environment for 28 days. After curing, samples were tested in compression at a rate of 0.05 in/min. Samples were not soaked prior to testing. The results of the Class C fly ashes are shown in Figure 30, and a summary of data is provided in Table 13.

The results do not show an immediate loss of strength for delay times of 0.5 hours. All but one of the ten Class C fly ashes exhibit a strength gain for one or two hours, and then show a strength decrease. The strengths start to decrease when the mixture is compacted after the fly ash set time. One set of samples, CB New, did not show a strength loss at any delay time. This is probably due to the fact that the fly ash did not set up in four hours, therefore allowing reaction products to continue forming over the duration of testing, instead of flocculating and agglomerating the soil particles.

Figure 31 shows the strength versus compaction delay results for the non-Class C fly ashes used in these tests. The PC3+4 samples don't show much strength loss even though it had a set time of 18 minutes. The Ames and SGS samples do show an immediate strength loss. The final losses were 37 psi and 45 psi for the SGS and Ames, respectively.

A second set of tests was conducted to measure strength loss with compaction delay. In these tests 20% (by dry weight) PC3+4 fly ash was mixed with the paleosol at varying moisture contents. Samples were compacted using the ISU 2-in x 2-in apparatus (L/D=1.0) at times of 0, 4, and 24 hours. Three samples were compacted for each moisture content at each delay time. After molding, samples were cured for 28 days in a humidity room. Prior to compression testing at a rate of 0.05 in/min, samples were soaked for 1 hour in a bath of room temperature water. The test results are shown in Figure 32.

For the most part, samples showed an overall decrease in strength with increased compaction delay time. The exception was noted for the samples at 18.4% moisture at 4 hours. The samples converted moisture to structural water during the delay.

Flocculation and agglomeration were pronounced on the 13.9% moisture 24-hour samples. These samples had little water to carry on chemical reactions and the samples slaked during soaking. All of the 24-hour samples are shown after soaking in Figure 33. The second set of samples from the top show some cracking due to swelling, which also contributed to lower strength. The strength loss was thought to be greater due to the high clay content of the paleosol, but not as pronounced as expected. Overall there is a continued strength decrease at 24 hours.

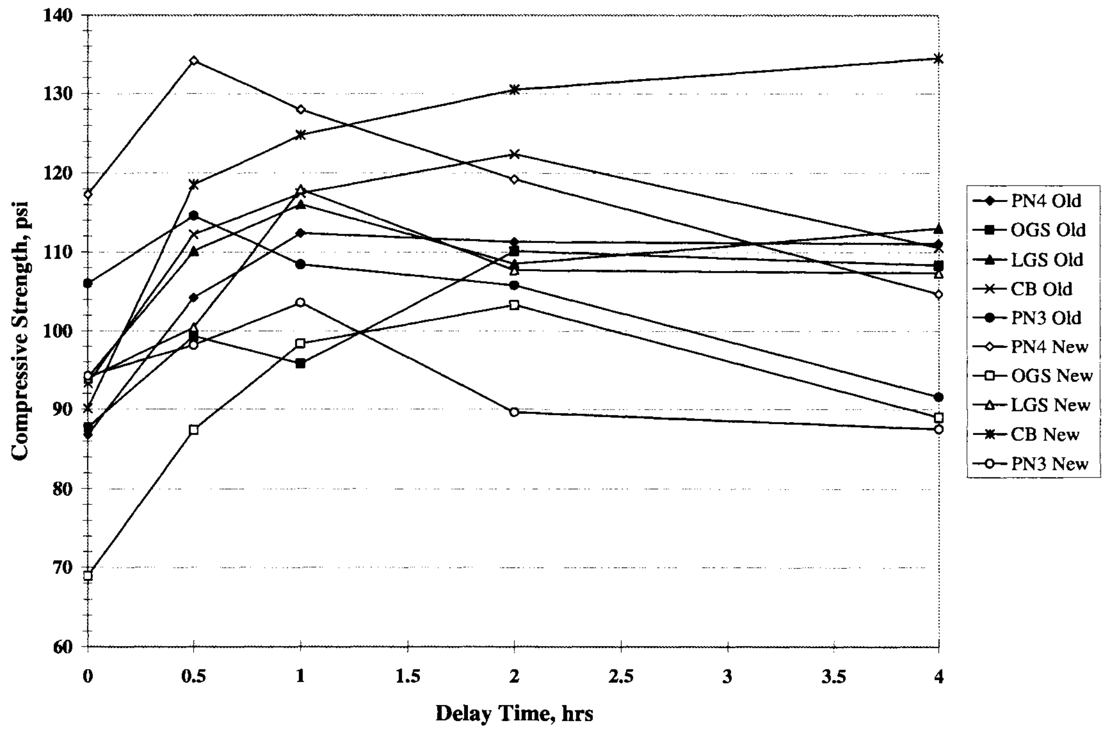


Figure 30. Influence of Compaction Delay on Strength Gain of Class C Fly Ash and Turin Loess

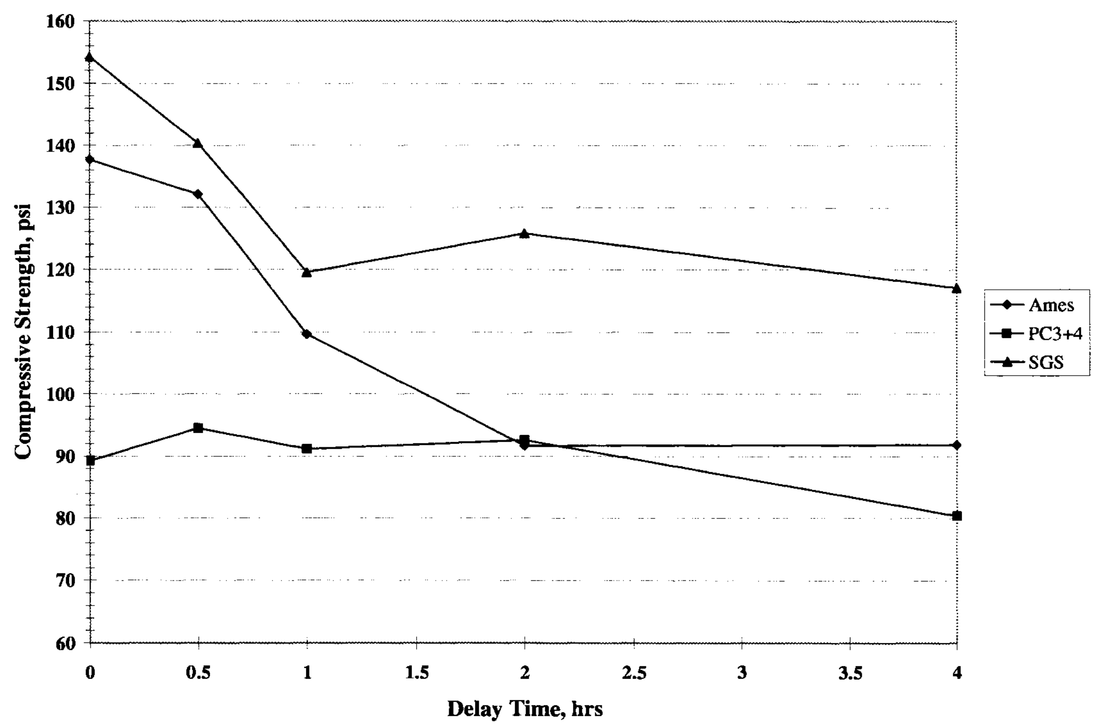


Figure 31. Influence of Compaction Delay on Strength Gain of Non-Class C Ash and Turin Loess

Table 13. Summary of Compaction Delay Data

	Delay Time				
Ash	0.0	0.5	1.0	2.0	4.0
PN4 New	117	134	128	119	105
OGS New	69	87	98	103	89
LGS New	94	100	118	108	107
CB New	90	119	125	131	134
PN3 New	94	98	104	90	88
PN4 Old	87	104	112	111	111
OGS Old	88	99	96	110	108
LGS Old	94	110	116	109	113
CB Old	93	112	117	122	111
PN3 Old	106	115	108	106	92

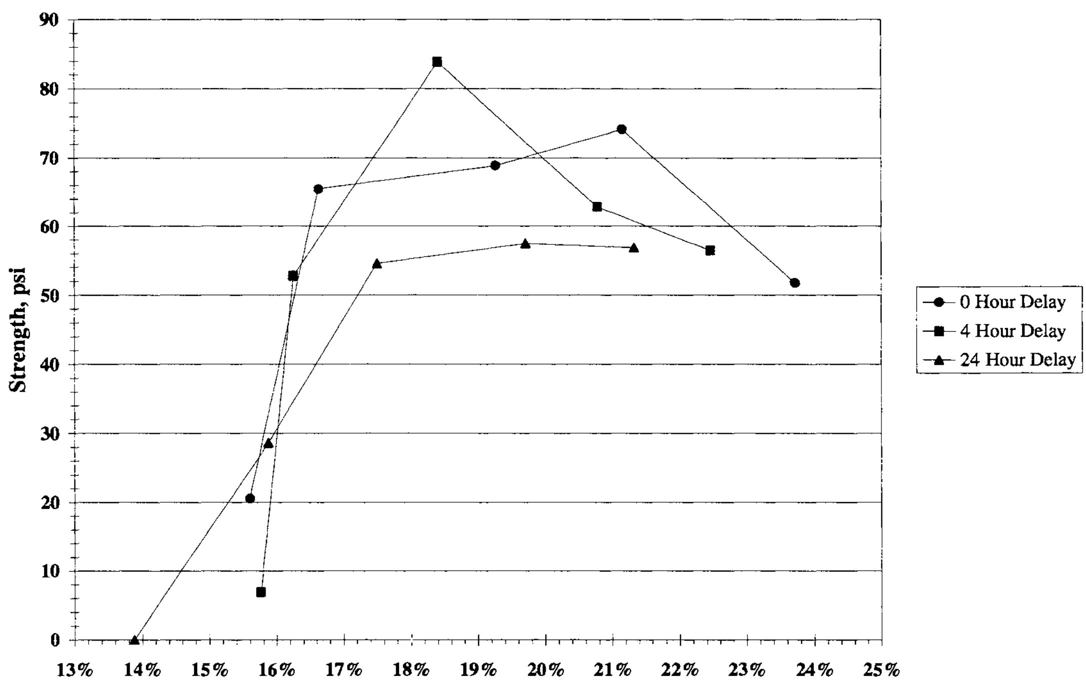


Figure 32. Influence of Long-Term Compaction Delay on Strength Gain of Paleosol and 20% PC3+4 Fly Ash

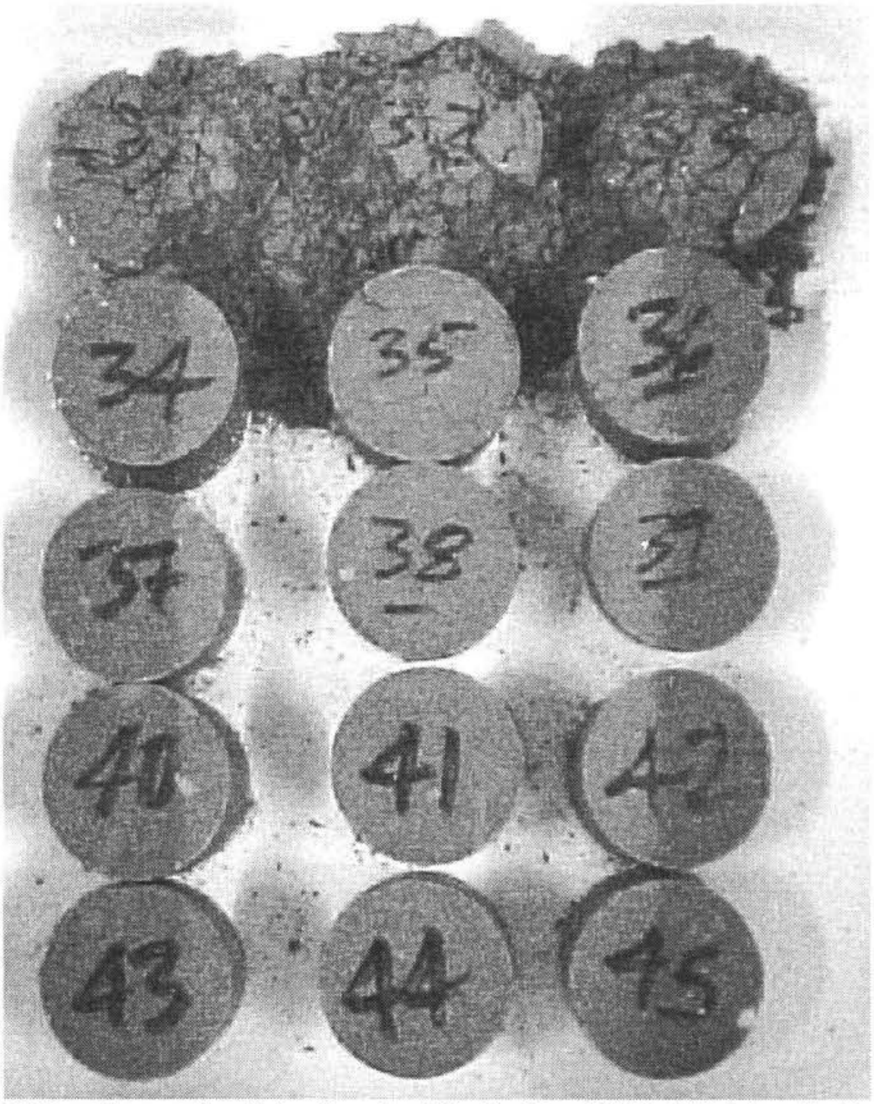


Figure 33. 24-Hour Compaction Delay Samples After Soaking (Molded Moisture Content Decreases From Top to Bottom)

Long-Term Strength Gain of Soil Stabilized with Self-Cementing Fly Ash

On August 29, 2002, 2-in x 2-in samples were measured to determine volume change and tested for compressive strength. These samples were molded in March and April of 2000 as part of a previous research project. The fly ash was mixed at different rates with the same glacial till used in the more recent studies. The strength results are presented in Figure 34.

All of the samples, except for the UNI AFBC and PC stoker samples, showed a continued increase in strength throughout the curing time. Some of the samples have leveled off due to excess moisture being used up to form reaction products. The UNI AFBC fly ash contained 29% sulfur while the PC stoker fly ash had 6% sulfur. Due to these high sulfur contents, ettringite formed in the samples. This ettringite led to volume expansion and deterioration of the samples. A scanning electron microscope (SEM) image of the UNI AFBC samples is shown in Figure 35. The ettringite crystals have formed on the fly ash spheres, which are completely covered. Conversely an SEM image of the PC3+4 20% samples is shown in Figure 36. In this picture, some crystals of ettringite and cementitious reaction products are evident. Also the fly ash spheres are primarily intact, therefore allowing continued long-term strength gain.

The percentage of volume increase was computed based on an initial sample that was 2 inches in diameter and 2.05 inches in height. The volume increase data is shown in Table 14. The UNI AFBC and PC stoker ash samples showed the greatest volume increases at 30% and 10%, respectively. The PC stoker samples had begun to show cracking and delineation. Also shown in Table 14 is the final percent strength gain (or loss) calculated as a percentage of the 7 day strength.

The PC3+4 mixed at 10% samples show a strength loss, most likely due to available water being used up. The PC stoker ash samples show a 308% strength gain; however, these samples were on the decline when tested. Soils stabilized with self-cementing fly ash continue to gain strength through pozzolanic reactions, as long as water is available and large amounts of sulfur are not present in the fly ash.

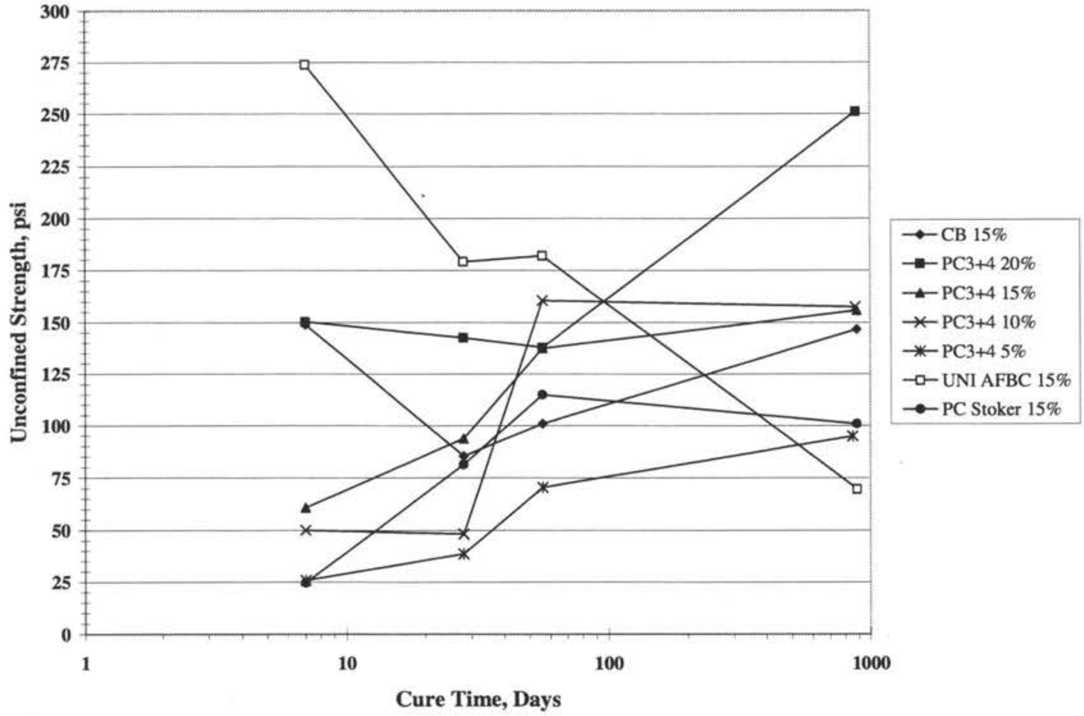


Figure 34. Long-Term Strength Gain of Glacial Till and Fly Ash

Figure 35. SEM Image Showing Ettringite Formation in AFBC Stabilized Glacial Till

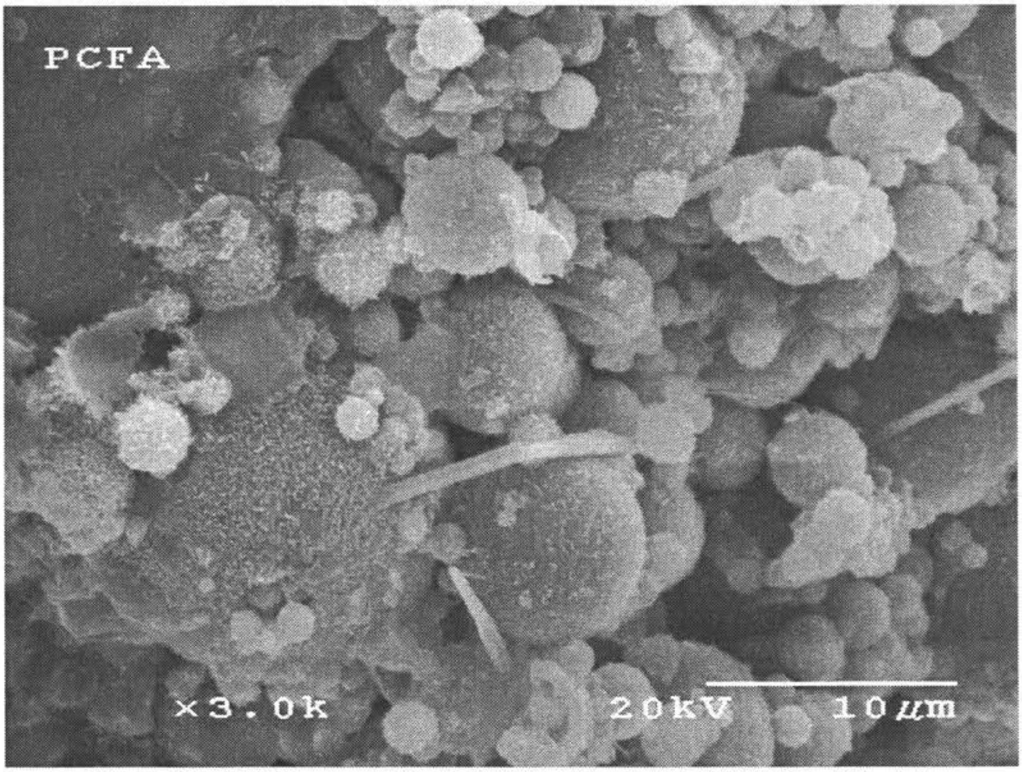


Figure 36. SEM Image of Glacial Till Stabilized with 20% PC 3+4 Fly Ash

Table 14. Volumetric Expansion and Strength Gain of Stabilized Glacial Till

Ash Type	Addition Rate	New Volume, in^3	Initial Volume, in^3	Volume Change, in^3	Volume Change, %	Strength Gain, %
AFBC	15%	8.43	6.44	1.99	30.97%	-75% *
PC3+4	20%	6.91	6.44	0.47	7.32%	67%
PC3+4	5%	6.65	6.44	0.21	3.29%	156%
CB	15%	6.87	6.44	0.42	6.60%	214%
PC3+4	15%	6.83	6.44	0.39	6.11%	266%
PC3+4	10%	6.86	6.44	0.42	6.59%	-2% *
PC Stoker	15%	7.13	6.44	0.69	10.67%	308%

* Negative Values Indicate Strength Loss

Unconfined Compressive Strength of Soil Stabilized with Self-Cementing Fly Ash

Unconfined compressive strength tests were performed on three different sizes of samples. True unconfined compressive strength is to be tested on samples having a length to diameter ratio of 2.0. Some samples were molded with a diameter of 2.8 inches and a length greater than 5.6 inches. Compressive strength was also measured on samples compacted in a standard 4-inch diameter Proctor mold with height of 4.584 inches. These samples had an L/D=1.15. Molding of the samples followed ASTM D1633 [Standard Test Method for Compressive Strength of Molded Soil-Cement Cylinders] (12). The last samples were compacted using the ISU 2-in x 2-in apparatus having an L/D of approximately 1.0. The ISU 2-in x 2-in method is described by Chu and Davidson (21) and the material was mixed in the same manner as the other samples.

Most samples were produced from the Le Grand and Turin loess soils, although some samples were molded using the alluvium and paleosol. True unconfined, Proctor size, and 2-in x 2-in samples were molded at fly ash contents of 5% (LGS), 10% (OGS), and 20% (PN4) for the Le Grand loess. These three samples sizes were also made for the alluvium and 15% CB fly ash. Proctor and 2-in x 2-in samples of Turin loess and 5% (PN3), 10% (PN4), 15% (OGS), and 20% (LGS) fly ash were molded. The last set of samples were made with the paleosol and 12 PC3+4 fly ash, in 2.8-in x 5.6-in and 2-in x 2-in sizes.

Soils were mixed in a range of moisture contents in order to produce moisture-strength relationships for the various addition rates of fly ash. One Proctor sized, two 2-in x 2-in cylinders, and two 2.8-in x 5.6-in specimens were molded for each moisture content.

The samples were compacted with a delay of less than 20 minutes. After molding, each sample was extruded, sealed in plastic wrap, and cured.

The Proctor and 2-in x 2-in samples were cured for 7 days in a 100° F oven, while the true unconfined samples were cured for 28 days in a 100% humid environment at 70° F. At the end of cure times, the samples were soaked in a water bath then tested in unconfined compression at a rate of 0.05 in/min. Proctor and unconfined samples were soaked 4 hours while the 2-in x 2-in specimens were soaked for one hour. In addition to moisture-strength relationships for each fly ash content, a strength correlation was developed between the three different sizes of samples. ASTM D1633 section 7.1 states that multiplying the strength of samples with L/D=2 by 1.10 has shown correlation to Proctor sized strength for soil-cement mixtures (12).

The 2-in x 2-in and Proctor sized strength results for the Le Grand loess and Turin loess are shown in Figures 37 and 38, respectively. The overall trend in both figures is that strength decreases when moisture content increases. Ferguson (26) stated that optimum moisture content for strength was 0% to 8% below optimum moisture for density. The figures show evidence that this may not be an accurate assumption. For the most part strength is lower at moisture contents below optimum for density.

An exception is in the case of each set of samples that contained 20% fly ash. The trend in these samples is that moisture content has no effect on strength below optimum for density. The 20% ash samples are strong enough to offset the deterioration effects of soaking which produces moisture-strength relationships similar to unsoaked soils. For both soil types it is observed that at high moisture contents, the amount of fly ash added has little influence, as the moisture-strength relationships tend to converge. The UCS results for the Le Grand loess are plotted in Figure 39. The samples with L/D=2 also show the moisture-strength relationships exhibited by the 2-in x 2-in and Proctor samples. There is low strength at low moisture contents, a peak in the middle of moisture range, followed by a decrease in strength at higher moisture content. The samples treated with 20% fly ash also seem to be free from the effects of soaking.

The data for alluvium and 15% CB fly ash are shown in Figure 40. These samples all exhibited the typical soaked moisture-strength relationship. It also appears as though these samples will converge at high moisture contents. The paleosol samples were used as another data set in the correlation of 2-in x 2-in strength to unconfined compressive strength. This set is presented in Figure 41. The figure shows the 2-in x 2-in samples had higher strength than the UCS samples. This is due to the fact that their L/D was 1.0, requiring more strength to mobilize the strength of the soil. Overall, soaking the samples produces lower strength than testing without soaking. The soaking was used to measure the strength of the material in its weakest condition. A limited number of each size of sample was produced at the beginning of each testing session, and testing was discontinued because each natural soil dissolved during soaking.

All of the data collected was plotted and correlations between samples sizes were attempted. Figure 42 shows the correlation plot between the 2-in x 2-in and Proctor sized samples. This data set produces an R^2 of 0.87. This shows a strong correlation between the two sizes of samples. This was expected due to the fact that the L/D values are close to each other, i.e., 1.0 and 1.15.

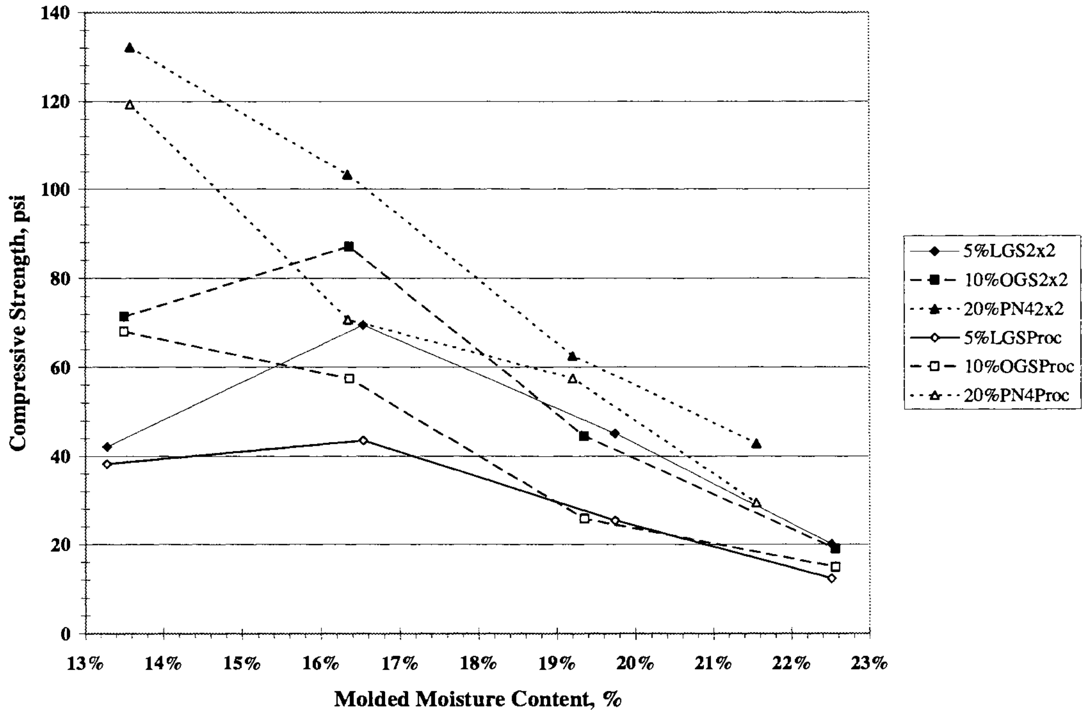


Figure 37. Compressive Strength of 2-in x 2-in and Proctor Samples of Le Grand Loess and Fly Ash

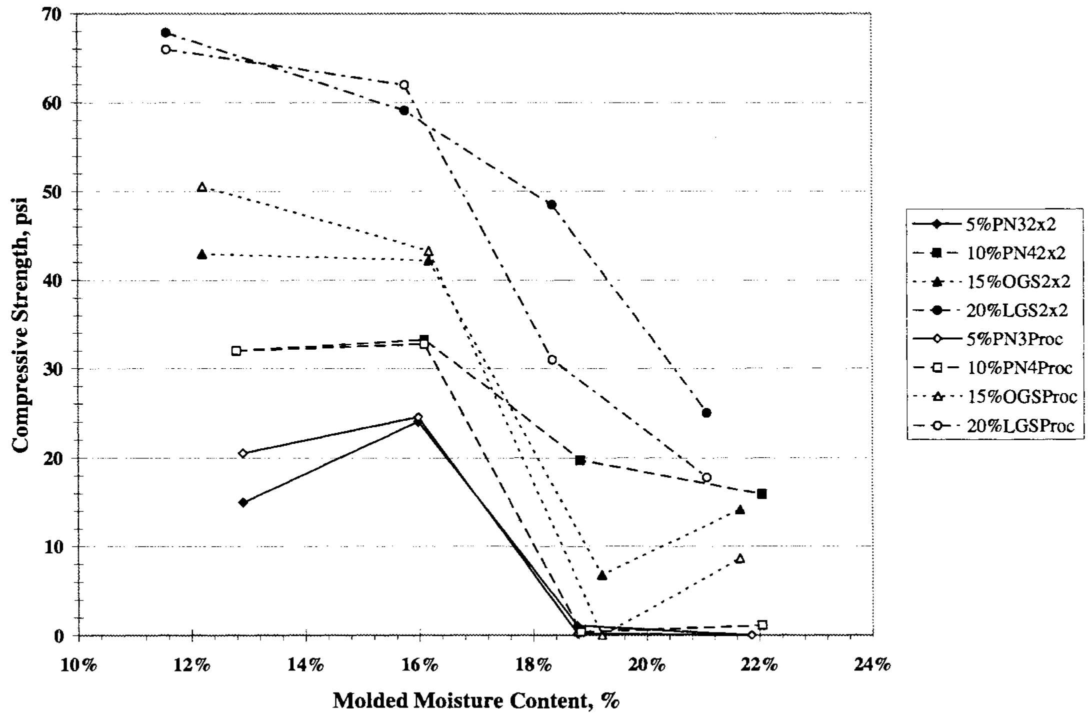


Figure 38. Compressive Strength of 2-in x 2-in and Proctor Samples of Turin Loess and Fly Ash

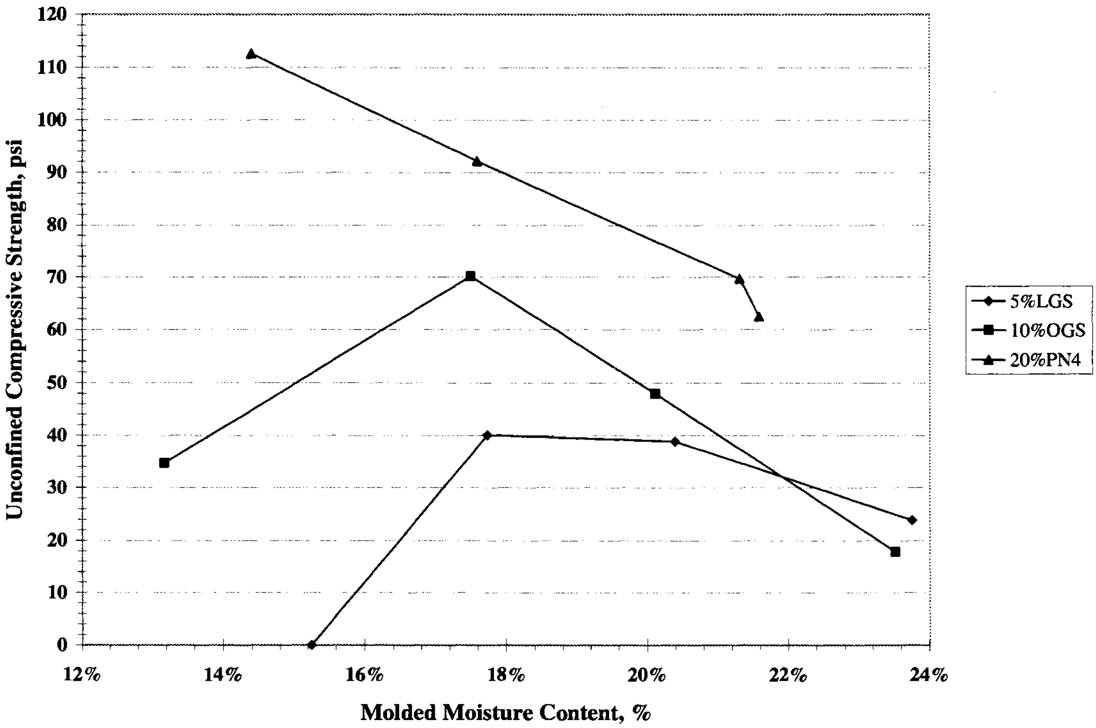


Figure 39. Unconfined Compressive Strength of Le Grand Loess and Fly Ash

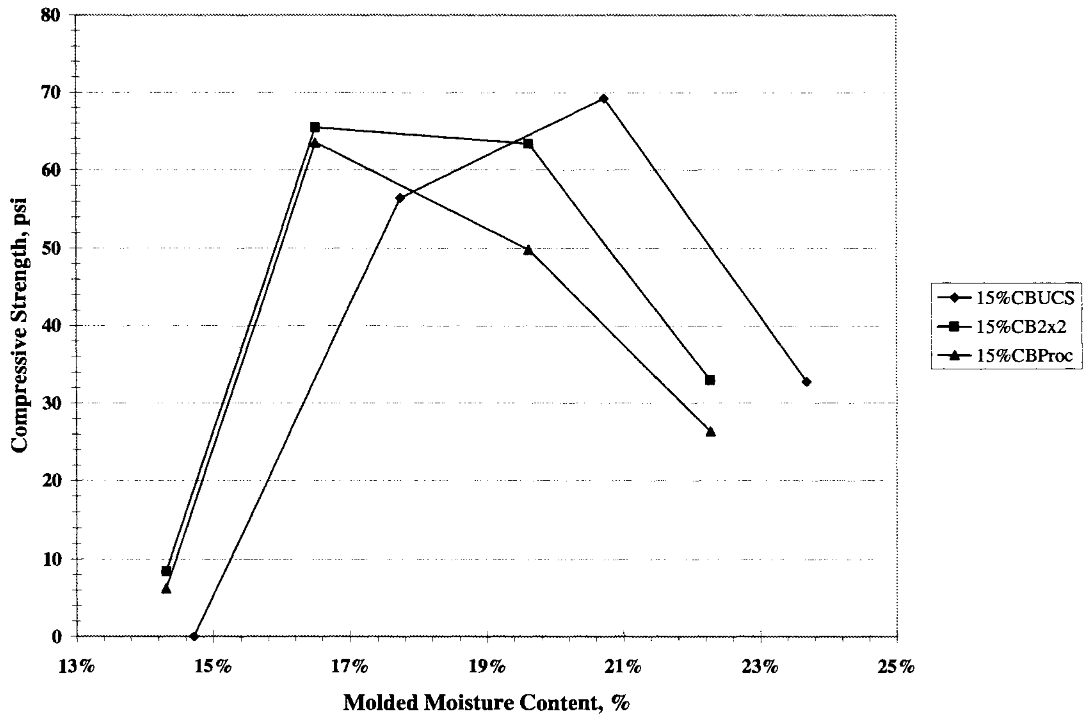


Figure 40. Strength Results of Alluvium Stabilized with CB Fly Ash

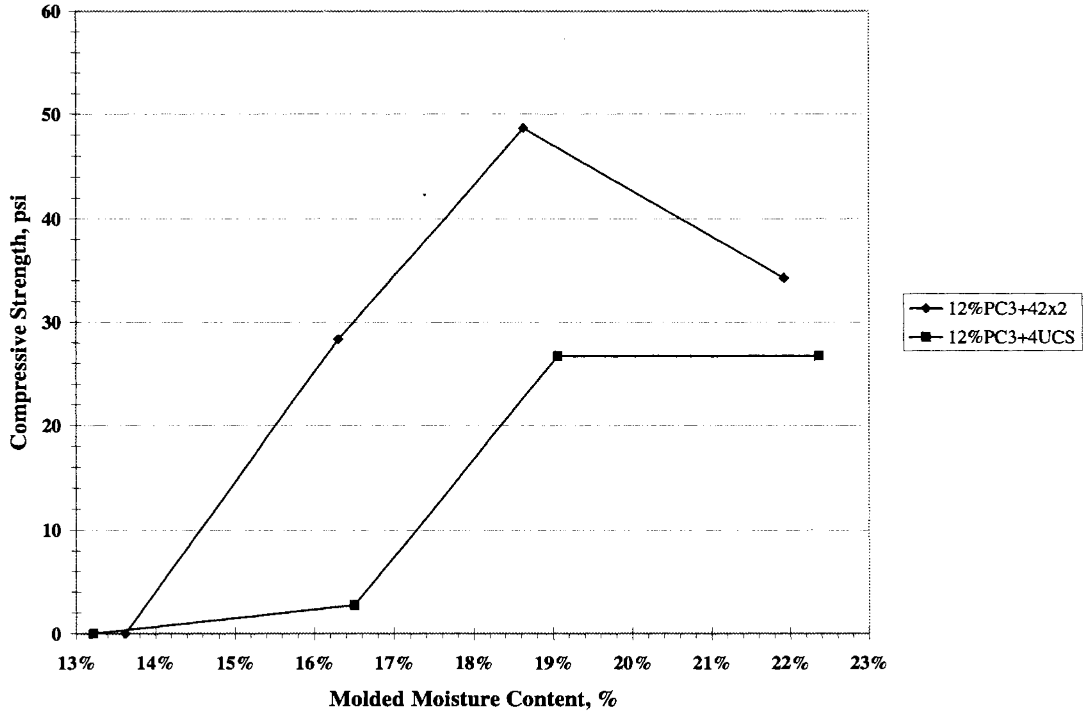


Figure 41. 2-in x 2-in and UCS Strength of Paleosol and Fly Ash

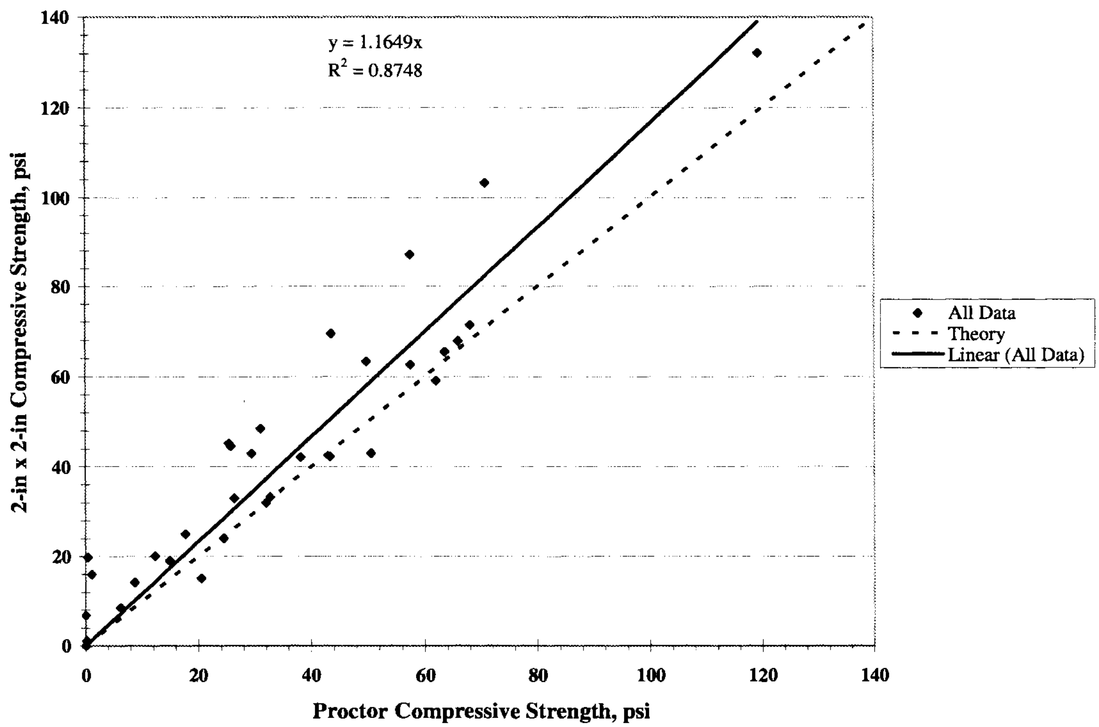


Figure 42. Correlation Between 2-in x 2-in and Proctor Compressive Strengths

The correlations of 2-in x 2-in and Proctor versus true UCS are shown in Figures 43 and 44, respectively. The 2-in x 2-in samples have an R^2 of 0.70 while the R^2 of the Proctor samples is 0.60. There is a lot of scatter in both figures. The best fit equations show that the UCS must be reduced to meet Proctor strengths and increased to meet 2-in x 2-in strengths. ASTM D1633 (12) states that multiplying the UCS of soil-cement by 1.10 gives an approximation of the Proctor compressive strength for the same material As a comparison to ASTM D1633 the UCS strengths were corrected by multiplying by 1.10 and these strengths and correlations are plotted in Figures 45 and 46. Figure 45 shows the 2-in x 2-in data and Figure 46 shows the Proctor data. The R^2 values are the same as before for each set of data, but in this case the corrected UCS would have to be reduced even further to match the Proctor strengths because it was to be reduced before the correction was applied. The 1.10 correction factor does not appear to be applicable for soil-fly ash Proctor and UCS sized samples, but shows a good relationship between UCS and 2-in x 2-in samples. The linear trend line for 2-in x 2-in and corrected UCS strengths shows these values are approximately in a one to one relationship. Overall, a correlation between seems to exist between the sample sizes, but more data is needed to more accurately predict the true correlations. This may be somewhat difficult as fly ash and soil are variable.

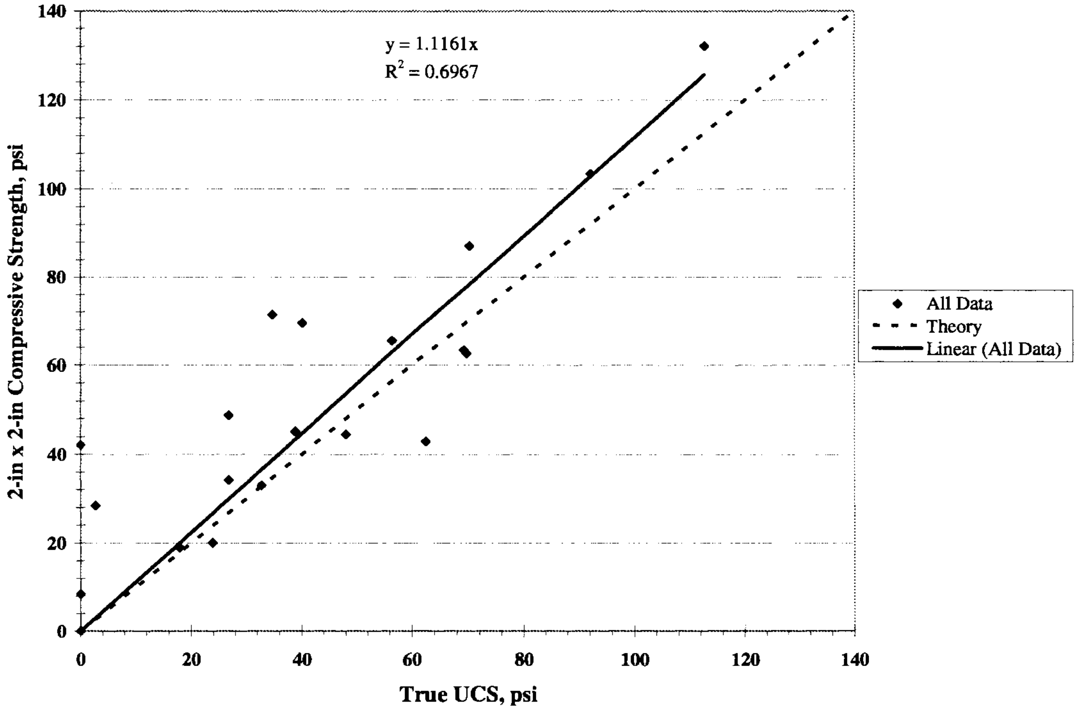


Figure 43. Correlation Between 2-in x 2-in Compressive Strength and True UCS

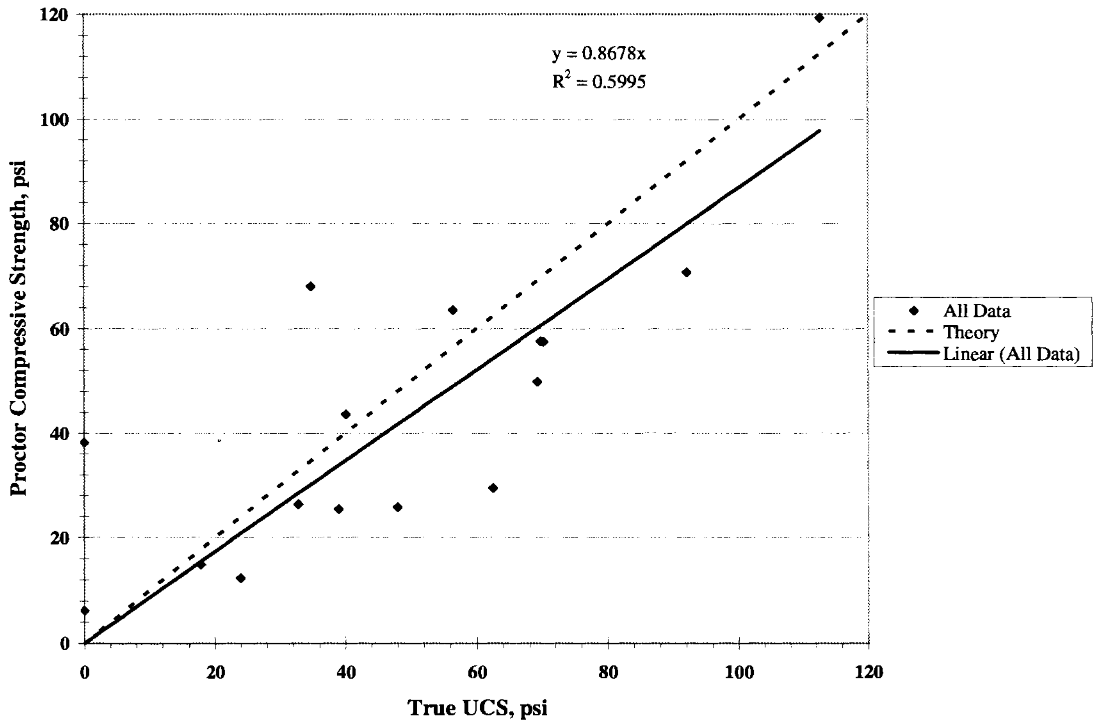


Figure 44. Correlation Between Proctor Compressive Strength and True UCS

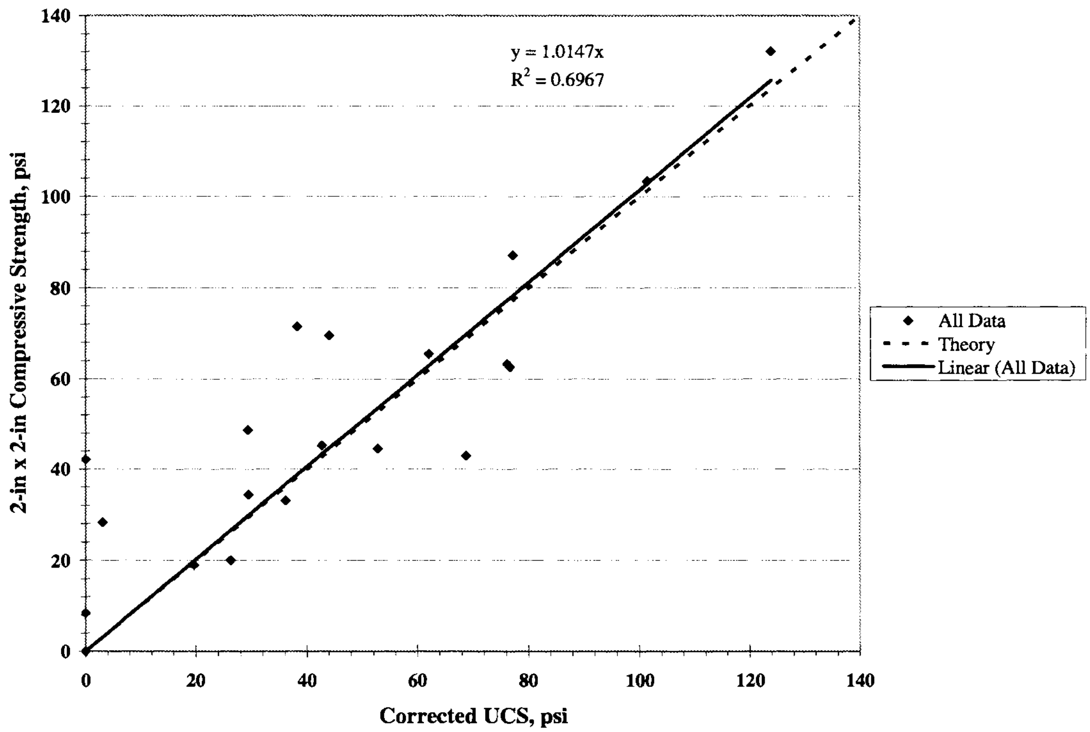


Figure 45. Correlation Between 2-in x 2-in Compressive Strength and Corrected UCS

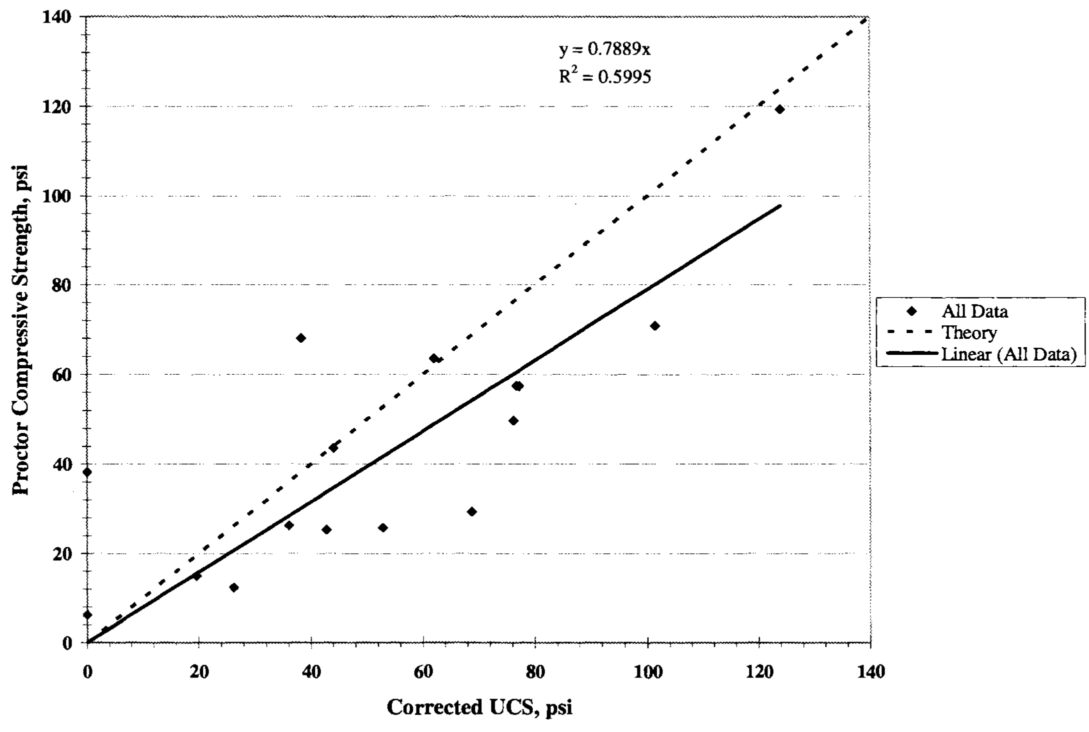


Figure 46. Correlation Between Proctor Compressive Strength and Corrected UCS

California Bearing Ratio of Self-Cementing Fly Ash Stabilized Soil

A series of CBR tests were conducted to determine CBR-moisture relationships for soil stabilized with fly ash. A correlation between CBR and the three unconfined compressive strength test methods was attempted. The Le Grand loess was the soil used in this study. The testing was carried out in compliance with ASTM D1883 [Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils] (13). After proper moisture was added to the natural soil, the soil was allowed to cure for 24 hours. The fly ash was added and the materials were mixed to a homogeneous mixture, as stated in ASTM D560 section 5.1.3 (8). The compacted CBR specimens were cured in a humidity room for 28 days, and allowed to soak for 96 hours prior to loading. The sample height was determined prior to and directly after soaking to measure swell. A summary of the CBR values and swell percentages is shown in Table 15. The CBR values are also plotted against molded moisture content in Figure 47.

Table 15. Summary of Swell Values and CBR of Le Grand Loess and Fly Ash

Sample #	Molded Moisture Content, %	Ash Type	Addition Rate, %	Soaked CBR, %	Swell, %
1	13.9%	No Ash	0%	1	1.68%
2	17.2%	No Ash	0%	2	0.80%
3	19.9%	No Ash	0%	3	0.34%
4	23.4%	No Ash	0%	3	0.13%
5	14.0%	LGS	5%	5	0.46%
6	17.2%	LGS	5%	13	-0.02%
7	20.2%	LGS	5%	18	0.02%
8	23.4%	LGS	5%	7	0.04%
9	14.3%	OGS	10%	26	0.11%
10	17.2%	OGS	10%	26	0.02%
11	20.4%	OGS	10%	20	0.00%
12	23.2%	OGS	10%	10	-0.02%
13	14.4%	PN4	20%	70	0.02%
14	17.2%	PN4	20%	75	0.02%
15	20.4%	PN4	20%	44	0.02%
16	23.1%	PN4	20%	24	0.02%

The samples that were not treated with fly ash experienced the greatest volume change, as was expect. Also, samples compacted near 14% moisture exhibited the most swell. This is typical of fine grained soils compacted at lower moisture contents. The values listed as -0.02%, 0.02%, and 0.04% were likely due to variations in placement of the deflection gauge after the samples had soaked. The samples may not have shown any swell at all. In any case, swell of -0.02%, 0.02%, and 0.04% is very minimal.

The moisture-CBR curves show the relationship of increasing CBR with increased fly ash addition, similar to other moisture-strength relationships. The Le Grand loess alone exhibited CBR values from 1 to 3%, typical for saturated Iowa soils. These low values indicate this material would provide very little support for overlying pavement structures. With 5% addition of LGS fly ash to the loess, the CBR increased to values that are found for select fine grained soils, typically in the range of 5 to 15% (40). The CBR versus moisture plot for loess and 10% OGS ash has higher values at low moisture content and then begins to decrease at higher moisture content. The range of CBR values of 10 to 26% is typically found in clayey or poorly graded sands (40). The relationship for the loess stabilized with 20% fly ash is similar to the other samples mixed with 20% fly ash. Enough fly ash was present so that at lower moisture contents samples did not weaken after soaking. And as with the other samples, CBR decreased at higher moisture contents. The 20% ash samples exhibited soaked CBR values near 75%, which are very uncommon for fine grained soils. These samples showed strengths similar to gravels (40).

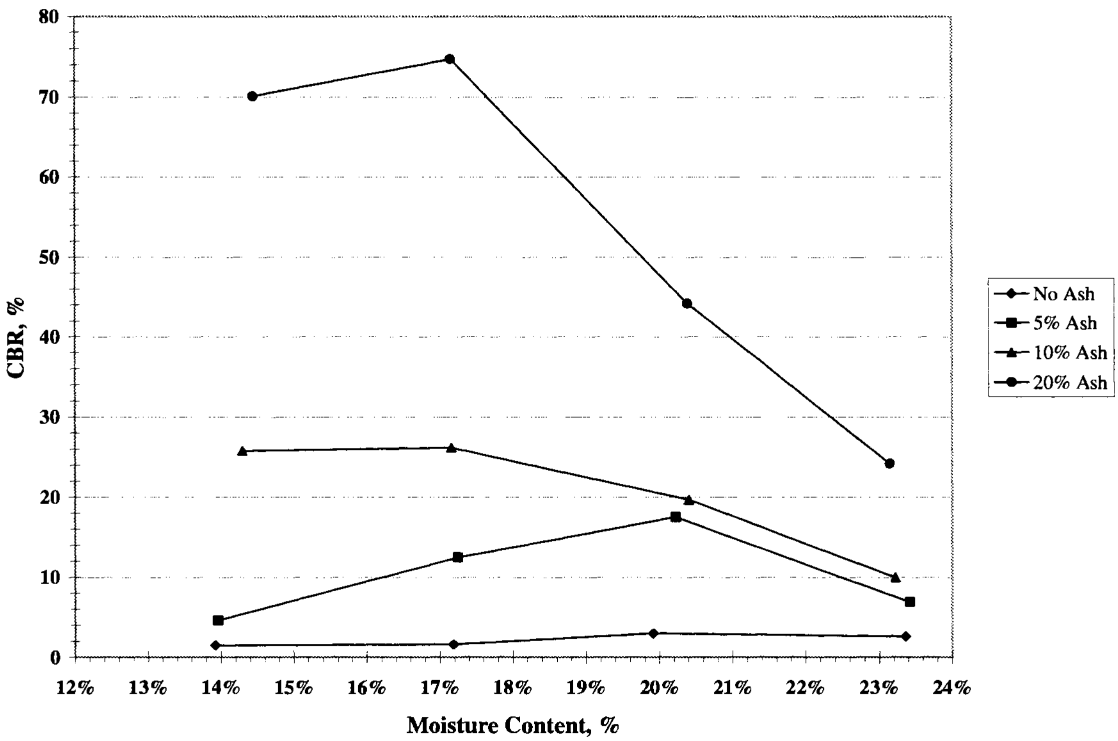


Figure 47. CBR Versus Moisture Content of Le Grand Loess and Fly Ash

Correlation plots between CBR and unconfined compressive strength, compressive strength of 2-in x 2-in cylinders, and compressive strength of standard 4-inch diameter Proctor sized samples are shown in Figures 48, 49, and 50, respectively. As expected, there was a general trend of increasing CBR with increasing compressive strength of all sample sizes. The relationship is not linear, but rather parabolic. This could partially be due to the fact that the CBR samples are confined during testing, therefore gaining strength due to lateral stress resistance. The UCS correlation had the largest R^2 value, which is 0.86, while the Proctor-sized samples only provided an R^2 of 0.66. The 2-in x 2-in correlation R^2 was 0.73. Typically values of R^2 less than 0.8 shows there is a moderate correlation between parameters. The R^2 values are on the low side of acceptability, but an overall trend is present. More data points are needed to fully understand the relationship between CBR and unconfined compressive strength, and compressive strength of materials with $L/D<2$.

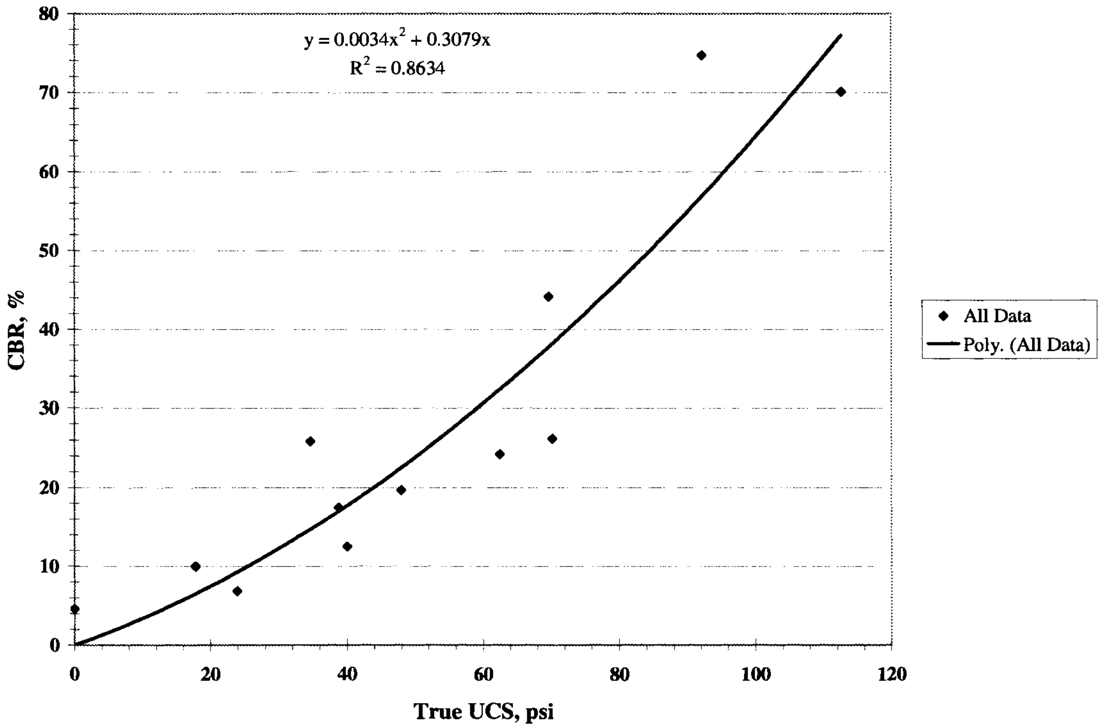


Figure 48. Correlation Between CBR and UCS for Le Grand Loess and Fly Ash

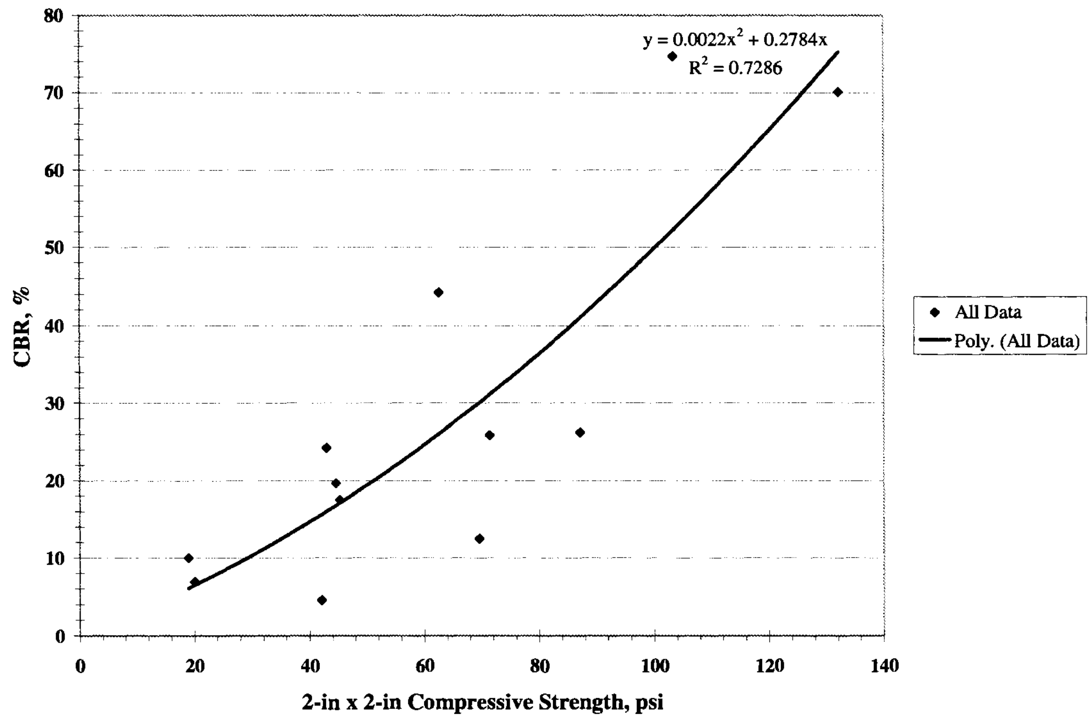


Figure 49. Correlation Between CBR and 2-in x 2-in Compressive Strength for Le Grand Loess and Fly Ash

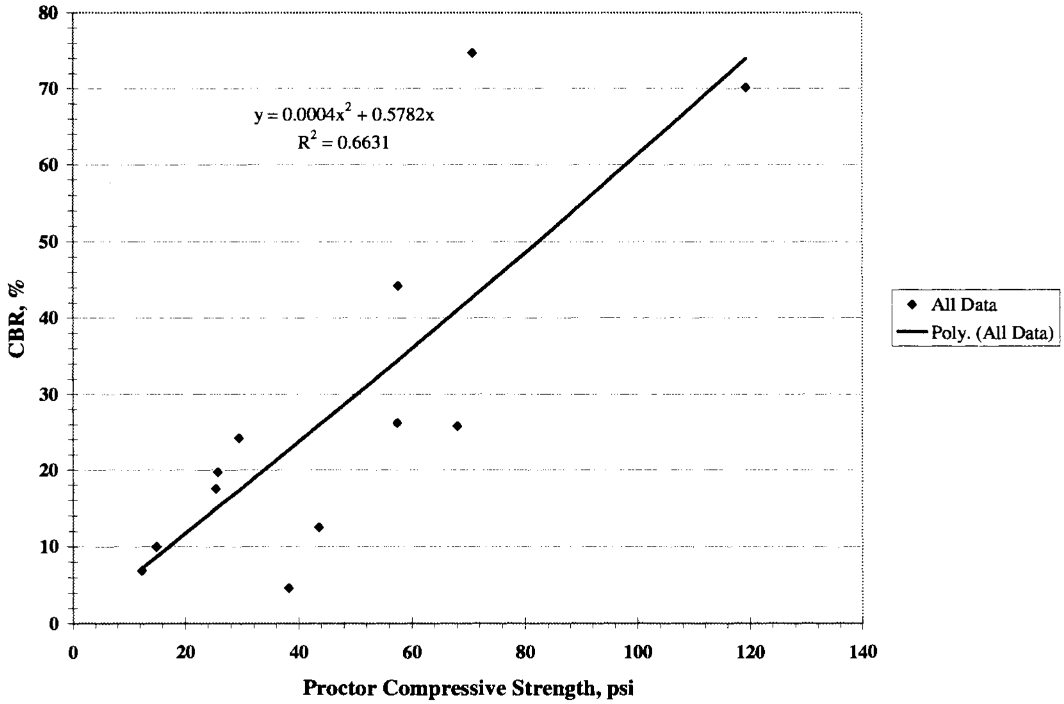


Figure 50. Correlation Between CBR and Proctor Compressive Strength for Le Grand Loess and Fly Ash

SOIL MODIFICATION DUE TO THE ADDITION OF SELF-CEMENTING FLY ASH

Self-cementing fly ash can be used to modify engineering properties of soil. Fly ash changes the plasticity characteristics of soil, thereby reducing swell potential. Addition of fly ash to expansive soils has been shown to reduce swell not only by lowering the plasticity, but also by cementing soil particles together. Self-cementing fly ash can be used to dry and add strength to wet soils in order to facilitate construction operations.

Self-Cementing Fly Ash as a Drying Agent

In most instances earthwork operations are halted due to extremely wet soil conditions. A way to correct this problem is to mix self-cementing fly ash with wet soils. Fly ash addition increases the volume of dry material, therefore reducing the moisture content. As fly ash uses soil water to hydrate, the mixture gains strength to provide a stable working platform for construction equipment.

OGS, PN4, SGS, and PC3+4 fly ashes were mixed with saturated Turin loess in this study. Water was added to Turin loess to raise the moisture content to approximately 33%. Each fly ash was then mixed with the soil at 10, 20, and 30% by dry weight. After adding and mixing fly ash, the moisture content of the mixture was determined and the soil-fly ash mixture was placed in a shallow dish. Readings were taken with the pocket penetrometer to measure strength gain of the mixture, as in the fly ash set time test.

The amount of moisture content change is shown in Table 16 for all tests. The moisture content differences are also shown in Figure 51, with a theoretical relationship for moisture content difference. The theoretical relationship was calculated based on the following equation:

$$\Delta w\% = \frac{Ww}{Ws}\left(1 - \frac{1}{1 + \frac{\%FA}{100}}\right) \quad \textbf{Equation 1}$$

All of the test results lay above the theoretical line except for the SGS 10% result. The reason for this is unknown, but it is believed this is due to sample variability. It is thought that the actual moisture content difference varies from theoretical difference because fly ash not only increases the volume of dry material but that water is used during initial cementitious reactions. Overall the results are in good agreement with each other at each fly ash addition rate.

Table 16. Summary of Moisture Decrease With Addition of Self-Cementing Fly Ash

Ash Type	Addition Rate, %	Initial Moisture, %	Final Moisture, %	Moisture Change, %
SGS	10%	32.5%	29.9%	2.6%
SGS	20%	33.8%	27.1%	6.7%
SGS	30%	33.7%	24.8%	8.9%
PC 3+4	10%	33.3%	28.8%	4.5%
PC 3+4	20%	32.7%	25.7%	7.0%
PC 3+4	30%	32.7%	23.7%	9.0%
OGS	10%	33.3%	29.1%	4.2%
OGS	20%	32.3%	25.7%	6.6%
OGS	30%	33.1%	23.7%	9.4%
PN4	10%	32.9%	28.6%	4.3%
PN4	20%	32.8%	26.6%	6.2%
PN4	30%	32.4%	23.7%	8.7%

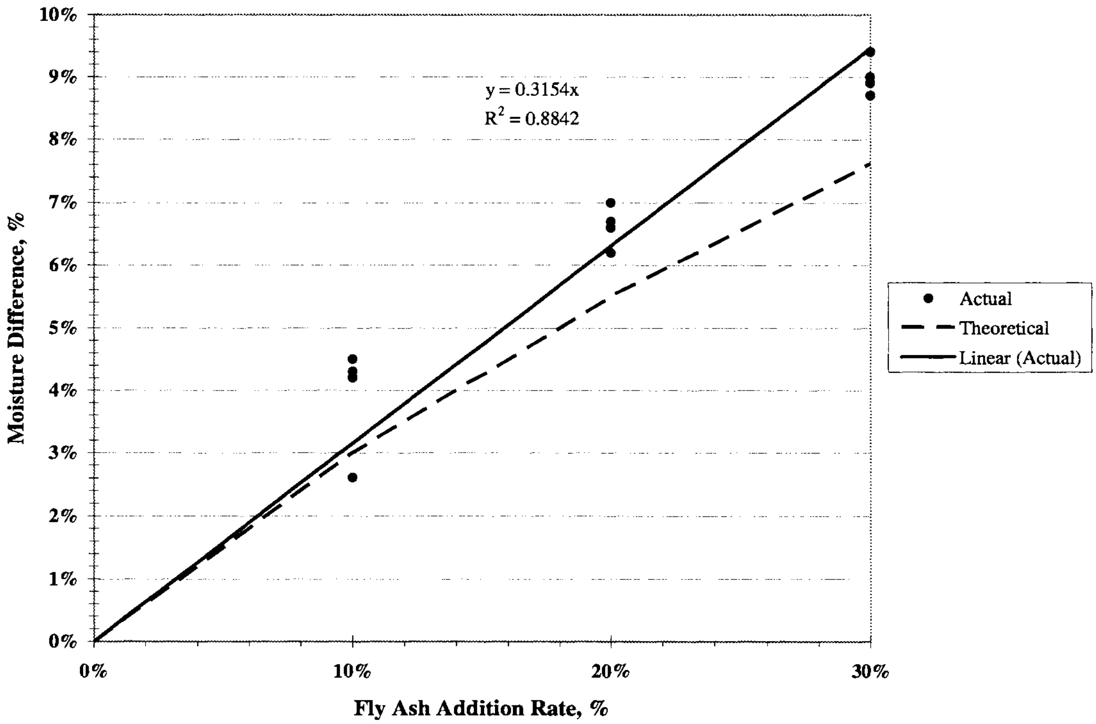


Figure 51. Actual Moisture Difference Versus Theoretical Moisture Difference

The strength gain results for the SGS, PC3+4, OGS, and PN4 fly ash-loess mixtures are shown in Figures 52, 53, 54, and 55, respectively. It was observed that all fly ashes provided some strength to the soil, as was expected. Also each set of tests for each fly ash shows the relationship of higher ash addition providing more strength. The SGS fly ash tests showed the largest strength gains overall for each ash content, closely followed by PN4 tests. The 30% tests for these two fly ashes both reached a refusal reading of 4.50 tsf during testing. Twenty percent SGS and PN4 reached four hour penetration resistances of 4.35, and 4.00 tsf, respectively. The PC3+4 30% test shows a strength of 4.1 tsf after four hours, and a moderate strength gain of 1.75 tsf was observed for 20% fly ash addition. The OGS tests show the soil was dried but little strength gain occurred. The strength results for the OGS are 0.45, 1.00, and 2.00 tsf, for 10, 20, and 30% ash addition, respectively. The 10% ash tests had a range of 0.45 to 1.35 tsf, meaning that 10% fly ash addition did not provide much strength.

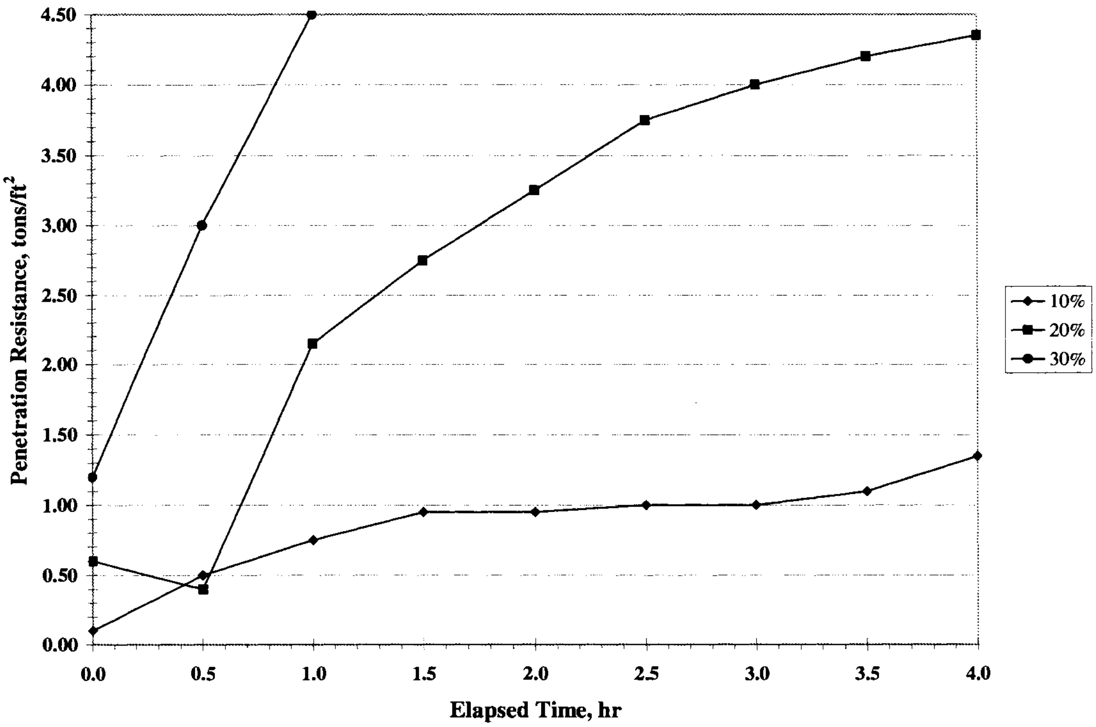


Figure 52. Strength Gain of Turin Loess Dried with SGS Fly Ash

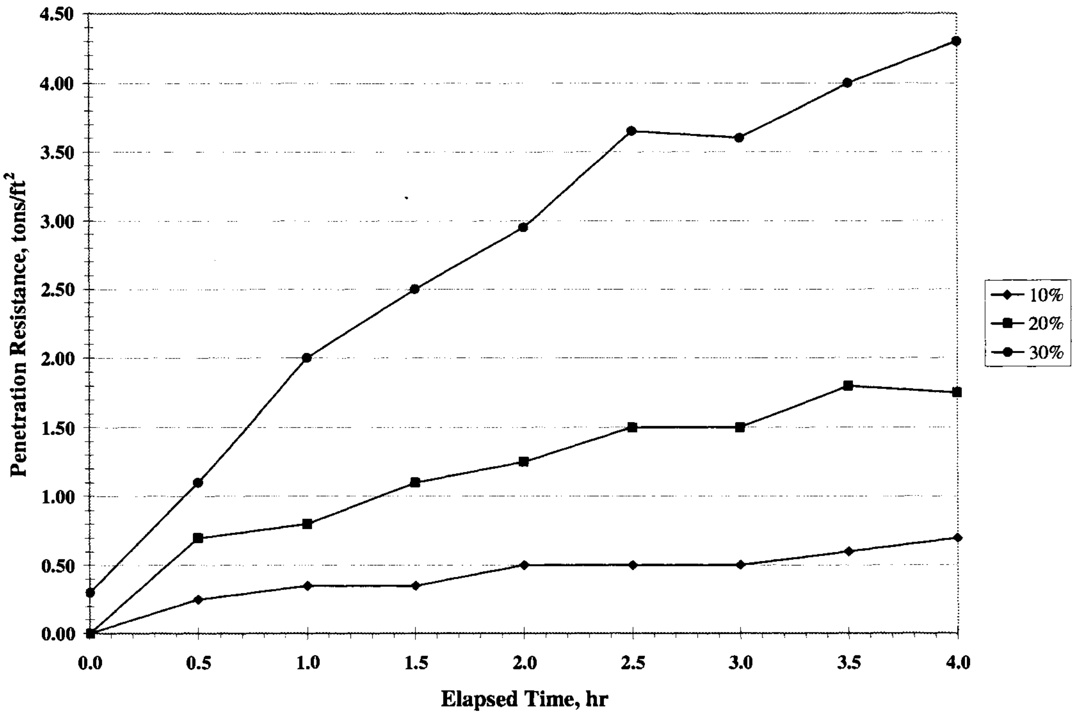


Figure 53. Strength Gain of Turin Loess Dried with PC3+4 Fly Ash

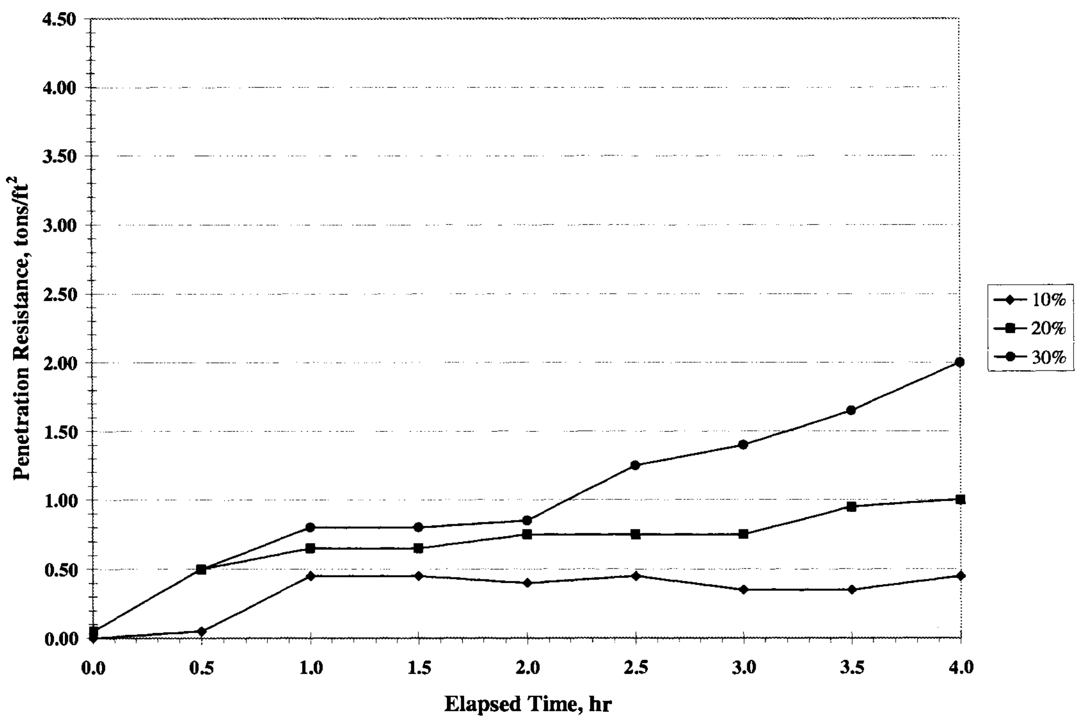


Figure 54. Strength Gain of Turin Loess Dried with OGS Fly Ash

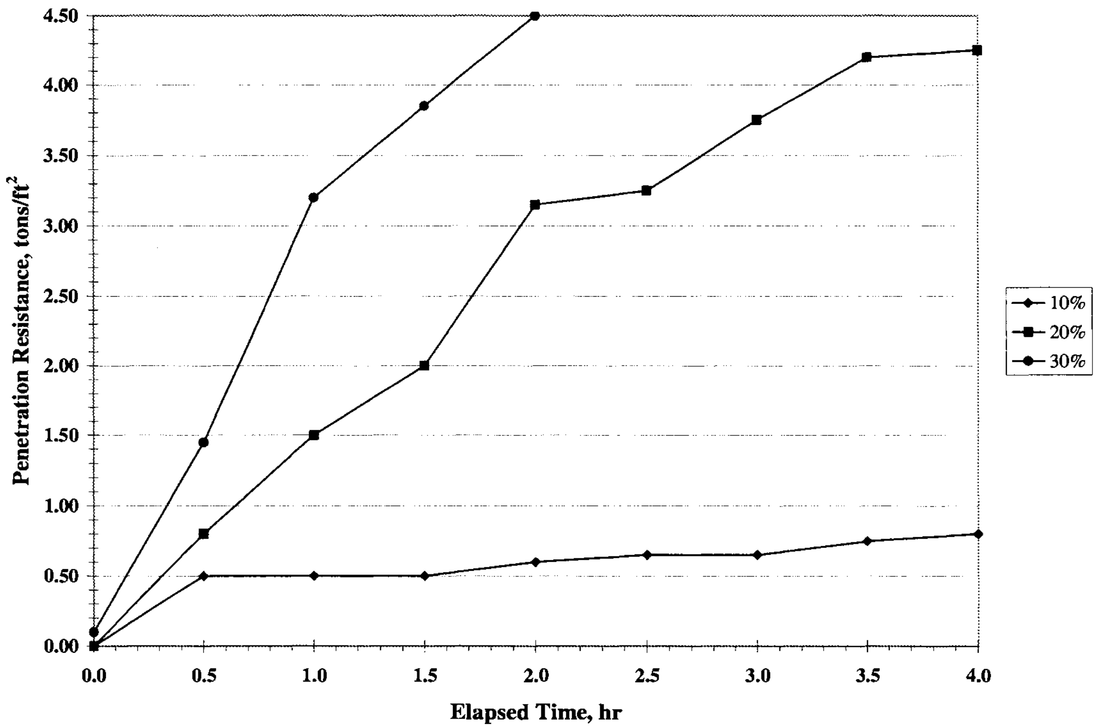


Figure 55. Strength Gain of Turin Loess Dried with PN4 Fly Ash

Reduction of Swell Potential with Self-Cementing Fly Ash Addition

Self-cementing fly ash can reduce swell potential of expansive soils. ASTM D4829 [Standard Test Method for Expansion Index of Soils] (16) was followed for this series of tests. Tests varied from ASTM in that samples were molded in a 4-inch diameter Proctor mold, extruded, and then the ring from the expansion test apparatus was placed around the molded samples, and the samples were then trimmed. This was done in order to provide more uniform compaction to the samples. Expansion index (EI) is calculated by multiplying the change in sample height by 1000 and dividing this by the original sample height. The test method is used for saturation values of 50%, and the calculated EI is adjusted based on the original sample saturation. Saturation of 50% is used because the samples take in more water which leads to swelling. In practice material is not compacted at 50% saturation so this test method provides a worst case scenario for design.

The till, paleosol, and Le Grand loess were tested by themselves, and mixed with 10 and 20% Ames fly ash. The samples were extruded and placed in the expansion test apparatus immediately after molding, therefore cementing effects due to fly ash hydration had not occurred. The fly ash acted as a mechanical stabilizer by reducing the volume of expansive clay particles. All stabilized samples exhibited their final swell within 24 hours of saturation. After 24 hours the fly had cemented the soil particles together, stopping expansion. Most of the swell exhibited by the natural soils was completed within 24 hours, but these samples exhibited minimal continued swell until the tests were stopped. The results are shown in Table 17. Overall, the till tests showed low swell potential. Swell potential should be further reduced as fly ash content increases, but this was not the case for the 10% fly ash-till sample. The EI is low but it is slightly higher than the natural soil. The variability of the soil could have been a cause of this observation. The paleosol exhibited the largest amount of swell. The paleosol itself has an EI of 84, while 10 and 20% ash treated samples had EI values of 66 and 62, respectively. EI values of 66 and 62 indicate the material still has medium potential to swell. The Le Grand loess itself exhibited low swell potential with an EI of 34, and fly ash addition lowered the EI to 19 for the 10% fly ash mixture and 8 for the 20% fly ash mixture. The addition of 20% Ames fly ash almost removed all swell potential of the Le Grand loess. The stabilized samples would have had lower EI values if the mixtures had been allowed to cure and harden before being saturated and tested.

Table 17. Expansion Indices of Soils Stabilized with Self-Cementing Fly Ash

Soil Type	Fly Ash	Expansion Index	Swell Potential
Glacial Till	No Ash	27	Low
	10%	31	Low
	20%	21	Low
Paleosol	No Ash	84	Medium/High
	10%	66	Medium
	20%	62	Medium
Le Grand Loess	No Ash	34	Low
	10%	19	Very Low
	20%	8	Very Low

A small study was initiated to monitor the effects of sulfur content and potential ettringite formation on EI. The SGS fly ash was used in this study along with the UNI AFBC and PC1+2 stoker fly ashes. The SGS fly ash meets the ASTM C618 (9) requirement of less than 5% sulfur, while the AFBC and PC1+2 stoker fly ashes have sulfur contents of 29% and 6%, respectively. The fly ashes were mixed with the paleosol and molded in the same manner as the previous tests. These tests are on-going and the results are reported after 60 days of testing. Table 18 shows the results of testing. All samples reduced the EI of the paleosol to values in the range for medium swell potential. The EI values were 53, 63, 67 for the AFBC, SGS, and PC1+2 ashes, respectively. The displacement readings are presented in Figure 56. The AFBC mixture has continued to slowly swell while the SGS mixture is beginning to consolidate. The PC1+2 sample is also continuing to swell, at a higher rate than the AFBC sample. It was noted earlier that the AFBC fly ash has a rapid initial strength gain, the reason for the smallest EI, but that ettringite formation decreases the strength of AFBC stabilized soil. It is believed that this phenomenon will also occur to the expansion sample and swelling will again commence.

Table 18. Expansion Indices of Paleosol and High Sulfur Fly Ashes

Fly Ash	Sulfur Content, %	Expansion Index*	Swell Potential
UNI AFBC	29	53	Medium
SGS	2.8	63	Medium
PC1+2	6	67	Medium

*Results at 60 days

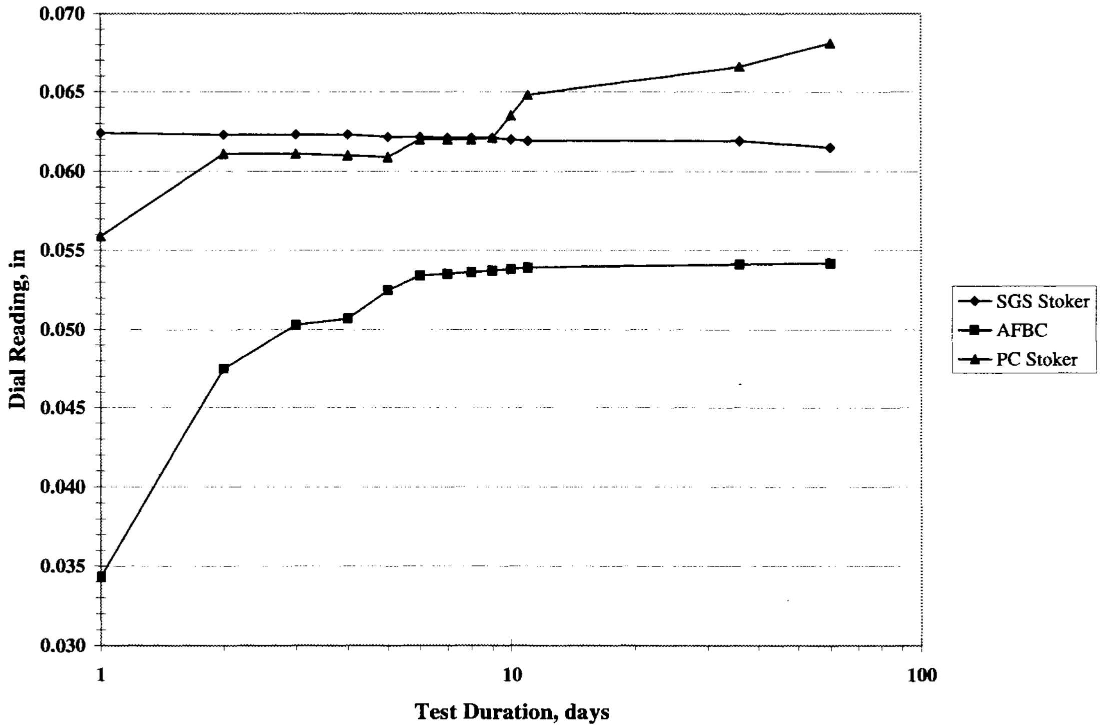


Figure 56. Swell of Paleosol-High Sulfur Fly Ash Mixtures

Modification of Plasticity Characteristics of Soils

As with other calcium based stabilizers, fly ash has been found to reduce the plasticity of fine-grained soils (26, 23, 38, 52, 37). Two sets of tests were conducted to monitor effects of fly ash addition on plasticity characteristics of soil. In the first test 20% (by dry weight) of five different fly ashes were mixed with the alluvium. The fly ash-soil mixture was compacted in a 4-inch diameter Proctor mold, extruded, sealed and cured in a 100% humid environment at 70° F. After curing for 7 and 28 days, samples were prepared and tested in accordance with ASTM D4318 (15). The second trial consisted of mixing paleosol with ash contents of 5, 10, 15, and 20% LGS fly ash. The samples were prepared and cured in the same manner as the alluvium samples. The paleosol-fly ash mixtures were allowed to cure 28 days before testing.

The test results of plasticity change with time are shown in Figure 57. The alluvium has a liquid limit of 47 and plasticity index of 25 and classifies as lean to heavy clay. After 7 days all of the fly ash treatments reduced the liquid limit to a range of 40 to 46. Plasticity index was also reduced from 25 to the range of 13 to 20. Twenty-eight days of curing produced liquid limit and plasticity index high values of 42 and 12, respectively. The results of this study along with the USCS symbols of the stabilized mixture are located in Table 19. After 7 days, plasticity had decreased enough that these materials classified as lean clays. Plasticity was further reduced to the point that the mixtures behaved as low plasticity silts to lean clays at 28 days. Cations (Ca^{+2}, Mg^{+2}, Na^{+}, K^{+}) from fly ash are attracted to the clay particles, reducing the thickness of the diffuse double layer and creating a flocculated structure. This flocculated structure leads to the soil particles becoming aggregated and they behave as a mass instead of individual particles. The reduction in diffuse double layer thickness and flocculated structure causes the soil to exhibit less plasticity. As soil-fly ash cures, more cations are attracted to clay particle surfaces and reduction in plasticity continues. The liquid limit data are shown in Figure 58. It is noted that for each respective fly ash, the slope of the best fit lines remain relatively constant. The lines shift down and left with increased cure time, showing effects of cations being attracted to the clay particle surfaces.

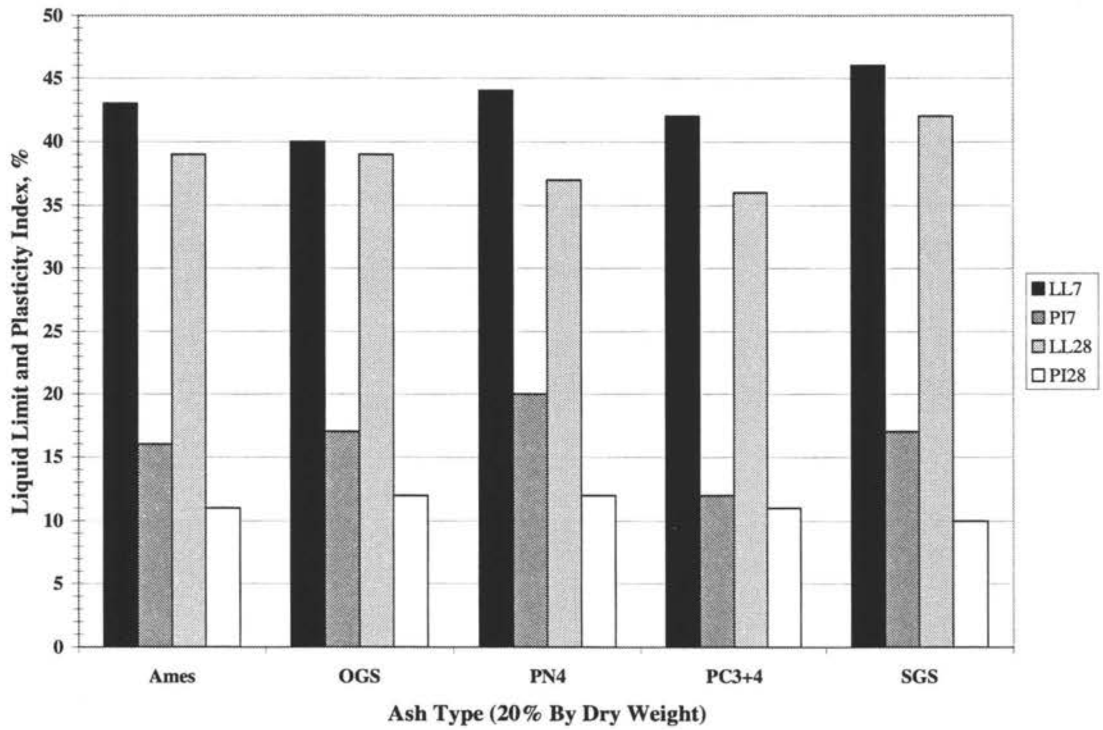


Figure 57. Liquid Limit and Plasticity Index of Alluvium and 20% of Various Self-Cementing Fly Ashes

Table 19. Atterberg Limits and USCS Symbols of Alluvium and 20% Various Self-Cementing Fly Ashes

Soil/Ash	**Alluvium**	**Ames**		**OGS**		**PN4**		**PC3+4**		**SGS**	
Cure, days	0	7	28	7	28	7	28	7	28	7	28
LL	47	43	39	40	39	44	37	42	36	46	42
PL	22	27	28	23	27	24	25	30	25	29	32
PI	25	16	11	17	12	20	12	12	11	17	10
USCS	CL-CH	CL	ML	CL	ML	CL	ML-CL	CL	CL-ML	CL	ML

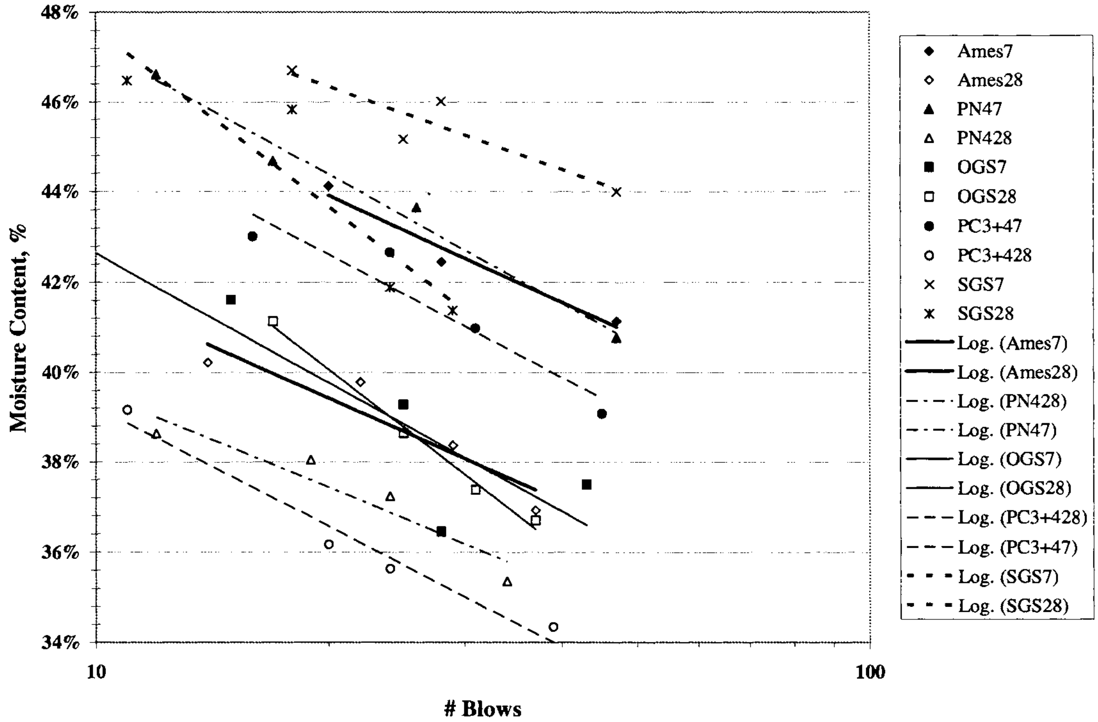


Figure 58. Liquid Limit Change of Alluvium and 20% of Various Self-Cementing Fly Ashes

The effects of fly ash addition rate are shown in Figure 59. It was theorized that as the fly ash content increased, the liquid limit and plasticity index would continue to decrease. This was true for fly ash contents of 5, 10, and 15%. The 20% fly ash-paleosol mixture exhibited higher plasticity compared to the other three addition rates. A possible explanation for this is that too many cations were available, therefore causing clay particles to repel each other, creating a dispersed structure. The results and USCS symbols are shown in Table 20. The plastic limit was increased to 19 and 20 for all fly ash treatments; the liquid limit was the defining parameter in the change of USCS classification. Each fly ash treatment modified the paleosol so it exhibited behavior of lean clay.

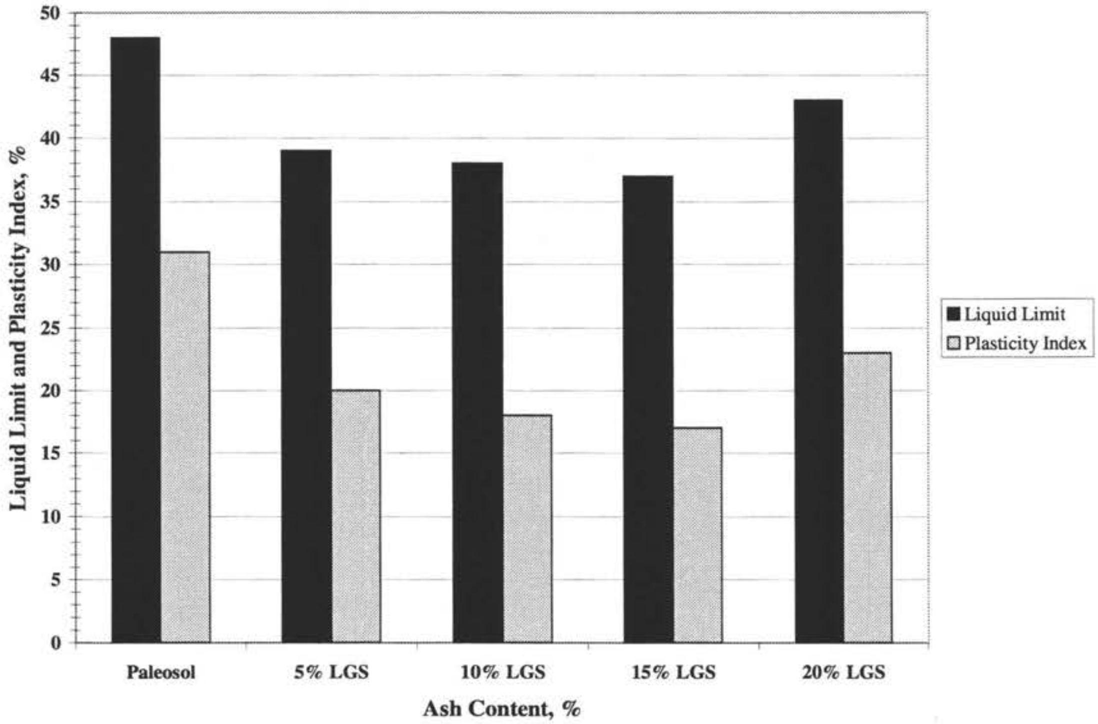


Figure 59. Influence of Fly Ash Addition Rate on Atterberg Limits of Paleosol

Table 20. Summary of Atterberg Limits of Paleosol and Various Amounts of LGS Fly Ash

Soil/Ash	**Paleosol**	**5% LGS**	**10% LGS**	**15% LGS**	**20% LGS**
Cure, days	0	28	28	28	28
LL	48	39	38	37	43
PL	17	19	20	20	20
PI	31	20	18	17	23
USCS	CL-CH	CL	CL	CL	CL

INFLUENCE OF CURING ENVIRONMENT ON STRENGTH GAIN AND FREEZE-THAW DURABILITY OF SOIL-FLY ASH

Curing Environment

ACAA (26) and Vandenbossche and Johnson (46) have recommended that soil-fly ash mixtures be cured at temperatures greater than 40° F in the field. The recommendations were made because fly ash does not gain strength as rapidly as temperature decreases. Effects of curing environment were evaluated in a series of tests. Short-term strength gain was evaluated for different fly ash contents. Turin loess and paleosol were both used in this study. Each soil was mixed with enough water to provide a final compacted moisture content of 19% for the soil fly ash mixtures and allowed to cure for 24 hours. After 24 hours, 10, 15, and 20% (by dry weight) fly ash were mixed with the soils. Turin loess was mixed with PN4 fly ash and paleosol was mixed with LGS fly ash. After mixing, samples were compacted immediately (< 15 minutes) using the ISU 2-in x 2-in (L/D=1.0) apparatus. Samples were extruded, sealed, and cured 7 days at 8°, 70°, and 100° F. The samples were allowed to reach room temperature and then soaked in a water bath for one hour before compression testing at a rate of 0.05 in/min. Results for the Turin loess and paleosol are shown in Figures 60 and 61, respectively.

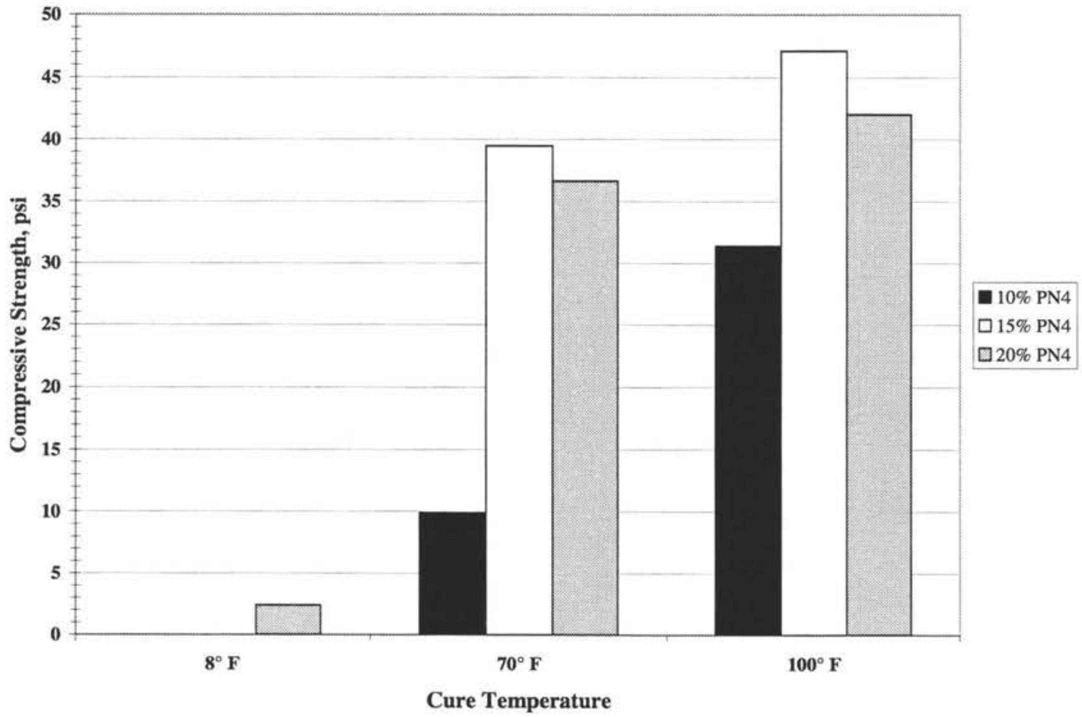


Figure 60. Influence of Curing Environment on Strength Gain of Turin Loess and PN4 Fly Ash

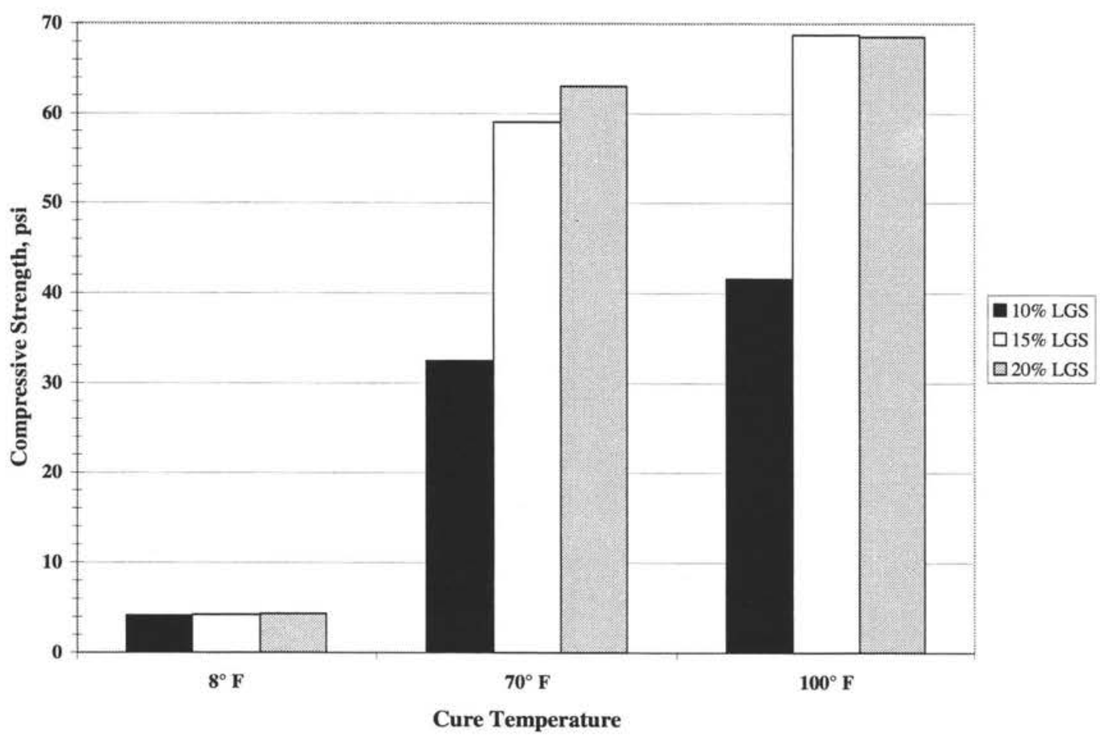


Figure 61. Influence of Curing Environment on Strength Gain of Paleosol and LGS Fly Ash

Typically, as fly ash content increases, so does strength. This was the case between 5% and 10% addition rates for the Turin loess. But the 20% sample strengths were actually less than the 15% samples for 70° and 100° F curing temperatures. The difference was minimal, approximately 5 psi. It is also noted that increased curing temperature did increase the 7-day strength of the stabilized soil. Another trend shown in Figure 60 is that the fly ash did not hydrate during curing in the 8° F environment. This was expected and all but one sample slaked during soaking, with only one 20% sample tested.

The paleosol samples exhibited higher strength gain at increased curing temperatures, as was expected. Samples cured in the freezer had approximate strength of 4 psi. These samples did not dissolve due to the higher clay content but did swell and crack, as shown in Figure 62. This is contrasted by Figures 63 and 64, which show the paleosol samples cured at 70° and 100° F after soaking, respectively. These samples did not exhibit any swelling or delamination during soaking. These samples also showed a trend of increased strength with increased fly ash content. Curing for 7 days at 100° F is said to simulate 28-day strength of materials cured at 70° F (5). No testing was completed to evaluate this assumption but from viewing the data for both soils it appears this is valid. Most of the initial cementitious reactions have taken place leaving pozzolanic reactions as the primary strength gain mechanism. Observation of 100° F strengths shows they are only slightly greater than the 70° F strengths.

By evaluating the Ottumwa-Midland Landfill Access Road and Sutherland Generating Station Access Road, the phenomenon of autogenous healing by pozzolanic reactions has been observed. As shown in Figures 60 and 61, fly ash stabilized soil does not gain strength below freezing temperatures. A small study was undertaken to evaluate autogenous healing. Le Grand loess (20% moisture) and 18% (by dry weight) LGS fly ash were used. The samples were prepared in the same manner as the previous samples. Two sets of samples were cured at 100° F and two sets were cured at 8° F. After 7 days one set of samples from each environment was tested in compression at 0.05 in/min. Again the samples were brought to room temperature and soaked for one hour before compression testing. Also at this time curing environments for the remaining samples were switched, i.e. the 100° F samples were placed in the 8° F environment and samples cured for 7 days at 8° F were switched to the 100° F environment. Samples were then allowed to cure for another 7 days and tested by the same means as the other samples. The results are shown in Figure 65.

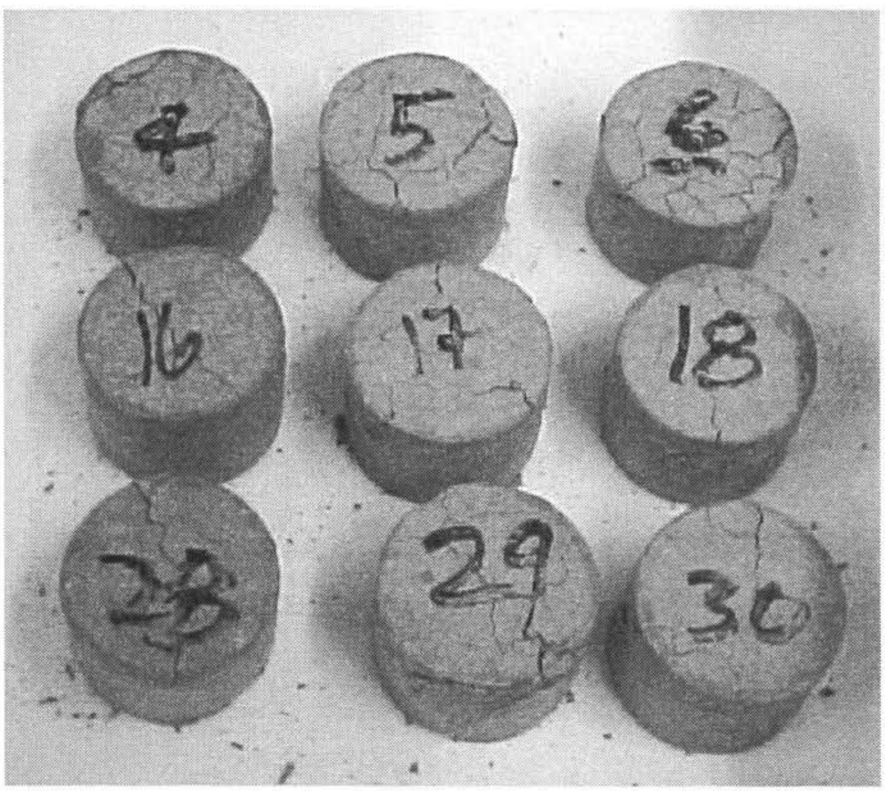


Figure 62. Swelling and Cracking After Soaking of Paleosol and Fly Ash Cured at 8° F

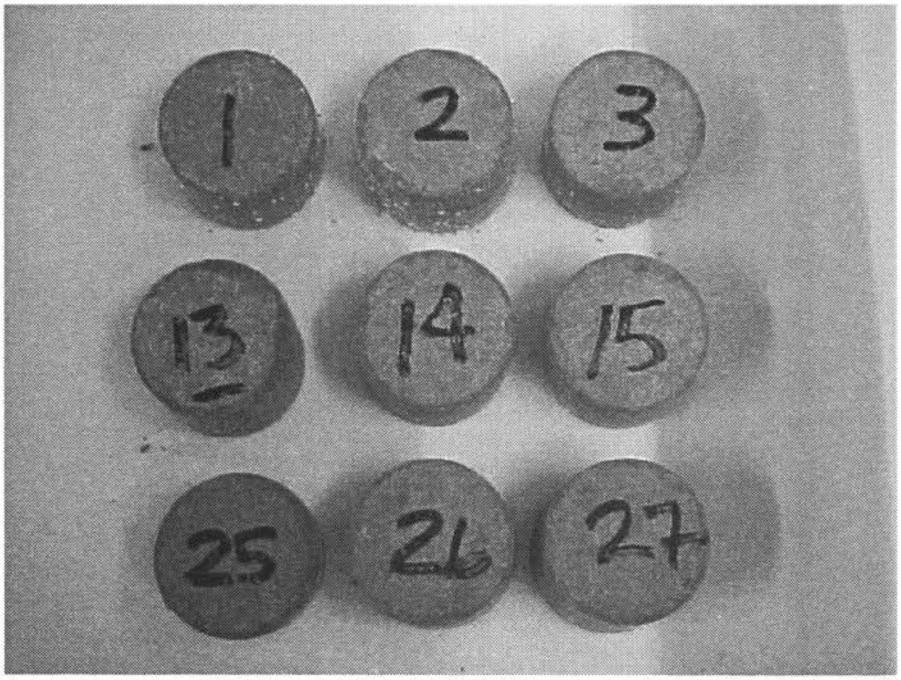


Figure 63. Paleosol and Fly Ash Cured at 70° F After Soaking

Figure 64. Paleosol and Fly Ash Cured at 100° F After Soaking

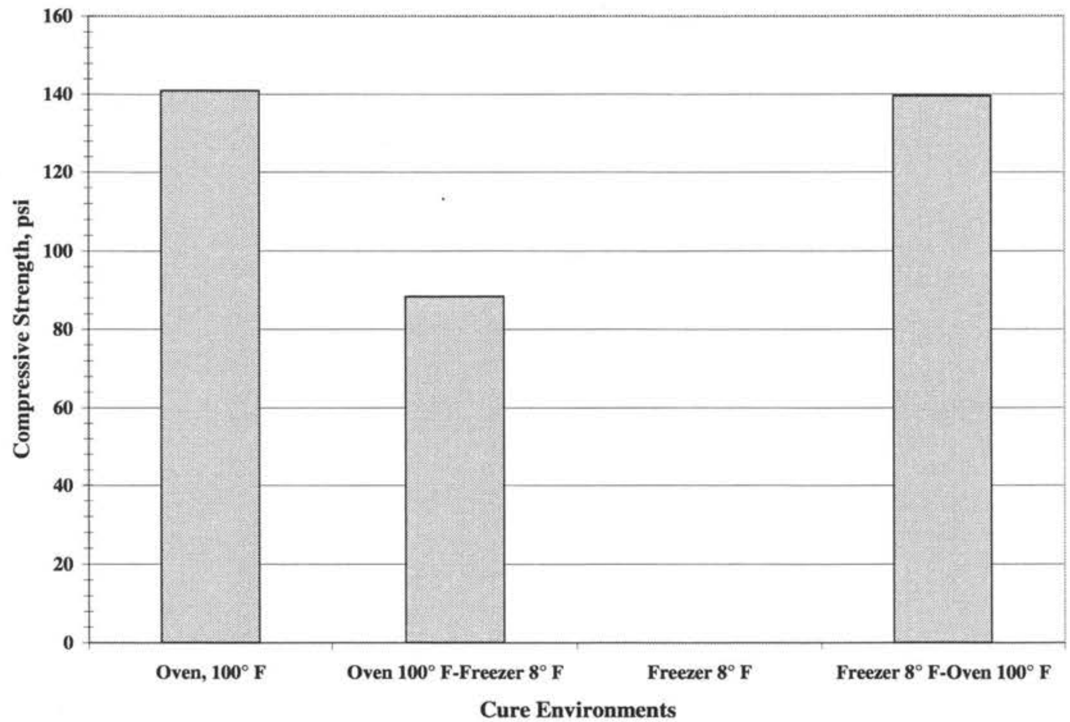


Figure 65. Role of Autogenous Healing in Self-Cementing Fly Ash-Soil Mixtures

The results for the 7-day samples were expected. The 8° F samples showed no strength gain while the 100° F samples had an average strength of 140 psi. Samples cured initially at 100° F and then transferred to the 8° F showed a decrease in strength to 88 psi compared to the 7-day 100° F samples. A decrease was expected, but not to this degree. Autogenous healing was exhibited in samples initially cured at 8° F and then transferred to the higher temperature environment. These samples had the same strength of samples cured only 7 days at 100° F. This shows evidence that after thawing the chemical reactions were re-initiated and strength increased. Fly ash stabilized soil does exhibit behavior that leads to the belief that the material could potentially heal after harsh winters.

Freeze-Thaw Durability

No ASTM test methods have been developed to test the freeze-thaw durability of soil stabilized with self-cementing fly ash alone. Typically ASTM C593 [Standard Specification for Fly Ash and Other Pozzolans for Use With Lime] (5) and ASTM D560 [Standard Test Methods for Freezing and Thawing Compacted Soil-Cement Mixtures] (8) have been used to test the durability of fly ash-soil mixtures. ASTM C593 was developed for combined Class F fly ash and lime stabilization, while ASTM D560 was developed for soil-cement mixtures.

The first testing program utilized ASTM C593. Only self-cementing fly ash was used as the stabilizer. Test specimens were prepared using Turin loess and 10, 15, and 20% PN4 fly ash (by dry weight) and paleosol with 10, 15, and 20% LGS fly ash (by dry weight). The soils were mixed to an initial moisture content of 19% and allowed to cure for 24 hours. Fly ash was then mixed with the soil and immediate compaction using the ISU 2-in x 2-in (L/D=1.0) apparatus was completed. Samples were extruded, sealed, and cured for 7 days in a 100° F environment. After curing, samples were brought to room temperature, and one set of samples was soaked in a water bath for one hour and a corresponding sample set was vacuum saturated for one hour. Compression testing at 0.05 in/min was performed after soaking. Vacuum saturation is supposed to simulate weakening from freeze-thaw action. ASTM C593 (5) states vacuum saturated samples must exhibit 90% of the strength attained by soaked samples and have a minimum compressive strength of 400 psi. Figure 66 shows the results of the Turin loess mixtures and Figure 67 shows the results of the paleosol mixtures.

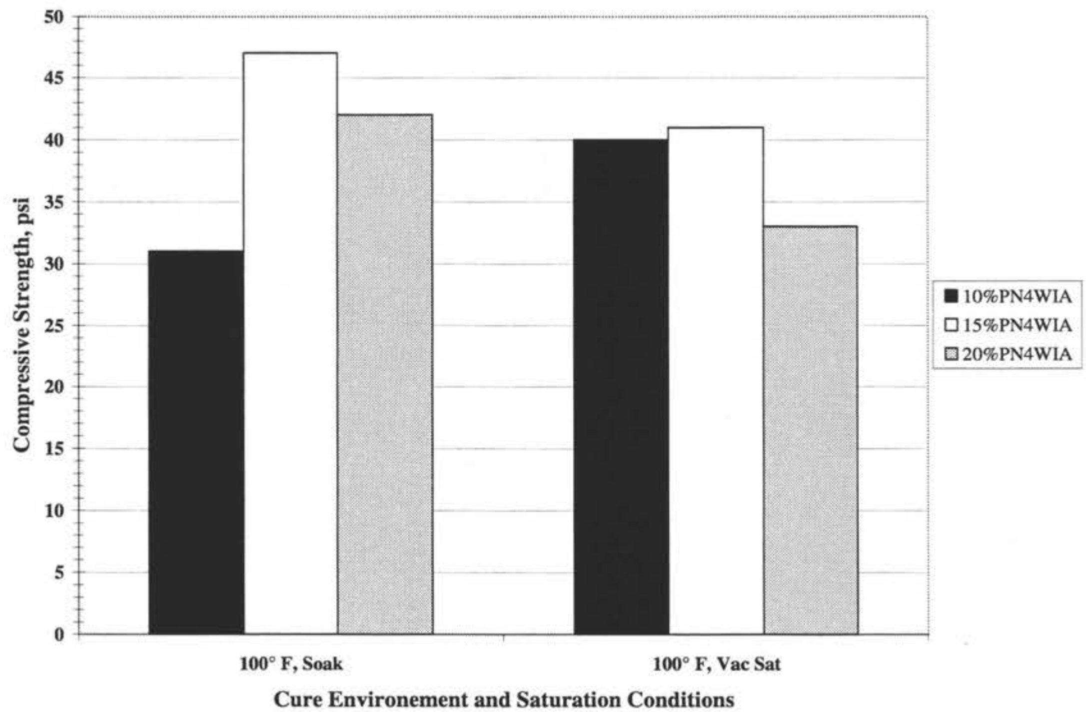


Figure 66. Freeze-Thaw Durability of Turin Loess and Fly Ash by ASTM C593

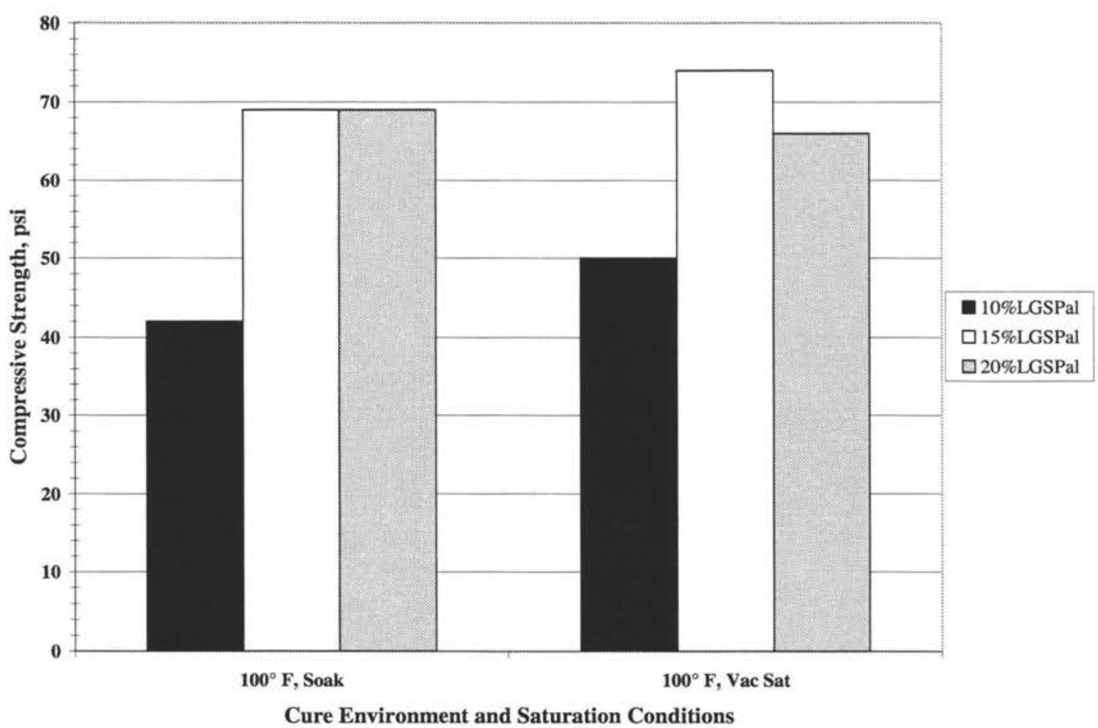


Figure 67. Freeze-Thaw Durability of Paleosol and Fly Ash by ASTM C593

Figure 66 shows the 10% fly ash samples passed the 90% strength difference test, while the 15 and 20% samples did not pass. Vacuum saturated 10% samples actually had higher strength than the soaked samples. Figure 67 shows the vacuum saturated 10 and 15% samples also exhibit this trend. The 20% paleosol samples show the theoretical relationship of strength loss due to vacuum saturation. Overall, each set of paleosol samples met the 90% strength criteria. Figures 68 and 69 show Turin loess and paleosol samples, respectively, after vacuum saturation. Note the samples do not exhibit any durability problems.

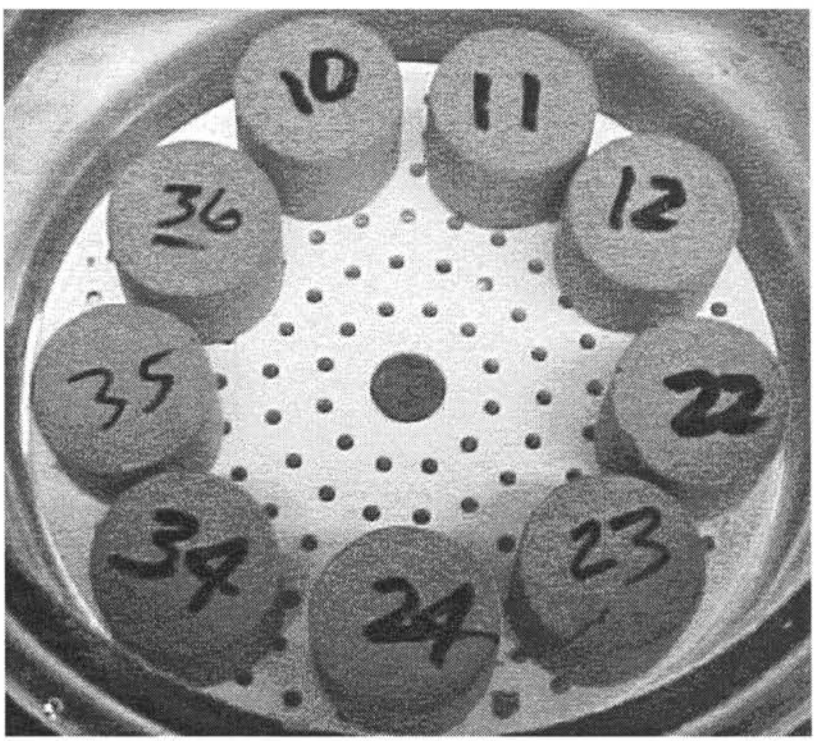


Figure 68. Vacuum Saturated Turin Loess and Fly Ash After Curing at 100° F

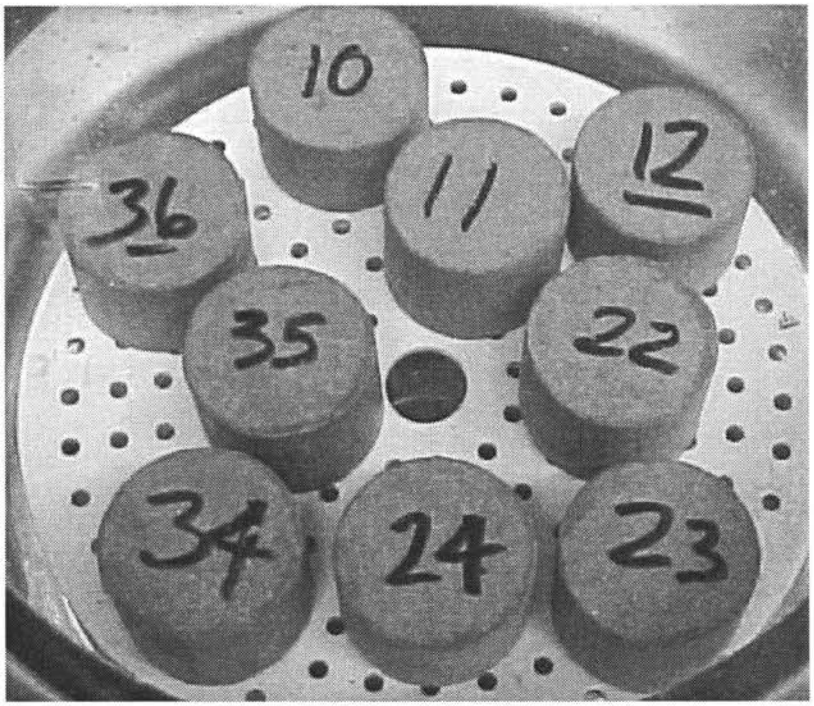


Figure 69. Vacuum Saturated Paleosol and Fly Ash After Curing at 100° F

The phenomenon of soaked samples having lower strength than vacuum saturated samples may be a result of sample variability as the differences were less than 10 psi. Also, the soaked samples may have contained entrapped air bubbles that were forced out of the sample during compression testing, causing excess internal damage, resulting in lower strengths. Although some of the sample sets passed the 90% strength criterion, no sample sets met the minimum strength requirement of 400 psi for fly ash-lime-soil mixtures. This would be difficult to attain without large amounts of self-cementing or the addition of lime. With lime addition, more calcium is available for cementitious reaction products, thus leading to a more rapid strength gain. Lime could be added to self-cementing fly ash and soil to facilitate this, but it defeats the purpose of using self-cementing fly ash by itself.

As stated before, ASTM D560 was developed for soil-cement mixtures. Identical specimens are molded, sealed, and cured for 7 days in a 100% humid environment. One sample is used to measure volumetric stability while the other is brushed to measure soil-cement loss. Samples in this study were molded in accordance with ASTM D560 (8). Proper moisture was added to the materials and they were allowed to cure as necessary before molding. A set, consisting of 2 samples, was molded using a standard 4-inch diameter Proctor mold and mechanical rammer. The samples were trimmed, extruded, weighed and measured, sealed, and cured for 7 days in a 100% humidity room at 70° F. After curing, each number 1 sample was measured to determine volume. The samples were then placed in a freezer (8° F) for 24 hours. Water was made available to the samples during all freeze-thaw cycles. After 24 hours, samples designated as number 1 were again measured for volume and all samples were then placed in a 100% humid environment to thaw for 24 hours. At the end of 24 hours number 1 samples were measured for volume and the number 2 samples were brushed with a damp sponge. ASTM D560 (8) states a wire brush must be used to perform the brushing but this was determined to not simulate field conditions, so a less destructive material, the sponge, was used. Twenty-four hours in the freezer followed by 24 hours in the humidity room constituted one complete cycle.

A range of materials was used in this testing. Unstabilized Le Grand loess was molded near optimum moisture content, and used as a control. Sets of samples composed of Le Grand loess stabilized with 10 and 20% PN4 fly ash (by dry weight) were also molded. OGS HFA samples along with SGS CFA samples were also tested. The HFA and CFA were tested due to concerns expressed by Iowa DOT staff.

The volumetric stability data are shown in Figure 70, and the maximum volume increase is shown in Figure 71. The results of the SGS CFA are not reported because the sample was unstable and completely fell apart before volume could be measured at the end of the first cycle. The previous volumetric measurements showed no volume increase for the number 1 SGS CFA sample. Figure 72 shows the number 1 SGS CFA sample before volumetric measurements were attempted. Bulging at the bottom of the sample under its own weight was observed. The Le Grand loess showed a maximum volume increase of nearly 8 in^3 compared to initial volume. This sample was lost after the second thaw period. After one cycle the sample has exhibited swelling and was beginning to crack as shown in Figure 73. Figure 74 was taken after the second freeze, showing a large crack propagating through the sample. Figure 75 is after the following thaw period; the crack had expanded and testing of this sample was discontinued. OGS HFA exhibited volumetric expansion similar to that of the Le Grand loess. The number 1 OGS HFA sample lost all strength before volume could be measured after the third thaw. Le Grand loess stabilized with 10% fly ash exhibited the lowest maximum volume change, approximately 7.5% of original sample volume. Measurements on this sample were discontinued after the seventh cycle due to mass loss on the outside of the sample. No deleterious freeze-thaw effects were observed at this time. The 20% fly ash-soil sample exhibited the largest volume change, approximately 9 in^3, or 15% greater than original volume. Water was available to the samples at all times, which is the cause for the constant volumetric instability.

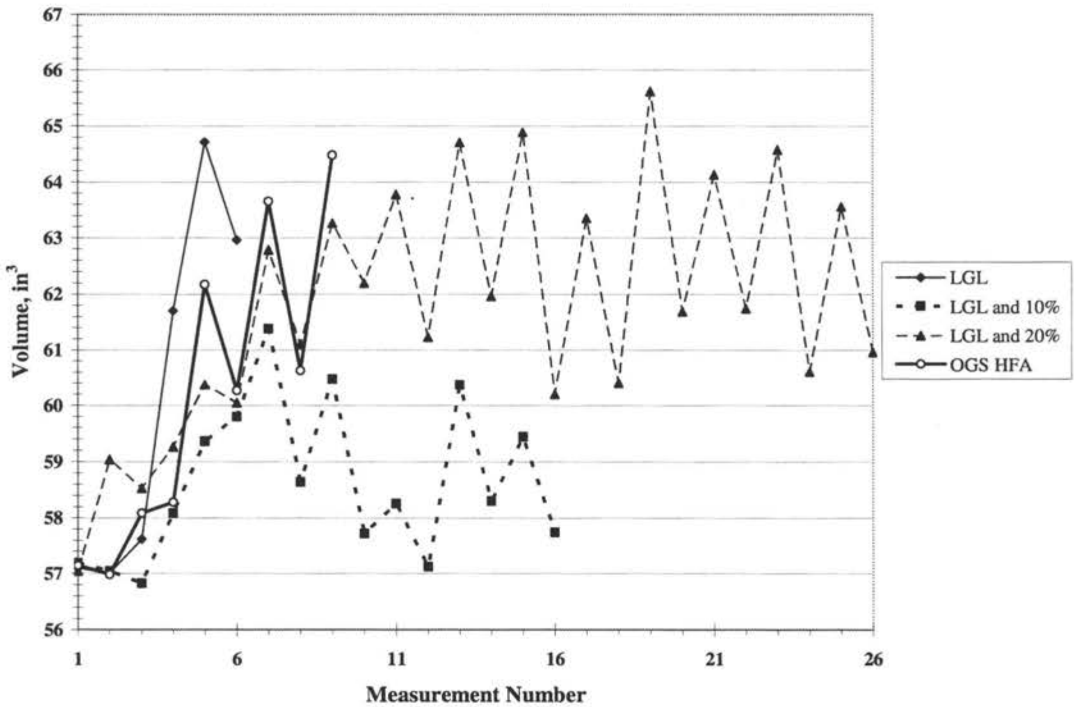


Figure 70. Volumetric Stability of Le Grand Loess (0, 10, 20% Fly Ash) and OGS HFA

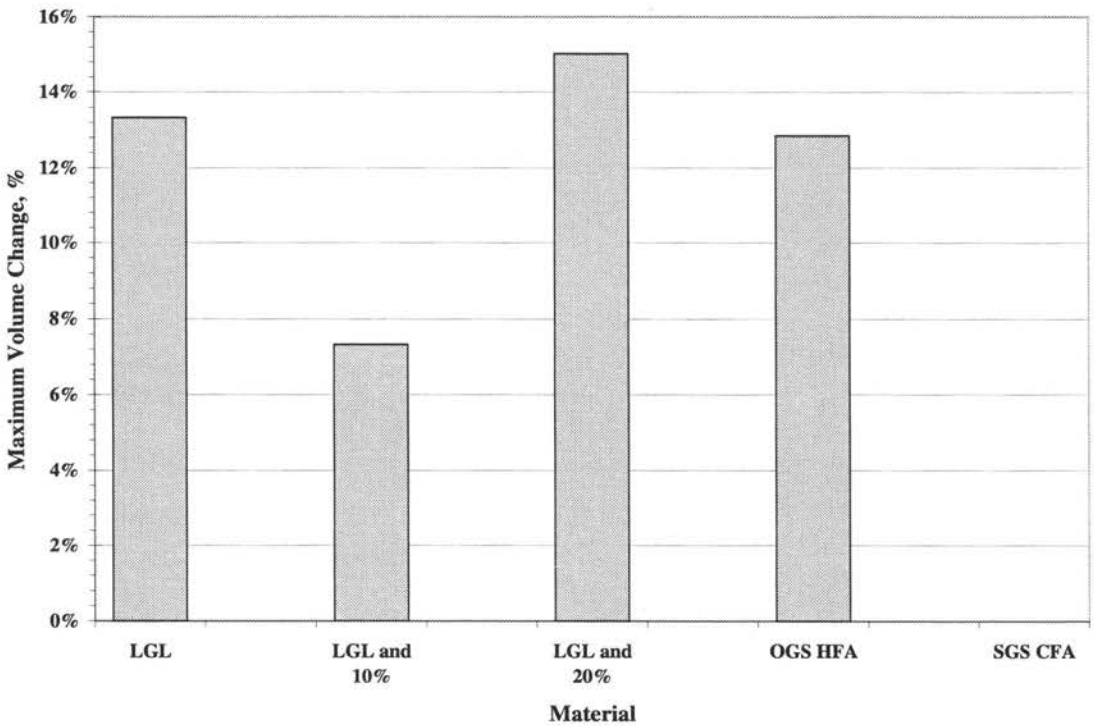


Figure 71. Maximum Volume Change of Le Grand Loess (0, 10, 20% Fly Ash), OGS HFA, and SGS CFA

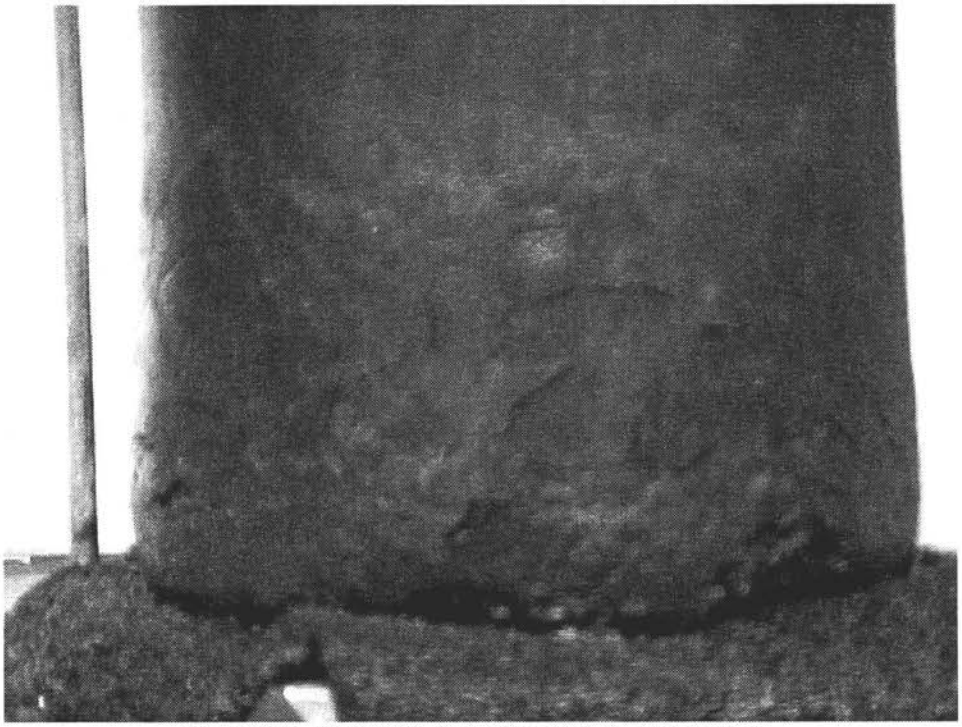


Figure 72. SGS CFA After One Freeze-Thaw Cycle

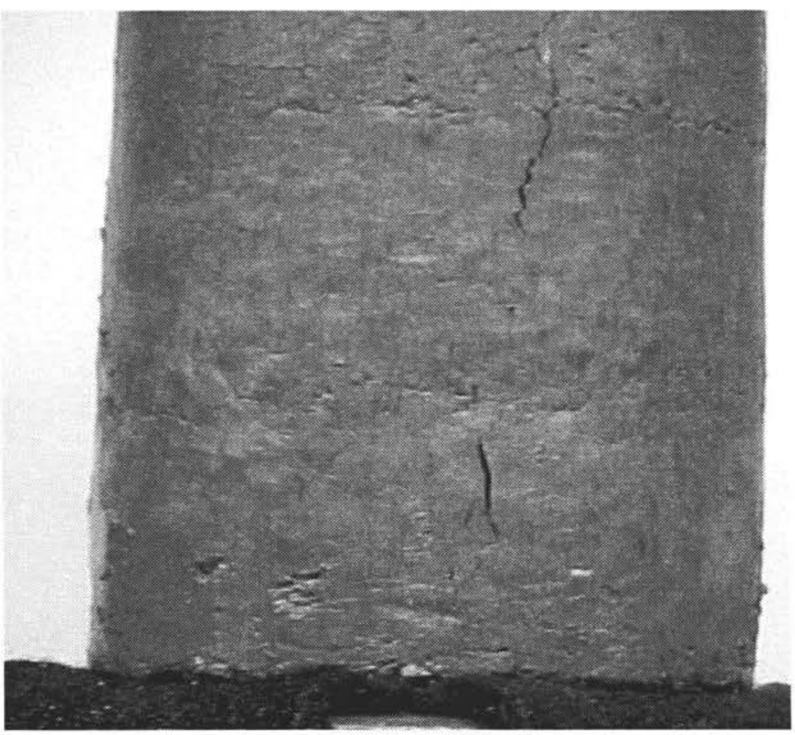


Figure 73. Le Grand Loess After One Freeze-Thaw Cycle

Figure 74. Le Grand Loess After Second Freeze Cycle

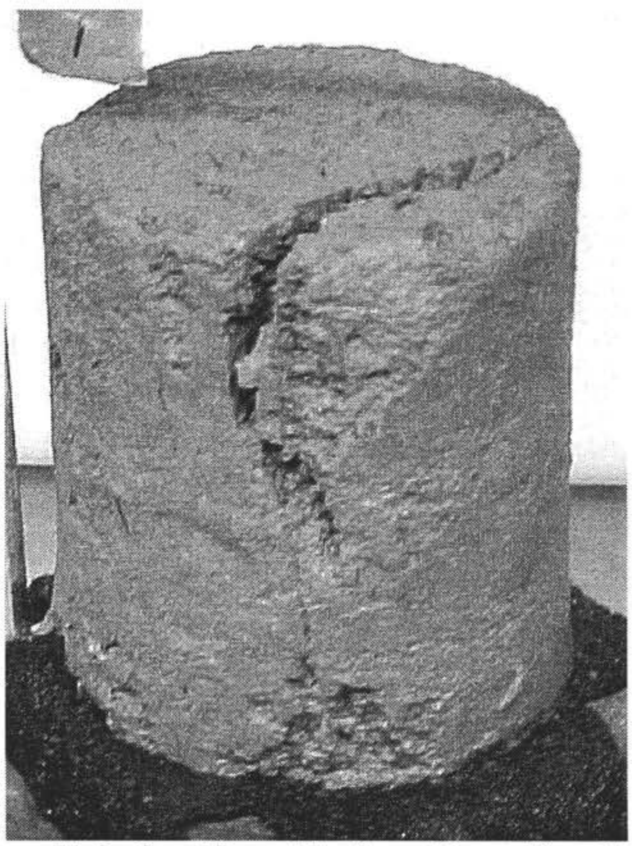


Figure 75. Le Grand Loess After Second Freeze-Thaw Cycle

Figure 76 shows the increase in moisture content of the number 1 samples at the time they were discontinued. The SGS CFA exhibited an increase in moisture of over 32%, giving the sample a final moisture content over 50%. No volume change was observed for this sample, even though it absorbed a large amount of water. This material probably contained a relatively large number of void spaces. The OGS HFA absorbed over 15% moisture, and this could have been greater if the sample had been subjected to more cycles. All samples containing Le Grand loess absorbed moisture in the range of 11 to 13%. Final moisture contents were approximately 27 to 30%. The fly ash stabilized samples did not exhibit cracking and soft texture, as observed in the unstabilized sample.

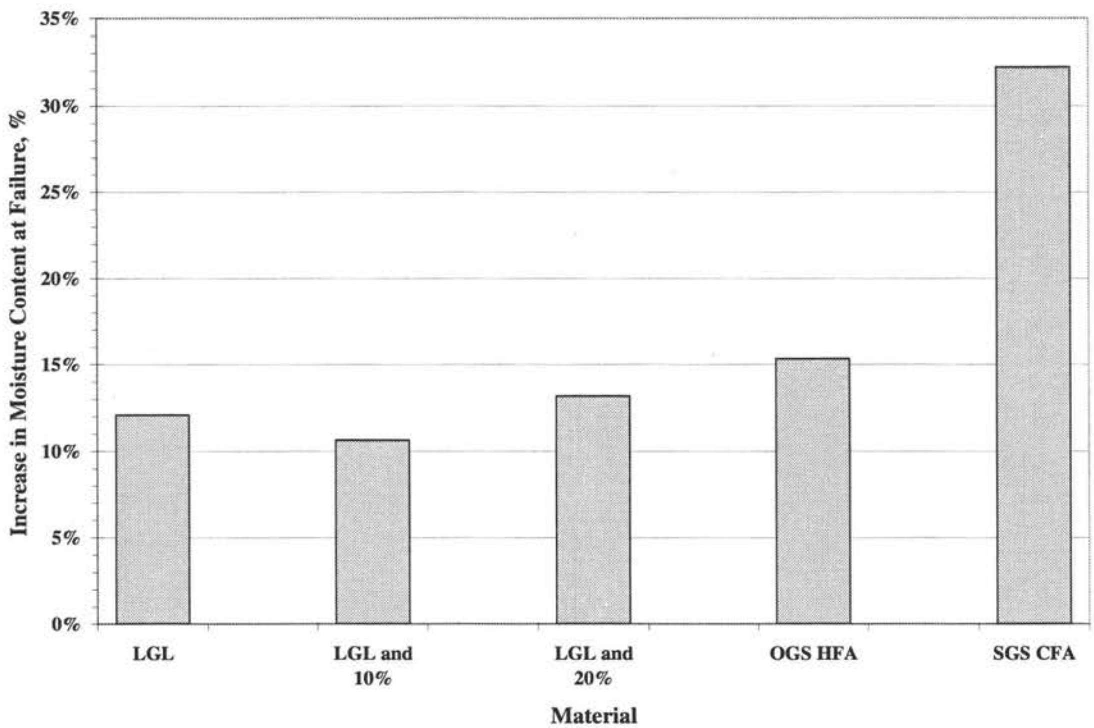


Figure 76. Moisture Content Increase of ASTM D560 Volumetric Stability Samples

The data for the second samples in each set is shown in Figure 77. Sample loss or gain was recorded as a percentage of molded mass with the following equation:

$$\%MoldedMass = \left(\frac{Mass - MoldedMass}{MoldedMass}\right) * 100 \qquad \textbf{Equation 2}$$

This allows the mass increase due to water absorption during freezing and thawing to be shown, as well as the mass loss due to brushing at the end of each cycle. During the first and second cycles, mass gain due to moisture absorption was prevalent in all samples. After initial saturation the samples did not absorb much water. The number 2 Le Grand loess sample became saturated quickly and testing was discontinued after one cycle. Figure 78 shows this sample after brushing. The Le Grand loess alone has very little freeze-thaw resistance, as typical of fine-grained materials. The 10% fly ash-soil sample delineated and fell apart during brushing after the twelfth cycle. Mass loss per cycle continued to increase up to the seventh cycle at which time it remained relatively constant until the end of testing. The 20% fly ash-soil sample was the most resistant to all 12 cycles of freeze-thaw and retained 58% of its original mass. It is believe this is mostly due to addition of 20% fly ash. The fly ash treated soil samples exhibited the best overall freeze-thaw durability. The OGS HFA and SGS CFA were destroyed after the fourth cycle. HFA and CFA gain strength due to pozzolanic reactions, which were unable to begin due to the change in curing environment every 24 hours. Figures 79 and 80 show SGS CFA before and after the second cycle brushing. This mass loss due to brushing was typical of both CFA and HFA until failure. The durability of the OGS HFA and SGS CFA was observed to be relatively low. This is due to lack of confinement of the material, and pozzolanic reactions not being able to be carried out, as would occur in actual field conditions. Also these materials had an affinity for water absorption, thereby accelerating freeze-thaw deterioration. An activator could increase durability as determined by Berg (19) and activated HFA and CFA have shown good durability in field conditions (50, 51).

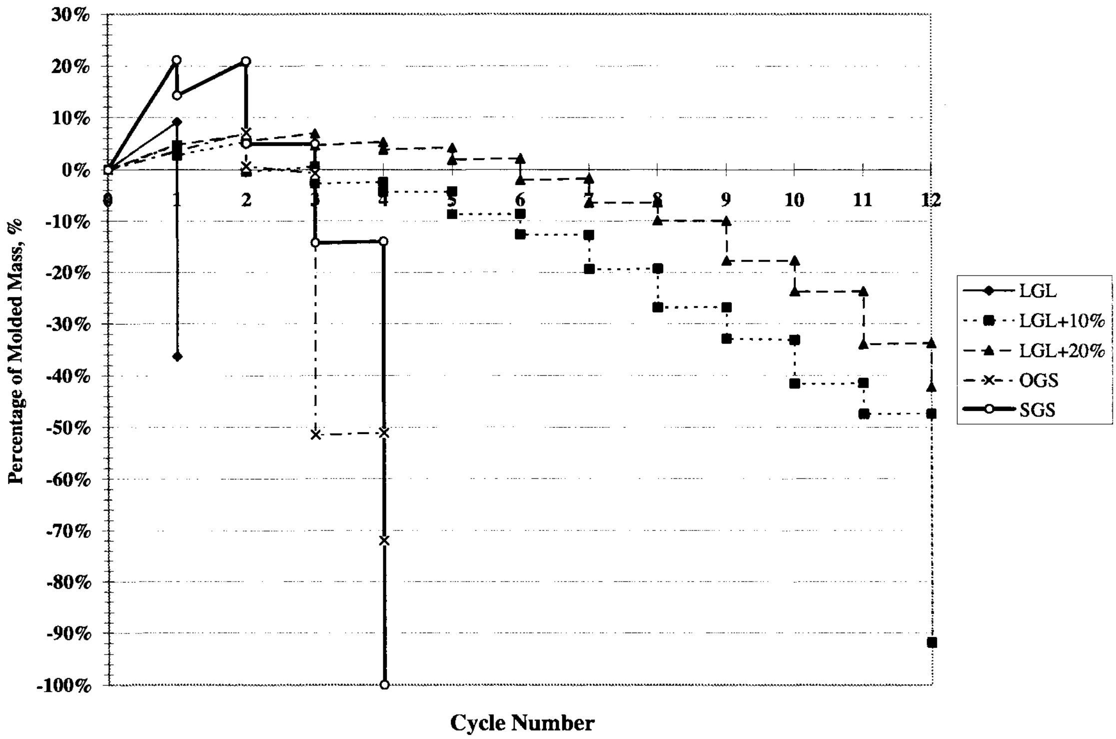


Figure 77. Percentage of Molded Mass Before and After Brushing for each Cycle

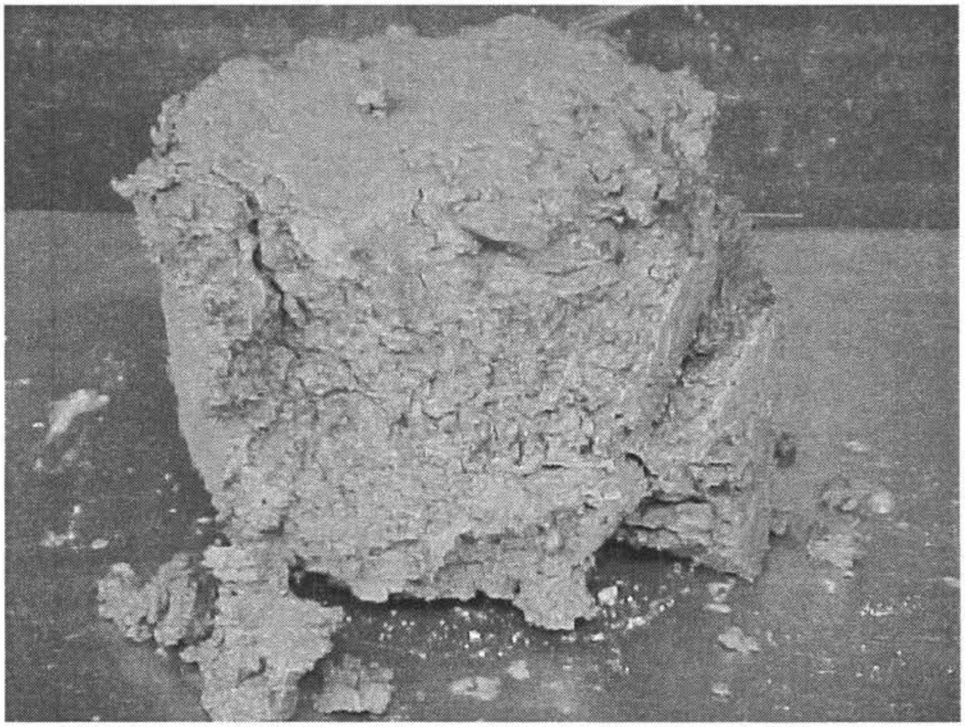


Figure 78. Le Grand Loess Number 2 Sample After First Cycle Brushing

Figure 79. SGS CFA Prior to Second Cycle Brushing

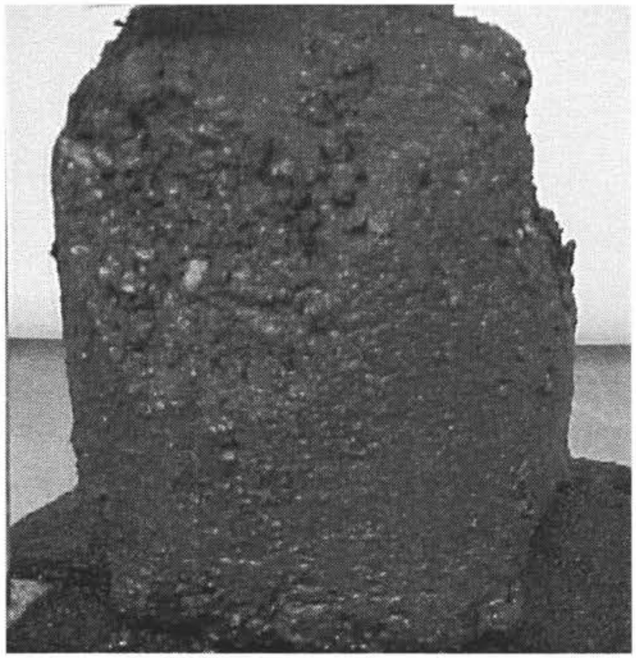


Figure 80. SGS CFA After Second Cycle Brushing

STRENGTH OF HYDRATED AND CONDITIONED SELF-CEMENTING FLY ASH

Influence of Moisture Content on Strength Gain

Molded moisture content is known to have an influence on the strength gain of pozzolanic materials. In this study samples (L/D=1.15) that were used to determine the moisture-density relationships for the HFA and CFA materials were extruded, sealed, cured for 28 days in a humidity room, and tested in compression. Before testing, samples were capped with high strength sulfur capping compound. The compressive loading rate was 0.05 in/min. The strength results for the HFA and CFA materials are shown in Figure 81.

For unsoaked materials, strength tends to decrease as molded moisture content increases. This relationship only holds for the OGS HFA and the PC CFA. The relationship for the LGS HFA strength is similar to a moisture-density relationship, and it is noted that the LGS HFA exhibited highest strengths. This may be the result of moisture not available for pozzolanic reactions until molded moisture content increased to initiate them. The same could also be said of the PN HFA. For the most part, moisture content didn't have a pronounced effect on the strength gain of the SGS CFA, where the strength trend appears to be related more to dry density than molded moisture content. The CB HFA was relatively unreactive and behaved as an aggregate when the samples were extruded. All but one sample fell apart while being extruded. This one sample, tested at 60 psi, had the lowest strength of all samples.

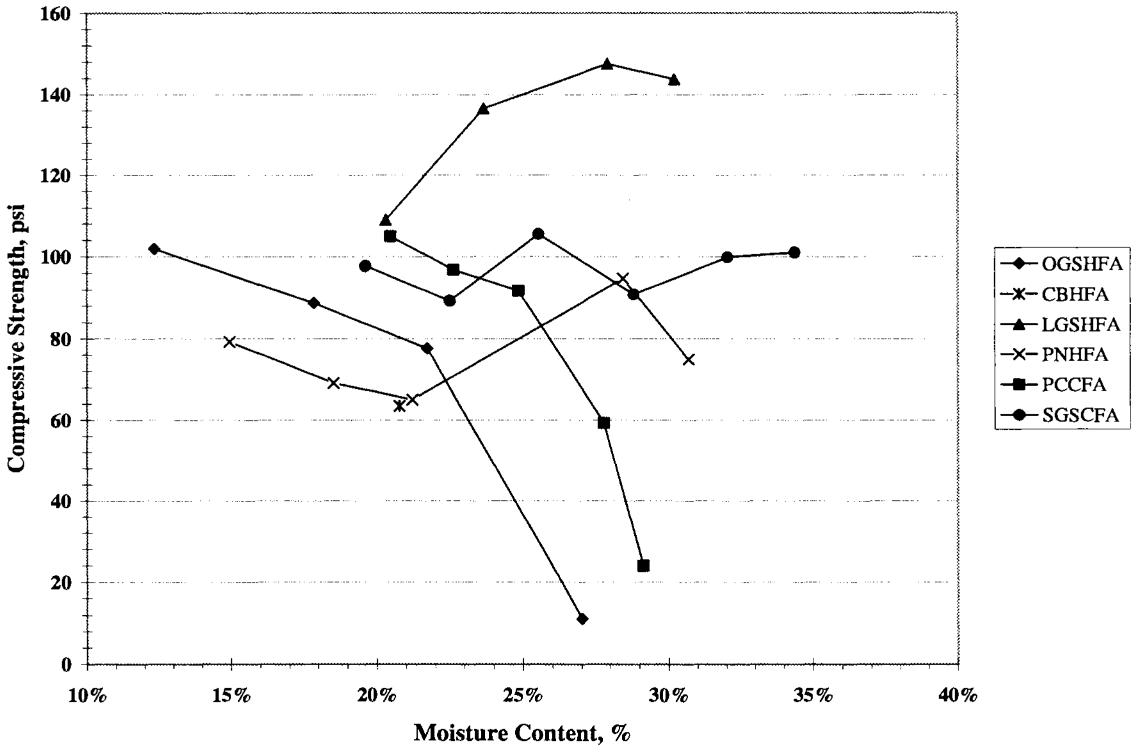


Figure 81. Influence of Moisture Content on Strength Gain of HFA and CFA

Another study was undertaken to monitor long-term strength gain of OGS HFA versus molded moisture content. Results of this testing are shown in Figure 82. OGS HFA was mixed at moisture contents of approximately 15%, 20%, and 25%. Samples were molded in a standard 4-inch diameter Proctor mold (L/D=1.15), extruded, sealed and allowed to cure in a 100% humid environment. After the specified cure time (7, 28, 90, and 365 days), the samples were capped with sulfur capping compound and tested in compression at a rate of 0.05 in/min.

At each cure time the samples follow the trend of higher strength at lower moisture content, typical of unsoaked samples. All samples continued to gain strength during the year long curing period. At 7 days, average strength values were 82 psi, 72 psi, and 38 psi for the 15%, 20%, and 25% moisture samples, respectively. After one year average strengths increased to 267 psi, 171 psi, and 120 psi for the samples. This equates to strength gains of 18.7%, 10.8%, and 14.1% per month based on 7 day strength.

Overall the 15% moisture samples had the highest strength with strengths decreasing somewhat at 20% moisture and even more noticeable decreases at 25% moisture. It is evident the 25% moisture samples were too wet to gain as much strength as other samples molded at lower moisture contents. There was plenty of water available for pozzolanic reactions, but at 365 days the samples still felt relatively saturated and soft, therefore not allowing complete bonding at the particle to particle contacts.

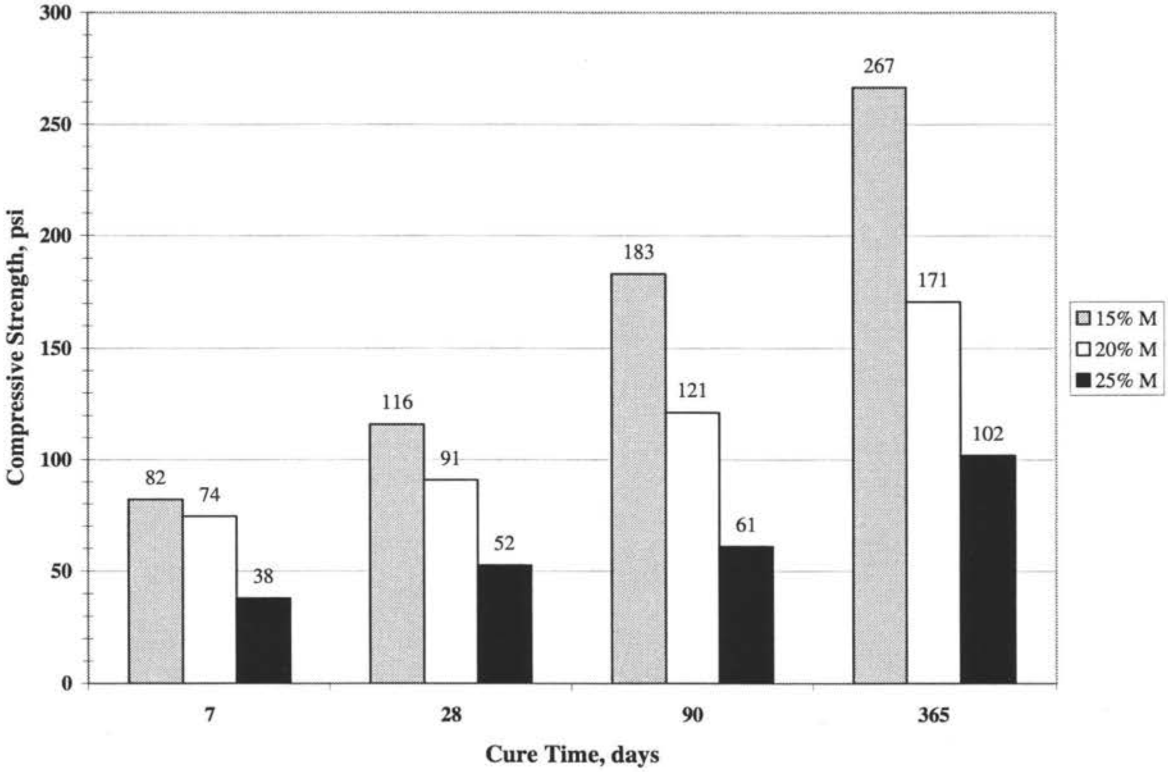


Figure 82. Long-Term Strength Gain of Ottumwa HFA Molded at Different Moisture Contents

Influence of Compaction Delay on Strength Gain

The object of this testing program was to see if compaction delay had the same effect on strength gain of HFA and CFA materials as it did with soil and self-cementing fly ash. The HFA tested was from Ottumwa, Port Neal, and Council Bluffs. Prairie Creek CFA was also tested.

The HFA and CFA was prepared to pass the #4 sieve to allow the samples to be molded with the ISU 2-in x 2-in apparatus (L/D=1). Initial moisture contents, close to optimum moisture content for each material, were 30.9%, 28.7%, 23.6%, and 26.7% for the PN, CB, OGS, and PC, respectively. Samples were molded at times of 0, 1, 2, 4, and 8 hours, then sealed and cured for 28 days in a humidity room before being tested in compression at a loading rate of 0.05 in/min.

Overall there is not a significant trend when comparing strength versus compaction delay time. Strengths of each material are fairly constant over the range of delay times, with the maximum difference being about 10 psi for the PC CFA. All materials showed a slight increase in strength at some delay time, which was followed by a small decrease. The results are plotted in Figure 83.

Influence of Curing Temperature on Strength Gain

This research was used to determine if curing temperature had an effect on strength gain of HFA materials. Curing temperatures used in the study were 8°, 40°, 70°, and 100° F. The OGS HFA was mixed at optimum moisture content based on the moisture-density relationship and compacted in a standard 4-inch diameter Proctor mold (L/D=1.15). Samples were extruded, sealed, and cured in the various environments for 7, 28, and 56 days. Samples were allowed to reach room temperature before being capped and tested in compression at a rate of 0.05 in/min. Comparison of results is shown in Figure 84.

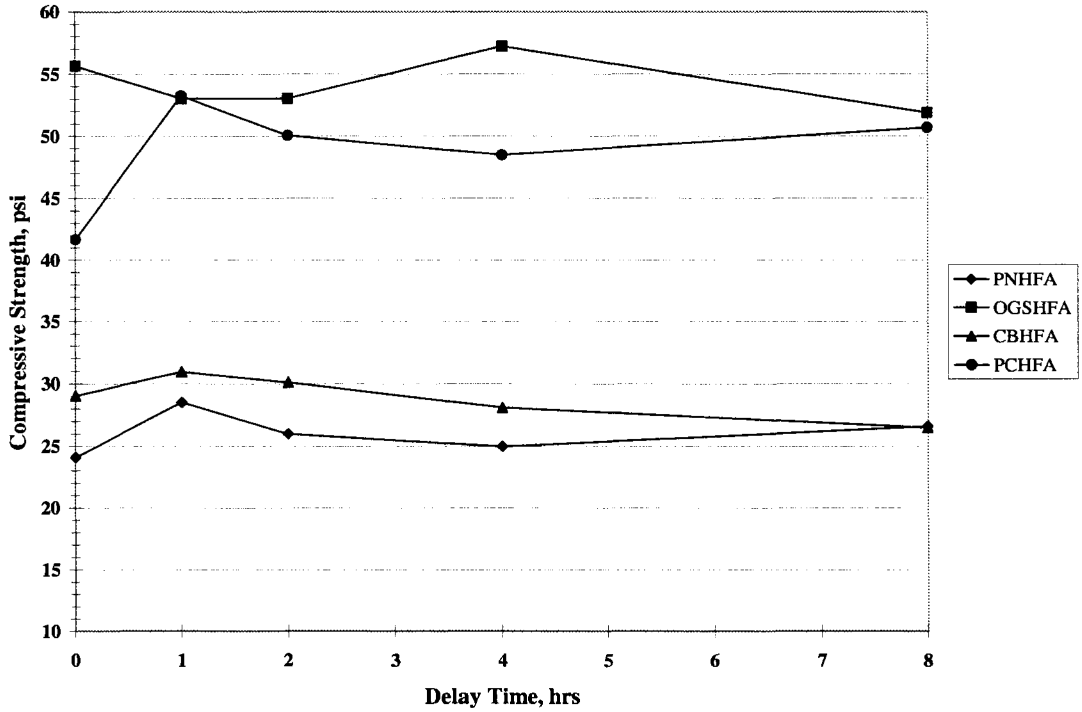


Figure 83. Influence of Compaction Delay on Strength Gain of HFA and CFA

At 7 days, all samples had approximately the same strength, on the order of 70 psi. This could be due to the fact that no pozzolanic reactions had occurred in any of the samples. One would expect the 100° F samples would have shown more activity than observed. Samples cured at 8° F should have shown similar strengths at all cure times because temperatures were below freezing. This was not the case at 56 days, where strength was about 20 psi higher than those at 7 and 28 days. Samples cured at 40° F did gain strength, but not until 56 days, increasing about 25 psi. The third set of samples, cured at 70° F, should have exhibited higher strengths than samples cured at 40° F and 8° F. This was not the case. The 70° F samples only gained 8 psi from 7 to 56 days. Reasons for this are not completely known.

It was hypothesized the 100° F samples would have had an accelerated strength gain due to the higher temperature. This was the case, with strength gains of 22 psi between 7 and 28 days and 25 psi between 28 and 56 days. Some results from this study are not what were expected. Both the 70° F and 100° F cured samples should have gained strengths at fairly constant rates. However, samples cured at 70° F did not gain much strength at all, while the 100° F samples did gain strength at a somewhat higher, constant rate once pozzolanic reactions began. It is unknown why the samples cured at 8° F showed any strength gain, or why the 40° F samples gained as much strength as they did at 56 days.

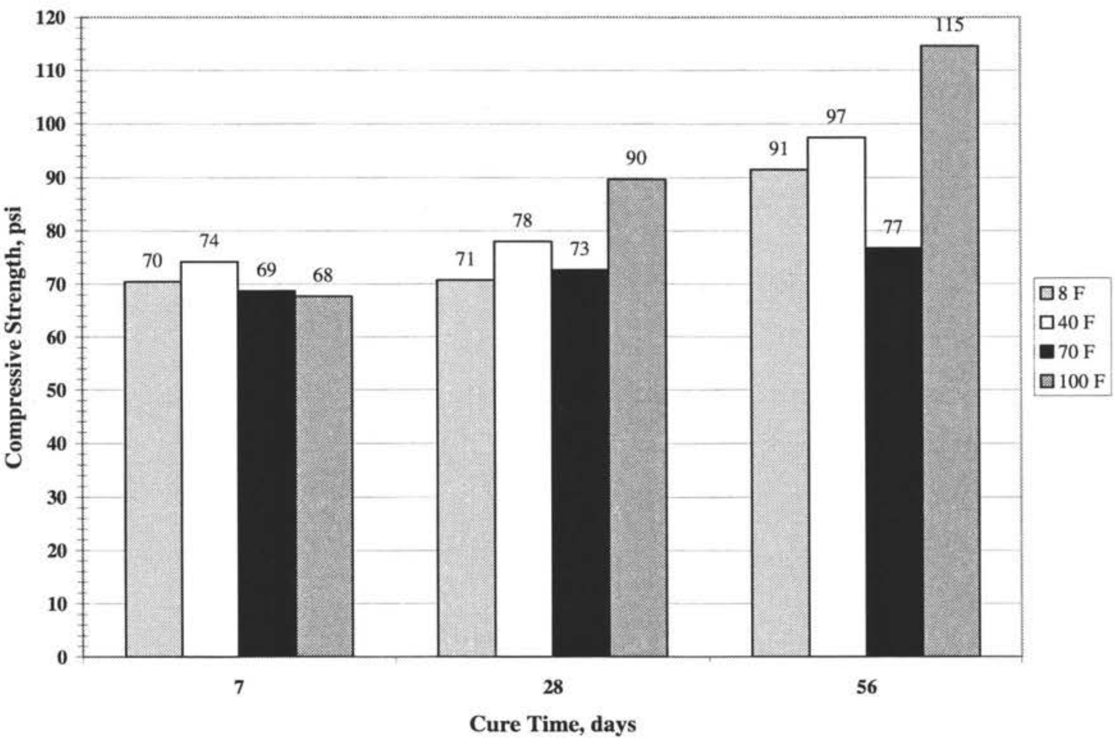


Figure 84. Influence of Curing Temperature on Strength Gain of Ottumwa HFA

Shear Strength Parameters of Hydrated and Conditioned Fly Ash

Consolidated drained (CD) triaxial compression tests were completed on the PC CFA, OGS HFA, and PN HFA. Materials were sieved through a three-quarter inch sieve, mixed at optimum moisture content based on moisture-density relationships, molded into 4" x 8" cylindrical (L/D=2) specimens, and cured in a humidity room for 7 and 28 days. Confining pressures of 10, 20, and 30 psi were used to produce the shear strength envelopes, which are in the form of p-q plots, shown in Figures 85, 86, 87, for the OGS HFA, PN HFA, and PC CFA, respectively. The results of these tests allow the drained cohesion (c') and friction angle (ϕ') to be calculated. Table 21 is a summary of c' and ϕ' at the two cure times for each material.

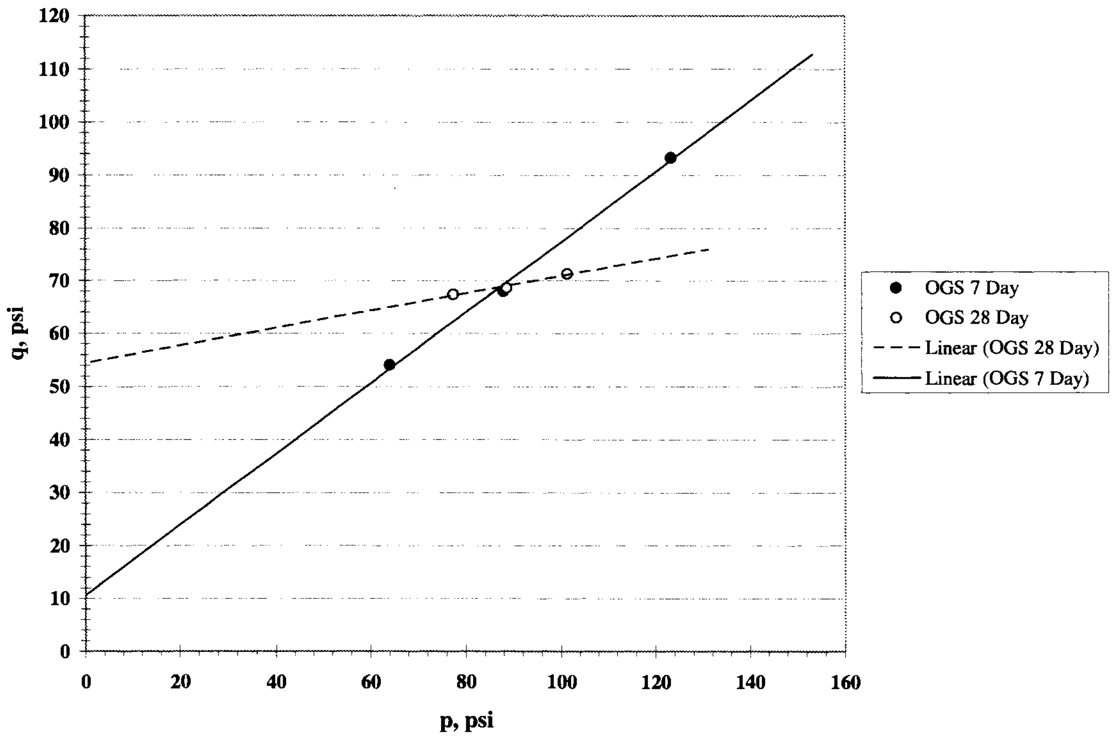


Figure 85. P-Q Plot of Ottumwa HFA

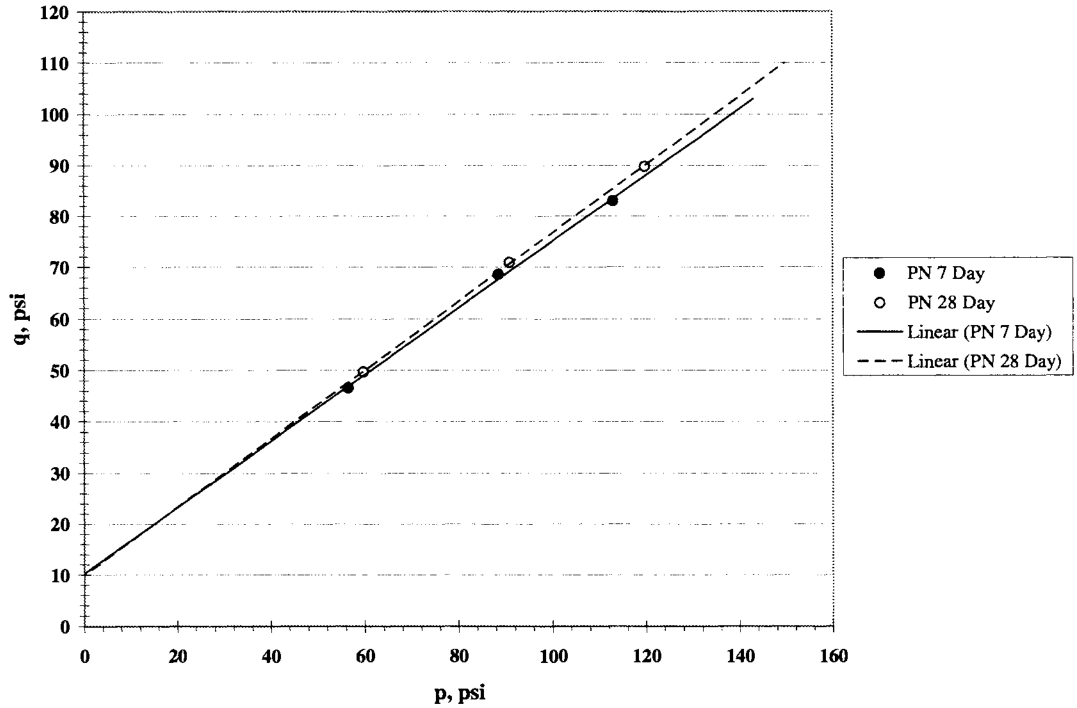


Figure 86. P-Q Plot of Port Neal HFA

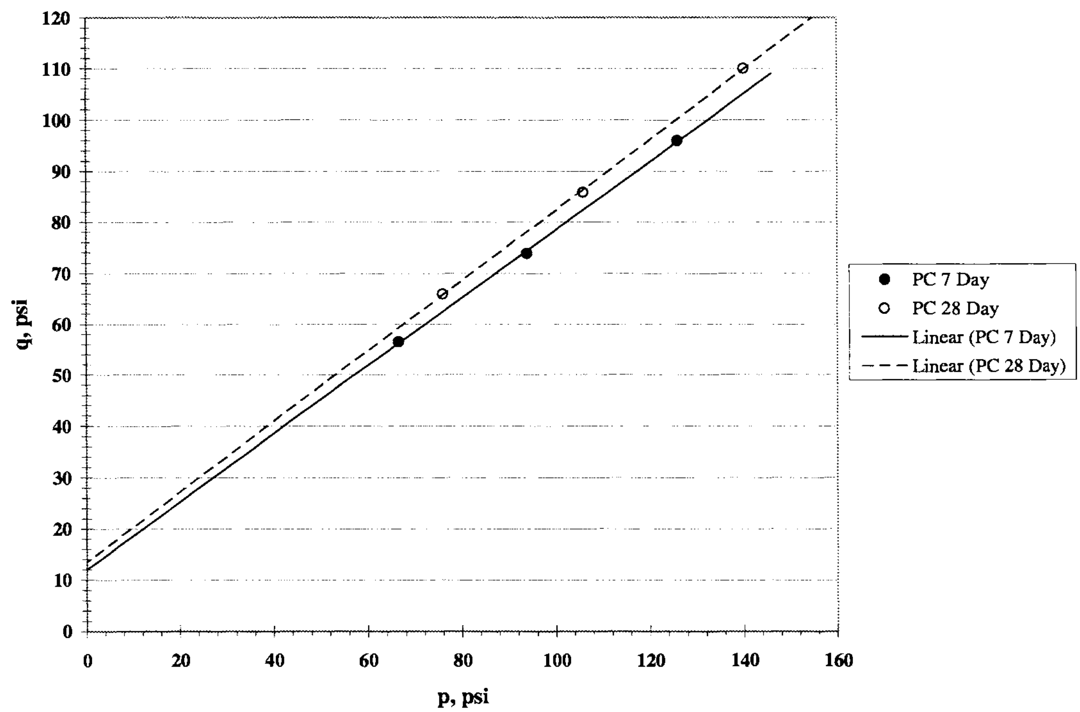


Figure 87. P-Q Plot of Prairie Creek CFA

At 7 days shear strength parameters for the OGS and PN materials are similar with c' values over 10 psi and ϕ'on the order of 32°. The PC CFA, containing some raw ash, was slightly more reactive. The 7 day values were c' equal to16 psi and ϕ' of 41.5°. Pozzolanic reactions that occurred during the 28 day cure time should have increased c' and ϕ' for each material. The PN samples tested at 28 days showed the same c' value of 10 psi but ϕ' increased to 34°. The PC material tested at 28 days continued to gain strength, producing c' of 18.6 psi and ϕ' of 43.5°.

The OGS HFA exhibited almost undrained behavior when tested at 28 days. The drained cohesion increased above 50 psi while the drained friction angle dropped to 8°. The reasons for these changes in shear strength are not understood at this time. One possible explanation for these results may be that loading was too rapid, therefore causing specimens to not exhibit fully drained behavior, i.e., lowering ϕ' and raising c'.

The PN HFA and PC CFA exhibit shear strength parameters making them suitable for use as fill materials in a wide range of earthwork construction. The OGS HFA showed parameters at 7 days that would make it acceptable for use in earthwork, but 28 day results show the material gets weaker with extended cure time. This would make the OGS HFA unacceptable as a fill material. However, past projects have shown that OGS HFA is suitable as a fill material in earthwork applications (50, 51).

Table 21. Summary of Shear Strength Parameters of HFA and CFA

	7 Day		28 Day	
Material	**c', psi**	**ϕ'**	**c', psi**	**ϕ'**
OGS HFA	10.6	32.0	54.4	8.0
PN HFA	10.3	32.0	10.0	34.0
PC CFA	16.1	41.6	18.6	43.5

STRENGTH GAIN OF SOIL STABILIZED WITH HYDRATED AND CONDITIONED FLY ASH

Soil and Hydrated Fly Ash

White has shown that HFA and CFA materials can be used to increase the strength of soil (48). The cementitious reactions are not as rapid as raw self-cementing fly ash, but the HFA/CFA-soil mixtures do gain strength due to pozzolanic reactions. To gain the required strength, the HFA and CFA should be mixed with soil at approximately 2 to 3 times the rate of raw fly ash (48).

OGS and CB HFA materials and PC CFA were used in this study to monitor the influence of addition rate on strength gain. The HFA and CFA products were crushed to provide material smaller than the #4 sieve, then mixed with Le Grand loess at rates of 10%, 20%, and 30% based on dry weight of soil. The loess was initially mixed to 17.5% moisture and the HFA and CFA were added at their original moisture contents. A total of nine 2-in x 2-in samples were molded for each addition rate and allowed to cure in a humidity room for 7, 28, and 90 days. Prior to testing, samples were soaked in water for one hour at room temperature in order to test the materials in their weakest condition. Additional samples were molded for testing at zero days, but dissolved during the soaking process, apparently the result of no strength gain from the HFA or CFA materials.

Ottumwa HFA and Soil

The OGS HFA used for this study was material sampled during the summer of 2002 near Eddyville, Iowa. This product was approximately one-year old and had been freshly reclaimed for the Highway 63 project. The results of the OGS testing are shown in Figure 88. The relationship of increased addition rate producing higher strengths is observed.

At 7 days, strengths were close to each other, generally in a range of 30 psi to 41 psi. The 28 day results show that the higher addition rates of 20% and 30% are starting to gain more strength than the 10% samples. By 90 days strengths were 60 psi, 72 psi, and 95 psi, for 10%, 20%, and 30%, respectively. While these strengths are lower than what would be expected for soils stabilized with raw fly ash, the soil stabilized with OGS HFA showed good strength gain due to pozzolanic reactions. This product was not as old as the other materials, which is the reason it gained more strength. This observation is explained later.

Council Bluffs HFA and Soil

Results of the CB HFA and Central Iowa loess are shown in Figure 89. At 7 days none of the addition rates produced measurable strength, with samples dissolving during the soaking stage of testing. This is due to the fact that the CB HFA is older than the OGS HFA used, requiring more time to reinitiate the pozzolanic reactions after the material had air dried. The CB HFA stabilized samples did show the typical strength versus addition rate relationship at 90 days. The 30% samples exhibited strengths near 50 psi. Another reason that the CB HFA samples failed to gain strength was that the raw ash used to make the HFA probably did not contain much calcium in the form of free lime.

Prairie Creek CFA and Soil

As observed with the CB HFA, the PC CFA stabilized soil did not show any strength gain at 7 days, as shown in Figure 90. These samples also dissolved during soaking. Strengths ranged from 35 psi to 50 psi at 28 days for the three addition rates. At 90 days the three addition rates are about 10 psi lower than the OGS HFA results with values of 50 psi, 65 psi, and 77 psi for addition rates of 10%, 20%, and 30%, respectively. During sample preparation it was noticed that the PC CFA had clumped up and the material had also carbonated. The carbonation of the previously unreacted ash may have contributed to the lesser strengths.

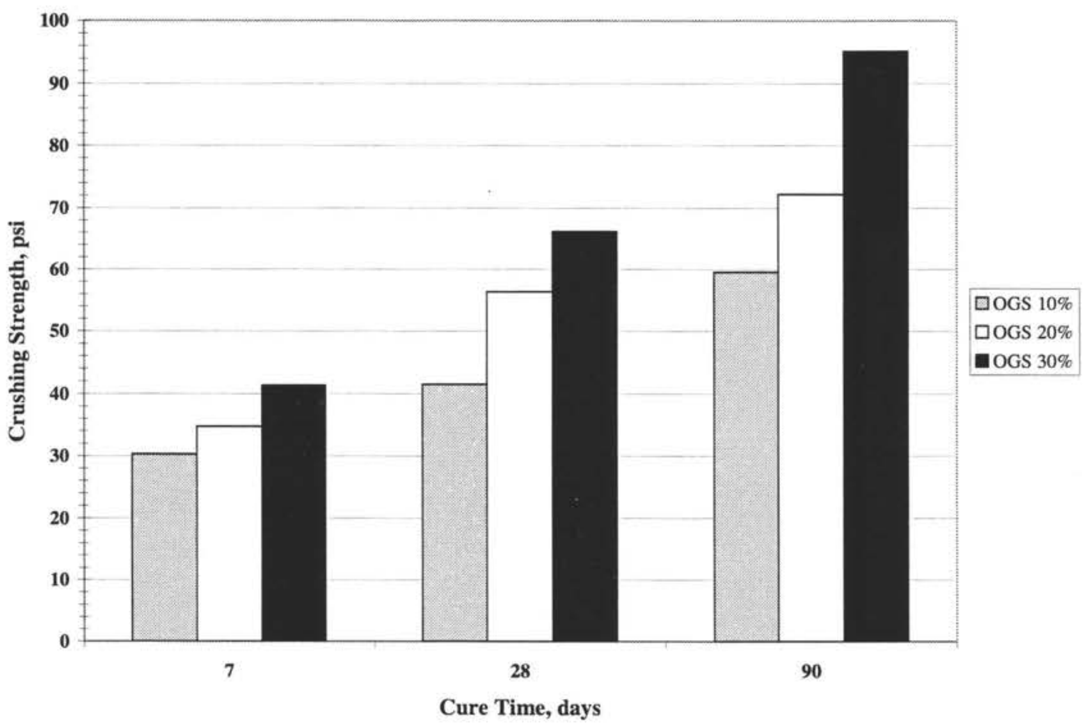


Figure 88. Strength Gain of Le Grand Loess Stabilized with OGS HFA

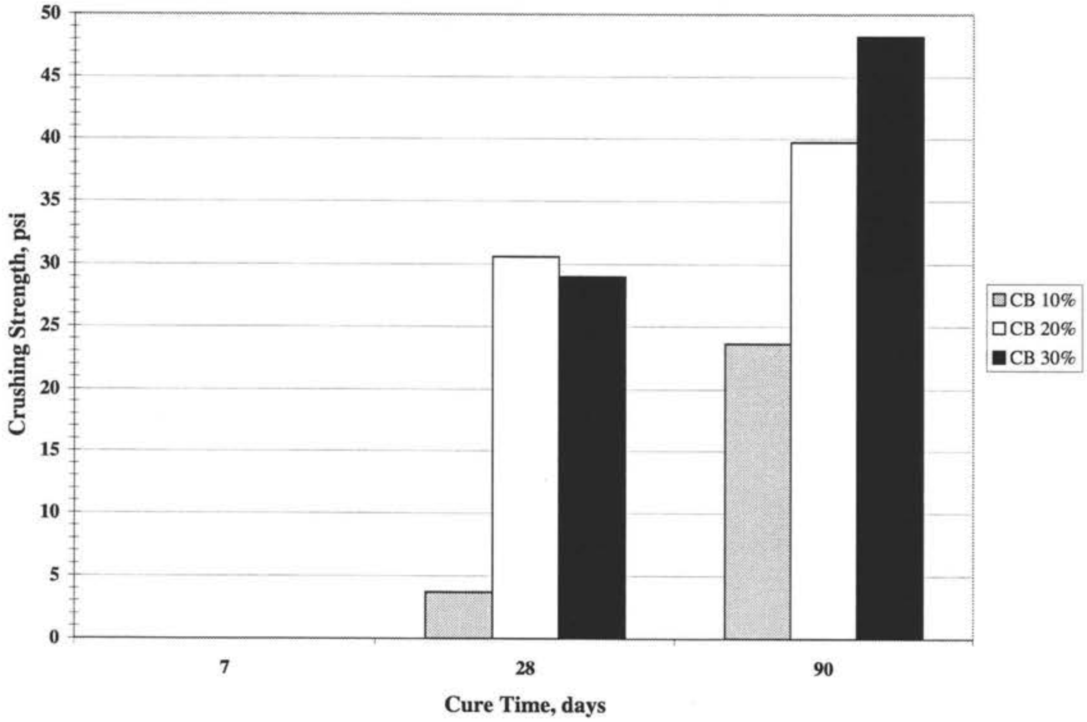


Figure 89. Strength Gain of Le Grand Loess Stabilized with CB HFA

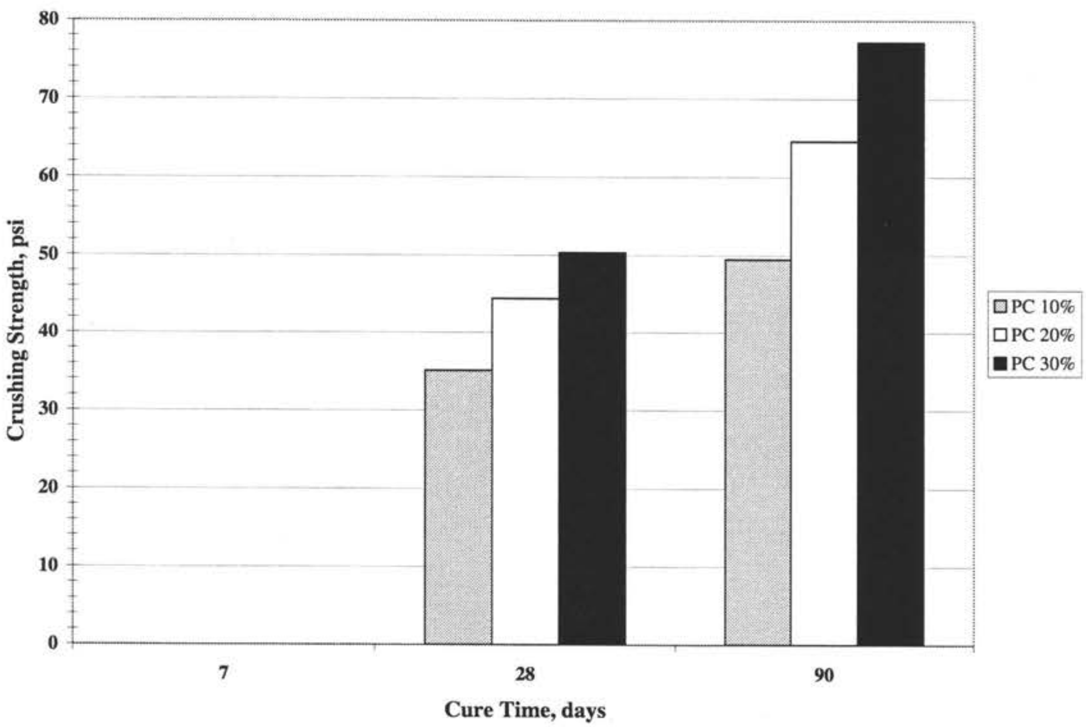


Figure 90. Strength Gain of Le Grand Loess Stabilized with PC CFA

STATISTICAL MODELLING OF SELF-CEMENTING FLY ASH CHEMISTRY AND SET TIME

Bergeson and Lapke (20) theorized fly ash set time is related to CaO content of fly ash. It is known that tricalcium aluminate is responsible for initial hardening of cement, so it was hypothesized that tricalcium aluminate would also have an influence on set time of fly ash. Chemical components from XRF and set time of 150 self-cementing fly ash samples from Nebraska power plants were determined by MARL personnel, from 1999 to 2001. The samples were collected from power plants that burn Powder River Basin subbituminous coal, similar to most power plants in Iowa. The data was obtained from MARL personnel during December 2002. Statistical analyses were used to gain an understanding of which chemical components were responsible for initial hardening of fly ash and how set could be predicted.

The computer program SAS was used in the analysis. Model selection was based on Mallow's Cp, which is estimated as the sum of squares error divided by the standard deviation squared of the sample set. The SAS program computes an initial regression on all input variables. The input with the largest Pr value is rejected and the regression is completed again. Pr is the probability the predicted value will lie about the t-value for that input in a normal distribution. T-value is calculated by dividing the estimated coefficient for each input by the corresponding standard error. The smaller the Pr-value, the closer the prediction is to the actual value. This is an iterative process, and several models are developed by the program. At this time the researcher can select the Cp model they would like to use. The Cp model is selected based on two main criteria. The first is that Cp must be minimized. Also Cp should be approximately equal to the number of inputs retained (k) plus one. Besides determining a prediction equation, Pr was used to determine which chemical constituents have the most effect on the set time. The Pr > absolute value of t is an indication of this. The smaller this value, the more influence a component has on set time.

The Cp selection used for the Nebraska fly ashes yielded the following equation:

ln(set time) = 18.6 + 7.2(ln Al_2O_3) + 3.7(ln SO_3) - 13.2(ln CaO) + 1.4(ln MgO) - 1.3(ln Na_2O) - 1.1(ln K_2O) + 2.4(ln P_2O_5) - 4.0(ln BaO) **Equation 3**

Figure 91 shows the data from the Nebraska fly ashes. If the prediction equation was 100% accurate, all points would lie on the dashed line in a one to one relationship. This is not the case. The R^2 value of the regression is 0.63. It appears the equation provides a reasonable estimate of set time for materials with set times under 40 minutes, while the relationship tends to underestimate set time for fly ashes with actual set times over 40 minutes.

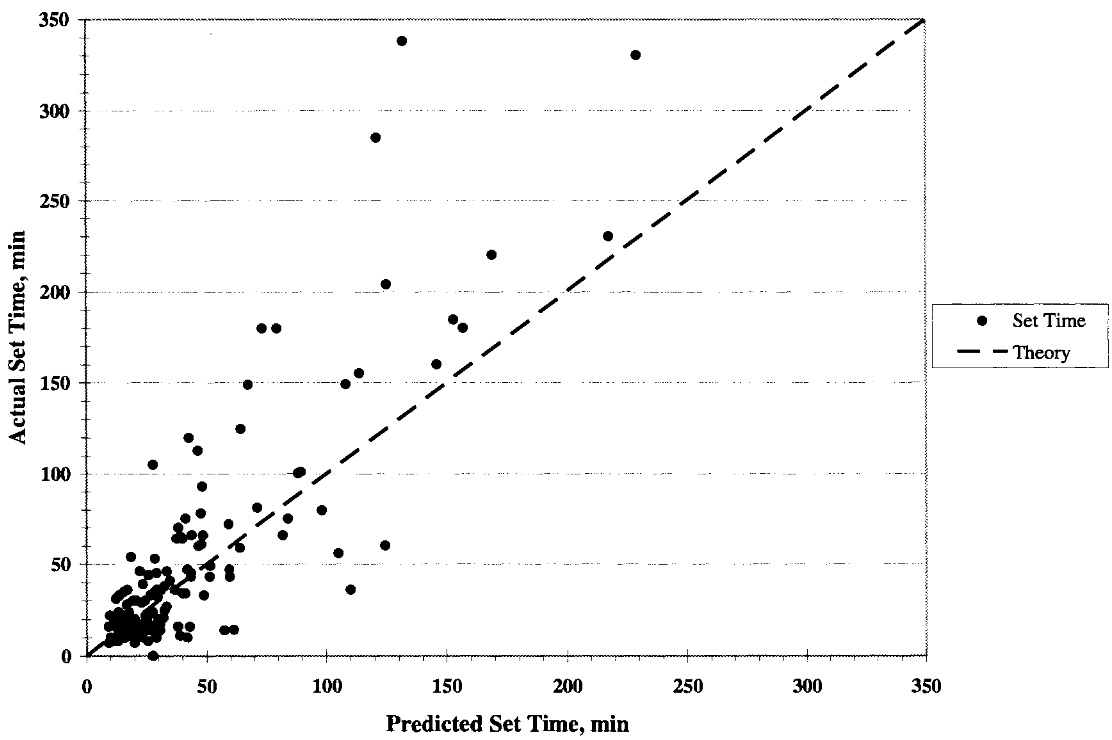


Figure 91. Actual Versus Predicted Set Time for Nebraska Self-Cementing Fly Ashes

The same type of model selection and analysis was performed on the Iowa fly ashes used in this study. More samples would have been preferred in order to be able to compare to the Nebraska sample set. Analysis of the Iowa ashes yielded the prediction equation of

$\ln(\text{set time}) = 2.2 + 8.1(SiO_2) - 10.0(Al_2O_3) + 7.2(Fe_2O_3) + 22.7(SO_3) - 5.9(CaO) + 0.9(MgO) - 7.1(Na_2O) - 106.2(K_2O) + 61.9(P_2O_5) - 35.2(TiO_2)$ **Equation 4**

Predicted set times are compared to actual set times in Figure 92. An R^2 value of 0.99 was computed for the regression, but this would have been lower if more data would have been available. Set times less than 50 minutes seem to be predicted well, while times in the middle of the test range are under estimated. No definite trend on prediction of high set times is observed due to only one data point in this range. A general trend is observed but more data points would aid in distinguishing a more well-defined trend.

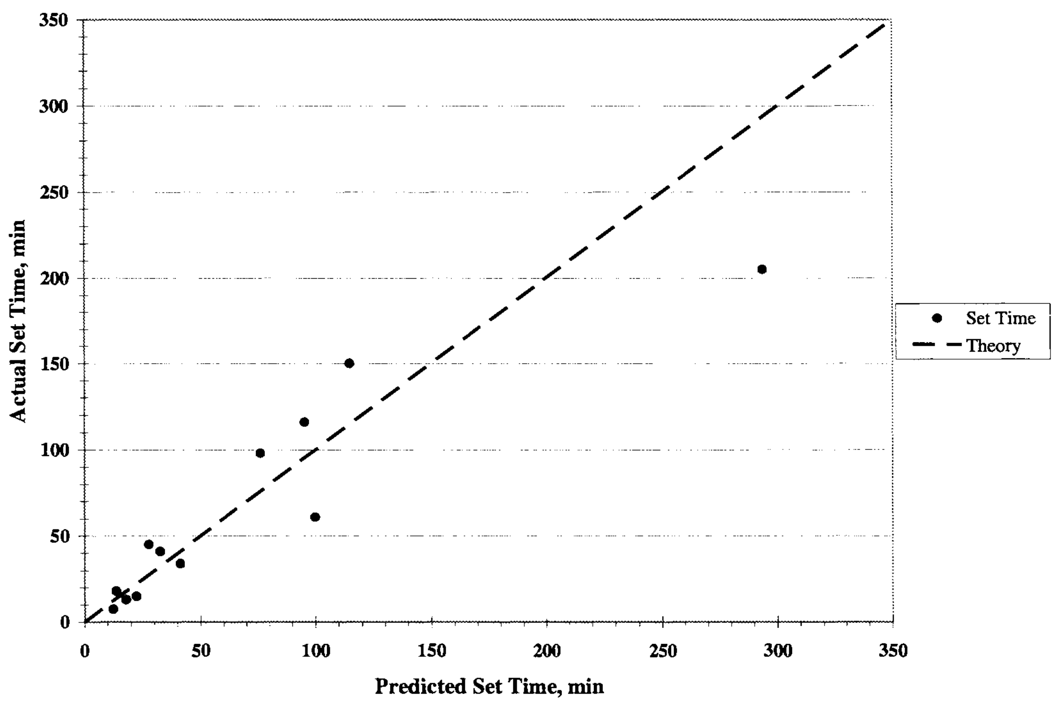


Figure 92. Actual Versus Predicted Set Time for Iowa Self-Cementing Fly Ashes

Table 22 compares the Pr values of the components used in each prediction equation. The number of indicators in the Nebraska sample set was 8 while the Iowa equation utilized 10 of the chemical components. As noted before, CaO and tricalcium aluminate were thought to be chemical components having the most influence on set time. The CaO would be reported in CaO, of course, while the tricalcium aluminate will be reflected in both Al_2O_3 and CaO. The results for the Nebraska sample set are in good agreement with the previous statements. Both the CaO and Al_2O_3 have an influence on set time. Sulfur and sodium also have a great influence on set time. Sulfur acts as a retarder, while the sodium is network modifier in the fly ash, modifying the crystal structure. The Iowa fly ash data exhibit much different characteristics than the Nebraska data. Both sets have Powder River Basin coal sources so the results should have been similar. A major possible difference is probably due to not having as many Iowa samples to evaluate. The only similarity is observed in the case of sulfur and sodium. Also both data sets indicate that MgO does not have an influence on fly ash set time. The Iowa results indicate that CaO and Al_2O_3 do not have much influence on set time. The cementitious reaction products discussed in the literature review indicate this assumption is most likely invalid. Again this is probably attributed to only analyzing 13 different samples, instead of 150.

Table 22. Probability Estimates on Chemical Component Influence on Set Time for Nebraska and Iowa Self-Cementing Fly Ashes

Nebraska Fly Ashes		Iowa Fly Ashes	
Element	Pr > abs(t)	Element	Pr > abs(t)
CaO	< 0.0001	TiO_2	0.0087
Al_2O_3	< 0.0001	Na_2O	0.0119
SO_3	< 0.0001	SO_3	0.0134
Na_2O	< 0.0001	Fe_2O_3	0.0138
P_2O_5	0.0007	P_2O_5	0.0142
K_2O	0.0290	SiO_2	0.0151
BaO	0.0292	K_2O	0.0184
MgO	0.0960	CaO	0.0186
		Al_2O_3	0.0196
		MgO	0.1888

SUMMARY AND CONCLUSIONS

The following observation and conclusions have been drawn from this research:

- Iowa self-cementing fly ashes can be an effective means of stabilizing Iowa soils.
- Fly ash addition increases the compacted dry density of soil and shifts optimum moisture content to a lower percentage than for the soil alone.
- Modified compaction energy further increases the compacted dry density of soil-fly ash at moisture contents less than optimum for the soil itself.
- Compactive effort does not have a pronounced influence on compacted dry density of soil-fly ash at high moisture content.
- Compaction delay decreases compacted unit weight of soil-fly ash, in some instances a 13 pcf decrease was observed after four-hour delay.
- Unsoaked soil-fly ash exhibits higher strength with greater compaction energy when compacted at low moisture contents.
- Compaction effort shows little influence on unsoaked soil-fly ash strength when compacted at high moisture contents.
- In most instances compaction delay has a negative influence on strength gain of soil-fly ash.
- Strength gain potential continues to occur until the fly ash used for stabilization has reached its set time.
- Samples compacted dry of optimum moisture content exhibit slaking when saturated.

- Soil-fly ash should be soaked prior to testing in order to test the material in its weakest state for design purposes.
- It was observed that most strength gain of soil-fly ash was recorded during the first 7 to 28 days of curing, and a less pronounced increase continued after this time due to pozzolanic reactions.
- Strength of soil-fly ash continues to increase in the long-term due to pozzolanic reactions as the fly ash spheres continue to break down and provide calcium, alumina, and silica for cementitious reaction products.
- SO_3 contents greater than 5% cause formation of ettringite in soil-fly ash and the ettringite reduces the strength of the material.
- Fly ash addition rates of 20% strengthen soil-fly ash against slaking when compacted at low moisture contents.
- ISU 2-in x 2-in sample strengths multiplied by 0.86 seem to correlate well with compressive strength of standard 4-inch diameter Proctor sized samples.
- ISU 2-in x 2-in sample strengths multiplied by 0.90 correlate well with true unconfined compressive strength measurements.
- Proctor sized samples multiplied by 1.15 provide an indication of true unconfined compressive strength, although this data needs further evaluation.
- Fly ash addition increases CBR of natural soils, and in the case of 20% fly ash addition, the CBR was similar to that of gravels.
- Compressive strength of samples with L/D=2, L/D=1.15, and L/D=1 exhibit a parabolic relationship to CBR for corresponding mixture properties.

- Fly ash decreases swell potential in the short-term due to replacing some of the volume previously held by expansive clay minerals and by cementing the soil particles together.
- In some cases swell potential remained medium to high for soils because the samples were tested without being allowed to cure.
- Fly ash addition dries wet soils and provides an initial rapid strength gain in most instances that would allow construction operations to continue.
- After 7 days, the plasticity of a CL-CH alluvial soil was reduced with 20% fly ash and the mixture became classified as a CL soil.
- Plasticity was further reduced after curing 28 days, and the mixtures classified as CL-ML or ML.
- A CL-CH paleosol stabilized with 5, 10, 15, and 20% fly ash exhibited an increasing reduction in plasticity for greater ash contents, 5, 10, and 15%, while plasticity began to rise for a 20% addition rate.
- Plasticity is reduced as the clay particles flocculate and agglomerate in to larger, less plastic aggregations.
- Soil-fly ash cured below freezing and soaked in water slaked and was unable to be tested for strength.
- Typically, compressive strength increases as fly ash content and curing temperature increase.
- Soil-fly ash mixtures exhibit behavior that leads to the belief that these mixtures could autogenous heal after harsh winters.

- Samples cured below freezing and subsequently cured at 100° F showed strength very similar to that of samples that had been cured at 100° F only.
- It is believed that current methods used to test freeze-thaw durability (ASTM C593 and D560) of soil-fly ash and HFA and CFA are too harsh and not representative of field conditions.
- Paleosol stabilized with fly ash exhibited increased freeze-thaw durability when tested in accordance with ASTM C593, but stabilized Turin loess did not meet the requirements of ASTM C593.
- Fly ash increased the resistance to mass loss due to freeze-thaw of Le Grand loess tested in accordance with ASTM D560.
- HFA and CFA did not perform well when tested in accordance to ASTM D560, this is partly due to the test method not allowing for pozzolanic reactions to occur and no confinement of the samples, which could occur in field conditions.
- Moisture-strength relationships of HFA and CFA do not show a definite trend, but most materials tested exhibited a strength decrease when molded moisture content increased.
- HFA shows a continued strength gain over long cure times, and lower moisture contents resulted in higher strengths for these samples.
- Compaction delay did not have pronounced negative effect on HFA and CFA strength.

- OGS HFA did not exhibit a strength increase for samples cured at 8°, 40°, 70° F between 7 and 28 days but the samples cured at 8° and 40° F did exhibit an increase after 56 days, although the reasons for this are not completely known.
- OGS HFA cured at 100° F exhibited a relatively constant strength gain over 56 days.
- PN HFA showed $c'=10$ psi and $\phi'=32°$ after curing 28 days.
- PC CFA had $c'=18.6$ psi and $\phi'=43.5°$ after curing 28 days.
- OGS HFA exhibited almost undrained behavior at 28 days, which was most likely due to too rapid of a deviatoric loading rate.
- Based on lab results and field observations, HFA and CFA are suitable fill materials for a wide range of earthwork applications.
- Strength of soil was increased with HFA and CFA addition, but the strength gain was not as pronounced as would be observed for raw fly ash, due to the fact that pozzolanic reactions occur much slower than initial hardening of raw fly ash.
- Due to fly ash variability, set time was unable to be accurately predicted from a statistical model at times greater than 40 minutes.
- A statistical regression showed that CaO, Al_2O_3, SO_3, and Na_2O had the most influence on hydration characteristics of Nebraska fly ash.
- The same regression analysis for Iowa fly ashes showed that Na_2O, TiO_2, and SO_3 had the greatest influence on set characteristics and that CaO and Al_2O_3 did not have much of an effect. This was most likely due to the small data set for the Iowa fly ashes.

- Raw self-cementing fly ash, HFA, and CFA can be used to modify engineering properties of soils and increase the strength of unsuitable or unstable soils.
- Construction recommendations based on review of the literature and this research are presented in Appendices D through F.

FUTURE RECOMMENDATIONS

In addition to the laboratory testing program described in this thesis, shear strength parameter values need to be determined for soil-fly ash mixtures. Measurements should investigate the effects of fly ash content and moisture content on cohesion, c, and friction angle, ϕ. Cohesion and friction angle values are useful for evaluating slope stability, retaining wall backfill, and foundations.

Further, many of the reaction mechanisms between self-cementing fly ash and soil, mainly the clay fraction, are not completely understood. Future work should be completed to evaluate the fly ash-clay particle interactions. This could be accomplished through the use of scanning electron microscopy imaging and elemental analysis. High resolution SEM images could be used to observe the interactions between fly ash spheres and soil particles. Clay particles may show accelerated decomposition and higher amounts of cementitious reaction products may be present near the clay particle surfaces. Elemental maps of the high magnification images could aid in determining the extent of reaction products near the particle surfaces. The maps may also show if silica and alumina from the clay particles is becoming mobile and contributing to formation of cementitious reaction products.

APPENDIX A:
X-RAY DIFFRACTOGRAMS FOR FLY ASH AND HYDRATED AND CONDITIONED FLY ASH

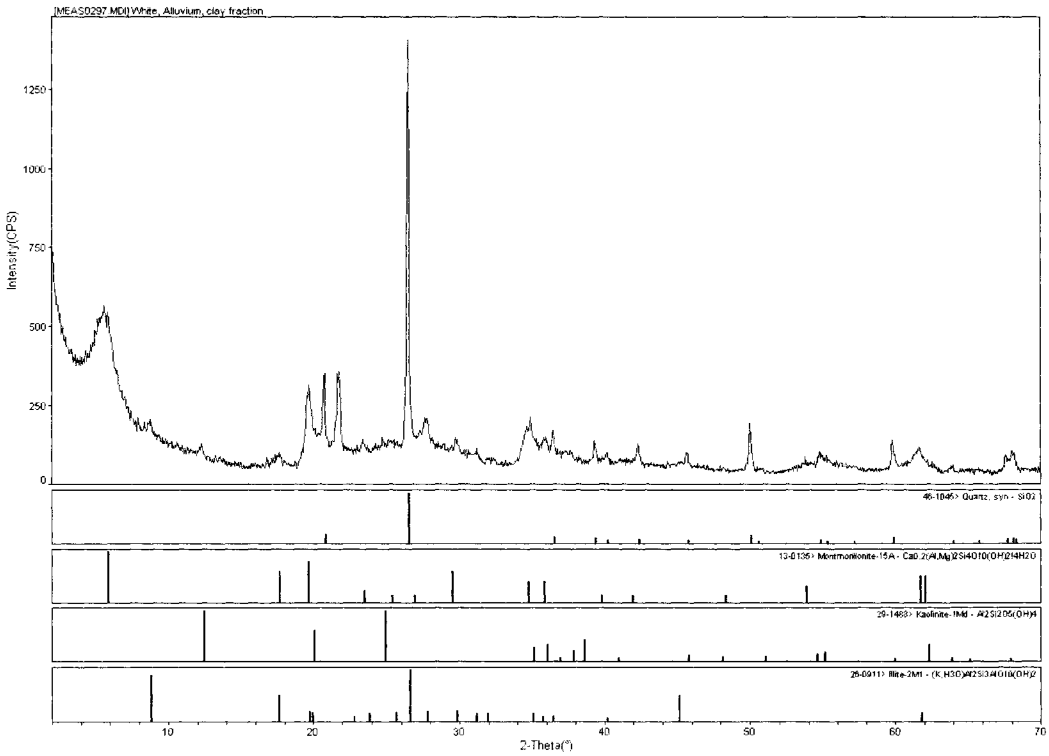


Figure 93. X-Ray Diffractogram for Neola Alluvium Clay Fraction (< 2 microns)

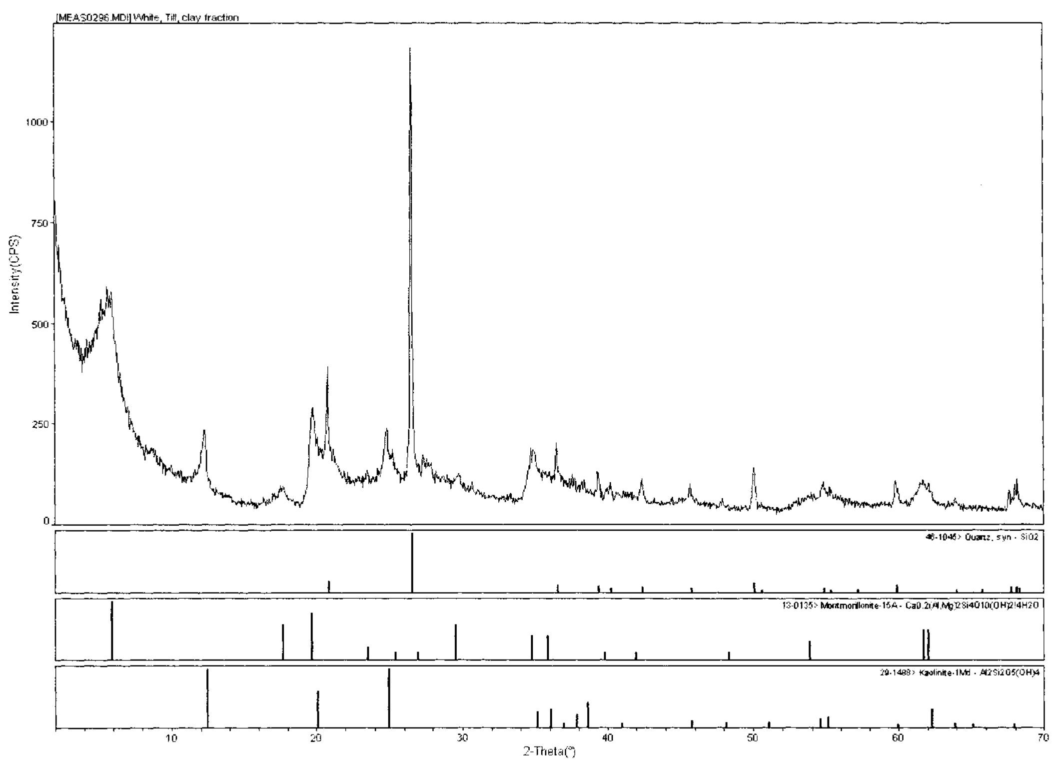


Figure 94. X-Ray Diffractogram for Cedar Rapids Till Clay Fraction (< 2 microns)

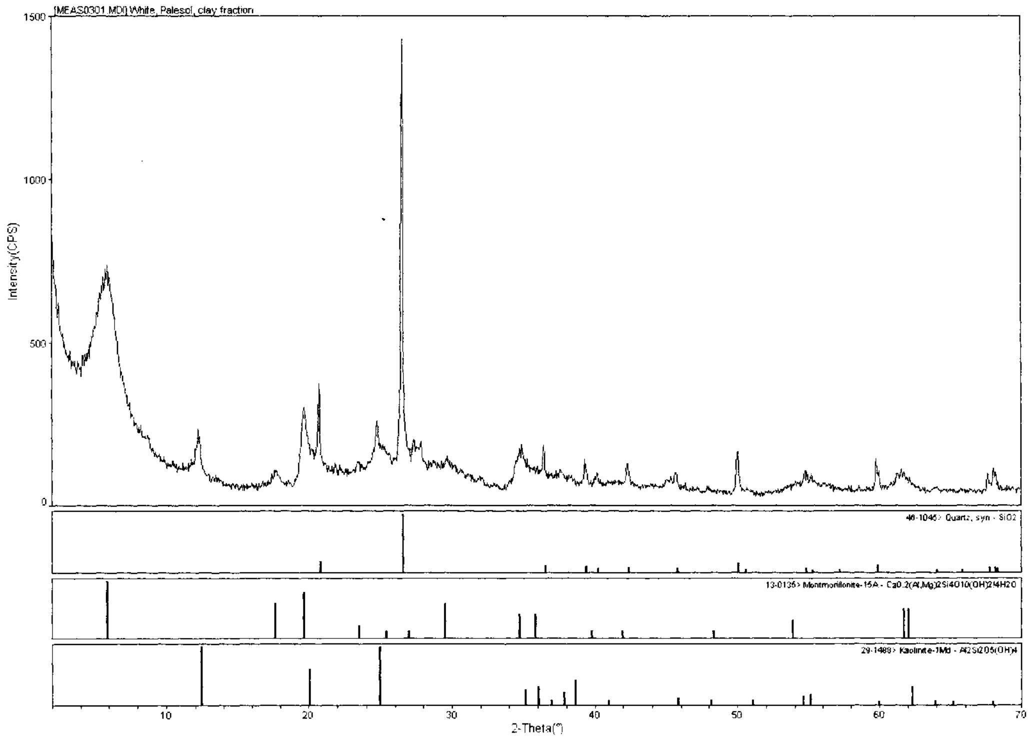


Figure 95. X-Ray Diffractogram for Argyle Paleosol Clay Fraction (< 2 microns)

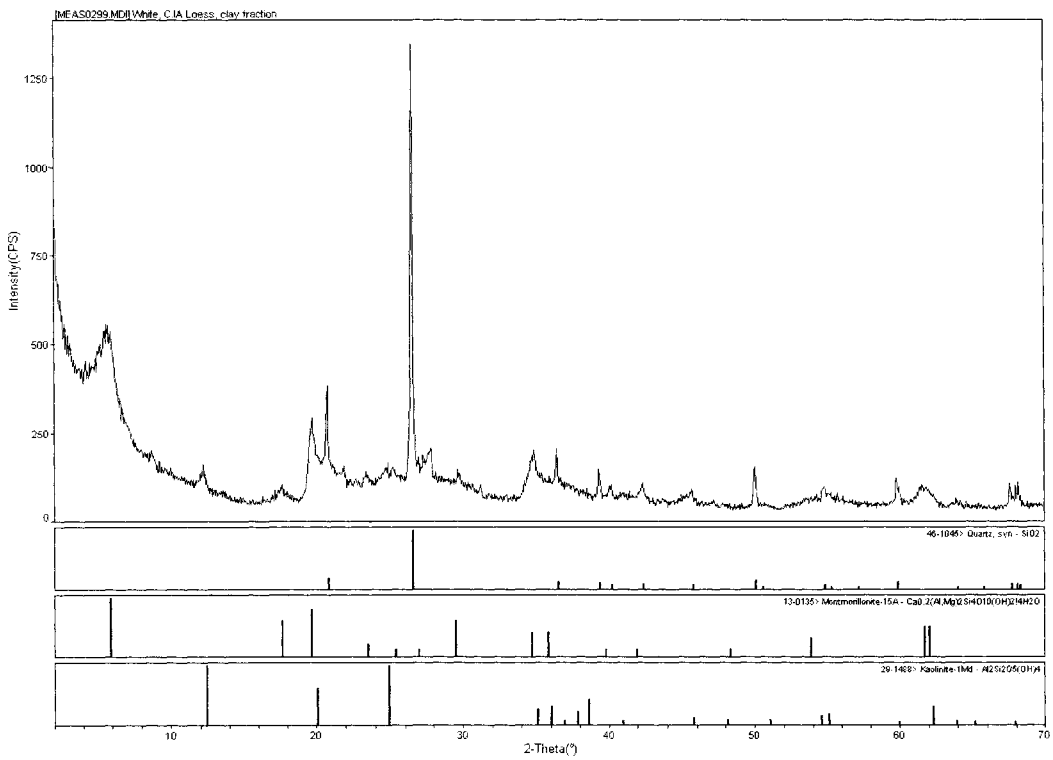


Figure 96. X-Ray Diffractogram for Le Grand Loess Clay Fraction (< 2 microns)

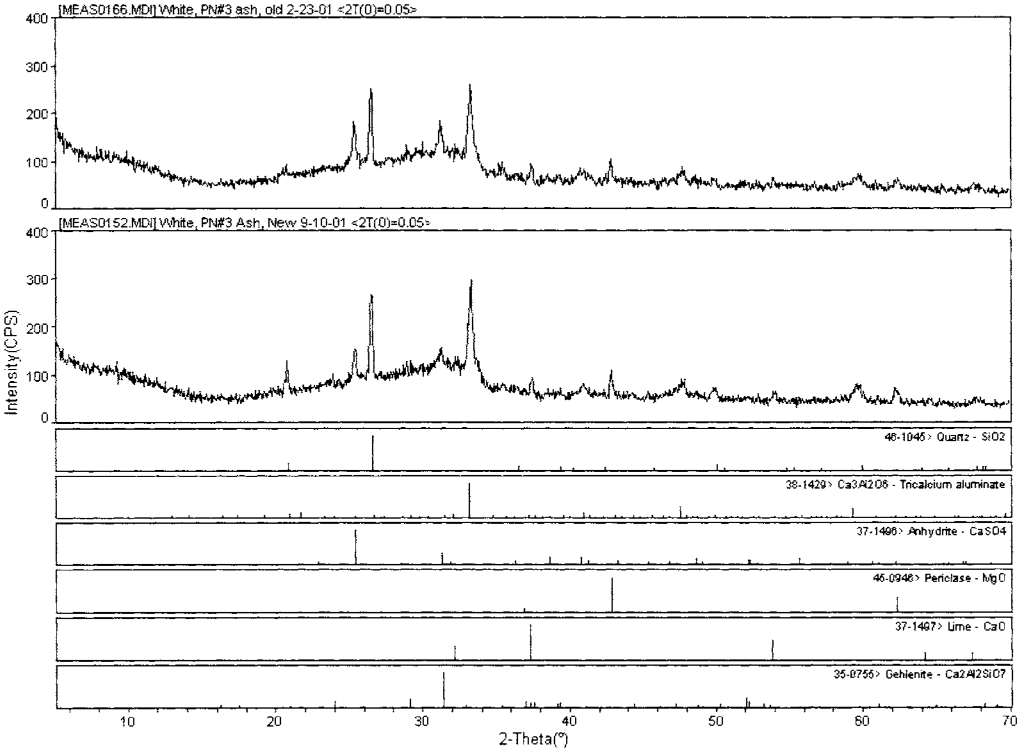


Figure 97. X-Ray Diffractograms for PN3 Old and PN3 New Fly Ash

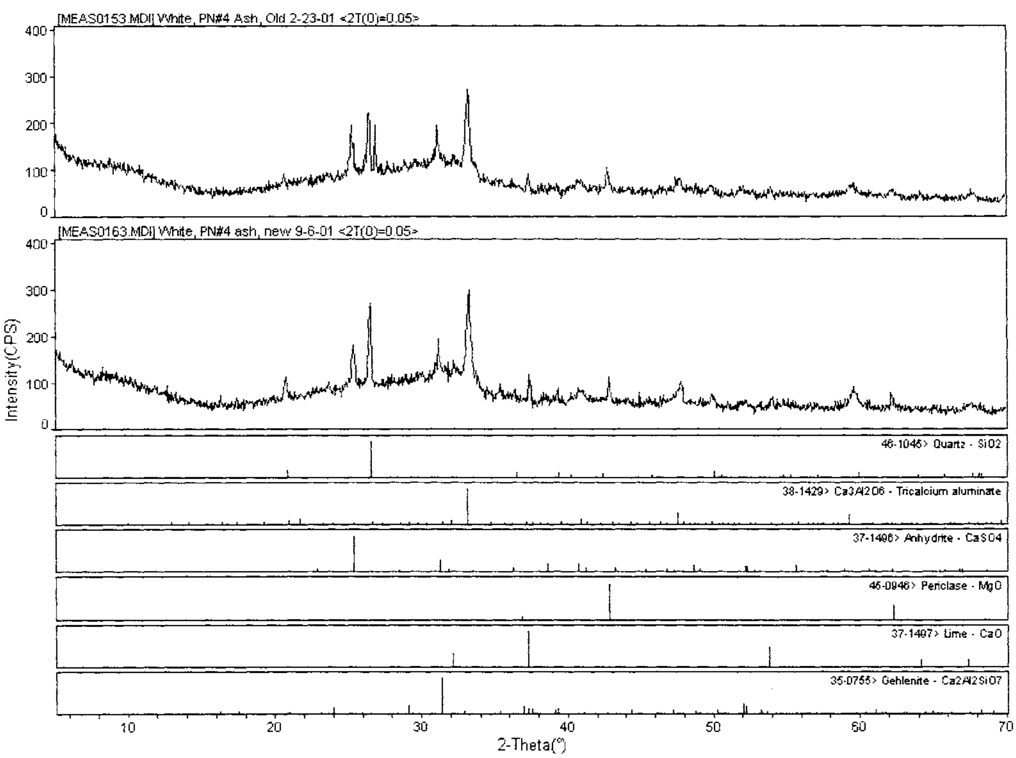


Figure 98. X-Ray Diffractograms for PN4 Old and PN4 New Fly Ash

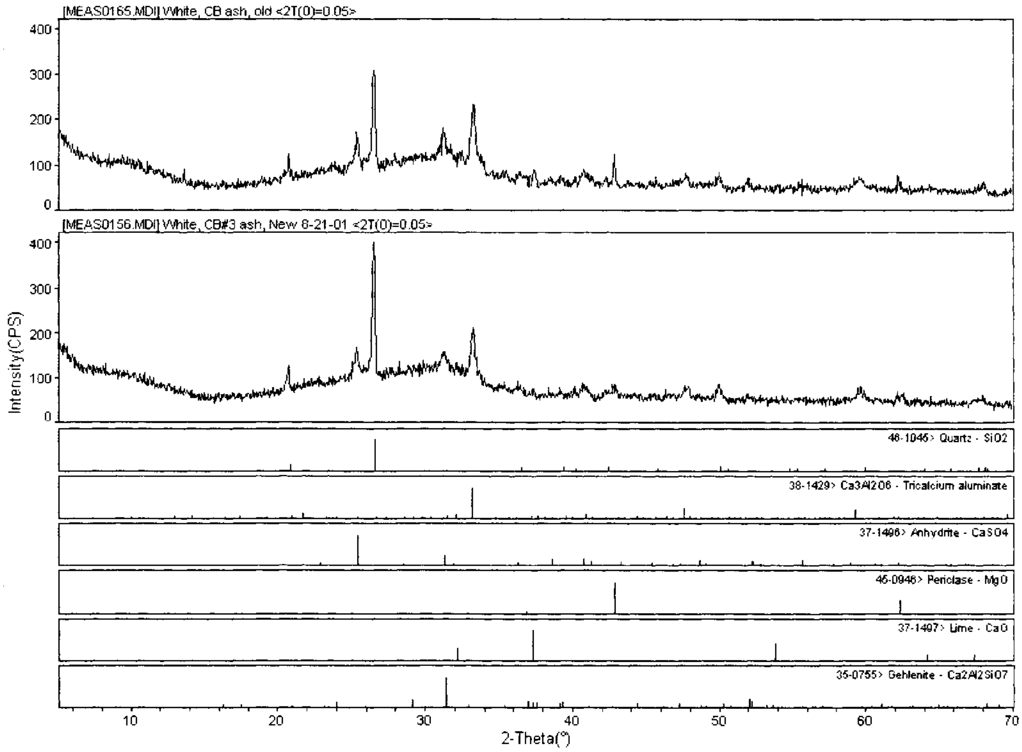


Figure 99. X-Ray Diffractograms for CB Old and CB New Fly Ash

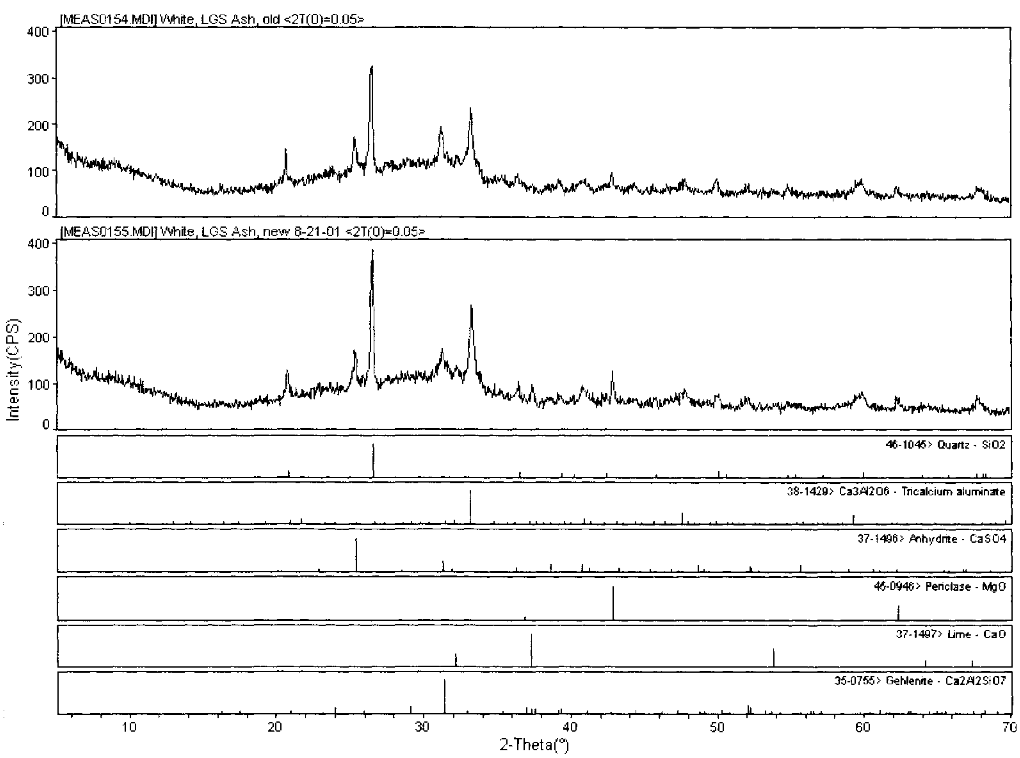


Figure 100. X-Ray Diffractograms For LGS New and LGS Old Fly Ash

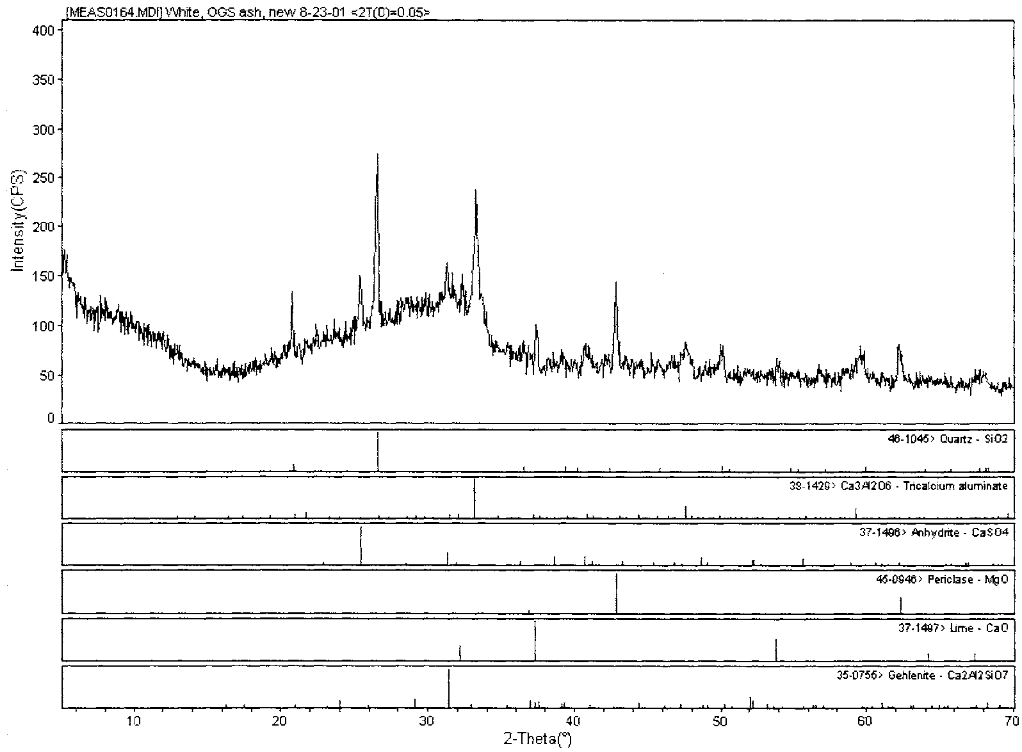


Figure 101. X-Ray Diffractogram for OGS New Fly Ash

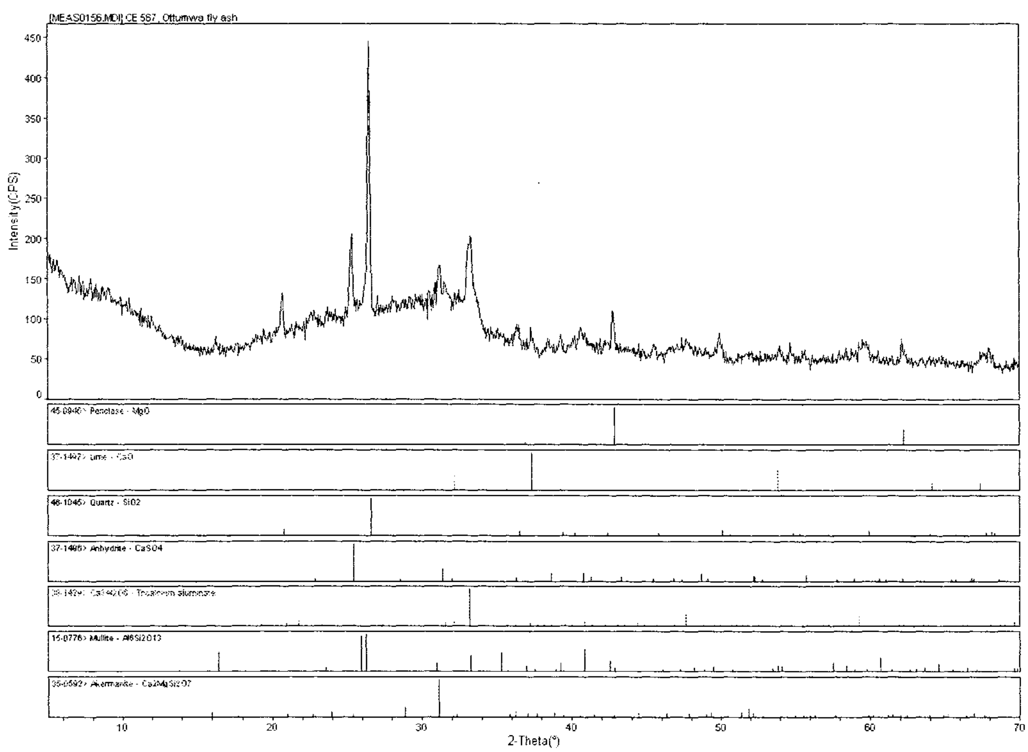


Figure 102. X-Ray Diffractogram for OGS Old Fly Ash

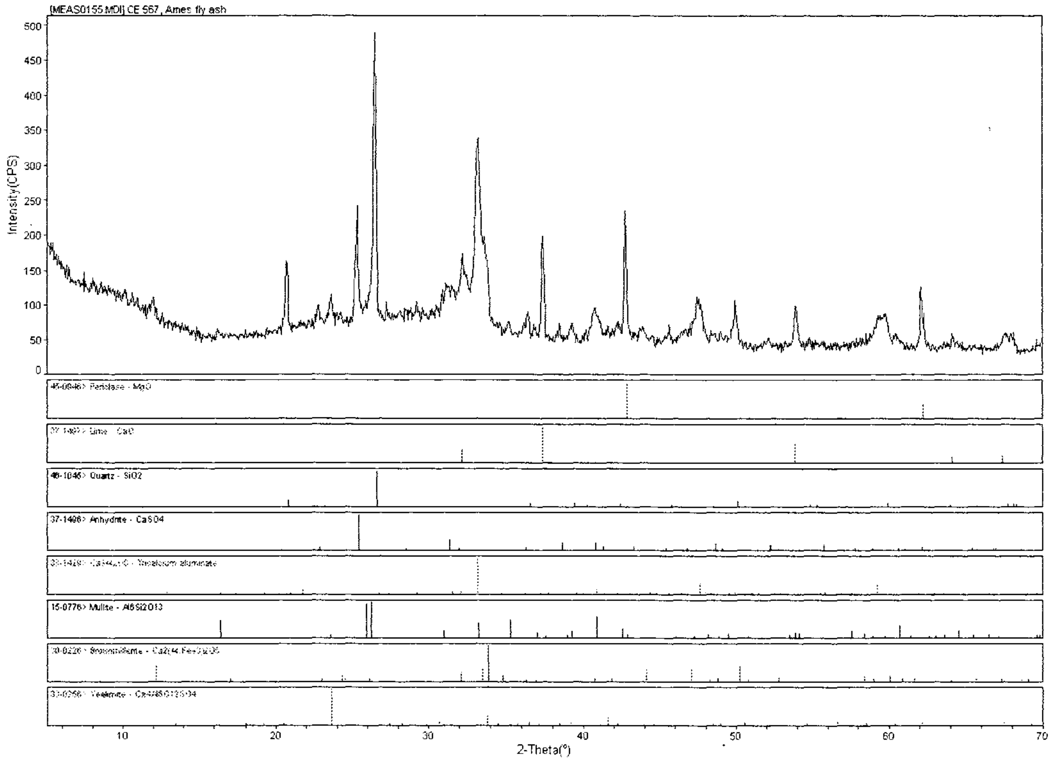


Figure 103. X-Ray Diffractogram for Ames Fly Ash

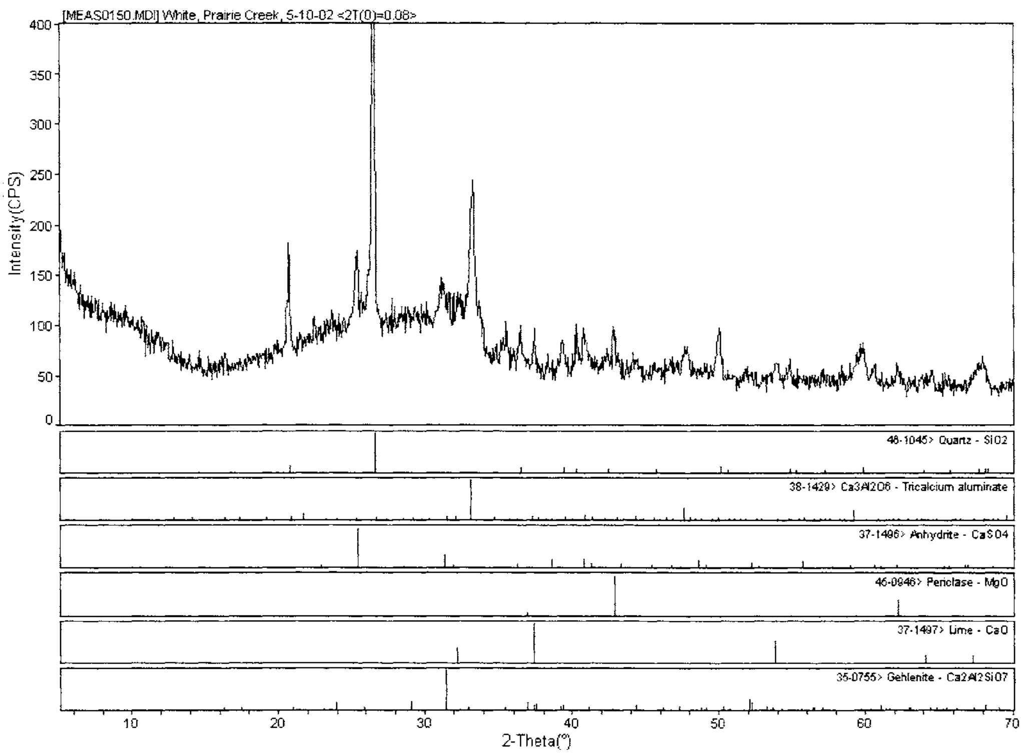


Figure 104. X-Ray Diffractogram for PC3+4 Fly Ash

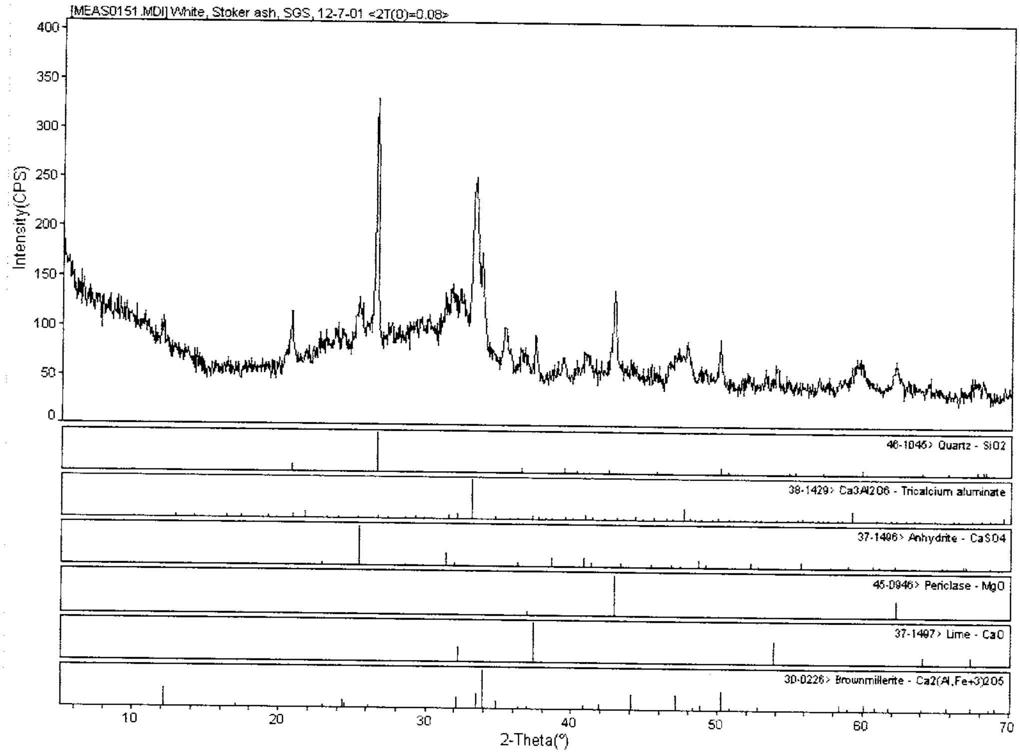


Figure 105. X-Ray Diffractogram For SGS Fly Ash

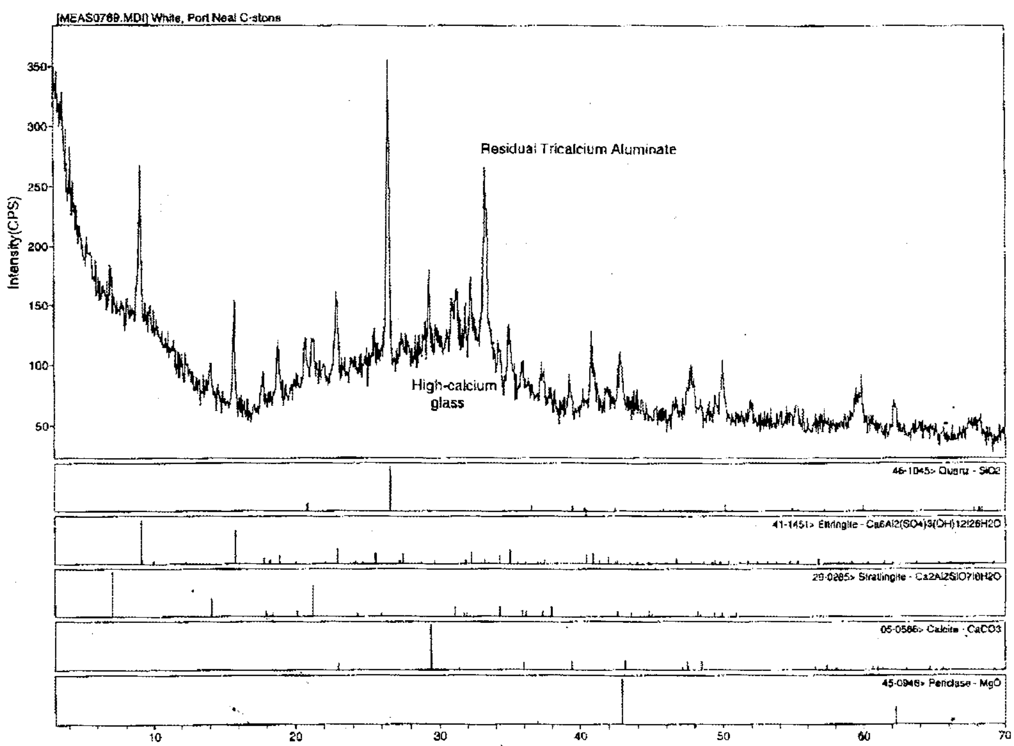


Figure 106. X-Ray Diffractogram for PN Hydrated Fly Ash

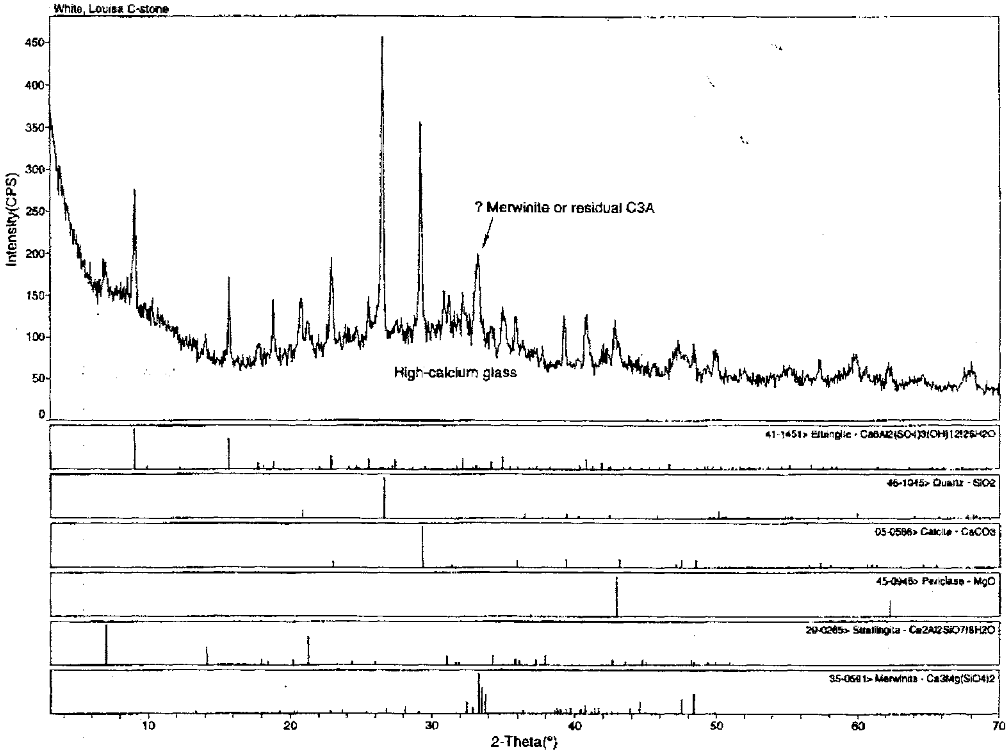


Figure 107. X-Ray Diffractogram for LGS Hydrated Fly Ash

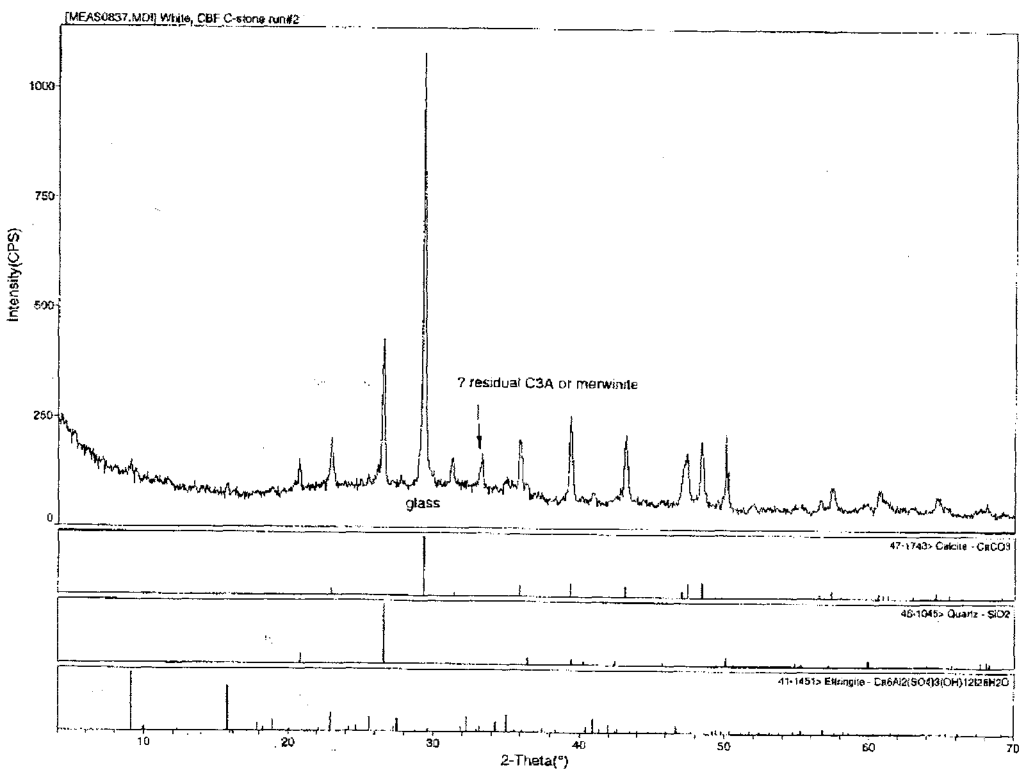


Figure 108. X-Ray Diffractogram for CB Hydrated Fly Ash

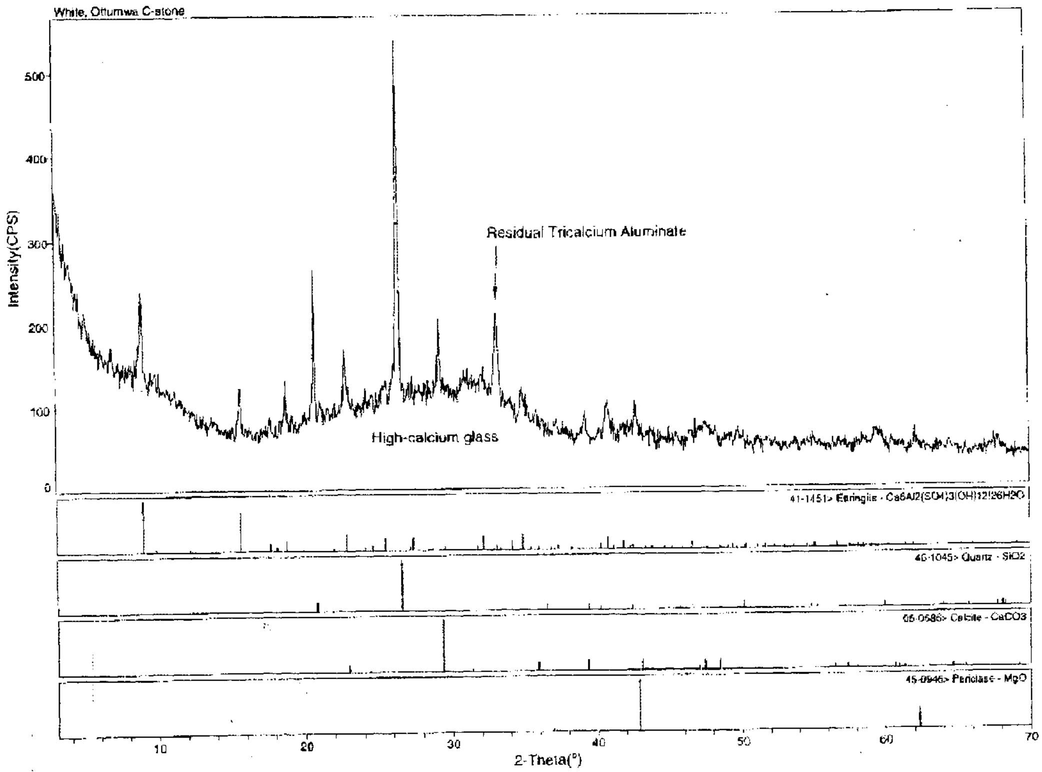


Figure 109. X-Ray Diffractogram for OGS Hydrated Fly Ash

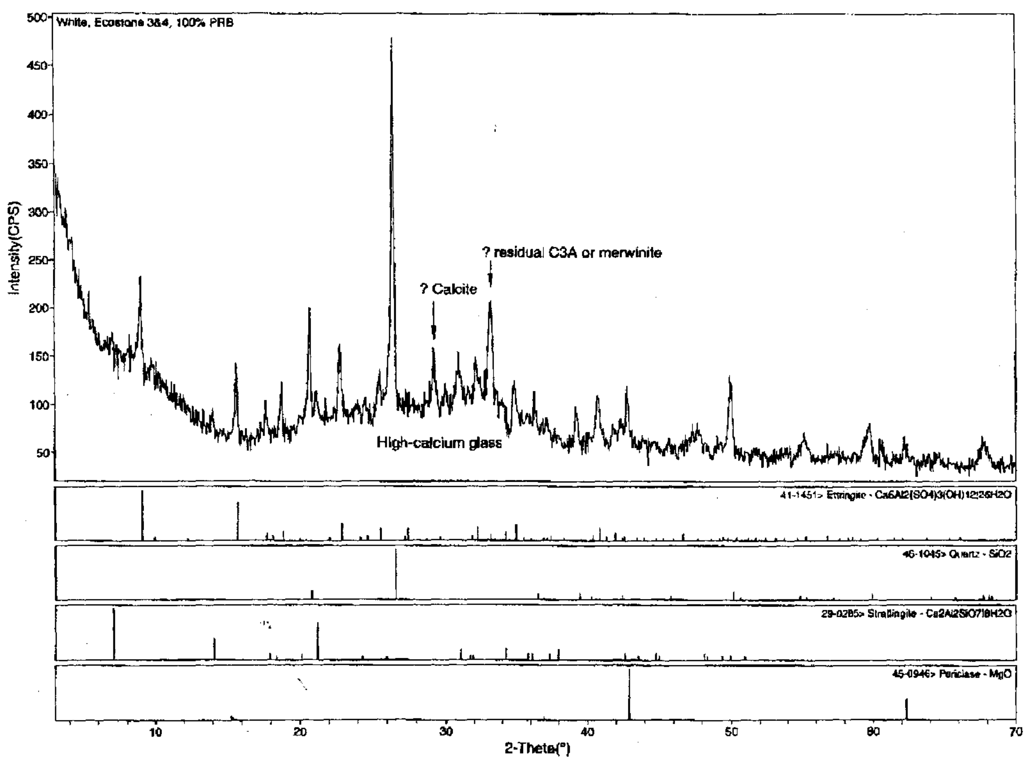


Figure 110. X-Ray Diffractogram for PC Conditioned Fly Ash

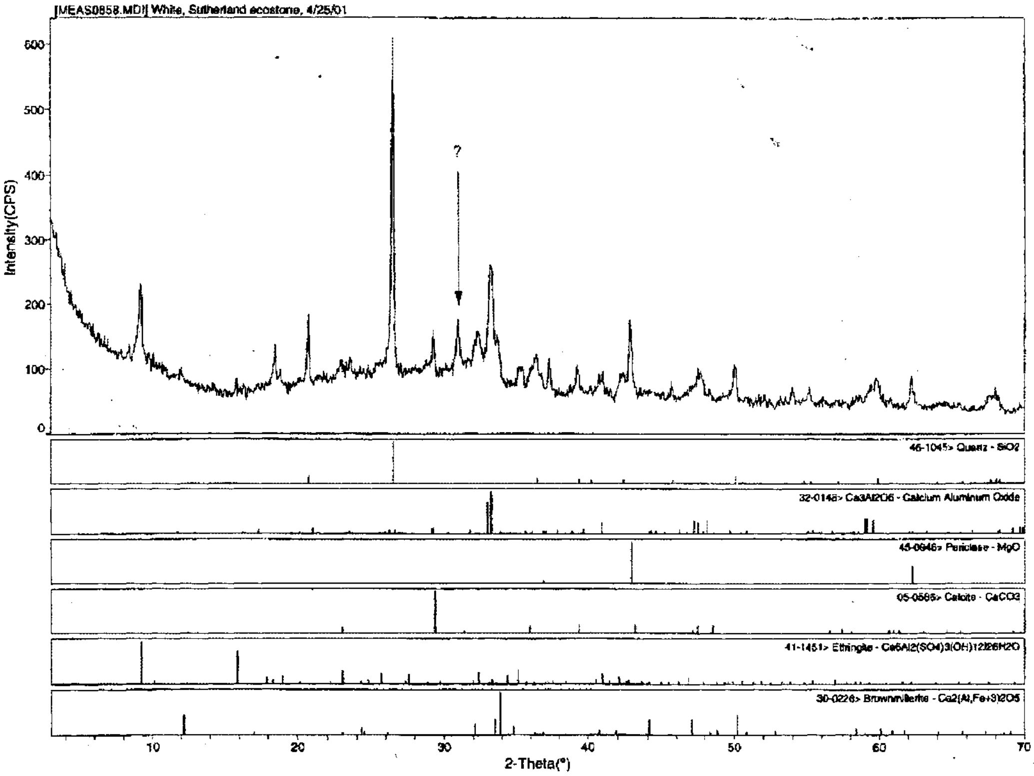


Figure 111. X-Ray Diffractogram for SGS Conditioned Fly Ash

APPENDIX B: STRESS-STRAIN DATA

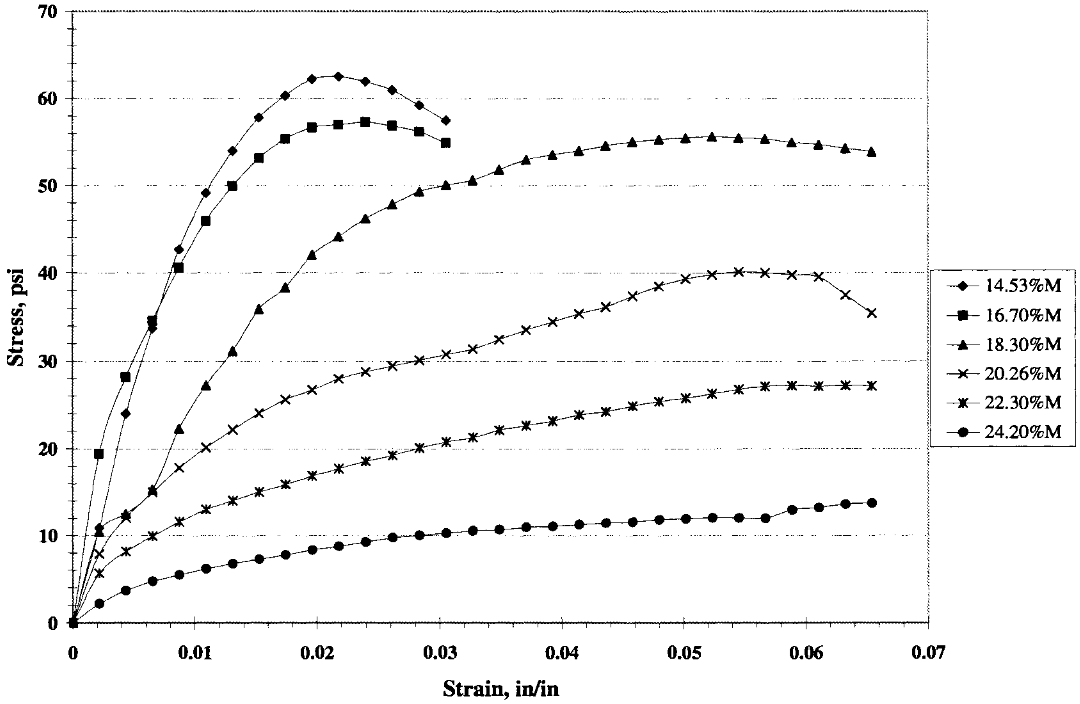


Figure 112. Stress-Strain Response of Neola Alluvium at Various Moisture Contents

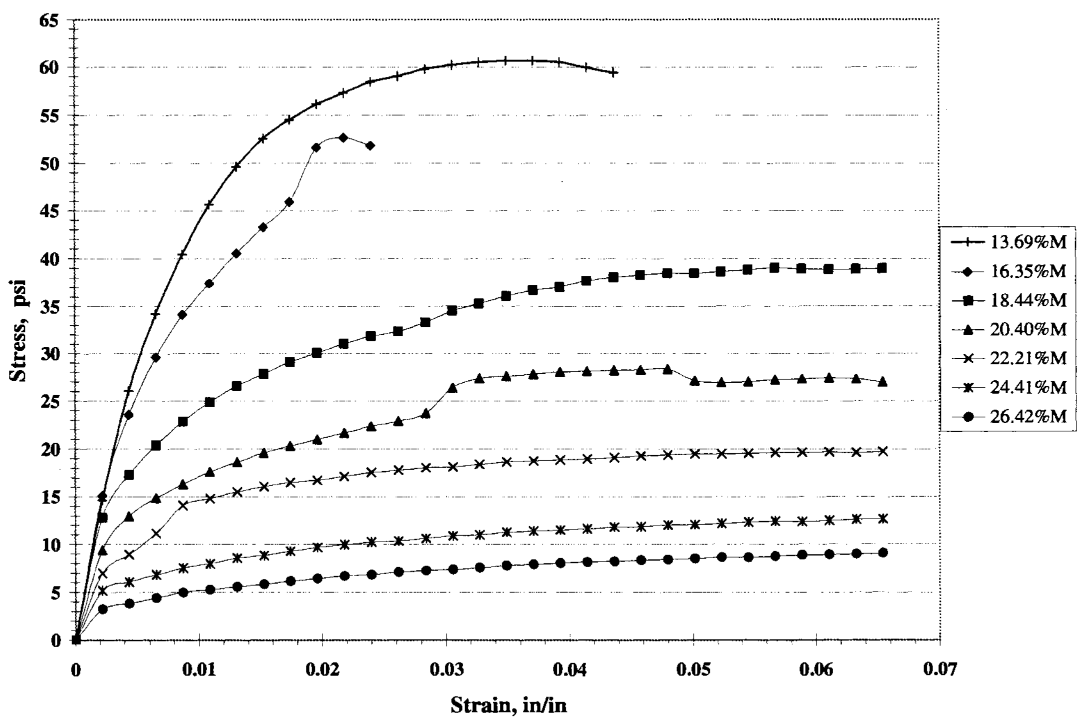


Figure 113. Stress-Strain Response of Argyle Paleosol at Various Moisture Contents

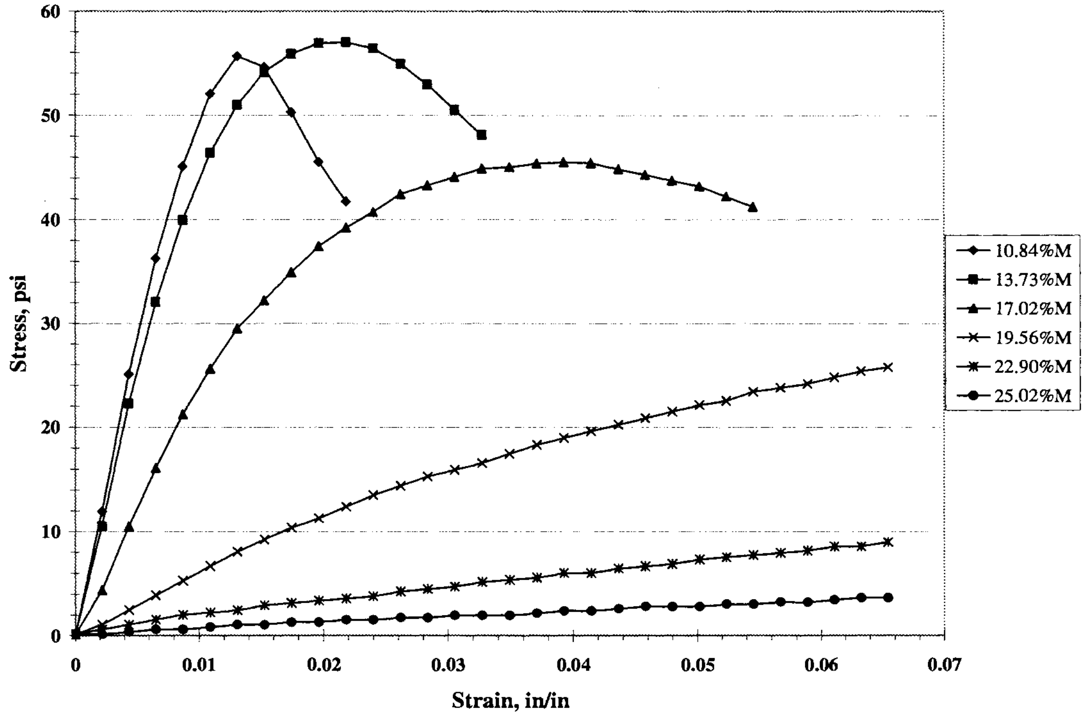


Figure 114. Stress-Strain Response of Le Grand Loess at Various Moisture Contents

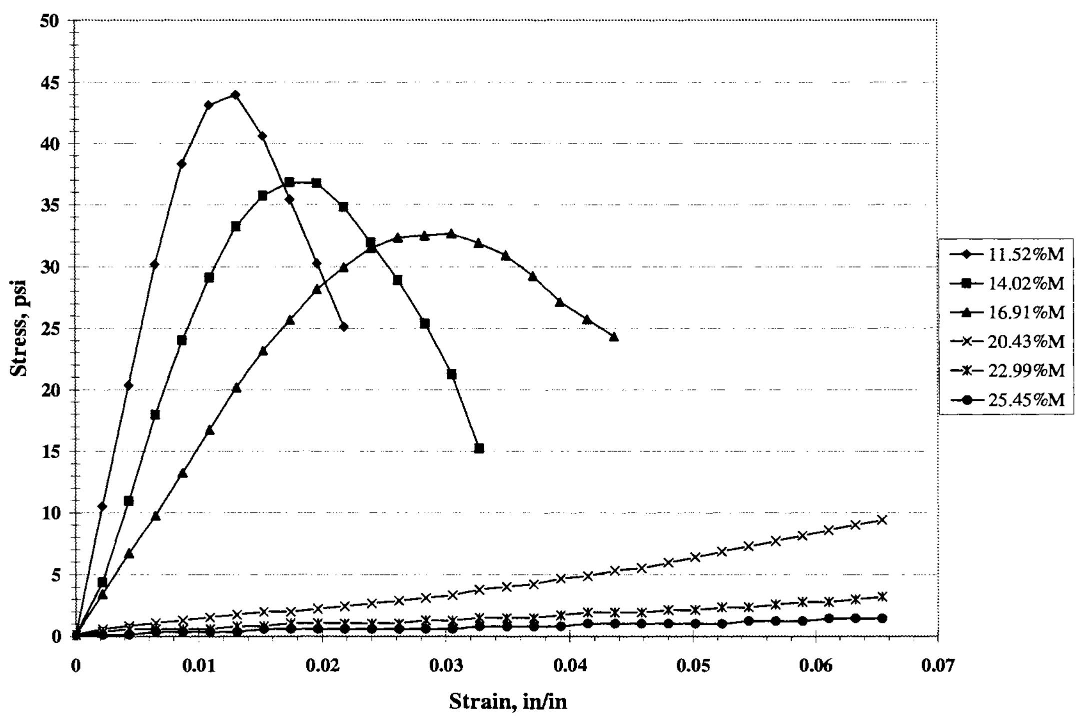


Figure 115. Stress-Strain Response of Turin Loess at Various Moisture Contents

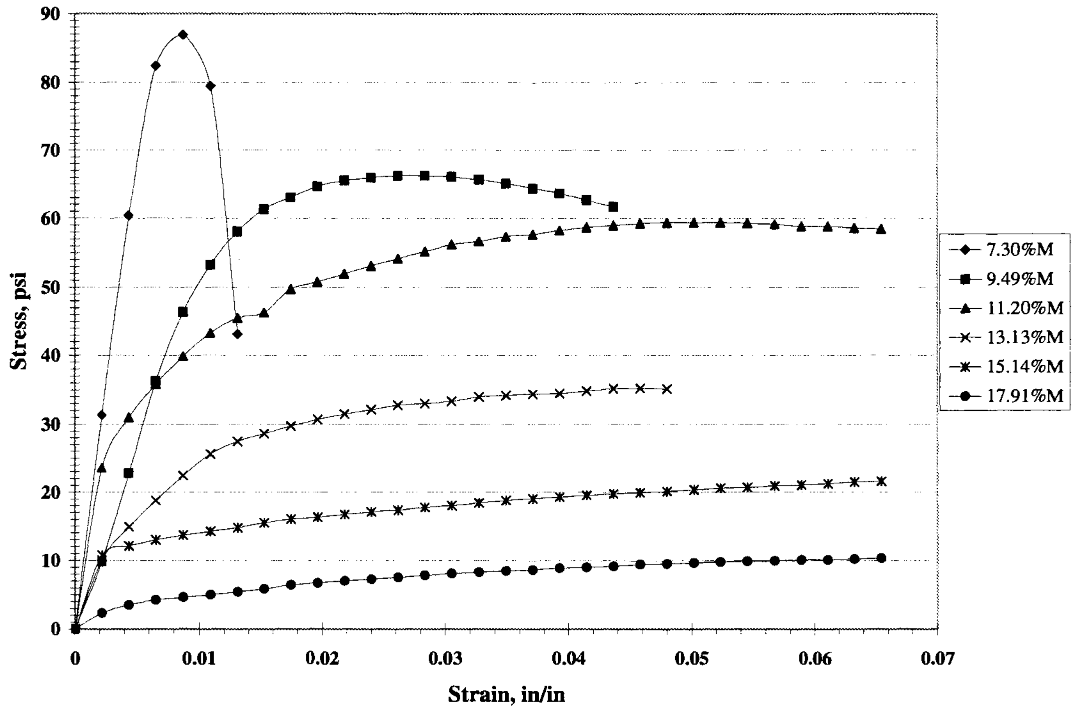


Figure 116. Stress-Strain Response of Cedar Rapids Glacial Till at Various Moisture Contents

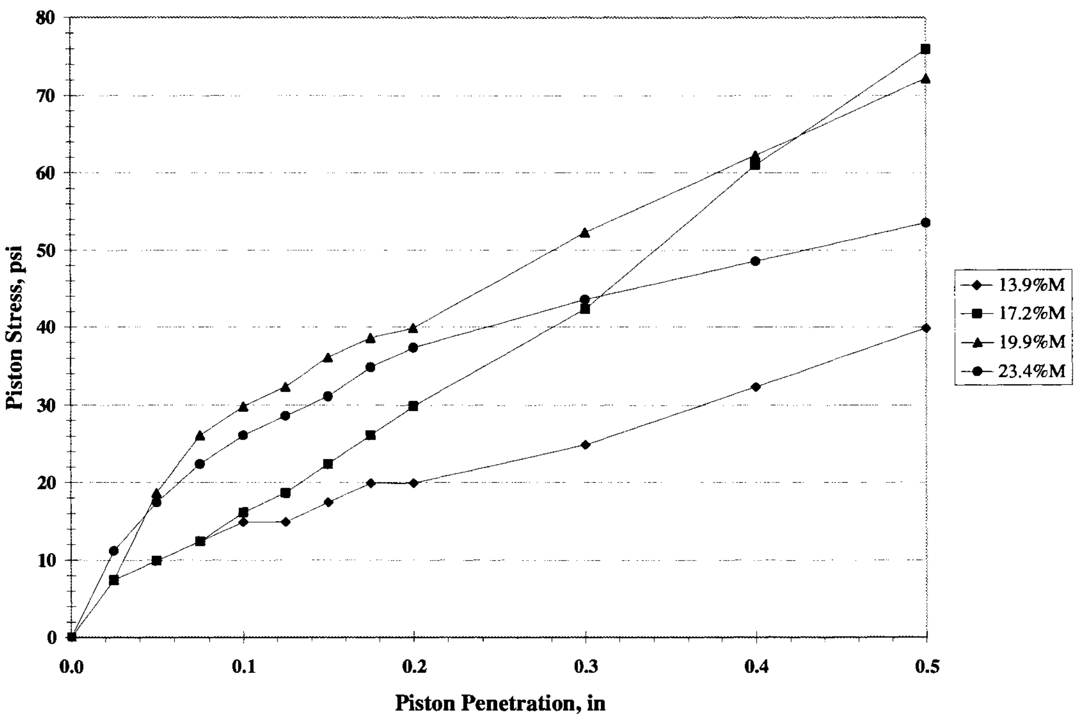


Figure 117. CBR Stress-Penetration Data for Le Grand Loess at Various Moisture Contents

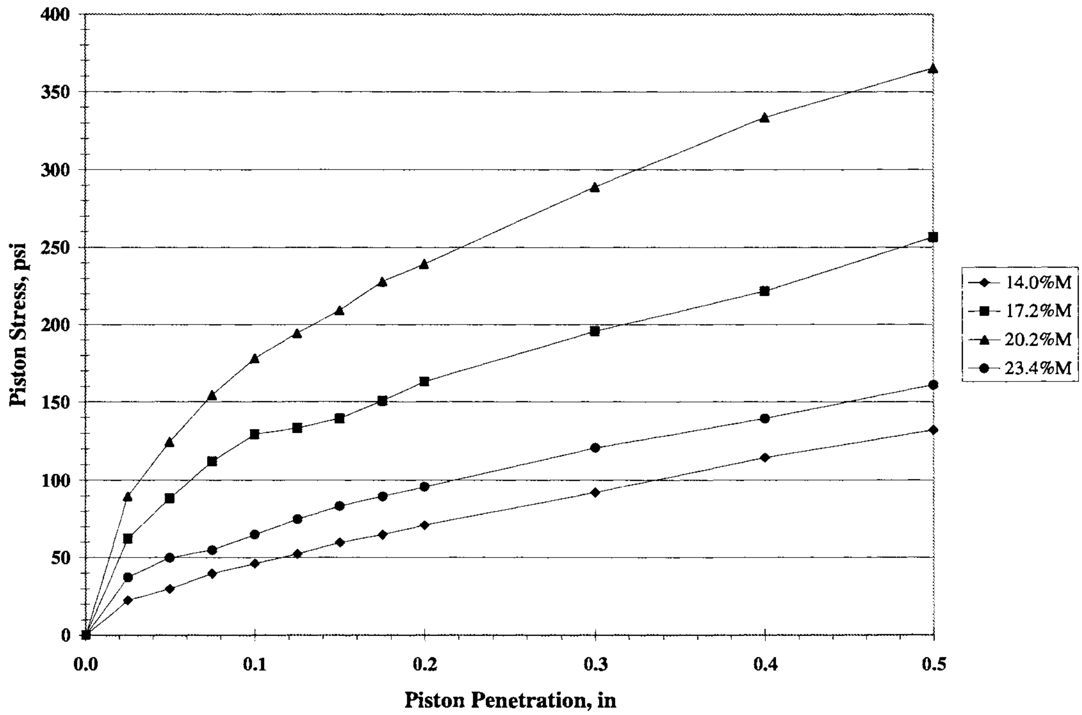


Figure 118. CBR Stress-Penetration Data for Le Grand Loess and 5% LGS Fly Ash

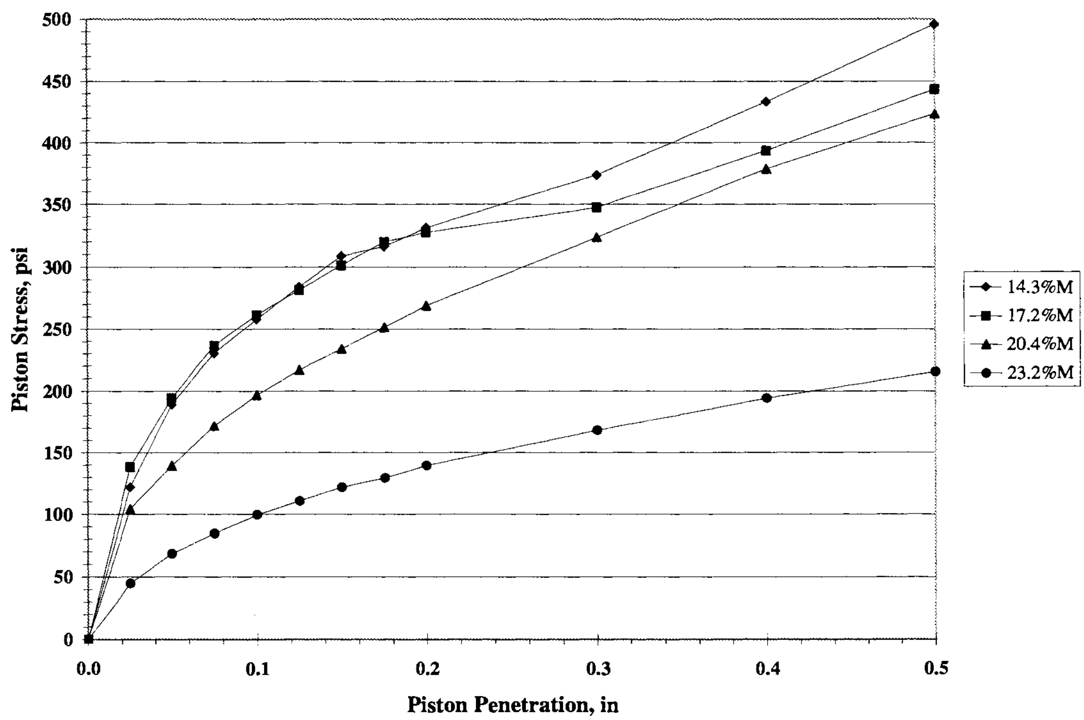


Figure 119. CBR Stress-Penetration Data for Le Grand Loess and 10% OGS Fly Ash

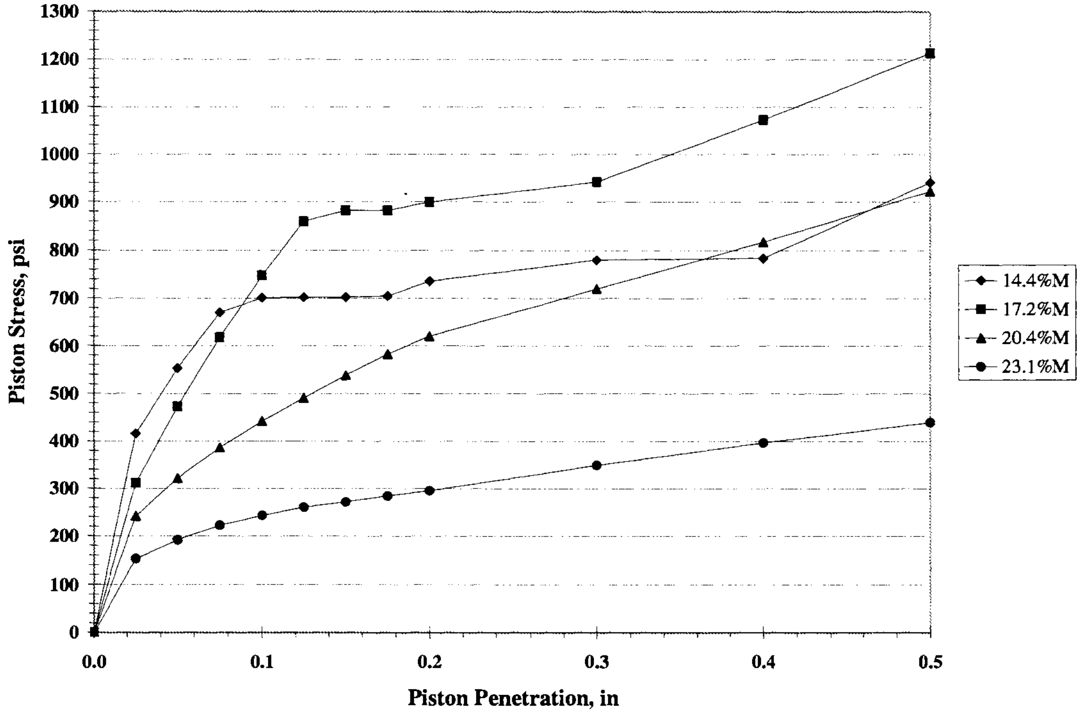


Figure 120. CBR Stress-Penetration Data for Le Grand Loess and 20% PN4 Fly Ash

APPENDIX C:
ELEMENTAL CHEMISTRY AND SET TIMES OF NEBRASKA SELF-CEMENTING FLY ASHES

APPENDIX D:
PROPOSED RECOMMENDATIONS FOR CONSTRUCTING SELF-CEMENTING FLY ASH STABILIZED SUBGRADE

Proposed Specification for Iowa DOT

USE OF SELF-CEMENTING COAL FLY ASH FOR SUBGRADE STABILIZATION

1. DESCRIPTION

This specification shall consist of the laboratory evaluation, field placement, moisture conditioning, compaction, and quality control testing of self-cementing fly ash stabilized subgrade to develop a sufficient subgrade section. This item shall be constructed as specified herein and in conformity with typical sections, lines and grades as shown on the plans or as established by the Engineer.

2. MATERIALS

2.1 Fly Ash

Self-cementing Class C fly ash complying with the chemical requirements of ASTM C 618, Table 1, [Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for use as a Mineral Admixture in Concrete] and meeting the physical requirements of ASTM D 5239, Section 6.4, [Standard Practice for Characterizing Fly Ash for Use in Soil Stabilization] with a compressive strength of at least 100 psi (3.45 MPa) at 7 days is approved for use. The source of the ash shall be identified and approved in advance of stabilization operations so the laboratory tests can be completed prior to commencing work.

Self-cementing fly ash produced from lignite or subbituminous coal that does not meet the chemical and physical requirements described above (i.e. compressive strength, LOI, etc.) may be approved for use if sufficient laboratory analysis (i.e. strength, durability, shrink/swell, etc.) is conducted to show adequate performance and is approved by the engineer. Based on dry weight, the fly ash soil mixture shall not contain more than 3.0 percent carbon.

Fly ash that is stored shall be stored in a weather tight facility to protect it from dampness. Fly ash that has become partially set or that contains hard lumps and cakes shall be discarded. Temporary storage (less than 12 hours) of fly ash in open pits will be allowed provided wetting of the fly ash is not allowed.

2.2 Mixing Water

Water used in fly ash soil mixtures shall be free from detrimental amounts of oil, salts, acids, alkali, organic matter, sulfur or other objectionable substances. Where the source of water is relatively shallow, it shall be maintained at a suitable depth, and the intake screened, to exclude objectionable amounts of silt, mud, grass, or other foreign material.

Water that contains suspended matter in excess of 2000 ppm shall be filtered or otherwise clarified.

Potable water obtained from a municipal supply, suitable for drinking, may be accepted without testing.

3. EQUIPMENT

3.1 General

The machinery, tools and equipment necessary for proper execution of the work shall be on the project and approved by the Engineer prior to beginning construction operations. Pulverization of the existing subgrade and blending of the fly ash modified subgrade mixture shall be accomplished with equipment such as a Bomag MPH 100, Caterpillar RM-350B, RR-250B or equivalent that has a recycling or mixing drum and a water proportioning system located in the drum. Initial compaction shall be achieved using a non-vibratory sheepsfoot roller or a vibratory padfoot roller. Final compaction of the stabilized section shall be achieved by the use of a pneumatic rubber tired roller or a smooth steel drum roller. All machinery, tools and equipment used during construction of the stabilized section shall be maintained in a workmanlike manner.

3.2 Trucking

Each truck that provides fly ash to the work site shall have the weight of fly ash certified on public scales or the Contractor shall place a set of standard platform scales or hopper scales at a location approved by the Engineer.

4. PLACEMENT

4.1 General

It is the primary purpose of this specification to construct a completed section of fly ash modified subgrade material which contains uniform moisture content with no loose or segregated areas; has a uniform density; and is well bound for its full depth. It shall be the responsibility of the Contractor to regulate the sequence of his work; to process a sufficient quantity of material to provide a completed section as shown on the plans; to use the proper amounts of fly ash; to achieve final compaction within the specified time; to maintain the work; and, to rework the lifts as necessary to meet the above requirements.

4.2 Weather Limitations

The soil temperature and ambient air temperature shall be at or above 40°F (4°C) for at least 24 hours prior to the time fly ash is placed, mixed and compacted. The Contractor shall be responsible for the protection and quality of the fly ash modified subgrade mixture under all weather conditions. Fly ash spreading, mixing, and compaction of the soil/fly ash mixture shall not proceed during periods of rain and snow or when rain and snow are possible before a stabilized section can be completed. Fly ash stabilization operations cannot begin when the subgrade material is frozen.

4.3 Preparation of Subgrade

Before the fly ash is placed, the area shall be cut and shaped in conformance with the lines and grades shown on the plans. The subgrade shall be firm and able to support the construction traffic associated with hauling, placing, and blending the fly ash. Soft subgrade areas shall be corrected and made stable by overexcavating, adding suitable material that may or may not contain fly ash, and compacting until the area is of uniform stability.

4.4 Moisture Control

Moisture control shall be achieved through the use of a pulvamixer equipped with a spray bar located inside the mixing drum. The spray bar apparatus shall be capable of applying sufficient quantities of water in a single pass to achieve the required moisture content for the fly ash modified subgrade mixture. The addition of water in the mixing drum shall be capable of being regulated to the degree necessary to maintain moisture contents within the range specified by the Engineer.

The Engineer shall establish required moisture contents based on laboratory tests conducted with the site-specific soil and the fly ash to be used during construction. Final moisture content of the fly ash modified subgrade shall not exceed ±2% (based on dry weight) the "optimum" moisture-strength relationship (see section 8). If the moisture content of the fly ash modified subgrade mixture is greater than the specified limit, additional fly ash may be added to lower moisture contents to within the specified limits. Once compacted, moisture conditioning or aeration will not be permitted.

4.5 Application of Fly Ash

Immediately prior to placement of fly ash, the area shall be graded to provide a uniform distribution of fly ash.

The fly ash may be hauled to the construction site in belly dump trucks or end dump trucks. The fly ash shall be hauled in such a manner as to reduce the loss of material during transportation. Fly ash shall not be applied using the slurry method.

After the fly ash has been applied to the construction site, it shall be spread to the required depth with a maintainer, bulldozer, box scraper, or any other means approved by the Engineer to minimize scattering of fly ash by wind. Fly ash shall not be placed on site when wind conditions, in the opinion of the Engineer, are such that blowing fly ash becomes objectionable to adjacent property owners or traffic or significantly reduces the amount of fly ash incorporated into the fly ash modified subgrade mixture. Between the time of fly ash placement on site and the beginning of mixing, fly ash that has become unacceptable because of excessive wetting can still be incorporated into the stabilized section but the section must be retreated with the appropriate amount of fly ash. The unacceptable ash could otherwise be removed from the stabilized section and be disposed of. Additional fly ash that is used for replacement purposes shall be at the sole expense of the Contractor.

Mixing of the soil and fly ash shall be completed within one half hour of fly ash placement on site.

4.6 Mixing

The subgrade soil and fly ash shall be thoroughly mixed by the approved pulvamixer and continue to be mixed until a homogeneous friable soil/fly ash mixture, free of clods or lumps is obtained. Any fly ash modified subgrade mixture containing clumps after initial mixing shall be remixed to remove clumps. The following size requirement shall be achieved before compaction begins.

Sieve Size, inches (mm)	Percent Passing
1.5 (38.1)	95
0.75 (19.1)	$\geq$ 50

If the temperature of the soil is between 40°F (4°C) and 45°F (7°C) prior to incorporation of the fly ash, a single pass of the pulvamixer will be required, unless not required by the Engineer. Incorporation of the fly ash and the second pass of the pulvamixer shall be completed within 15 minutes after initial mixing.

4.7 Compaction

Compaction of the fly ash modified subgrade mixture shall begin immediately after final mixing is completed and shall be completed within one hour of the beginning of initial mixing to prevent loss of strength and moisture. Compaction of the fly ash modified subgrade mixture shall start at the bottom of the layer and continue until the entire depth of the mixture is uniformly compacted to the specified density.

The fly ash modified subgrade mixture shall be compacted initially using a non-vibratory sheepsfoot roller or a vibratory padfoot roller. The stabilized mixture shall be compacted to ≥ 95 percent of maximum dry density as determined by ASTM D-698, [Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5-lb. Rammer and 12-in. Drop]. The stabilized subgrade shall not be placed more than 12 inches (30 cm) deep for any lift. Final compaction shall be achieved using a steel smooth drum roller or pneumatic rubber tired roller to seal the surface and reduce loss of moisture. Non-uniform sections shall be corrected immediately by scarifying the affected areas, adding or removing material as required by the Engineer and remixing and recompacting.

In addition to the requirements specified for density and moisture, the full depth of compacted stabilized subgrade shall remain firm and stable under further construction traffic. If during construction the compacted fly ash modified subgrade mixture is subjected to rain, areas of standing water shall be bladed off and reworked as deemed necessary by the Engineer.

Once the stabilized subgrade section is compacted, the Contractor and Engineer shall perform the necessary field quality control (QC) and quality assurance (QA) tests, respectively. If fly ash modified subgrade material fails to meet this standard, the Engineer may require reworking of the lift or a change in the Contractor's construction methods on the next section. Areas that are reworked must have additional fly ash added as established by the Engineer. If the material loses the required stability, density or finish for any reason before the Engineer accepts the work, it shall be reprocessed, recompacted, and refinished at the sole expense of the Contractor. The reprocessing of a failed lift shall follow the same procedures as initial stabilization, including the addition of fly ash.

Placement of fill over a stabilized section that has been accepted by the Engineer shall not begin for a period of at least 24 hours. Any unstable areas that result from the placement of succeeding layers of fill shall be removed to a depth below the initial stabilized section. The underlying soils and surrounding soils on successive lifts shall be stabilized with fly ash.

4.8 Finishing and Curing

Once the last lift is completed it must be brought to the required lines and grades in accordance with the typical sections.

After the fly ash modified subgrade section has been finished as specified herein, the surface shall be protected against rapid drying by one of the following curing methods until the succeeding lift or pavement section is placed:

(a) Maintain in a thorough and continuously moist condition by sprinkling with water.
(b) Apply a 2-inch layer of well-graded crushed aggregate on the completed course and maintain in a moist condition.
(c) Apply an asphalt seal coat consisting of cutback or emulsion asphalt.

5. QUALITY CONTROL/QUALITY ASSURANCE TESTING

The Contractor shall provide and maintain a Quality Control (QC) program, defined as all activities for sampling, testing, process control inspection, and necessary adjustments for construction of fly ash modified subgrade mixtures to meet the requirements shown on the plans or as established by the Engineer.

5.1 Test Strip Construction

Prior to the beginning of all construction operations, a test strip shall be constructed in order to verify proposed construction and testing methods. The test strip will be a minimum of 2.4 m (8 ft) in width and 30.5 m (100 ft) in length. The Engineer must approve changes that must be made due to the findings of the test strip construction. Additional test strips shall be required when the subgrade soil, construction methods, or fly ash changes, or as required by the Engineer. Construction of the test strip shall follow Section 4.

5.2 Quality Control Test Frequency During Construction

As directed on the plans or as established by the Engineer, compacted lift thickness, moisture content, density and/or strength/stability of compacted fly ash modified subgrade mixture shall be measured for the stabilized section being placed. Every 2.4 m (8 ft) in width by 30.5 m (100 ft) area of each compacted lift shall be tested. For test strip sections every 2.4 m (8 ft) in width by 7.6 m (25 ft) area of each compacted lift shall be tested.

5.3 Field Records

The Contractor shall be responsible for documenting all observations, records and inspection, changes in fly ash and soil classification, moisture content, fill placement procedures, and test results on a daily basis. The results of the observations and records of inspection shall be noted as they occur in a permanent field record. Copies of the field-test results, test strip construction procedures and production construction procedures shall be provided to the Engineer on a daily basis.

5.3 Control Charts

The Contractor shall maintain standardized control charts for field test measurements. The charts shall be posted at a location agreed upon by the Contractor and the Engineer. Test results obtained by the Contractor shall be recorded on the control charts the same day the tests are conducted. The results for the described field data shall be recorded on the standardized control charts for all randomly selected locations tested.

Both the individual test point and the moving average of four data points shall be plotted on each chart. The Contractor's test data shall be shown as black (filled) circles and the moving average in unfilled circles. Additional tests or retests, which have been randomly selected, shall be shown as black (filled) squares. Other means of chart plotting may be used when approved by the Engineer. Legends used on the control charts shall be consistent throughout the project.

5.4 Corrective Action

The Contractor shall notify the Engineer when a single moisture content test or a 4-point moving test average of density and/or strength/stability falls outside the specified control limits. All randomly selected tests shall be part of the project files and shall be included in the moving average calculations.

If a single moisture content of fly ash modified subgrade falls outside of the control limits, the material in the area represented by the test shall initially be considered unacceptable. In this case the Contractor may perform four additional randomly located re-tests within the specified test area. If the average of these four re-tests is within the specified moisture control limits, the test area will be considered acceptable. If the average moisture content of the four re-tests still falls outside of the control limits, the test section is considered unacceptable and correction action following section 4.7 shall be implemented.
If a 4-point moving average from the density and/or strength/stability tests fall outside of the specified control limits, the Contractor shall take corrective action(s) on the subsequent fly ash modified subgrade placed. The Contractor and Engineer shall discuss corrective action(s) to bring the fly ash modified subgrade material for the subsequent sections above the control limits.

If the corrective action improves the failed field test such that the new moving average, after a re-test, is within the control limit, the Contractor may continue fly ash modified subgrade placement.

If the new moving average point is still outside of the control limit after the re-test, the subgrade material in the recently tested area shall be considered unacceptable, and the Contractor shall perform additional corrective action(s) to improve the fill material until the new moving average, after a re-test, falls within the control limits.

5.5 Incorrect Data

If the Contractor's initial control data is later proven incorrect, which results in a corrected single moisture content or a corrected 4-point moving average of density and/or strength/stability falling outside of the control limits, the subgrade material represented by the incorrect test data shall be considered unacceptable. The Contractor shall employ the methods described above for corrective action of unacceptable materials.

5.6 Required Testing and Personnel Requirements

The Engineer will conduct assurance tests on split samples taken by the Contractor for fly ash and soil classification, moisture content limits determination, and laboratory compaction testing. These samples may be from sample locations chosen by the Engineer from anywhere in the process. The frequency of testing for the split samples will be equal to or greater than 10 percent of the tests taken by the Contractor. The referenced assurance test results will be provided to the Contractor within one working day after the Contractor's quality control test results have been reported.

The frequency of assurance testing for the field moisture and density and/or strength/stability tests will be equal to or greater than 10 percent of the tests required for the Contractor's quality control. The results of referenced testing and measurement will be provided to the Contractor on the day of testing.

A certified technician shall perform all field-testing and data analysis. The certified technician shall retain split samples from those obtained by the Contractor. The Engineer may select any or all of the Contractor-retained split samples for assurance testing.

The Engineer will periodically witness field-testing being performed by the Contractor. If the Engineer observes that the quality control field tests are not being performed in accordance with the applicable test procedures, the Engineer may stop production until corrective action is taken. The Engineer will notify the Contractor of observed deficiencies, promptly, both verbally and in writing. The Engineer will document all witnessed testing.

5.7 Testing Methods and Precision

5.7.1 Compaction

Field-testing to measure in-place density shall be determined in accordance with ASTM D-2167 [Standard Test Method for Density and Unit Weight of Soil in Place by the Rubber Balloon Method] or ASTM D-1556 [Standard Test Method for Density of Soil in Place by the Sand-Cone Method]. Further, ASTM D-2922 [Standard Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)] may also be used to measure in-place wet density, but must be calibrated with concurrent tests following ASTM D-2167 or ASTM D-1556. (Field-testing of fly ash modified subgrade soil has shown that wet density measurements from the nuclear density gauge are approximately the same as the in place wet density. The in place dry density measured by the nuclear density gauge, however, can differ from actual values due to the error in measured moisture content.)

5.7.2 Moisture

Moisture content shall be conducted in accordance with ASTM D-4959 [Standard Test Method for Determination of Water (Moisture) Content of Soil By Direct Heating Method, ASTM D-2216 [Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures] and ASTM D-4643 [Standard Test Method for Determination of Water (Moisture) Content of Soil by the Microwave Oven Method]. ASTM D-3017 [Standard Test Method for Water Content of Soil and Rock in Place by Nuclear Methods (Shallow Depth)] must be calibrated with concurrent tests following ASTM D-2216, ASTM D-4643 or ASTM D-4959. (The nuclear method moisture contents can differ from about ±2% of the actual moisture content; therefore the nuclear gauge must be calibrated with one of the above methods at the start of each day, when the material being used changes, and when ordered by the Engineer).

5.7.3 Strength/Stability

If shown to be feasible and adequately calibrated during test strip construction, the Engineer may approve use of strength/stability test methods in lieu of field density measurements. Example test devices that provide strength/stability testing include: Dynamic Cone Penetrometer; Clegg Hammer; and GeoGuage™, etc. Based on the specific test equipment used, the Engineer shall specify the acceptance values that must be achieved during or at a set time period after construction.

5.8 Referee Testing

If a difference in procedures for sampling and testing and/or test results exists between the Contractor and the Engineer which they cannot resolve, the Iowa DOT's Central Materials Laboratory in Ames or another mutually agreed upon independent testing laboratory will be asked to provide referee testing. The Engineer and the Contractor will abide by the results of the referee testing. The party found in error will pay service charges incurred for referee testing by an independent laboratory. Table D1 indicates allowable differences for various laboratory and field tests.

Table D1. Allowable differences for laboratory and field test measurements

Property	Reporting of Results	Acceptable Range for QC/QA Test Comparisons
Field Moisture Content	0.1 % (based on dry weight)	± 1.0 %
Field Density Tests for Compaction	1 lb/ft^3 (20 kg/m^3)	5 lb/ft^3 (80 kg/m^3)
Field Strength/Stability Test	—*	—*
Standard Proctor Laboratory "Optimum" Moisture Content (based on maximum dry density or maximum unconfined compressive strength)	0.1 % (based on dry weight)	± 1.5 %
Standard Proctor Laboratory "Maximum" Dry Density (based on dry density)	1 lb/ft^3 (20 kg/m^3)	5 lb/ft^3 (80 kg/m^3)
Standard Proctor Laboratory "Maximum" Unconfined Compressive Strength (based on strength)	5 lb/in^2 (30 kPa)	10 lb/in^2 (60 kPa)

* There is no uniformly accepted reference value for all field strength/stability tests. Bias values should be determined for the specific field test used (i.e. Dynamic Cone Penetrometer index test, GeoGauge™ vibration test, Clegg Hammer impact test, etc.)

5.9 Acceptance

The Engineer will base final acceptance of tests and materials on the results of the Contractor's quality control testing as verified by the Engineer's quality assurance.

6. METHOD OF MEASUREMENT

The fly ash provided for the project shall be measured by the ton (2000 lbs), based on the dry weight shown on the delivery tickets. Manipulation of the fly ash modified subgrade mixtures will be measured by the unit shown on the plans, completed in place.

7. BASIS OF PAYMENT

Work performed and materials furnished as prescribed by this item and measured as provided under Section 6 will be paid for as follows:

Fly ash will be paid for at the unit price per ton (2,000 lbs) dry weight, which shall be full compensation for furnishing all fly ash. The unit price bids shall be full compensation for all correction of secondary subgrade; for loosening, mixing, pulverizing, spreading, drying, application of fly ash, shaping and maintaining; for all curing including all curing water and/or other curing materials; for all manipulations required; for all hauling and freight involved; for all tools, equipment, labor and for all incidentals necessary to complete the work.

8. APPENDIX (NON MANDATORY INFORMATION)

The following laboratory methods are provided as a reference for evaluation and testing of fly ash stabilized materials, to determine "optimum" moisture content.

8.1 Moisture-Density Relationship

The maximum dry density of the fly ash stabilized material shall be determined in accordance with ASTM D-698 [Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5-lb Rammer and 12-in. Drop], or the Iowa State University 2" X 2" Moisture-Density Test Method (see Chu and Davidson 1955). The dry density of the stabilized material shall be determined over a range of moisture contents (10 to 15% by dry weight) based on the compaction characteristics of the soil that is to be stabilized. Two moisture-density relationships are determined as a function of compaction delay time (0.5 hours to establish the baseline reference and 1 hours to simulate field compaction delay). Results typically show that the dry unit weight of stabilized materials decreases as compaction delay time increases and as fly ash content increase. Moisture-density relationship can be plotted as shown in Figure A8.1. Influence of compaction delay time on dry unit weight is shown in Figure A8.2.

8.2 Moisture-Strength Relationship

Samples prepared for determination for "Moisture-Density Relationship" may be cured then tested for strength in compression. Strength samples should be wrapped in plastic wrap and aluminum foil, labeled, and sealed in a plastic bag immediately after being compacted. To simulate 28-day humid cure samples can be cured for 7 days in an oven at 100°F (38°C). Extra sample material should be prepared in order to determine the moisture content at time of compaction.

After the appropriate cure time, the samples should be removed from the wrappings and soaked in water at room temperature before testing. Standard 4-inch (10.2 cm) Proctor sized samples must be soaked four hours before testing. ISU 2-inch x 2-inch (5 cm x 5 cm) samples must soak for one hour before compression testing. Typically, target compressive strength for fly ash modified subgrade mixtures is about 50 lb/in^2 (345 kPa) to 100 lb/in^2 (690 kPa).

8.3 "Optimum" Moisture Content

In lieu of establishing "optimum" moisture content in terms of moisture-density relationships, "optimum" moisture content can be determined as a function of moisture-strength relationships, as shown in Figures A8.3. Typically "optimum" moisture content for strength varies from –5 to +5% of that determined for density.

8.4 Example Fly Ash Addition Rate Design Calculations

Specified Fly Ash Content	11% (by dry weight of subgrade soil) Note: Typically specify 1% greater than lab optimum
Standard Proctor Dry Unit Weight of Subgrade Soil	114 lb/ft^3
Depth of Stabilized Section	12 inches
Weight of Fly Ash	22 tons/truck load
Rate of Fly Ash Distribution	(114 lb/ft3)(11%)(1ft) = 12.54 lb/ft^2
Area to be covered by Truck Load of Ash	(22 tons x 2000 lb)/12.54 lb/ft^2 = 3509 ft^2
Length to Spread for 8ft Wide Section	3509 ft^2/8ft = 439 ft

9. REFERENCES

Chu, T.Y. and Davidson, D.T. (1955). "Some Laboratory Tests for the Evaluation of Stabilized Soils." Iowa Engineering Experiment Station, Project 283-S, Iowa Highway Research Board.

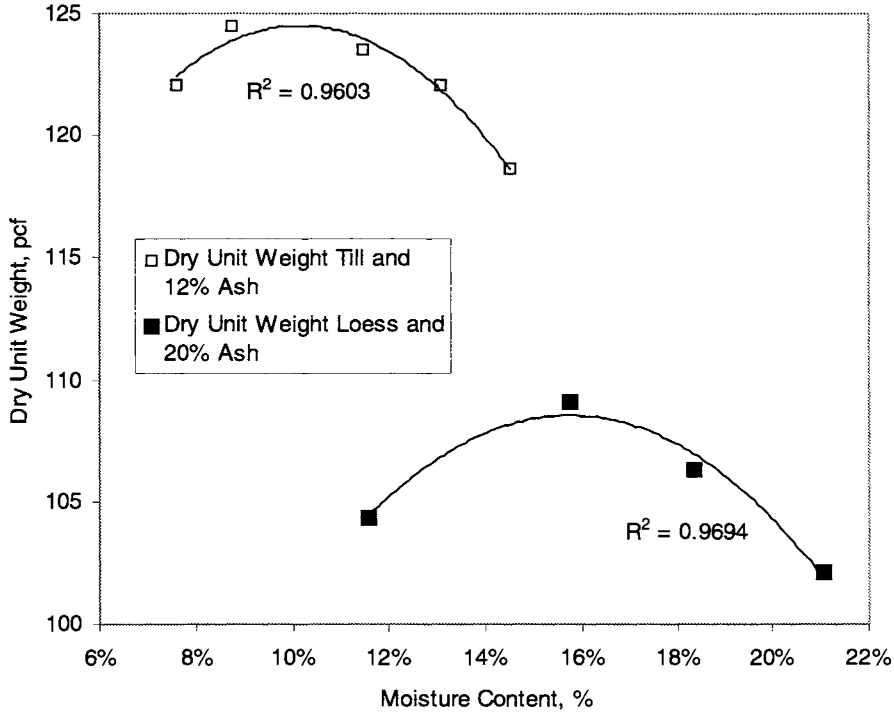


Figure D8.1 Typical Moisture-Density Relationships of Stabilized Soil

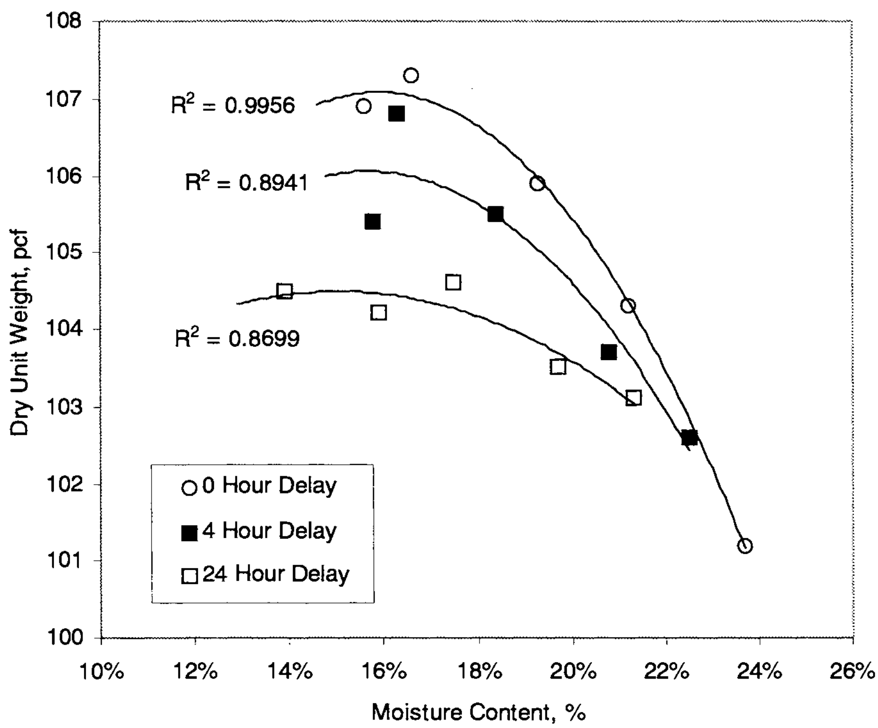


Figure D8.2 Influence of Compaction Delay on Dry Unit Weight

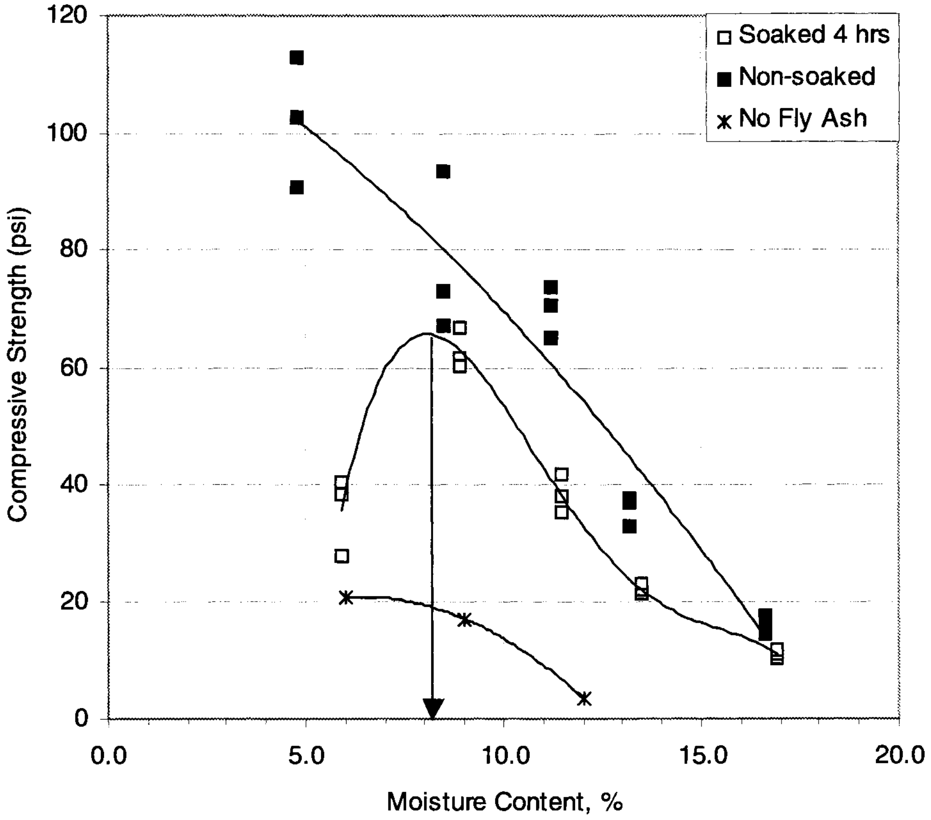


Figure D8.3 Typical Moisture-Strength Relationships for Soaked and Unsoaked Fly Ash Modified Subgrade. (For soaked samples optimum moisture content is approximately 8.2 %)

APPENDIX E:
PROPOSED RECOMMENDATIONS FOR CONSTRUCTING HYDRATED FLY ASH AS SUBGRADE MATERIAL

Suggested Specifications

USE OF RECLAIMED HYDRATED FLY ASH (HFA) AS SUBGRADE MATERIAL

1. DESCRIPTION

This specification shall consist of the laboratory evaluation, field placement, moisture conditioning, and compaction of reclaimed Class "C" hydrated fly ash (HFA), to develop a sufficient subgrade or subbase section. This item shall be constructed as specified herein and in conformity with typical sections, lines and grades as shown on the plans or as established by the Engineer.

2. MATERIALS

2.1 Hydrated Fly Ash (HFA)

Hydrated Fly Ash shall be defined as raw Class "C" fly ash that has been placed in thin lifts, watered, compacted, and mined back out using recycling/reclaiming equipment to produce a well-graded artificial aggregate. The standard Proctor maximum dry density [ASTM D698: Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5-lb Rammer and 12-in. Drop] shall be greater than or equal to 70 lb/ft^3 and the materials shall not be derived from fly ash that has been sluiced to a disposal pond. Based on oven-dry weight, the percentage of particles larger that 4 inches shall not be greater than 5.0 percent, and the percentage of particles smaller than the No. 200 sieve (0.003 in.) shall not be greater than 30.0 percent.

The parent fly ash shall be of the Class "C" type as set forth by ASTM C 618 [Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for use as a Mineral Admixture in Concrete]. The source of the HFA shall be identified and approved in advance of construction operations in order to allow for all necessary laboratory work to be completed and reviewed. HFA shall meet the following chemical requirements set forth by ASTM C618 when expressed on an LOI free (loss-on-ignition) basis (LOI for HFA materials is typically 15-40 percent due to chemically bound and free water):

- Silicon dioxide (SiO_2) + aluminum oxide (Al_2O_3) + iron oxide (Fe_2O_3) $\geq$ 50.0 and $\leq$ 70.0 percent
- Sulfur trioxide (SO_3) $\leq$ 5.0 percent

2.2 Mixing Water

Water used to bring the HFA to the required project specifications shall be clean, free of sewage, sulfates, organic matter, oil, acid, and alkali. Potable water may be used without testing. Non-potable sources of water shall be tested in accordance with AASHTO T-26 [Method of Test for Quality of Water to be Used in Concrete] and approved by the Engineer.

3. CONSTRUCTION PRACTICES

3.1 General

It is the primary purpose of this specification to construct a completed section of HFA which contains uniform moisture content with no loose or segregated areas; has a uniform density; and is well bound for its full depth. It shall be the responsibility of the Contractor to regulate the sequence of his work; to process a sufficient quantity of material to provide a completed section as shown on the plans; to use the proper amounts of HFA; to maintain the work; and, to rework the lifts as necessary to meet the above requirements. HFA and ambient air temperature shall be at or above 40°F (2°C) at the time HFA is placed.

3.2 Preparation of Subgrade

Before the HFA is placed, the area shall be cut and shaped in conformance with the lines and grades shown on the plans. The subgrade shall be firm and able to support the construction traffic associated with hauling and placing the HFA. Soft subgrade areas shall be corrected and made stable by overexcavating, adding HFA, and compacting until the area is of uniform stability. Dry fly ash that meets the chemical requirements of ASTM C618, Table 1, for Class C fly ash may also be used to stabilize soft subgrade (see Suggested Specifications for Treatment of Subgrade Materials with Class "C" Fly Ash).

3.3 Moisture Control

Moisture content of the HFA will be determined as the material is being reclaimed at the production site. If moisture conditioning is required, moisture shall be added during reclamation through the use of a pulvamixer equipped with a spray bar in the mixing drum or via other approved methods by the engineer either at the reclamation or construction site. The system shall be capable of being regulated to the degree necessary as to maintain moisture contents within the recommended ranges.

The Engineer shall establish required moisture contents based on laboratory tests conducted with the site specific HFA. Final moisture content of the HFA shall not exceed the moisture limits set forth by the Engineer at the time of compaction. Once compacted, moisture conditioning or aeration will not be permitted. However, the CFA surface should be maintained in a damp condition until surfacing is applied.

3.4 Transportation and Placement

The HFA fill may be hauled to the construction site in end dump trucks, side dump trucks, or belly dump trucks. The material shall be hauled in such a manner as to reduce the loss of moisture during transportation and minimize dusting.

After the HFA material has been unloaded on the construction site, it shall be spread to the required depth with a grader, maintainer, bulldozer, box scraper, or any other means approved by the Engineer.

3.5 Compaction

Compaction of the HFA shall begin immediately after placement to prevent loss of moisture. Compaction of the HFA shall start at the bottom of the layer and continue until the entire depth of the mixture is uniformly compacted to the specified density.

HFA shall be compacted initially using a vibratory padfoot roller. A non-vibratory sheepsfoot may be used if shown to meet required relative compaction level. The fill shall be compacted to ≥ 90 percent of maximum dry density as determined by ASTM D-698, [Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5-lb. Rammer and 12-in. Drop]. The fill shall not be placed more than 12 inches (30 cm) deep for any lift. Final compaction shall be achieved using a steel or pneumatic roller.

Compaction of the HFA shall begin as determined by the laboratory results based on compaction delay time. Compaction delay should not exceed 4 hours. The Engineer shall address any questions about a delay time that is not perceived to be in the best interest of the project time line.

Once the HFA is compacted, the Engineer shall perform the necessary field tests to ensure proper compaction (see Section 3.6). If HFA material is found to fail the density standard, the Engineer may require reworking of the lift. Once compacted the HFA shall support construction traffic. Any areas found to be soft and unstable shall be cored out and replaced with new HFA. If during construction the HFA fill is subjected to rain, areas of standing water shall be bladed off and reworked if deemed necessary by the Engineer.

When the Engineer has passed a previous lift of HFA fill, the next lift of HFA can immediately begin being hauled and placed on the site. Any completed lift that begins to surface dry must be watered in order to keep dust down and to ensure there is enough moisture in the HFA to meet required project limits.

3.6 Quality Control Field Testing

Field-testing to measure in-place density shall be determined in accordance with ASTM

D-2167 [Standard Test Method for Density and Unit Weight of Soil in Place by the Rubber Balloon Method] or ASTM D-1556 [Standard Test Method for Density of Soil in Place by the Sand-Cone Method]. Further, ASTM D-2922 [Standard Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)] may also be used to measure in-place wet density, but must be calibrated with concurrent tests following ASTM D-2167 or ASTM D-1556. (Field-testing of HFA has shown that wet density measurements from the nuclear density gage are approximately 5 to 10% higher than actual density and therefore should be calibrated with rubber balloon or sand cone density tests).

Moisture content shall be conducted in accordance with ASTM D-4959 [Standard Test Method for Determination of Water (Moisture) Content of Soil By Direct Heating Method, ASTM D-2216 [Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures] and ASTM D-4643 [Standard Test Method for Determination of Water (Moisture) Content of Soil by the Microwave Oven Method]. Note that ASTM D-3017 [Standard Test Method for Water Content of Soil and Rock in Place by Nuclear Methods (Shallow Depth)] is not an acceptable method for determination of in-place moisture content.

3.7 Finishing and Curing

Once the last lift is completed it must be brought to the required lines and grades in accordance with the typical sections.

After the HFA has been finished as specified herein, the surface shall be protected against rapid drying by either of the following curing methods until the pavement section is placed:

(d) Maintain in a thorough and continuously moist condition by sprinkling with water
(e) Apply and compact a minimum 2-inch layer of well-graded crushed limestone on the completed course
(f) Build the section 2-inches high and trim prior to paving

A minimum of 3 inches of crushed limestone cover/wearing surface shall be immediately placed over the last lift for HFA applications that will involve direct interaction with vehicle traffic, or as deemed necessary by the Engineer. The crushed limestone may be removed to bring the project to the required lines and grades before paving begins.

4. MEASUREMENT

The HFA fill shall be measured by the ton (2000 lbs), based on the as received weight.

5. PAYMENT

Work performed and materials furnished as prescribed by this item and measured as provided under "Measurement" will be paid for as follows:

HFA material will be paid for at the unit price per ton (2,000 pounds) as received, which shall be full compensation for furnishing all HFA. The unit price bids shall be full compensation for all correction of secondary subgrade; for loosening, mixing, pulverizing, spreading, drying, application of HFA, shaping and maintaining; for all curing including all curing water and/or other curing materials; for all manipulations required; for all hauling and freight involved; for all tools, equipment, labor and for all incidentals necessary to complete the work.

6. APPENDIX

When the fly ash source of the HFA is not identified and approved in advance of construction operations, the HFA shall meet the chemical requirements set forth by ASTM C618 when expressed on a LOI-free basis.

Table E.6 shows a typical bulk analytical chemical analysis of an HFA sample via X-ray fluorescence. The assay is expressed on an as-received basis. This is done because the drying processes (105 to 110°C) destroy the hydrates present in the sample. No attempt is made to determine the ratio of chemically bound water to free moisture. The assay expressed on a LOI-free basis is given because it is similar to the bulk parent ash composition produced at the power station.

Table E.6. Analytical chemical composition of a typical HFA expressed on an LOI-free basis

Constituent	As Received (mass %)	LOI-free basis (mass %)
Na_2	1.18	1.76
MgO	4.12	6.16
Al_2O_3	11.78	17.62
SiO_2	22.26	33.29
P_2O_5	0.63	0.94
SO_3	1.65	2.47
K_2O	0.23	0.34
CaO	18.78	28.08
TiO_2	0.91	1.36
Mn_3O_4	0.04	0.06
Fe_2O_3	4.12	6.16
SrO	0.31	0.46
BaO	0.54	0.81
LOI	32.97	0
Total	99.5	99.5

7. APPENDIX (NONMANDATORY INFORMATION)

The following laboratory methods are provided as a reference for evaluation and testing of HFA select fill materials, to determine "optimum" moisture content.

7.1 Moisture-Density Relationship

The maximum dry density of HFA shall be found by following ASTM D-698 [Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5-lb. Rammer and 12-in. Drop]. Prior to compaction all material shall be passed through a 0.75-inch sieve. All material not passing the sieve shall be crushed to pass the 0.75-inch sieve and remixed with the sample. (Crushing of the material to pass the 0.75-inch sieve simulates crushing of the material while being compacted in the field.) The wet and dry density of the HFA material shall be found at moisture contents ranging from approximately 10% to 40%. The interval between the moisture contents in the specified range shall not exceed 5% with no less than 5 moisture contents evaluated. Results can be plotted as shown in Figure A7.1.

7.2 Moisture-Strength Relationship

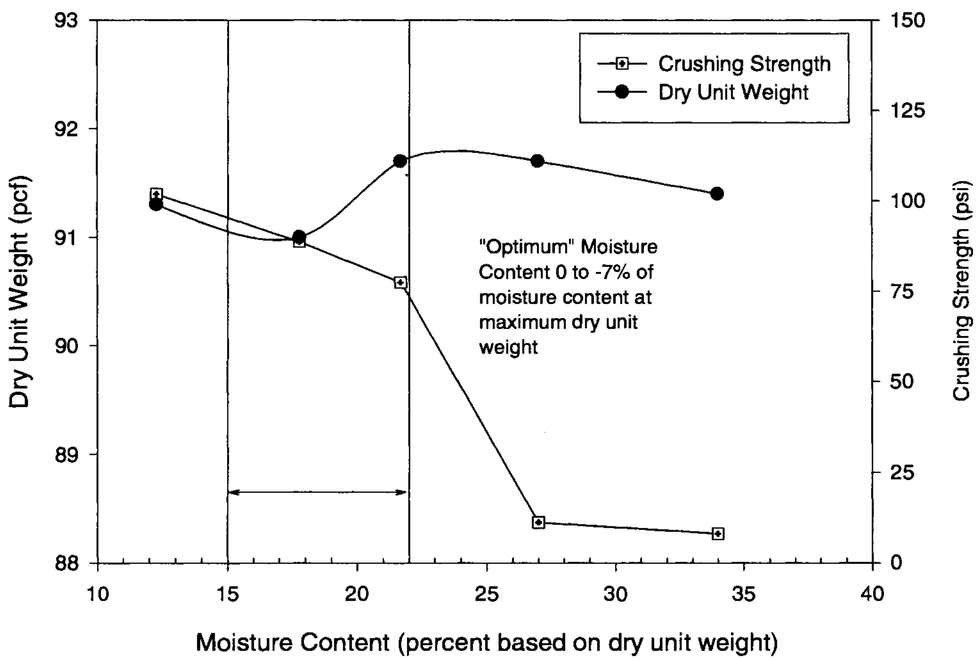
As a measure of strength, samples prepared for "Moisture-Density Relationship" may be cured (typically 3, 7, 28, 56, or 90 days) then tested for strength in compression. Samples for strength testing shall be compacted in a Proctor mold of the split type. Once compacted the sample should be removed from the mold, weighed, and measured for height. To cure, the specimen is wrapped in plastic wrap and aluminum foil, labeled, sealed in an air free plastic zip-lock bag and then stored in a humidity room. Extra sample material should be prepared for each group of samples in order to evaluate the moisture content at time of compaction.

Strength shall be tested compared to moisture content and to curing temperature. For strength versus moisture content, three samples should be prepared per moisture content per cure time. The moisture contents should be in the range of 10% to 40%, with an interval no greater than 5%. These samples should be cured in a 100% moist environment at 72°F +/- 2°F.

Typical HFA materials exhibit a maximum crushing strength of up to 200 psi at 28 days. Before samples are tested for strength it is required that the ends of each sample be capped with high-strength, non-shrink sulfur capping compound. This is to ensure an even stress distribution during testing. The samples that were prepared alike should have representative samples of each one taken after compression testing and combined to determine the moisture content after the specified cure time.

7.3 "Optimum" Moisture Content

"Optimum" moisture content to satisfy strength and density requirements may typically be defined as from about -8 to +4 percent of the moisture content at the maximum dry unit weight. Sample laboratory test results are shown in Figure E7.1.



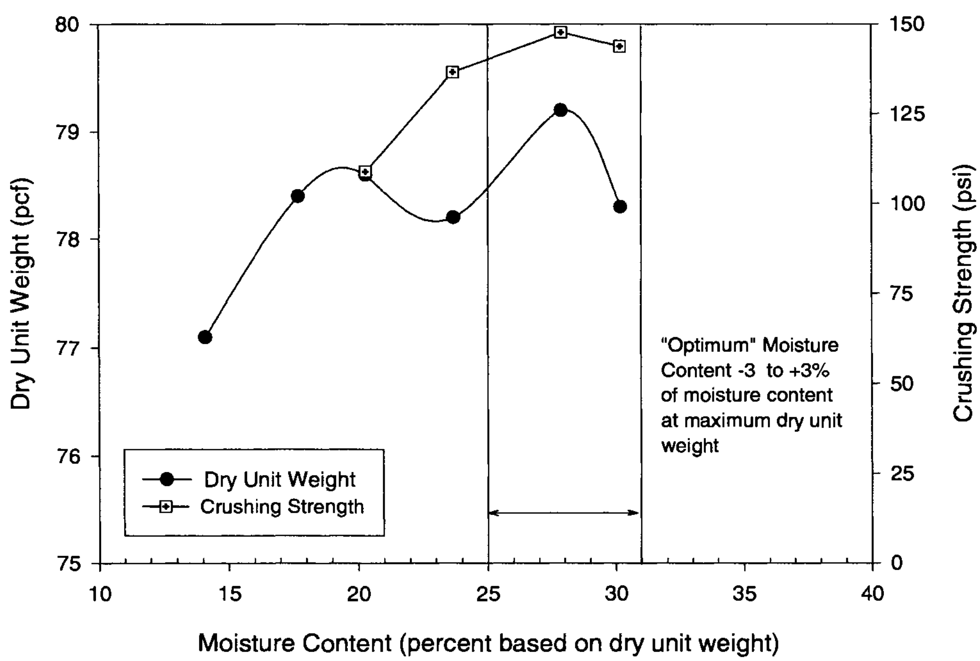


Figure E7.1 Typical 28-Day Moisture-Density and Moisture-Strength Relationships for HFA Materials

APPENDIX F:
PROPOSED RECOMMENDATIONS FOR CONSTRUCTING CONDITIONED FLY ASH AS SUBGRADE MATERIAL

Suggested Specifications

USE OF RECLAIMED CONDITIOINED FLY ASH (CFA) AS SUBGRADE MATERIAL

1. DESCRIPTION

This specification shall consist of the laboratory evaluation, field placement, moisture conditioning, and compaction of reclaimed Class "C" conditioned fly ash (CFA), to develop a sufficient subgrade or subbase section. This item shall be constructed as specified herein and in conformity with typical sections, lines and grades as shown on the plans or as established by the Engineer.

2. MATERIALS

2.1Conditioned Fly Ash (CFA)

Conditioned Fly Ash shall be defined as raw Class "C" fly ash that has been wetted in a pug mill mixer and stockpiled to be mined back out using a reclaimer, front end loader, or backhoe to produce a well-graded artificial aggregate. The standard Proctor maximum dry density [ASTM D698: Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5-lb Rammer and 12-in. Drop] shall be greater than or equal to 70 lb/ft^3 and the materials shall not be derived from fly ash that has been sluiced to a disposal pond. Based on oven-dry weight, the percentage of particles larger that 6 inches shall not be greater than 10.0 percent, and the percentage of particles smaller than the No. 200 sieve (0.003 in.) shall not be greater than 50.0 percent.

The source of the CFA shall be identified and approved in advance of construction operations in order to allow for all necessary laboratory work to be completed and reviewed. CFA shall meet the following chemical requirements set forth by ASTM C618 [Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for use as a Mineral Admixture in Concrete] when expressed on an LOI free (loss-on-ignition) basis (LOI for CFA materials is typically 15-40 percent due to chemically bound and free water):

- Silicon dioxide (SiO_2) + aluminum oxide (Al_2O_3) + iron oxide (Fe_2O_3) = 50.0 to 70.0 percent
- Sulfur trioxide (SO_3) $\leq$ 5.0 percent

2.2 Mixing Water

Water used to bring the CFA to the required project specifications shall be clean, free of sewage, sulfates, organic matter, oil, acid, and alkali. Potable water may be used without testing. Non-potable sources of water shall be tested in accordance with AASHTO T-26 [Method of Test for Quality of Water to be Used in Concrete] and approved by the Engineer.

3. CONSTRUCTION PRACTICES

3.1 General

It is the primary purpose of this specification to construct a completed section of CFA which contains uniform moisture content with no loose or segregated areas; has a uniform density; and is well bound for its full depth. It shall be the responsibility of the Contractor to regulate the sequence of his work; to process a sufficient quantity of material to provide a completed section as shown on the plans; to use the proper amounts of CFA; to maintain the work; and, to rework the lifts as necessary to meet the above requirements. CFA and ambient air temperature shall be at or above 40°F (2°C) at the time CFA is placed.

3.2 Preparation of Subgrade

Before the CFA is placed, the area shall be cut and shaped in conformance with the lines and grades shown on the plans. The subgrade shall be firm and able to support the construction traffic associated with hauling and placing the CFA. Soft subgrade areas shall be corrected and made stable by overexcavating, adding CFA, and compacting until the area is of uniform stability. Dry fly ash that meets the chemical requirements of ASTM C618, Table 1, for Class C fly ash may also be used to stabilize soft subgrade (see Suggested Specifications for Treatment of Subgrade Materials with Class "C" Fly Ash).

3.3 Moisture Control

Moisture content of the CFA will be determined as the material is being reclaimed at the production site. If moisture conditioning is required, moisture shall be added during reclamation at the source or prior to compaction on site using water truck or other approved methods by the engineer. The system shall be capable of being regulated to the degree necessary as to maintain moisture contents within the recommended ranges.

The Engineer shall establish required moisture contents based on laboratory tests conducted with the site specific CFA. Final moisture content of the CFA shall not exceed the moisture limits set forth by the Engineer at the time of compaction. Once compacted, moisture conditioning or aeration will not be permitted. However, the CFA surface should be maintained in a damp condition until surfacing is applied.

3.4 Transportation and Placement

The CFA fill may be hauled to the construction site in end dump trucks, side dump trucks, or belly dump trucks. The material shall be hauled in such a manner as to reduce the loss of moisture during transportation and minimizing dust.

After the CFA material has been unloaded on the construction site, it shall be spread to the required depth with a maintainer, bulldozer, box scraper, or any other means approved by the Engineer.

3.5 Compaction

Compaction of the CFA shall begin immediately after placement to prevent loss of moisture. Compaction of the CFA shall start at the bottom of the layer and continue until the entire depth of the mixture is uniformly compacted to the specified density.

CFA shall be compacted initially using a vibratory padfoot roller. A non-vibratory sheepsfoot may be used if shown to meet required relative compaction level. The fill shall be compacted to ≥ 90 percent of maximum dry density as determined by ASTM D-698, [Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5-lb. Rammer and 12-in. Drop]. The fill shall not be placed more than 12 inches (30 cm) deep for any lift. Final compaction shall be achieved using a steel or pneumatic roller.

Compaction of the CFA shall begin as determined by the laboratory results based on compaction delay time. Compaction delay should not exceed 4 hours. The Engineer shall address any questions about a delay time that is not perceived to be in the best interest of the project time line.

Once the CFA is compacted, the Engineer shall perform the necessary field tests to ensure proper compaction (see section 3.6). If CFA material is found to fail the density standard, the Engineer may require reworking of the lift. Once compacted the CFA shall support construction traffic. Any areas found to be soft and unstable shall be cored out and replaced with new CFA. If during construction the CFA fill is subjected to rain, areas of standing water shall be bladed off and reworked if deemed necessary by the Engineer.

When the Engineer has passed a previous lift of CFA fill, the next lift of CFA can immediately begin being hauled and placed on the site. Any completed lift that begins to surface dry must be watered in order to keep dust down and to ensure there is enough moisture in the CFA to meet required project limits.

3.6 Quality Control Field Testing

Field-testing to measure in-place density shall be determined in accordance with ASTM D-2167 [Standard Test Method for Density and Unit Weight of Soil in Place by the Rubber Balloon Method] or ASTM D-1556 [Standard Test Method for Density of Soil in Place by the Sand-Cone Method]. Further, ASTM D-2922 [Standard Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)] may also be used to measure in-place wet density, but must be calibrated with concurrent tests following ASTM D-2167 or ASTM D-1556. (Field-testing of CFA has shown that wet density measurements from the nuclear density gage are approximately 5 to 10% higher than actual density and therefore should be calibrated with rubber balloon or sand cone density tests).

Moisture content shall be conducted in accordance with ASTM D-4959 [Standard Test Method for Determination of Water (Moisture) Content of Soil By Direct Heating Method, ASTM D-2216 [Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures] and ASTM D-4643 [Standard Test Method for Determination of Water (Moisture) Content of Soil by the Microwave Oven Method]. Note that ASTM D-3017 [Standard Test Method for Water Content of Soil and Rock in Place by Nuclear Methods (Shallow Depth)] is not an acceptable method for determination of in-place moisture content.

3.7 Finishing and Curing

Once the last lift is completed it must be brought to the required lines and grades in accordance with the typical sections.

After the CFA has been finished as specified herein, the surface shall be protected against rapid drying by either of the following curing methods until the pavement section is placed:

(g) Maintain in a thorough and continuously moist condition by sprinkling with water
(h) Apply and compact a minimum 2-inch layer of well-graded crushed limestone on the completed course
(i) Build the section 2-inches high and trim prior to paving

A minimum of 3 inches of crushed limestone cover/wearing surface shall be immediately placed over the last lift for CFA applications that will involve direct interaction with vehicle traffic, or as deemed necessary by the Engineer. The crushed limestone may be removed to bring the project to the required lines and grades before paving begins.

4. MEASUREMENT

The CFA fill shall be measured by the ton (2000 lbs), based on the as received weight.

5. PAYMENT

Work performed and materials furnished as prescribed by this item and measured as provided under "Measurement" will be paid for as follows:

CFA material will be paid for at the unit price per ton (2,000 pounds) as received weight, which shall be full compensation for furnishing all CFA. The unit price bids shall be full compensation for all correction of secondary subgrade; for loosening, mixing, pulverizing, spreading, drying, application of CFA, shaping and maintaining; for all curing including all curing water and/or other curing materials; for all manipulations required; for all hauling and freight involved; for all tools, equipment, labor and for all incidentals necessary to complete the work.

6. APPENDIX

When the fly ash source of the CFA is not identified and approved in advance of construction operations, the CFA shall meet the chemical requirements set forth by ASTM C618 when expressed on a LOI-free basis.

Table F.6 shows a typical bulk analytical chemical analysis of an CFA sample via X-ray fluorescence. The assay is expressed on an as-received basis. This is done because the drying processes (105 to 110°C) destroy the hydrates present in the sample. No attempt is made to determine the ratio of chemically bound water to free moisture. The assay expressed on a LOI-free basis is given because it is similar to the bulk parent ash composition produced at the power station.

Table F.6. Analytical chemical composition of a typical CFA expressed on an LOI-free basis

Constituent	As Received (mass %)	LOI-free basis (mass %)
Na_2	1.85	2.55
MgO	4.04	5.58
Al_2O_3	12.8	17.7
SiO_2	23.6	32.6
P_2O_5	0.89	1.23
SO_3	1.83	2.53
K_2O	0.39	0.54
CaO	19.8	27.4
TiO_2	1.07	1.47
Fe_2O_3	5.7	7.8
SrO	0.26	0.35
BaO	0.58	0.80
LOI	27.5	0

7. APPENDIX (NONMANDATORY INFORMATION)

The following laboratory methods are provided as a reference for evaluation and testing of CFA select fill materials, to determine "optimum" moisture content.

7.4 Moisture-Density Relationship

The maximum dry density of CFA shall be found by following ASTM D-698 [Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5-lb. Rammer and 12-in. Drop]. Prior to compaction all material shall be passed through a 0.75-inch sieve. All material not passing the sieve shall be crushed to pass the 0.75-inch sieve and remixed with the sample. (Crushing of the material to pass the 0.75-inch sieve simulates crushing of the material while being compacted in the field.) The wet and dry density of the CFA material shall be found at moisture contents ranging from approximately 10% to 40%. The interval between the moisture contents in the specified range shall not exceed 5% with no less than 5 moisture contents evaluated. Results can be plotted as shown in Figure A7.1.

7.5 Moisture-Strength Relationship

As a measure of strength samples prepared for "Moisture-Density Relationship" may be cured (typically 3, 7, 28, 56, or 90 days) then tested for strength in compression. Samples for strength testing shall be compacted in a Proctor mold of the split type. Once compacted the sample should be removed from the mold, weighed, and measured for height. To cure, the specimen is wrapped in plastic wrap and aluminum foil, labeled, and sealed in an air free plastic zip-lock bag. Extra sample material should be prepared for each group of samples in order to evaluate the moisture content at time of compaction.

Strength shall be tested compared to moisture content and compared to curing temperature. For strength versus moisture content, three samples should be prepared per moisture content per cure time. The moisture contents should be in the range of 10% to 40%, with an interval no greater than 5%. These samples should be cured in a 100% moist environment at 72°F +/- 2°F.

Typical CFA materials exhibit a maximum crushing strength of up to 200 psi at 28 days. Before samples are tested for strength it is required that the ends of each sample be capped with high-strength, non-shrink sulfur capping compound. This is to ensure an even stress distribution during testing. The samples that were prepared alike should have representative samples of each one taken after compression testing and combined to determine the moisture content after the specified cure time.

7.6 "Optimum" Moisture Content

"Optimum" moisture content to satisfy strength and density requirements may typically be defined as from about -8 to +4 percent of the moisture content at the maximum dry unit weight. Sample laboratory test results are shown in Figure F7.1.

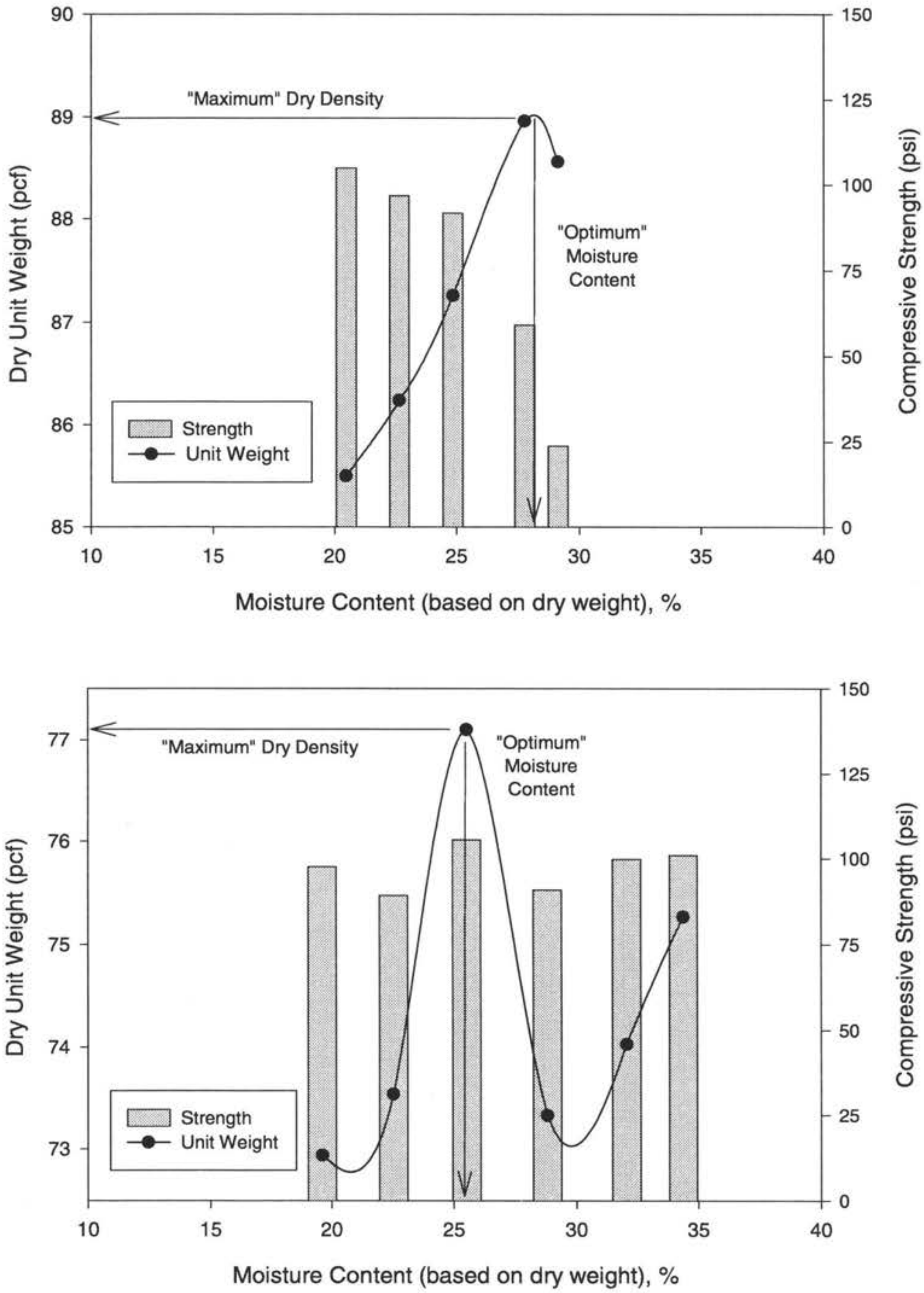


Figure F7.1 Typical Moisture-Density and Moisture-Strength Relationships for CFA Material

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