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P A V E M E N T T E C H N O L O G Y

Fly Ash Soil Stabilization for  
Non-Uniform Subgrade Soils, Volume I:  
Engineering Properties and  
Construction Guidelines

Final Report  
April 2005

IOWA STATE UNIVERSITY

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the Iowa Highway Research Board (Project TR-461),  
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# **FLY ASH SOIL STABILIZATION FOR NON- UNIFORM SUBGRADE SOILS, VOLUME I**

## **ENGINEERING PROPERTIES AND CONSTRUCTION GUIDELINES**

**Final Report**

**April 2005**

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## EXECUTIVE SUMMARY

Soil treated with self-cementing fly ash is increasingly being used in Iowa to stabilize fine-grained pavement subgrades, but without a complete understanding of the short- and long-term behavior. To develop a broader understanding of the engineering properties of fly ash, mixtures of five different soil types, ranging from ML to CH, and several different fly ash sources (including hydrated and conditioned fly ashes) were evaluated. Results show that soil compaction characteristics, compressive strength, wet/dry durability, freeze/thaw durability, hydration characteristics, rate of strength gain, and plasticity characteristics are all affected by the addition of fly ash. Specimens for strength and durability testing were produced using the Iowa State University 2 x 2 apparatus, standard Proctor, and California bearing ratio (CBR) test methods. To simulate harsh Iowa field conditions, several specimens were subjected to freeze/thaw and wet/dry curing environments prior to testing. Some specimens were cured for up to 2.5 years. The morphology of soil fly ash mixtures and the soil clay mineralogy were also studied using x-ray diffraction and scanning electron microscopy techniques.

Some of the key findings from this study are as follows:

- Iowa self-cementing fly ashes at addition rates of 10% to 20% (by dry weight of soil) are effective at stabilizing fine-grained Iowa soils for earthwork and paving operations.
- With regard to the influence of self-cementing fly ash on density and compaction, test results show that fly ash increases the compacted dry density and reduces the optimum moisture content.
- Strength gain in soil-fly ash mixtures is dependent on cure time and temperature, compaction energy, and compaction delay. To develop a mix design, soaking laboratory samples before compressive strength testing is recommended for evaluating samples in a saturated condition.
- Sulfur contents can cause formation of expansive minerals in soil-fly ash mixtures, which severely reduces the long-term strength and durability. Tests should be performed to determine the sulfur contents of the fly ash, soil, and mix water.
- Fly ash increases the CBR of fine-grained soils, and in the case of 20% fly ash addition, the CBR can be increased up to values simulating compacted gravel (~75%).
- Fly ash effectively dries wet soils and provides an initial rapid strength gain, which is useful during construction in wet, unstable ground conditions.
- Fly ash decreases swell potential of expansive soils.
- Soil-fly ash mixtures cured below freezing temperatures and then soaked in water are highly susceptible to slaking and strength loss.
- Soil stabilized with fly ash exhibits increased freeze-thaw durability.
- Strength of soil can be increased by adding hydrated fly ash (HFA) and conditioned fly ash (CFA), but at higher quantities and not as effectively as self-cementing fly ash.

Based on the results of this study, three proposed specifications were developed for using self-cementing fly ash, HFA, and CFA. The specifications describe laboratory evaluation, field placement, moisture conditioning, compaction, quality control testing procedures, and basis of payment.

## 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 (ACAA 2003). Although utilization of fly ash is continuing to grow, less than 32% of coal combustion by-products are recycled (ACAA 2003). The remainder of fly ash and other coal combustion by-products are wasted in sludge ponds or landfills, taking up valuable space. The most widely used application of fly ash is as a partial replacement for cement in Portland cement concrete (PCC). States such as Iowa have allowed up to 15% replacement of cement with fly ash, which can improve various concrete mix properties and strength gain (Kosmatka and Panarese 1994).

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 in unstable soils. The volume of fly ash currently used for soil stabilization is less than that used for cement replacement in concrete. As knowledge is gained about the mechanisms of stabilization involving self-cementing fly ash, the volume used in soil stabilization applications will increase.

The primary benefits of using self-cementing fly ash for soil stabilization are (1) environmental incentives, because material used does not have to be wasted; (2) cost savings, because fly ash is typically cheaper than cement and lime; and (3) availability, because fly ash sources are distributed geographically across the state. When fly ash production exceeds demand in the construction industry, the material is typically hydrated or conditioned with water and stored on site. The hydrated and conditioned materials can later be reclaimed and used as soil stabilizers or as select fill under pavement structures.

As subgrade material, Iowa soils generally rate from fair to poor. The majority of 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 about soil stabilization using self-cementing fly ashes, hydrated fly ashes (HFA), and conditioned fly ashes (CFA) to improve soil properties and strength, as well as uniformity under pavement sections. Twelve power plants in the state of Iowa produce self-cementing fly ash, and over half of these power plants have sources of hydrated or conditioned fly ash (Table 1).

The primary objective of this research study was to evaluate the effects of self-cementing fly ash addition on the engineering properties of several Iowa soils. In the material evaluation phase of this project, tests included strength (unconfined compressive strength and California bearing ratio [CBR]), the influence of curing temperature on strength gain, and the 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 were also evaluated. Research results are also provided for the 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 using self-cementing fly ash to stabilize soils, using

hydrated or conditioned fly ash to stabilize soils, and using hydrated or conditioned fly ash as select fill under pavement structures.

**Table 1. Iowa fly ash sources and production (Bergeson and Mahrt 1999)**

<b>Fly Ash Source</b>	<b>Fly Ash Type</b>	<b>Raw Ash Production (tons/year)</b>	<b>Reclaimed Ash Status</b>
Ames	C	10,000	Potential
Clinton	F	15,000	NA
Council Bluffs	C	100,000	Active
Cedar Rapids	C	30,000	Active
Bettendorf	C	25,000	Active
Burlington	C	25,000	Active
Ottumwa	C	100,000	Active
Lansing	C	40,000	Potential
Louisa	C	120,000	Active
Marshalltown	C	10,000	Active
Sioux City (units 2, 3, and 4)	C	150,000	Active

## LITERATURE REVIEW

A literature review examined the construction operations for using self-cementing fly ash as a soil stabilizer. Procedures for mixing, moisture control, compaction, and curing were investigated. Several methods of quality control testing, including field density and moisture, stability, and in-service performance-based tests, were also reviewed. A review of fly ash chemical properties and reaction mechanisms is provided and suggestions for fly ash construction specifications are summarized. Lastly, case histories describing various uses of fly ash as a stabilizer or as a fill material are summarized.

### 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 soil 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 with on-site mixing. A disadvantage in using self-cementing fly ash is that it can exhibit a relatively rapid set (as little as ten minutes), which can decrease strength with delayed compaction (ACAA 1991). Most off-site mixing operations have been used for combining 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 the 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 consistent interval. Batch-type operations are similar to batch plants used for Portland cement concrete. Enough material for a single truck is mixed and then loaded into the truck. Terrel *et al.* (1979a) have suggested that for large scale projects, a continuous mixing plant is preferred because material can be generated at a higher rate than that of a batch-type mixing operation.

As use of self-cementing fly ash expands for stabilization practices, the American Coal Ash Association (ACAA) reports that the preferred mixing method is on-site mixing (ACAA 1991). This approach does not require the establishment of a mixing plant and takes better 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 directly onto the subgrade. The mixing operation can then be completed using a pulvamixer or disc (Vandenbossche and Johnson 1994). An example of a 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 one-inch sieve and a minimum of 50% passing the #4 sieve (ACAA 1991). In some cases, as when fly ash is used as a drying agent, discing with a construction disc has been effective in fine grained soils. The 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 (Ferguson and Levenson 1999).



**Figure 1. Pulv mixer used for on-site mixing**

### *Application of Water*

The process of adding and monitoring the mixing water during the stabilization operation 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% of the optimum moisture content, based on the moisture-density relationship of the stabilized mixture, to obtain proper density at time of compaction (ACAA 1991). As mentioned before, however, this process is giving way to the on-site mixing of self-cementing fly ash, subgrade soils, and water.

Before adding fly ash, water can be added to the subgrade soils (Vandenbossche and Johnson 1994). However, a disadvantage to this approach is that the subgrade may become unstable, complicating the rest of the construction process (Ferguson and Levenson 1999). 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 the hydration of the fly ash prior to final compaction (Ferguson and Levenson 1999). Ferguson and Levenson (1999) further report that the most effective method for controlling mixing water has been to add the water directly into the mixing drum of the pulv mixer. This procedure produces the most uniform mixing and the least amount of delay in the construction process (Ferguson and Levenson 1999).

Moisture control also includes the properties of the water to be used on the project. The water should 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) (Kosmatka and Panarese 1994). 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, dependant on soil type. Due to its self-cementing properties, fly ash can be an effective stabilizer for granular and fine grained materials. Terrel *et al.* (1979a) classify 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 (ACAA 1991). Terrel *et al.* (1979a) suggest initial compaction with a sheepsfoot or padfoot roller and using a pneumatic roller to finish rolling 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 (Ferguson and Levenson 1999).

Compaction delay time should also be considered 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 four hours has been specified (Terrel *et al.* 1979a, Vandenbossche and Johnson 1994). 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, compaction should 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 (Terrel *et al.* 1979a, Ferguson and Levenson 1999). In most cases, the initial compaction begins with a padfoot-type roller directly behind the pulv mixer and can be finished within 15 minutes after final mixing (Ferguson and Levenson 1999).

### *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 (Terrel *et al.* 1979a, Vandenbossche and Johnson 1994). Availability of moisture, temperature during curing, and length of cure time all affect the strength gain of fly ash-stabilized soils (ACAA 1991; Vandenbossche and Johnson 1994; Ferguson and Levenson 1999). Typically, mixtures are cured by sprinkling with water or by coating with a thin layer of emulsion or cutback asphalt. The Joint Departments of the Army and Air Force and the Federal Highway Administration (FHWA) recommend that the sealer be applied within one day of completing the section and that multiple coats may be required (Terrel *et al.* 1979a; Joint Departments of the Army and Air Force 1994; Vandenbossche and Johnson 1994). Completed sections can also be cured with water for a short time and then sealed with thin coats of asphalt products (Klassen and Jones 1985).

According to Terrel *et al.* (1979a) and Johnson and Vandenbossche (1994), before heavy traffic or pavement sections are placed, the completed sections should be cured for three to seven days. From observations by the Joint Departments of the Army and Air Force (1994), paving can begin within a day or two after completing 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 (Klassen and Jones 1985). Terrel *et al.* (1979a) also recommend that a protective layer of crushed

stone be applied to areas where traffic will be present before paving is completed; however, the protective layer can delay the release of the volatiles in an asphalt seal coat. Reportedly, the volatiles react negatively with the stabilized base and inhibit strength gain during curing.

According to the ACAA (1991), Ferguson and Levenson (1999), and Vandebossche and Johnson (1994), fly ash stabilization operations should not proceed when the air temperature is below 40° F. As with most chemical reactions, the 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 (Ferguson and Levenson 1999). An example from the Portland Cement Association (Kosmatka and Panarese 1994) 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 Vandebossche (1994) further recommend that frozen soils should not be used in stabilization processes and that completed sections should be allowed to cure at temperatures above 40° F for at least seven days before freezing temperatures occur. Another concern arises from the fact that highly plastic soils 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 (Ferguson and Levenson 1999).

## **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 have been 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 primary 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). In practice, tests used to measure in situ density are the sand cone, rubber balloon, nuclear gauge, and drive cylinder (ACAA 1991, Seals \_\_\_\_, Terrel et al. 1979a). 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 of these tests are that they take a long time 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 calculated dry density is variable at best when compared to measurements determined by the rubber balloon method (Mahrt 2000).

### Moisture Content Determination

One of the most difficult parameters to measure accurately in the field, yet one of the most important, is the moisture content of the stabilized soil. Without proper moisture, typically +/- 4% of the optimum based on maximum density (Seals \_\_\_\_), the fly ash-stabilized material

cannot reach the specified limits of relative compaction. Ferguson and Levenson (1999) state 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 heating them directly (ACAA 1991; Seals \_\_\_; Terrel *et al.* 1979a). Oven drying takes longer than direct heating, which can cause construction delays.

The “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 determine moisture content in the field quickly (ACAA 1991). While Seals (\_\_\_) reported that results for the “Speedy Moisture Tester” have proven to be somewhat unreliable and variable, studies in Virginia and West Virginia have produced acceptable results. Seals (\_\_\_) also states that the “Speedy Moisture Tester” should be calibrated with oven or direct heating tests.

The quickest and easiest way to determine moisture content in the field is by using a nuclear moisture gauge. A disadvantage of using the nuclear moisture gauge is that moisture measurements are usually subject to errors (ACAA 1991; Terrel *et al.* 1979a; Ferguson and Levenson 1999). Mahrt (2000) states that differences between actual and nuclear moisture measurements may stem from the elemental and compound structure of the fly ash, which affects the readings from the neutron radioactive source. Seals (\_\_\_) recommends against using the nuclear gauge entirely. Alternatively, Ferguson and Levenson (1999) suggest that the nuclear gauge may be used as long as calibrations and correction factors are determined. These calibrations are mixture-specific. The relative quickness of this test method has proven useful when measuring 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 (CIV) is correlated to the CBR of in-place materials. The measured values are taken at the surface of the stabilized layer, with the test taking less than one minute to perform (Mahrt 2000). A general correlation between CIV and CBR exists, but ASTM suggests calibrating CIV and CBR for site-specific materials to be used on each project. Mahrt (2000) determined a correlation between CIV and CBR for Iowa hydrated fly ash used as select fill:

$$CBR = 12.241e^{0.0572(CIV)} \quad (1)$$

Unlike the Clegg Impact Test, which is a shallow test, the DCP test provides a plot of material stability versus depth. The results of the DCP have been converted to CBR for hydrated fly ash (White 2002a). A common correlation between DCP depth in mm/blow and CBR is

$$CBR = \frac{292}{DCP^{1.12}} \quad (2)$$

### *In-Service Performance Testing*

#### Non-Destructive Testing

On a project in Wapello County, Iowa, Iowa State University researchers have used two forms of non-destructive testing to determine the in-service characteristics of a PCC pavement supported by hydrated fly ash. 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. The deflections are converted to a structural rating number, which in turn is used to calculate the modulus of subgrade reaction (k-value) for the subgrade. Road Rater testing has been used on numerous projects by the Iowa Department of Transportation (IDOT) (Klassen and Jones 1985).

The other test that has been used for pavement evaluation is roughness testing, which provides International Roughness Index (IRI) values for the pavement. Testing is conducted by pulling a trailer with a vehicle tire riding on 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 reflect subgrade deterioration over the lifetime of a pavement. Results for a PCC pavement over a hydrated fly ash subbase are reported in White (2002a).

#### Destructive Testing

Iowa State University researchers have also been monitoring the compressive strength of stabilized subgrade materials on two projects in which HFA and CFA were used as fill materials. In-service testing involved coring the pavement and subgrade to recover samples for unconfined compression testing (White 2002b ). However, sample recovery has proven 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 (Klassen and Jones 1985). Coring and sample extraction allow 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. White (2002a) reports the use of the DCP for a project in Wapello County, Iowa to measure the stability of the shoulders and select fill underlying the pavement. In DCP testing, cores of the pavement are taken to expose the subgrade and allow DCP testing to be completed. DCP testing can be completed in five to ten minutes, depending on the test depth and strength of the material.

#### *Construction Method Specifications*

Several authors (ACAA 1991; Ferguson and Levenson 1999; Terrel *et al.* 1979a) have developed specifications that provide suggestions for stabilizing soil with fly ash. These specifications cover topics from construction of the stabilized section through quality control testing. Most

specifications are broken up into some or all of the following areas: (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. These specifications form the basis of the proposed specifications reported herein for Iowa soil conditions.

## **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 (Terrel *et al.* 1979b). Since the onset of lime-fly ash stabilization, burning of Western United States coal has resulted in the 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 (White and Bergeson 1998, Klassen and Jones 1985, Ferguson and Levenson 1999, Çoçka 2001, Ferguson 1993, Khoury and Zaman 2002, Misra 1998, Nalbantoglu and Gucbilmez 2002, Parsons 2002, Rupnow 2001, Senol *et al.* 2002, Zia and Fox 2000).

### 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 (Ferguson 1993). Çoçka (2001) found that plasticity and swell potential decrease with increasing fly ash contents, and that ash addition rates greater than 20% are comparable to lime addition rates of 8% for reducing plasticity and ultimately swell potential in a soil consisting of 85% kaolinite and 15% bentonite. Ferguson (1993) notes that the decrease in plasticity and swell potential is generally less than that of lime because fly ash does not provide as many calcium ions that modify the surface charge of clay particles. According to Ferguson (1993), 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. Ferguson and Levenson (1999) recommend that careful laboratory evaluation of different fly ash contents for a given soil is necessary to find the optimum ash addition rate. However, Parsons (2002) reports 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 incorporating self-cementing fly ash.

Nalbantoglu and Gucbilmez (2002) report on the swell potential and compressibility of Degirmenlik soil (LL=67.8, PI=45.6) stabilized with 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 ash addition, respectively. Thirty days of curing reduced the swell potential to nearly zero for both addition rates. The authors also note that compression ( $C_c$ ) and rebound ( $C_r$ ) indices decreased as curing time and fly ash content increased. Zia and Fox (2000) 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, however, that the swell magnitude for the 10% samples increased with increased compaction. Samples containing 15% fly ash actually exhibited a 255% increase in swell potential over the loess soil. Zia and Fox (2000) attribute this behavior to the formation of ettringite (fly ash contains 3.6%  $\text{SO}_3$ ).

### Strength Gain Due to Addition of Self-Cementing Fly Ash

The most widely used application for self-cementing fly is in increasing 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 (Ferguson 1993, Ferguson and Levenson 1999, Parsons 2002, White and Bergeson 1998). Generally, clay soils have soaked CBR values from 1.5% to 5% (Rollings and Rollings 1996), which provides very little support to the pavement structure. Ferguson (1993) has shown that the 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 (Rollings and Rollings 1996). Zia and Fox (2000) also found that the CBR of loess increased five times with 10% fly ash addition, but an ash addition rate of 15% showed lower CBR than the 10% mixtures. They theorized that the reduction in CBR at 15% was 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 (Ferguson 1993, Ferguson and Levenson 1999). White and Bergeson (2000) compacted oxidized glacial till soil and 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 the samples to cure for 28 days. The data show that at 2% drier than the optimum moisture content 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 moisture 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% wetter than optimum moisture. 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. According to Ferguson and Levenson (1999) the optimum moisture content needed for maximum strength is typically 0% to 8% lower than the optimum moisture content for maximum density.

Misra (1998) states that long-term strength gain is expected for Class C fly ash stabilized soils. However, shrinkage cracks may occur over time and may be detrimental to strength development. Studies involving kaolinite mixed with 0%, 2%, 4%, and 6% bentonite showed that the largest strength gains occurred within 24 hours, and smaller increases were evident up to 7 to 14 days, but after 7 to 14 days strength gain was retarded and strength actually began to decrease. Zia and Fox (2000) also report that the majority of strength development occurs within 7 days of compaction for Indiana loess–fly ash mixtures, and that between 14 and 28 days the strength of stabilized loess decreases when compared to the strength of the loess alone. Misra (1998) theorized that the strength degradation is due to the fly ash content and the amount of smectite material in the soil. It was observed that higher ash and smectite contents slowed the strength loss. Zia and Fox (2000) 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 (2002) report on the effect of wet-dry cycles on resilient modulus ( $M_r$ ), elastic modulus (E), and unconfined compressive strength (UCS) for Class C fly ash-stabilized soft limestone aggregate.  $M_r$  increased 55% for specimens cured for three days and then subjected to 30 wet-dry cycles. Several cured specimens exhibited an increase in  $M_r$  for up to 12 cycles, at which time  $M_r$  began to decrease. The authors also observed that E and UCS values increased as the number of wet-dry cycles increased.

The strength gain and compacted density of self-cementing fly ash and soil are sensitive to compaction delays. Ferguson (1993) found that compaction delay can markedly decrease 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 compaction effort to break up the cemented particles (Ferguson and Levenson 1999). Ferguson (1993) has observed decreases in densities 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 the loss of interparticle contact points that result from a lower compacted density (Ferguson and Levenson 1999). 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 an increase in strength three to five times that of non-stabilized soils. This strength loss can be as much as 50% compared to the material compacted without delay (Ferguson and Levenson 1999). enol *et al.* (2002) report that the UCS and CBR of low plasticity clay with 20% fly ash decreased after a two-hour compaction delay. Ferguson (1993) and Ferguson and Levenson (1999) suggest that self-cementing fly ash-stabilized materials be compacted within two hours of initial mixing, and the authors have observed delay times of less than one hour during well-planned construction operations. enol *et al.* (2002) suggest that strength can be maximized by stabilizing 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 used in other industries, some power plants store this material on-site rather than transport it to landfills. This material can be stored in 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). Later this material 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, producing conditioned fly ash (CFA). CFA is considered to be more reactive than HFA because only some of the raw ash has been hydrated. 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 also act in a similar way to a lightweight aggregate (Barnes 1997). Iowa State University has conducted extensive research on the engineering properties and uses of HFA and CFA. The engineering properties evaluated were moisture-density relationships, compressive strength, CBR, freeze/thaw durability, hydraulic conductivity, and volumetric stability. Results of this testing are summarized in White (200x).

Barnes (1997) and later Mahrt (2000) have reported that the moisture-density relationships of HFA and CFA are typically flat curves without a pronounced peak as shown in typical Proctor curves for cohesive materials. Four sources of HFA materials and one source of CFA from Iowa show a range of maximum dry unit weights from 74 to 94 pcf, and optimum moisture contents from 22% to 37%.

The majority of strength testing for HFA and CFA materials has been through the use of CBR tests, both soaked and unsoaked tests, and in some cases the HFA and CFA have been activated by CKD, lime, or raw self-cementing fly ash. Mahrt (2000) reports that molded moisture content does not appear to affect CBR, but the level of compaction effort does. Sub-standard compaction effort (~95% of standard Proctor) produces unsoaked and soaked CBR values around 40%, while modified compaction effort yields values between 80% and 90%. Barnes (1997) conducted research on the influence of calcium activators on CBR and unconfined compressive strength of HFA and CFA. An activator of 10% raw fly ash was shown to increase CBR values up to 110%, and 2.5% lime addition produced CBR values to over 200%.

Test results show that unconfined compressive strengths for HFA and CFA materials are generally between 50 and 100 psi after 7 days, and gradually increase at 56 days due to pozzolanic reactions (Barnes 1997). HFA and CFA mixtures with 10% raw fly ash and 2.5% lime have shown compressive strength increases as much as 300% and 700% greater than those of untreated material, respectively (Barnes 1997).

Adding a calcium activator (CKD, lime, or raw fly ash) or leaving the material untreated has no bearing on the volume change potential of HFA and CFA. Furthermore, Berg (1998) 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 increase the freeze/thaw durability greatly (Berg 1998).

The hydraulic conductivity of untreated HFA and CFA is in the range of  $1 \times 10^{-3}$  to  $5 \times 10^{-6}$  cm/s, which is comparable to silty clay soils (Mahrt 2000). Mahrt (2000) has shown that differences in hydraulic conductivity depend on compaction 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 (2000) and White and Bergeson (2000) have investigated the use of HFA and CFA as soil stabilizers. Adding 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 (2000) has shown that HFA mixtures with lean and fat clays can increase strength two to three times that of the soil, when compacted dry of optimum moisture content. The strength gain of HFA-stabilized soils is material dependant, as Ottumwa HFA showed strength increases of 8 to 12 times when compacted wet of optimum moisture content with fat

clay, whereas Neal 3 HFA did not produce significantly more strength gain than the Ottumwa HFA (White 2000). White's (2000) results also show that lean clay stabilized with HFA fines and prepared wet of optimum moisture content gained strength over about two times that of the natural soil. A limited compaction delay study was completed with non-oxidized glacial till and 20% Prairie Creek CFA. This study showed that it was actually beneficial for a long compaction delay (~ eight hours) to facilitate maximum strength gain, although the reasons for this are not understood (White and Bergeson 2000).

With the addition of HFA fines, expansive clays that have a high plasticity index can show a dramatic decrease in plasticity index and therefore swell potential. In an expansive clay reported by White (2000) with an original plasticity index of 47%, the plasticity index was reduced to 22% after HFA addition and a cure time of 28 days (White 2000).

## **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 (ASTM C618). 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 (Barnes 1997). 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. The chemical requirements of ASTM C618 and typical Iowa Class C fly ash composition are shown in Table 2.

**Table 2. Typical chemical compositions of Iowa Class C fly ash and ASTM C 618 chemical requirements for Class C fly ash (Barnes 1997)**

Oxide	Self Cementing Fly Ash (% of Total Weight)	ASTM C618
SiO <sub>2</sub>	20-40	Summation between 50% and 70%
Al <sub>2</sub> O <sub>3</sub>	10-30	
Fe <sub>2</sub> O <sub>3</sub>	3-10	
CaO	10-32	
MgO	0.5-8	
Na <sub>2</sub> O	0.5-6	
K <sub>2</sub> O	0.5-4	
TiO <sub>2</sub>	0.5-2	
SO <sub>3</sub>	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 (Barnes 1997). Fly ash is a pozzolanic material that ASTM C618 defines as “materials rich in silica and alumina which in themselves have little or no 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 (1997) reports that the pozzolanic reactivity of fly ash mainly depends 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. The initial formation of cementitious reaction products is due to the hydration of tricalcium aluminate, which Ferguson and Levenson (1999) report is the cause of problems during long compaction delay times. Strength gain during 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 Sulfur*

Due to environmental concerns, some power plants have converted to fluidized bed combustion (FBC) or flue gas desulfurization (FGD) systems to help remove SO<sub>2</sub> 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. Ferguson and Levenson (1999) report that FBC ashes can contain up to 35% SO<sub>2</sub> while FGD ashes contain SO<sub>2</sub> contents greater than 35%.

Problems are encountered when crystals composed of sulfate compounds grow after the 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, producing long-term expansion. Ettringite and thaumasite are formed by the reactions of

calcium, sulfates, alumina, silica, and water. Ettringite forms initially and occupies a volume over 200% of the volume of the original constituents. The secondary formation of expansive crystals is the conversion of ettringite to thaumasite, which takes a longer time and results in an additional 200% volume expansion (Ferguson and Levenson 1999). Thaumasite is formed at a lower temperature than ettringite, and by way of the 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. Ferguson and Levenson (1999) state that as the pH of a soil reaches about 10.5, the alumina and silica in clay particles become 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 Iowa State University. White (2000) 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% over the 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 (Ferguson and Levenson 1999). Ferguson and Levenson (1999) provide the following guidelines for stabilization with high sulfur ashes:

- Ashes with sulfur contents 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 the 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 have had catastrophic results when calcium-based stabilizers, usually lime or portland cement, were used in the following situations: (1) to stabilize sulfate bearing clay soils, (2) in areas where a supply of fresh groundwater continues to provide available sulfates, or (3) when the mixing water for the project contained high amounts of soluble sulfates (Hunter 1988, Kota *et al.* 1996, Rollings *et al.* 1999). Hunter (1988) reported 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 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 via 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 ( $\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$ ), which is a precursor to the formation of ettringite and thaumasite. One project reported ettringite formation as rain water infiltrated the subgrade that had been stabilized with lime and Type II portland cement. Areas on the same project that had not been stabilized did not exhibit swelling-related problems (Kota *et al.* 1996).

The other major project that experienced problems with calcium-based stabilizers involved a double application of lime. Heaving of the subgrade stabilized by the first lime treatment was observed within six months of project completion. Kota *et al.* (1996) 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 six months, destroying the pavement structure. Kota *et al.* (1996) provide some suggestions 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
- Stabilization of 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.* (1999) examined a project in Georgia that involved a cement-stabilized, sand-based course material that was mixed off-site at the sand borrow pit. As in the Texas examples, sulfate-induced heave was evident within six months after construction. A preliminary investigation provided no definitive answers as to why the base course heaved. Sulfur was not present in the cement used or in the sand. Closer inspection showed that the mixing water used at the off-site mixing plant contained over 10% sulfur, and that the water was 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 a low CBR value of 3.5. The initial pavement design included 12 inches of full-depth asphalt. However, reducing the pavement thickness by improving the subgrade was desired.

Laboratory testing using a fly ash content of 15% from Hawthorne Power Station showed decreases in the liquid limit and plasticity index to 45 and 18, respectively. The 28 day unconfined compressive strength of the stabilized clay soil was seven 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 (GAI Consultants 1992).

#### *Low Cost Fly Ash-Stabilized Sand, Des Moines County, Iowa*

The main objective of this Iowa Department of Transportation project had been to develop a low cost fly ash-stabilized roadway using locally available unprocessed sand. The project is located on county road H-40 in Des Moines County. The roadway is 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. The 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. However, the main objective, a 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 (Klassen and Jones 1985).

#### *Recycled Pavement, Shawnee County, Kansas*

This 1.5-mile section of 93<sup>rd</sup> Street is considered a rural road but carries a high volume of truck traffic. The existing thickness of the road material varied from one to six inches for the asphalt surface and one to eight inches for the granular base over a clay subgrade. The mix design intended 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 were pulverized to a six-inch depth and lightly compacted. The fly ash was spread on the surface and mixed 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 five 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 four years of service (GAI Consultants 1992).

#### *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 came from Kansas Power and Light and the Jeffrey Energy Center and had calcium oxide contents 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 zero and two 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 the 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 (Ferguson 1993).

#### *Northwest Highway Fly Ash Stabilization, Oklahoma*

The material on this site was sandy clay and required stabilization. The design engineers specified an ash addition rate of 15%, which was initially to be mixed to a depth of eight inches with the existing subbase. After preliminary mixing, water was sprinkled on the mixture and a second pass of the mixing equipment immediately followed. The compaction window on the project was four to six hours. The stabilized subgrade was finished with a ten-inch hot-mix asphalt layer (CMI Corp. 1994).

#### *Power Plant Access Road, Marshalltown, Iowa*

Construction began in June 1994 on a 1,700-foot-long by 22-foot-wide access road to the Sutherland Generating Station in Marshalltown, Iowa. The road was constructed on a ten-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 1,000 feet of the access road. For this portion of the project, the CFA was placed on-site and the CKD was then 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 first compacted using a padfoot roller 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 for the CKD section, and the compacted AFBC section was also kept in a moist condition. Final surfacing was a two-inch chip seal. Beginning in November 1994, ISU personnel 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 extend down through the samples. The cause of the cracks is believed to be high vehicle loads and freeze/thaw damage. The freeze/thaw damage is probably due to a decrease in the permeability of the stabilized material. Currently, the materials are behaving similarly 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 needing to have been resurfaced with hot mix asphalt in early 2002 (White 2002b).

#### *Landfill Access Road, Ottumwa, Iowa*

The Ottumwa-Midland Landfill is located five miles north of Ottumwa, Iowa. Construction of the road base took place from May 30 to June 1, 1995. The road is 2,500 feet long and had 1,800 feet of CKD-stabilized (10% by dry weight) HFA base and 700 feet of AFBC-stabilized (15% by dry weight) 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 over the compacted HFA and a reclaimer mixed the materials to a depth of eight inches. A loader was then used to stockpile the reclaimed material. The mixtures were then hauled to the site and spread over 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 using a smooth drum roller. The compacted material was kept moist by using an asphalt prime coat. A 1.5-inch asphalt concrete surface was applied after the base had been allowed to cure for one week. The base has been cored annually since August 1995 by ISU personnel. A maximum compressive strength of 2,235 psi was reached in 1997. The strength has decreased since 1997, but the 2002 cores still had an average compressive strength of 2,055 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 (White 2002b).

#### *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. Because no select soils were available for construction of the road, 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 used. Two sections of the road that totaled 3.1 miles were constructed on HFA, while the remainder was constructed on Class 10 soil, 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 closely agrees with the value of 55% used in the pavement thickness design. The Iowa DOT has

performed Road Rater testing on this project annually, except in the year 2002. Based on the Road Rater testing, the HFA sections are behaving like the PCC slab is between 10.4 to 12.2 inches thick, while the equivalent thickness for the control section is between 9.1 and 10.5 inches. The HFA sections are providing very good structural support to the slab. Roughness testing results for the pavement range from 1.47 to 1.58 m/km. This is in the high range for a new PCC pavement. It is expected that results comparing the HFA and control sections will be more useful in the future. Overall, the pavement is performing very well (White 2002a).

#### *Fly Ash Stabilization of Recycled Asphalt Pavement, Waukesha County, Wisconsin*

Highway JK in Waukesha County, Wisconsin is a 0.75-mile-long county road in a low area with very silty subgrade soils. The silty nature of the underlying soil and available water had led to problems with frost heave. Construction of the new road base began in October 2001. The existing asphalt was pulverized to a depth of 6 inches, a water truck was then 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 recycled asphalt pavement (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. During the following winter, no problems from frost heave were observed (Gantenbein 2002).

#### *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 between 140% and 350% of the original strength, as measured by the unstabilized underlying soils. No correlation existed between the CBR values 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 (Parsons 2002).

#### *Fly Ash RAP Stabilization of Parking Lots, Ames, Iowa*

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 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 was begun and completed in the summer of 2002. The existing asphalt 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. As the reclaimer made the final pass, initial compaction was immediately completed with a vibratory padfoot roller. Compaction was finished with a smooth drum steel roller that sealed 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. Detailed results of this project are described in Rupnow (2002).

## **SOURCES AND MATERIAL PROPERTIES OF SOILS, RAW FLY ASHES, AND HYDRATED AND CONDITIONED FLY ASHES**

### **Material Sources**

#### *Soils*

##### Turin, Iowa–Loess

Silty Western Iowa loess was sampled from containers located in Town engineering building at Iowa State University. 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 area of Western Iowa.

##### Neola, Iowa–Alluvium

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

##### Le Grand, Iowa–Loess

Central Iowa loess had been sampled previously by Iowa State University 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

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

##### Argyle, Iowa–Paleosol

During October of 2001, Iowa State University personnel collected 50 gallons of Southeast Iowa paleosol. The material came from an area about two miles north of Argyle, Iowa in Lee County, and was part of a Highway 218 earthwork construction project. 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. The Ottumwa Generating Station burns subbituminous coal from the Powder River basin in Wyoming. Four additional five-gallon containers were delivered to Iowa State University on August 21, 2001.

### Council Bluffs Fly Ash

Class C fly ash from the 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. This material is formed from burning Wyoming subbituminous coal. Four additional five-gallon containers were also delivered to Iowa State University on August 21, 2001.

### Louisa Fly Ash

The 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. An additional twenty gallons was delivered to Iowa State University on August 21, 2001.

### Ames Fly Ash

One five-gallon container of Ames power plant self cementing fly ash was delivered to Iowa State University in the early spring of 2002 by ISG Resources personnel. The Ames power plant burns Wyoming subbituminous coal, but 10% of the fuel stream also consists 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 that allow 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 (PC3) is reportedly marginal Class C fly ash due to the type of coal burned. Unit #4 (PC4) at the station burns subbituminous coal from Wyoming. The 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 PC 3 and PC4 was delivered to Iowa State University on May 10, 2002.

### 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 were delivered to Iowa State University on September 10, 2001.

### Sutherland Fly Ash

Sutherland Generating Station (SGS) is located in Marshalltown, Iowa. The Sutherland station produces marginal Class C fly ash due to the stoker-fired boilers, which cause the fly ash to have sulfur contents higher than 5%. A five-gallon sample was delivered to Iowa State University on December 13, 2001. Sutherland station fly ash does exhibit self-cementing properties.

### *Hydrated and Conditioned Fly Ashes*

#### Ottumwa Hydrated Fly Ash

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

#### Port Neal Hydrated Fly Ash

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

#### Council Bluffs Hydrated Fly Ash

Iowa State University personnel collected Council Bluffs Generating Station HFA on March 23, 1999 from a stockpile 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 Iowa State University personnel. The material was sampled from a previously milled stockpile located at the power plant. Sixty gallons of material were available for this study.

## Prairie Creek Conditioned Fly Ash

Twenty gallons of Prairie Creek CFA were delivered to Iowa State University in early June 2001. The Prairie Creek 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.

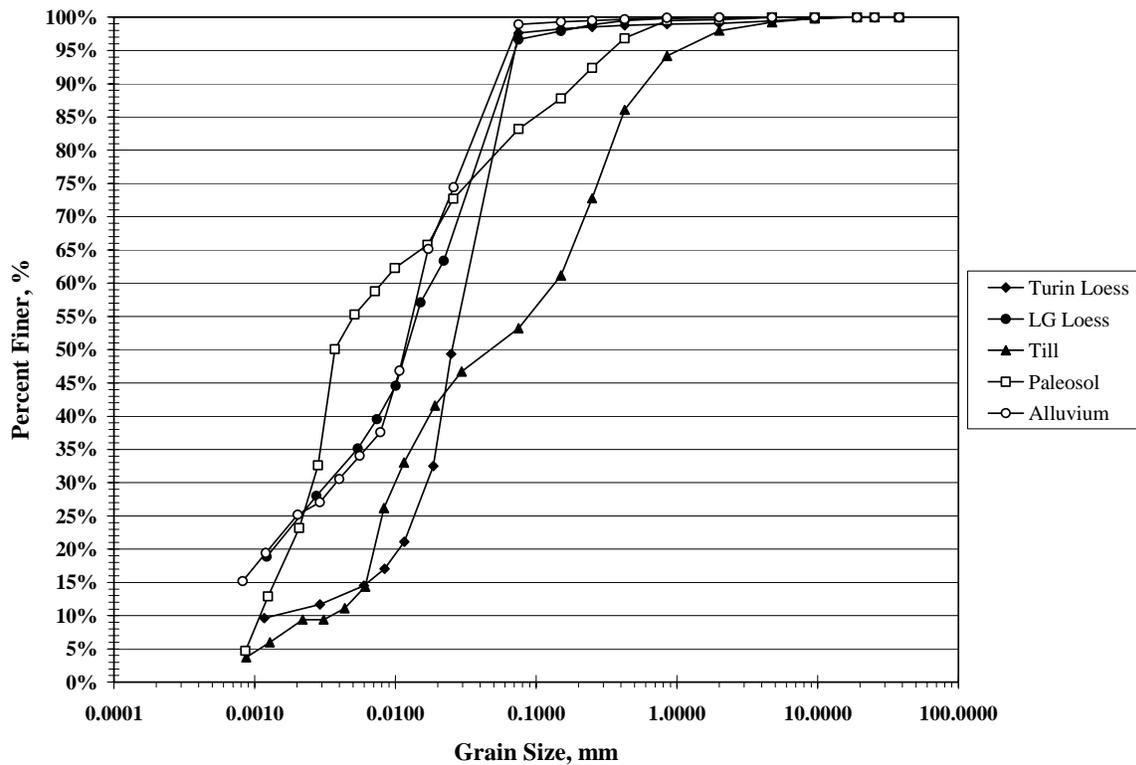
## Sutherland Conditioned Fly Ash

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 in general accordance with ASTM D422 (Standard Test Method for Particle-Size Analysis of Soils). Results are shown in Figure 2. The percentages of gravel, sand, silt, and clay for each material are summarized in Table 3.



**Figure 2. Soil particle size distribution**

**Table 3. Summary of soil properties**

<b>Property</b>	<b>Turin Loess</b>	<b>Le Grand Loess</b>	<b>Cedar Rapids Till</b>	<b>Argyle Paleosol</b>	<b>Neola Alluvium</b>
<b>LL</b>	33	40	40	48	47
<b>PL</b>	29	21	17	17	22
<b>PI</b>	4	19	23	31	25
<b>G<sub>s</sub></b>	2.74	2.68	2.70	2.74	2.80
<b>% Gravel</b>	1%	0%	2%	1%	0%
<b>% Sand</b>	1%	3%	45%	16%	1%
<b>% Silt</b>	87%	70%	44%	60%	74%
<b>% Clay</b>	11%	27%	9%	23%	25%
<b>Group Symbol</b>	ML	CL	CL	CL-CH	CL-CH
<b>Group Name</b>	Low Plasticity Silt	Lean Clay	Lean Clay	Lean to Heavy Clay	Lean to Heavy Clay
<b>AASHTO (Group Index)</b>	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). Atterberg limits results are provided in Table 3. The Turin loess has the lowest liquid limit (LL) and plasticity index (PI) while the paleosol has the highest LL and PI. The paleosol is the remnant of an ancient B horizon formed in glacial till. The Le Grand loess has the highest clay size content. The soils were classified in general accordance with ASTM D2487 (Standard Classification of Soils for Engineering Purposes [Unified Soil Classification System]) and AASHTO (Classification of Materials for Subgrades and Granular Type Roads). The USCS and AASHTO classification symbols as well as the USCS group names and AASHTO group index values are provided in Table 3.

#### *Specific Gravity*

Soils were tested in accordance with ASTM D854 (Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer) to determine specific gravities. The specific gravity values shown in Table 3 range from 2.68 to 2.80.

#### *Clay Mineralogy*

A 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 microscopy (SEM), and energy dispersive spectrometry (EDS). All tests were conducted on material smaller than about two  $\mu\text{m}$ . 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 the remaining clay sludge was allowed to air dry.

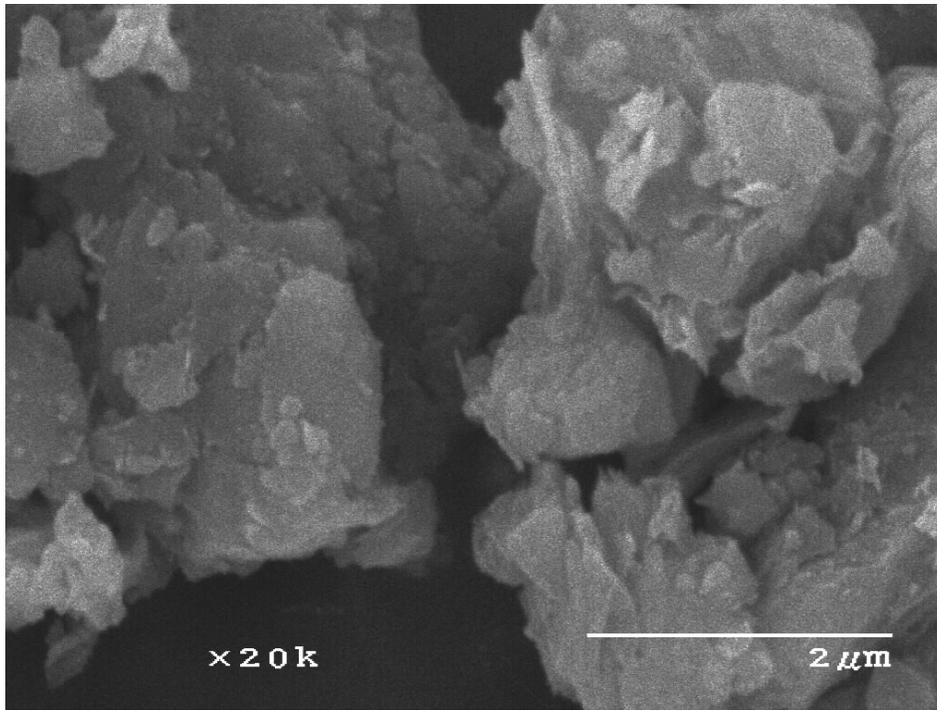
The diffractograms from the XRD analyses are provided in Appendix A. All clay fractions show that quartz has the largest intensity. Montmorillonite was the dominant clay mineral in all samples. Samples also contained lesser amounts of kaolinite and illite.

Calculated CEC results are provided in Table 4. It is assumed that all exchange sites were filled before testing was conducted and that the exchangeable cations consisted only of potassium, magnesium, calcium, and sodium. Duplicate tests were completed for each soil and the results agreed well.

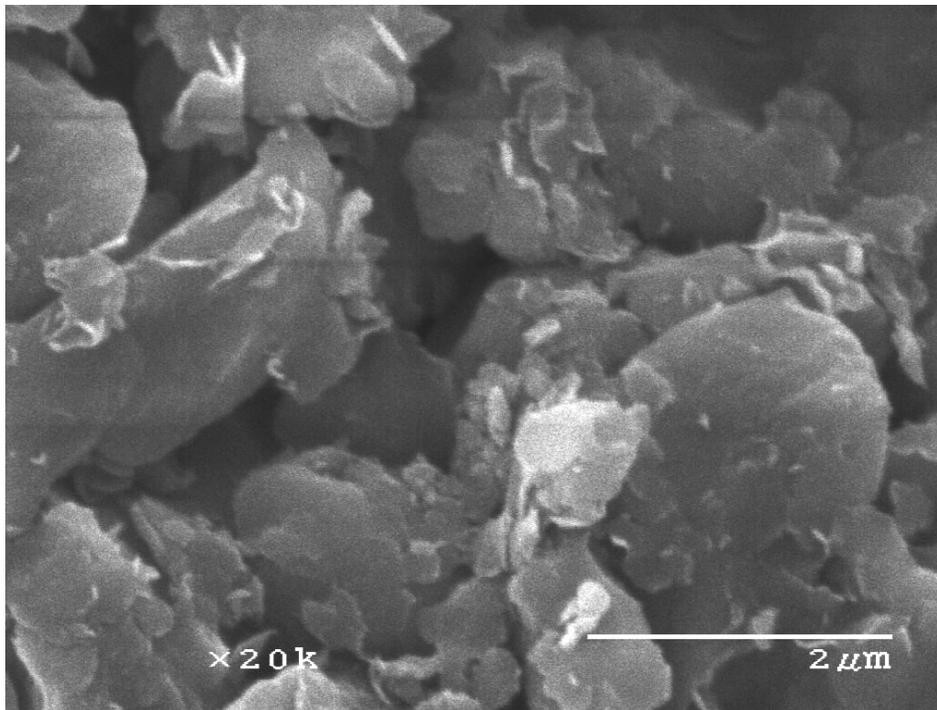
SEM images provide a visual indication of the type of clay minerals present in the soil samples. Figures 3, 4, 5, and 6 show the Le Grand loess, alluvium, till, and paleosol clay fractions, respectively. These images were taken at 20,000x magnification. All samples show some thin, wavy particles, typical of montmorillonite, as well as thicker, blockier type particles, which are typical of kaolinite. Illite clay minerals are not identified in the images as easily and often occur as very small (0.1  $\mu\text{m}$  or less) platy particles mixed with other clay minerals.

**Table 4. Summary of cation exchange capacity for soil clay fractions (<2 microns)**

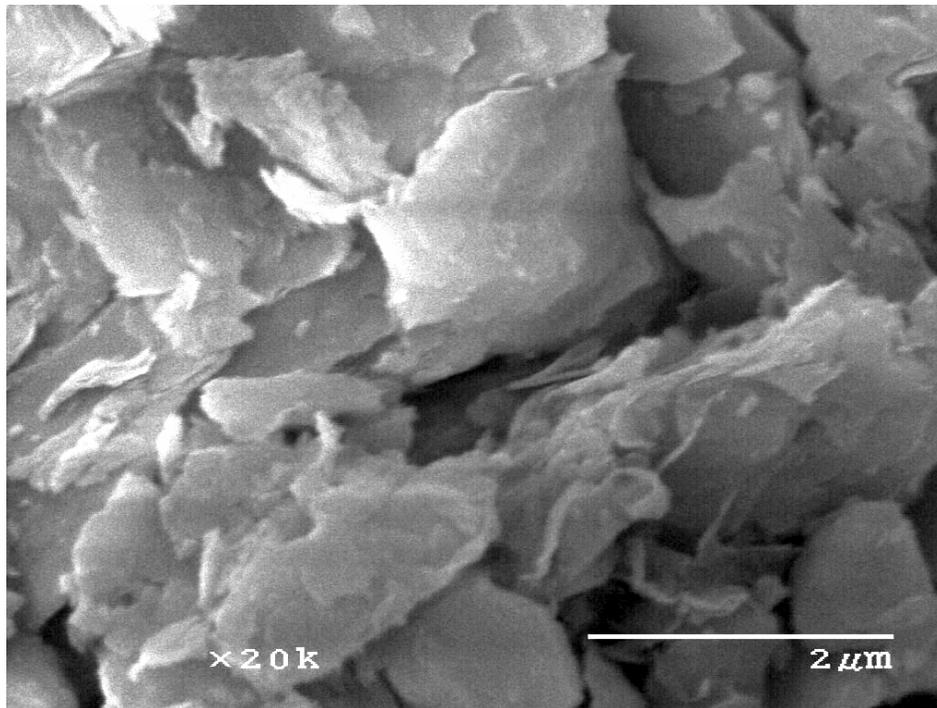
<b>Soil</b>	<b>K, ppm</b>	<b>Ca, ppm</b>	<b>Mg, ppm</b>	<b>Na, ppm</b>	<b>CEC, meq/100g</b>
Glacial Till	306	14	2.3	8	9
Glacial Till	308	14	2.3	8	9
Alluvium	688	15	3.4	12	19
Alluvium	702	16	3.5	13	20
Paleosol	451	13	4.8	28	14
Paleosol	435	13	4.7	28	13
Le Grand Loess	354	12	3.1	9	10
Le Grand Loess	330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



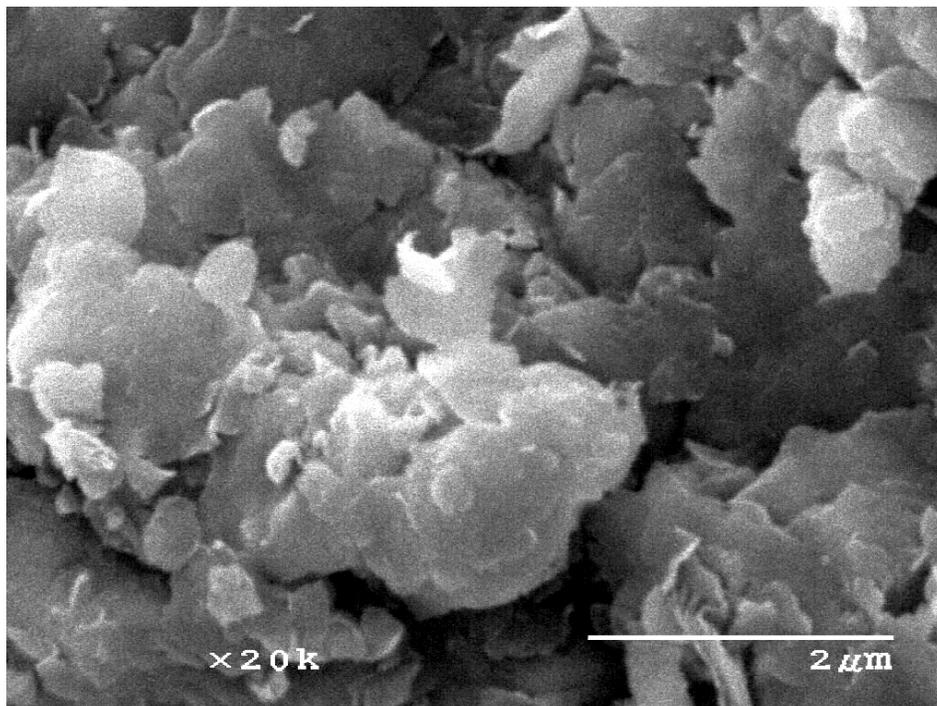
**Figure 3. SEM image at 20,000x of Le Grand Loess clay fraction (<2 microns)**



**Figure 4. SEM image at 20,000x of Alluvium clay fraction (<2 microns)**



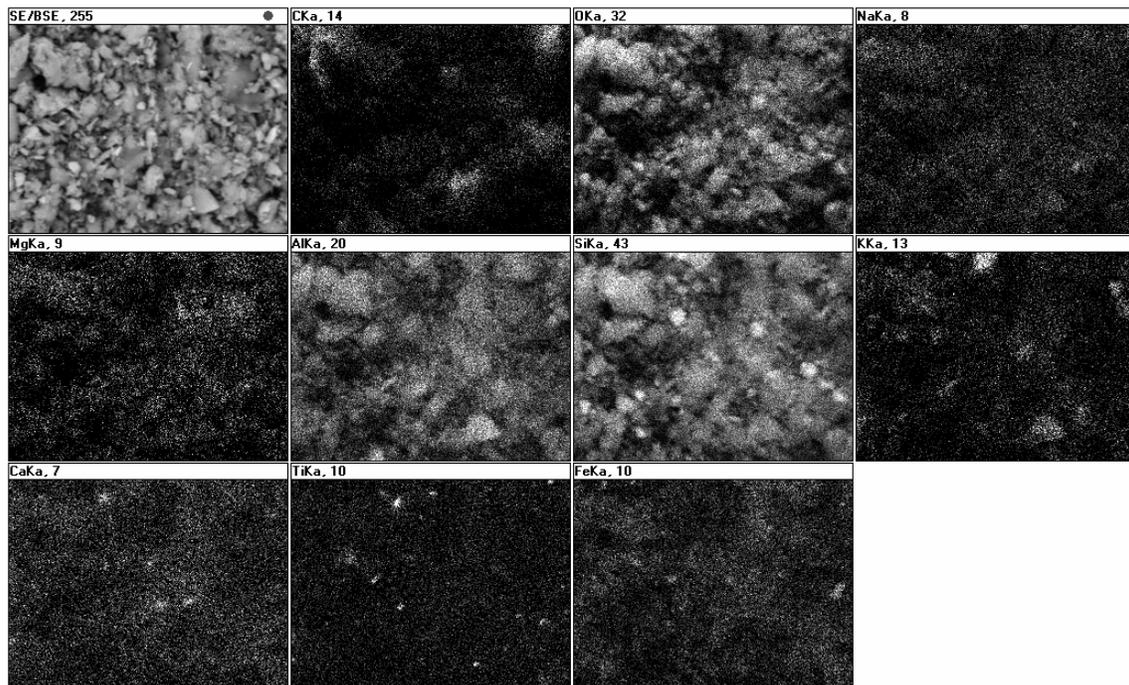
**Figure 5. SEM image at 20,000x of Glacial Till clay fraction (<2 microns)**



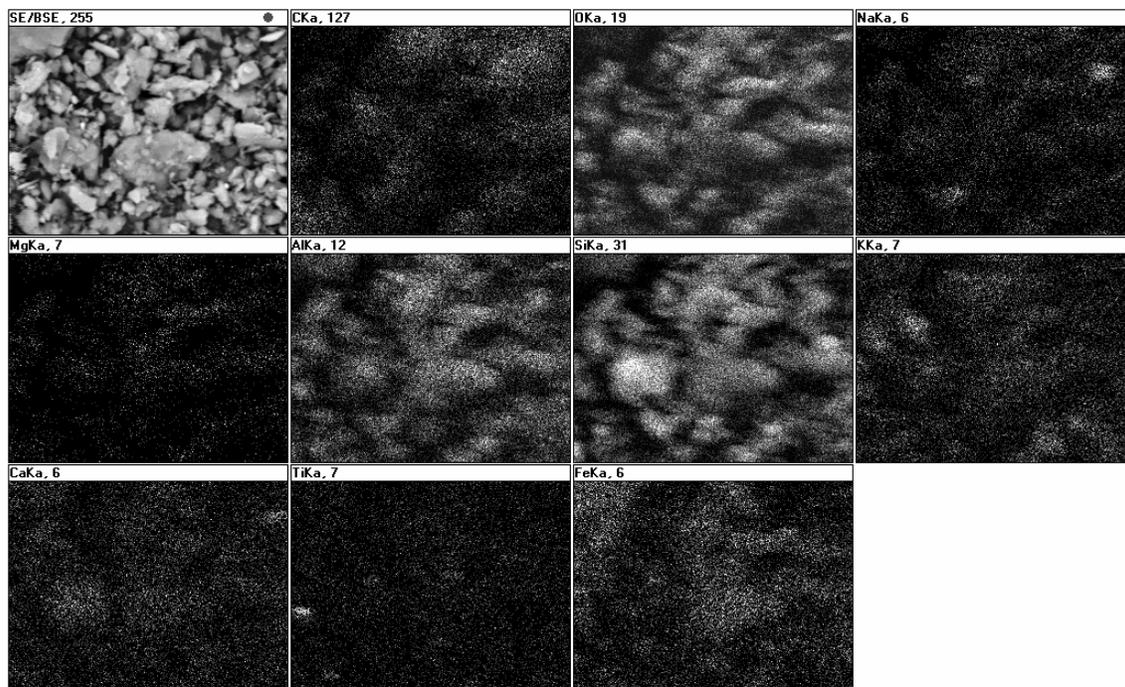
**Figure 6. SEM image at 20,000x of Paleosol clay fraction (<2 microns)**

Elemental maps of the clay fractions were produced using an EDS technique. The maps are shown in Figures 7, 8, 9, and 10. Each map was produced at 3,000x power. As shown, the clay minerals are comprised mainly of silicon, aluminum, magnesium, and oxygen. Each map shows that these are the elements most common in each soil. Some quantities of calcium, sodium, and potassium are also present. The carbon is indicative of organic matter. Small amounts of titanium and iron oxides are also present.

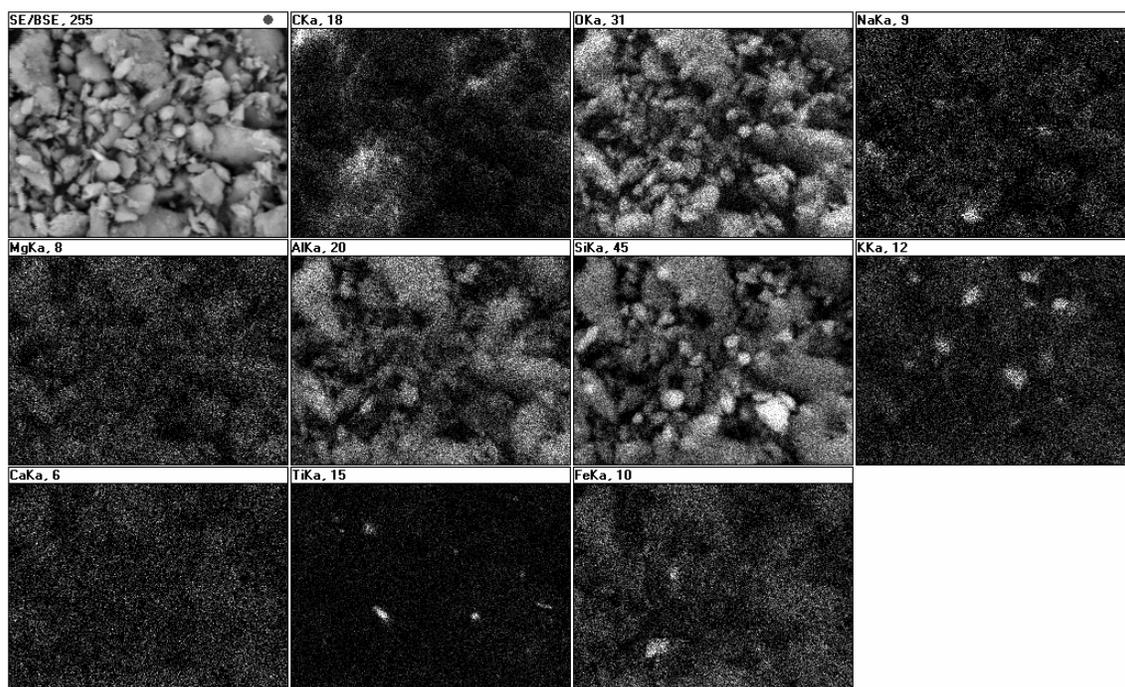
Thermal analyses techniques were also conducted to characterize the clay fractions. Sample heating was carried out in an inert nitrogen atmosphere at 10° C/min. TGA testing measures the mass loss of the sample during heating, while DTA compares the temperature difference between an inert sample and the soil sample. Figures 11 and 12 show the TGA and DTA results, respectively. Duplicate tests were conducted for each sample and the results were compared. The TGA results show that the clay fractions lost weight due to water loss up to about 100° C. The weight loss increased between about 375° and 500° C. The DTA results are somewhat difficult to interpret. Endothermic peaks are evident at 100° and 180° C for all samples. Other than these two peaks, no other large peaks are clearly evident. All samples with the exception of the alluvium showed a small endothermic peak at about 500° C, which is indicative of both illite and kaolinite. Small exothermic peaks are present between 880° C and 920° C which are indicative of illite and montmorillonite. The thermal analysis results ultimately show that there is a mixture of clay minerals present in the soils.



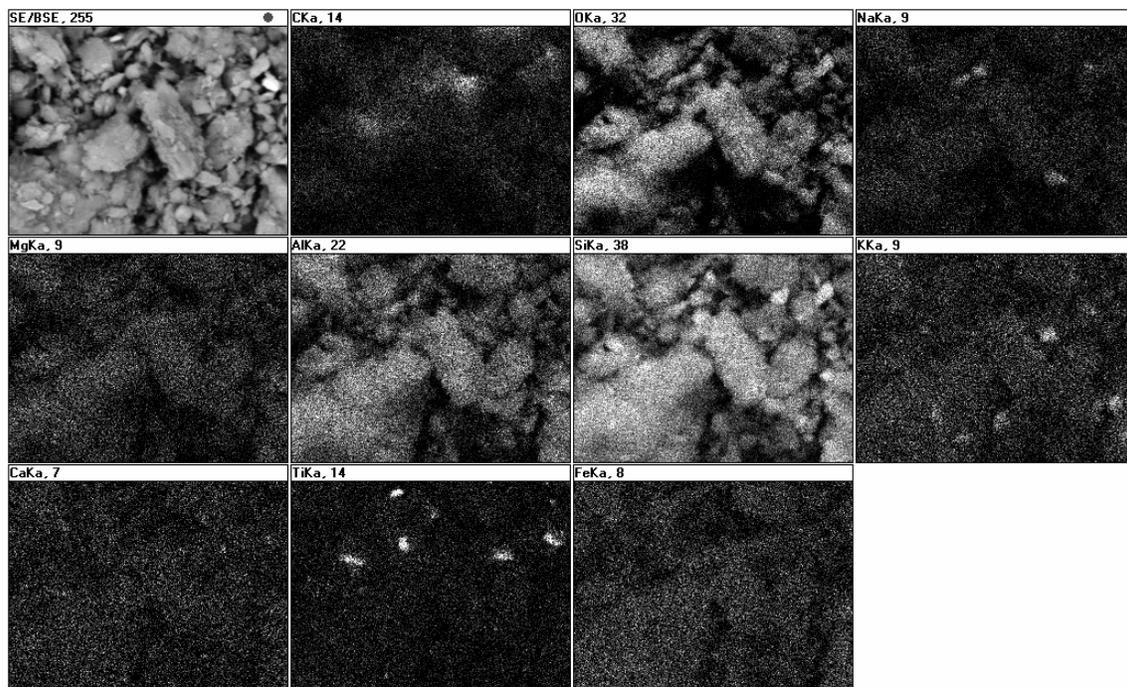
**Figure 7. EDS elemental map of Le Grand Loess clay fraction (<2 microns)**



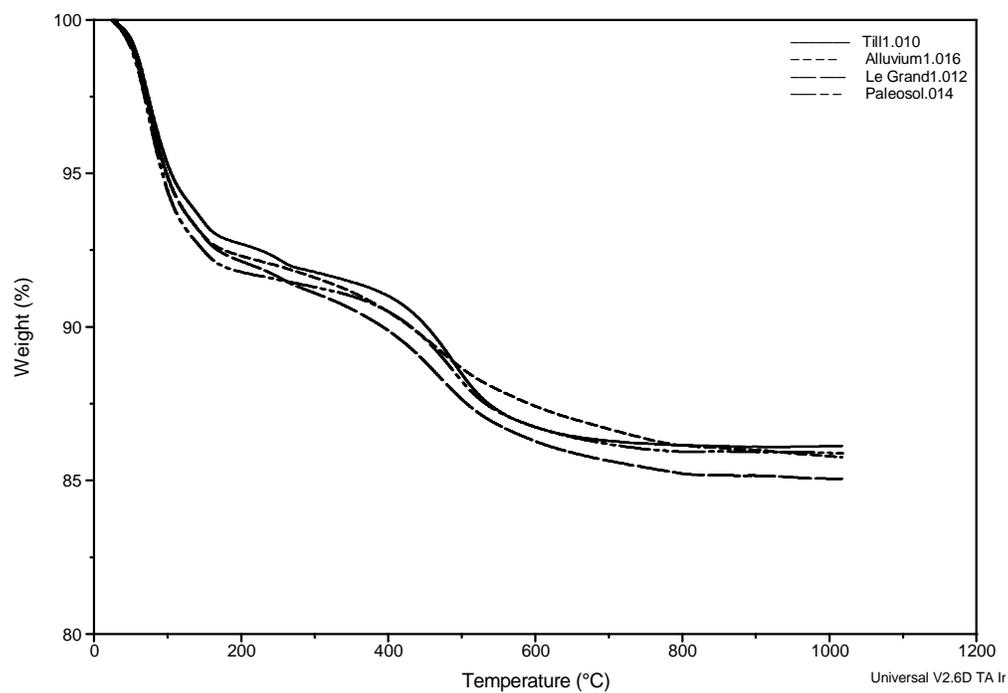
**Figure 8. EDS elemental map of Alluvium clay fraction (<2 microns)**



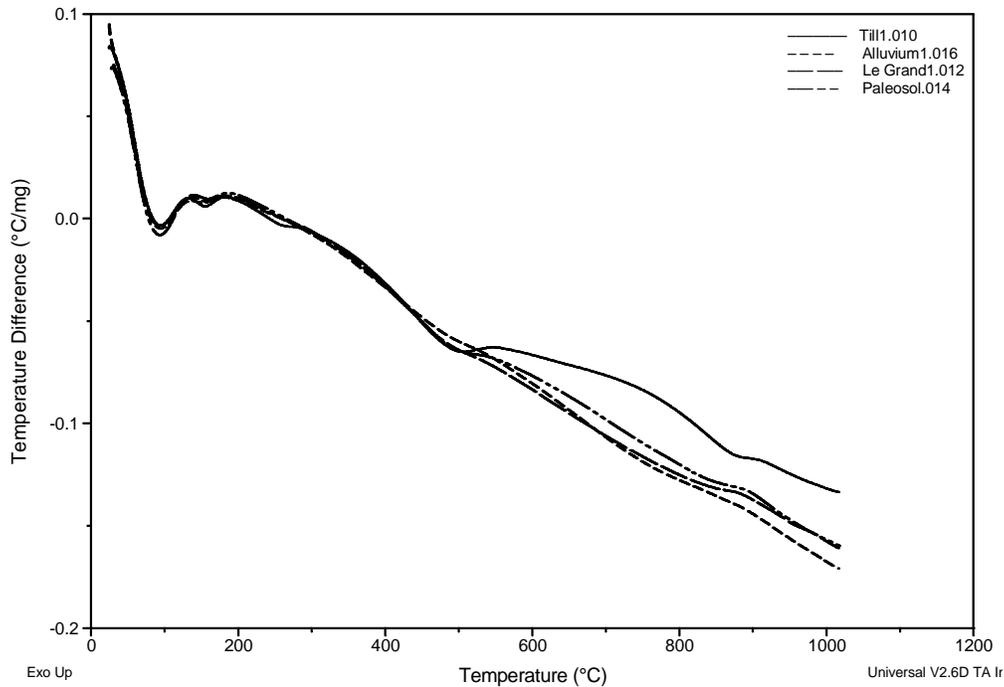
**Figure 9. EDS elemental map of Glacial Till clay fraction (<2 microns)**



**Figure 10. EDS elemental map of Paleosol clay fraction (<2 microns)**



**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 in general accordance with ASTM D698 Method A (Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort [12,400 ft-lb<sub>f</sub>/ft<sup>3</sup>]). A wide range of maximum densities and optimum moisture contents were determined. Results for all soils are shown in Figure 13. Table 5 summarizes the optimum moisture contents and maximum dry densities. 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%).

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 of 1.15. Figure 14 shows the unconfined compressive strength (UCS) results plotted against molded moisture content. The samples were not soaked prior to testing. It is estimated that soaking the samples would have resulted in lower strength and possibly deterioration of the samples compacted dry of optimum. The glacial till has the highest UCS at 87 psi. The alluvium exhibited the next highest strength, with 62 psi at 14.5 % moisture. The paleosol and Le Grand loess have similar UCS values, with the paleosol's results being approximately 5 psi greater throughout the moisture content range. Turin loess is predominately composed of silt-sized particles, and has the lowest overall strength. Table 6 presents the UCS values at optimum moisture content along with the relative strength decrease as a function of moisture content. The Turin loess has a UCS of about 33 psi at optimum moisture content, while

the paleosol has a UCS of about 48 psi. The other soils have approximately the same UCS of about 44 psi at optimum moisture content. The glacial till's UCS decreases approximately 7 psi/m% while the other soils decrease between 3 and 5 psi/m%, thus indicating the sensitivity of soil UCS to compaction water content.

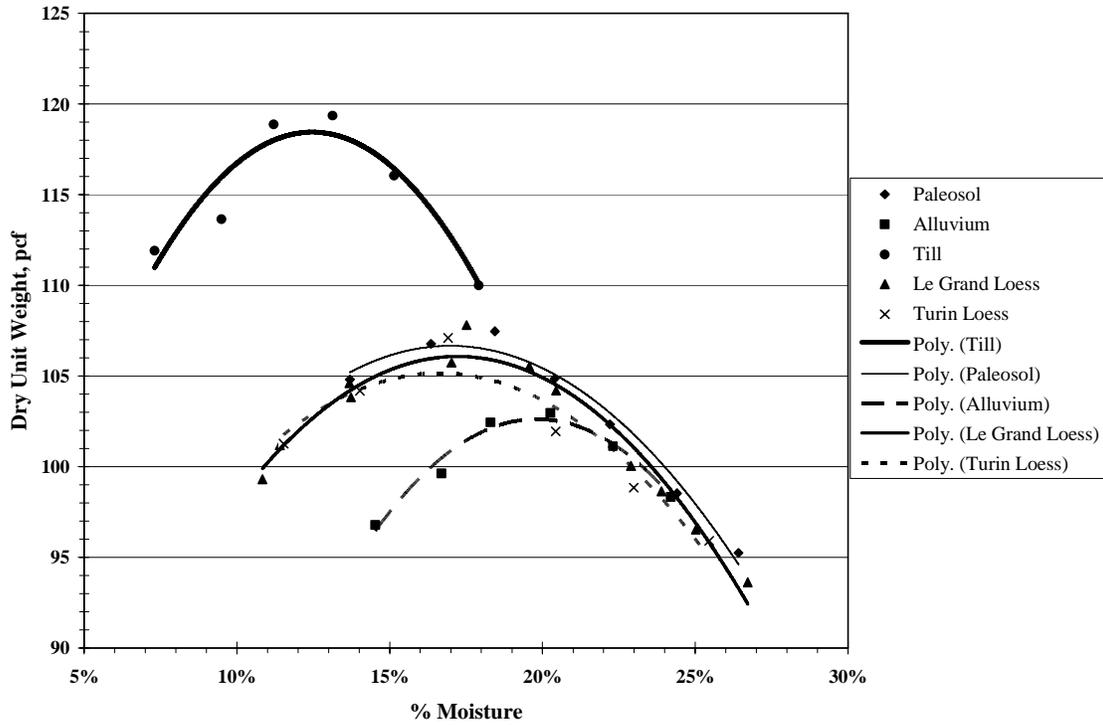


Figure 13. Moisture-density relationships of soils used in the study

Table 5. Summary of soil optimum moisture contents and maximum dry unit weights

Soil	Optimum Moisture Content (%)	Maximum Dry Unit Weight, $\gamma_d$ , (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

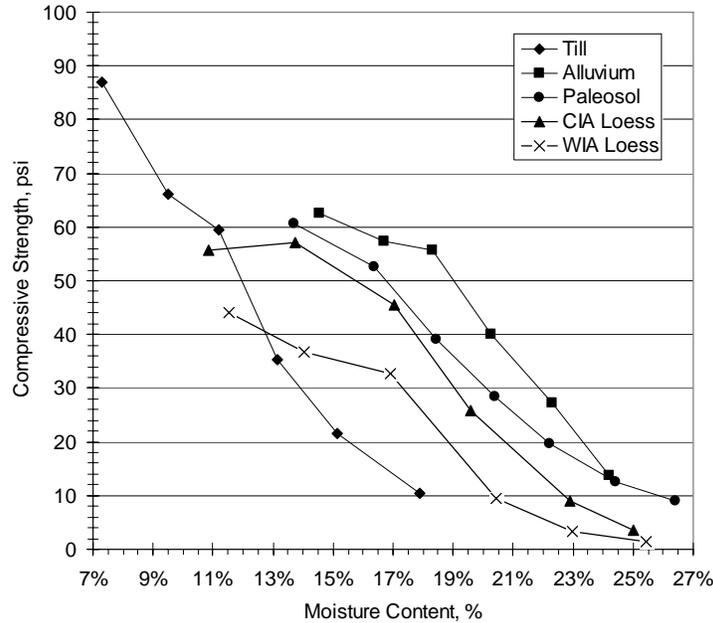


Figure 14. Moisture-strength relationships of soils used in the study

Table 6. Compressive strength at optimum moisture content and strength decrease of soils used in the study

Soil	Optimum Moisture Content, %	Strength at 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*

To determine the mineralogy and elemental composition of the fly ashes, samples of the fly ash were prepared for XRD and x-ray fluorescence (XRF) analyses by grinding the samples to a fine powder using a shatterbox. The bulk specimens for XRD analysis were back-loaded to avoid preferred orientation. The samples for XRF analysis were ignited to a constant mass at 950°C. The ignited samples were fused into a glass disk using a lithium borate flux. 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), and testing

was conducted by personnel at the Materials Analysis and Research Laboratory at Iowa State University during the spring of 2003. The major minerals identified by XRD are summarized in Table 7. The complete XRD diffractograms are provided 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) Class C, while the ones that do not meet the requirements are still self-cementing and effective for soil stabilization.

The minerals summarized in Table 7 are listed in order of decreasing intensity, as reported in the diffractograms (Appendix A). The fly ashes have large amounts of quartz and tricalcium aluminate. Other common minerals are lime (CaO), anhydrite (CaSO<sub>4</sub>), and periclase (MgO). Tricalcium aluminate is important because it contributes to initial hardening when hydrated. The free lime also contributes to the formation of cementitious reaction products in combination with silicon and aluminum in the fly ash and clay minerals. Anhydrite is a precursor to ettringite formation, although ettringite formation is minimal due to the low sulfur content detected in the XRF analyses. Periclase is also evident in the fly ash samples and is composed of MgO.

Table 8 summarizes the XRF results. Some of the key parameters to consider in the XRF results are the loss on ignition (LOI), sulfur (SO<sub>3</sub>) content, calcium (CaO) content, and silica (SiO<sub>2</sub>), alumina (Al<sub>2</sub>O<sub>3</sub>), and iron oxide (Fe<sub>2</sub>O<sub>3</sub>) contents. The sum of silica, alumina, and iron oxide must total between 50% and 70% for an ash to be classified as Class C according to ASTM C618. The sum of oxides for the Sutherland Generating Station ash is the lowest, at 50.98%, while the largest sum of oxides, 63.52%, was obtained in the older Ottumwa Generating Station fly ash. The literature states that sulfur content greater than 5% can have detrimental effects on soil stabilization. All fly ashes tested have sulfur contents less than 5%. Moisture contents of the ashes were determined prior to testing. The measured values are typically structural water absorbed from the environment. LOI is a measure of the unburned carbon content of fly ash. Sutherland Generating Station ash has the largest LOI, at 9.2%. The CaO contents of the ashes range from 22.2% to 28.5%.

### *Fly Ash Set Time*

Self-cementing fly ashes form cementitious reaction products when hydrated. Fly ash set time can have a significant bearing on strength gain/loss and density due to compaction delay, as discussed earlier. To determine the fly ash set times, samples were mixed into a paste with water at a water/ash ratio of 0.275. The paste was then placed in a shallow brass dish and a pocket penetrometer was used to measure penetration resistance versus time. When the reading on the pocket penetrometer reaches 4.5 tsf, the fly ash has achieved its final set time and the time is recorded. The initial set time is the time at which the fly ash begins to increase its rate of strength gain. Lapke and Bergeson (1993) reported that set time is related to the CaO content of the ash. Figures 15 and 16 show the complete set time test results for the ashes used in this study, and Table 9 summarizes the initial and final set times for each ash. Results show that fly ash can be a highly variable material. In one case, the Council Bluffs fly ash did not set within the four-hour test period, whereas the Ames fly ash reached final set in under eight minutes. Although the set times are much different, both ashes can be effective soil stabilizers.

**Table 7. XRF summary of major minerals in fly ash**

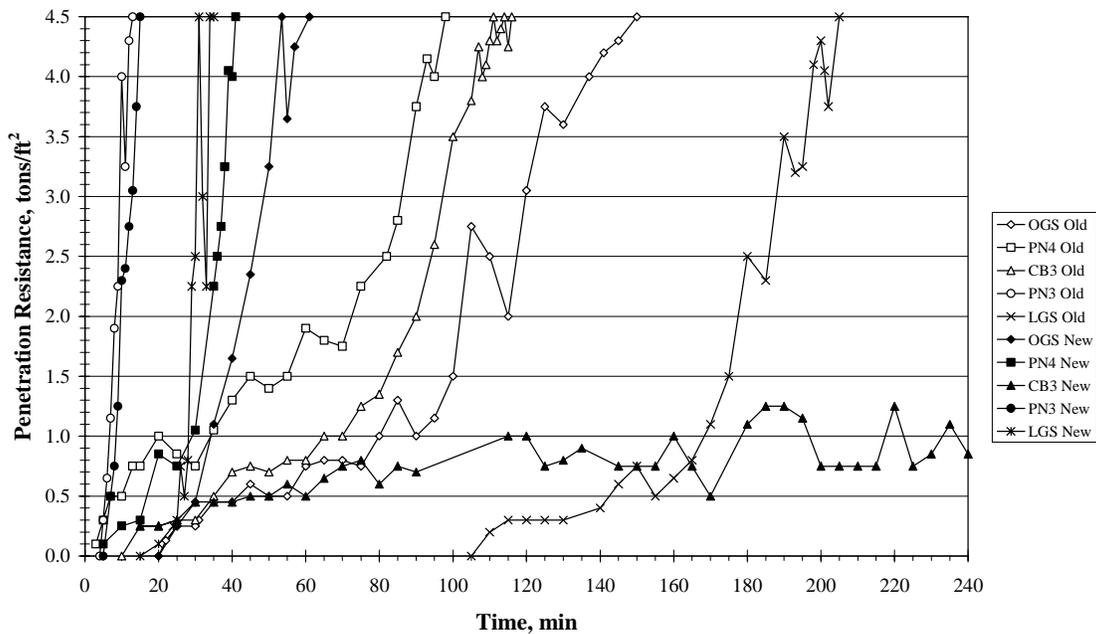
<b>Ash Source</b>	<b>Date Sampled</b>	<b>Notable Minerals</b>
CB3	8/21/01	Quartz, Tricalcium Aluminate, Anhydrite, Lime, Periclase
LGS	8/21/01	Quartz, Tricalcium Aluminate, Anhydrite, Periclase, Lime
PN3	2/23/01	Quartz, Tricalcium Aluminate, Anhydrite, Periclase, Lime
PN3	9/10/01	Tricalcium Aluminate, 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

*Fly Ash Paste Strength*

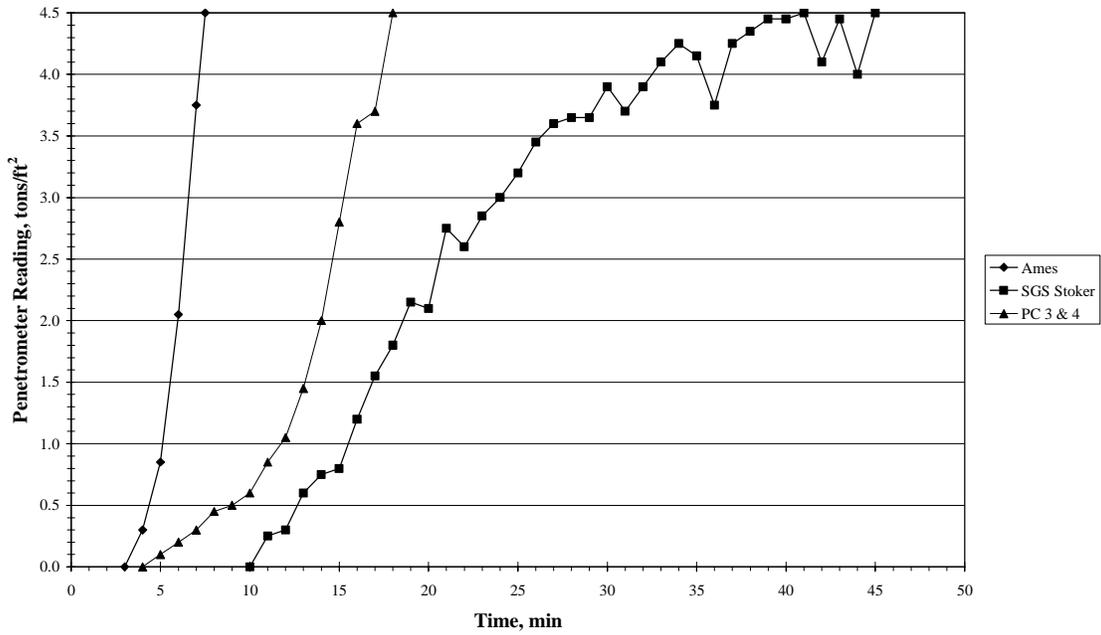
ASTM D5239 (Standard Practice for Characterizing Fly Ash for Use in Soil Stabilization) was developed to indicate the effectiveness of fly ash as a soil stabilizer. In this test, the fly ash is mixed with water at a water/ash ratio of 0.35. The paste is then formed into two-inch x two-inch cubes and allowed to cure for seven days, at which time they are tested in compression. Non-self-cementing fly ashes have strengths less than 100 psi, moderately-self-cementing ashes have strengths between 100 and 500 psi, and extremely-self-cementing fly ashes have strengths 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 Figures 17 and 18. Portland cement shows an exponential decay as the water/cement ratio increases (Kosmatka and Panarese 1994). This is not the case for fly ash, whose strengths are highly variable. At 28 days, some of the ashes did not have strengths greater than 100 psi for the higher water/ash ratios, while others reached compressive strengths of up to 3,000 psi.

**Table 8. XRF results for fly ashes**

Ash Source	CB3	LGS	PN3	PN3	PN4	OGS	SGS	PC3+4	CB	LGS	PN4	OGS	AMES
Date	8/21/0	8/21/	2/23/0	9/10/0	9/10/0	8/21/	12/13		2/23/	2/23/	2/23/0	2/23/	
Sampled	1	01	1	1	1	01	/01	5/10/02	01	01	1	01	2/15/02
Moisture	0.40	0.17	0.43	0.19	0.43	0.14	0.46	0.32	0.36	0.12	0.43	—	—
LOI	0.53	0.40	0.69	0.36	0.75	0.45	9.20	3.73	0.53	0.40	0.64	—	—
Al <sub>2</sub> O <sub>3</sub>	20.69	19.43	18.00	18.22	18.17	18.11	16.15	19.58	18.87	19.94	17.95	21.22	17.60
SiO <sub>2</sub>	34.94	34.78	32.30	32.30	32.04	34.76	26.10	37.78	35.30	35.93	32.28	36.66	33.58
Fe <sub>2</sub> O <sub>3</sub>	5.87	5.89	6.80	7.46	5.96	5.80	8.74	4.92	5.60	5.92	6.81	5.64	5.92
Sum of Oxides	61.50	60.10	57.10	57.98	56.16	58.66	50.98	62.28	59.77	61.79	57.04	63.52	57.10
Na <sub>2</sub> O	1.61	1.76	2.01	1.87	1.81	3.42	2.84	1.51	1.77	1.71	2.02	3.23	2.42
MgO	4.48	5.06	5.05	5.04	5.03	5.46	4.92	4.08	5.34	4.72	5.07	4.22	5.93
P <sub>2</sub> O <sub>5</sub>	1.52	1.30	1.04	1.25	1.21	1.10	1.63	1.04	1.11	1.33	1.05	1.42	1.09
SO <sub>3</sub>	2.14	2.14	3.08	2.35	3.04	2.29	2.82	1.71	2.43	2.00	3.10	2.17	3.48
K <sub>2</sub> O	0.44	0.38	0.34	0.32	0.30	0.38	0.63	0.52	0.44	0.46	0.34	0.52	0.52
CaO	24.54	25.90	27.51	27.81	28.47	25.43	23.97	22.23	25.54	24.69	27.56	22.25	26.78
TiO <sub>2</sub>	1.62	1.54	1.54	1.57	1.54	1.36	1.47	1.53	1.47	1.56	1.54	1.52	1.65
Mn <sub>3</sub> O <sub>4</sub>	0.02	0.02	0.06	0.02	0.03	0.02	0.02	0.03	0.03	0.04	0.06	—	—
SrO	0.44	0.45	0.38	0.42	0.44	0.46	0.34	0.38	0.44	0.42	0.38	0.41	0.30
BaO	0.76	0.78	0.76	0.82	0.80	0.83	0.73	0.64	0.76	0.76	0.76	0.74	0.73
Total	100	100	100	100	100	100	100	100	100	100	100	100	100



**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 9. Initial and final set times of fly ashes used in the study**

<b>Fly Ash</b>	<b>Initial Set, min</b>	<b>Final Set, min</b>
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

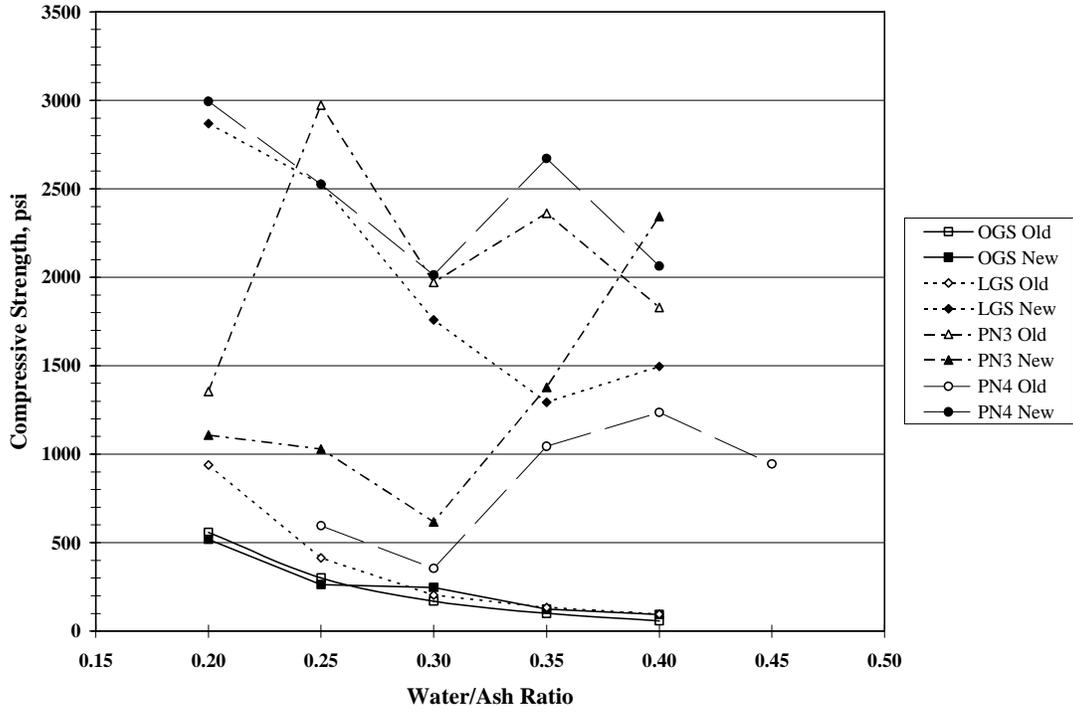


Figure 17. Class C fly ash paste strength

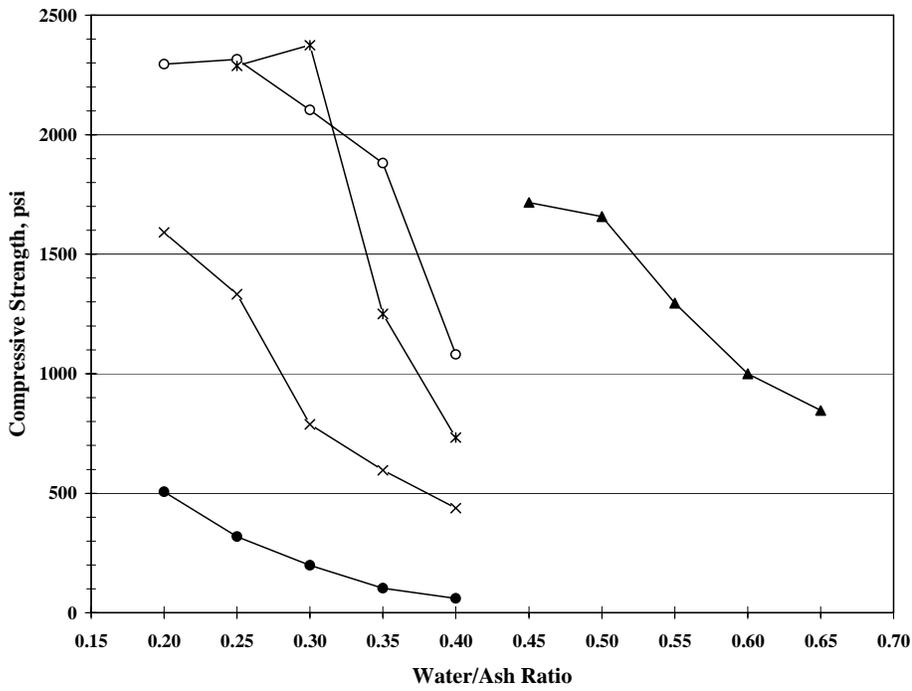


Figure 18. Class C and non-Class C fly ash paste strength

## Engineering Properties of Hydrated and Conditioned Fly Ash

### Grain Size Distribution

The grain size distributions of the HFA and CFA material were determined in accordance with ASTM C136 (Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates). Each sample was oven dried to a constant weight and then agitated through a nest of sieves using a mechanical shaker. The results are shown in Figure 19. The data show that the CFA materials have between 20% and 40% material passing the No. 200 sieve, while the HFA materials have only 4% to 14% passing the No. 200 sieve. CFA materials have higher fines content because they are not fully hydrated and cemented, as are the HFA materials. All of the CFA and HFA materials can be considered well-graded based on the grain-size distributions.

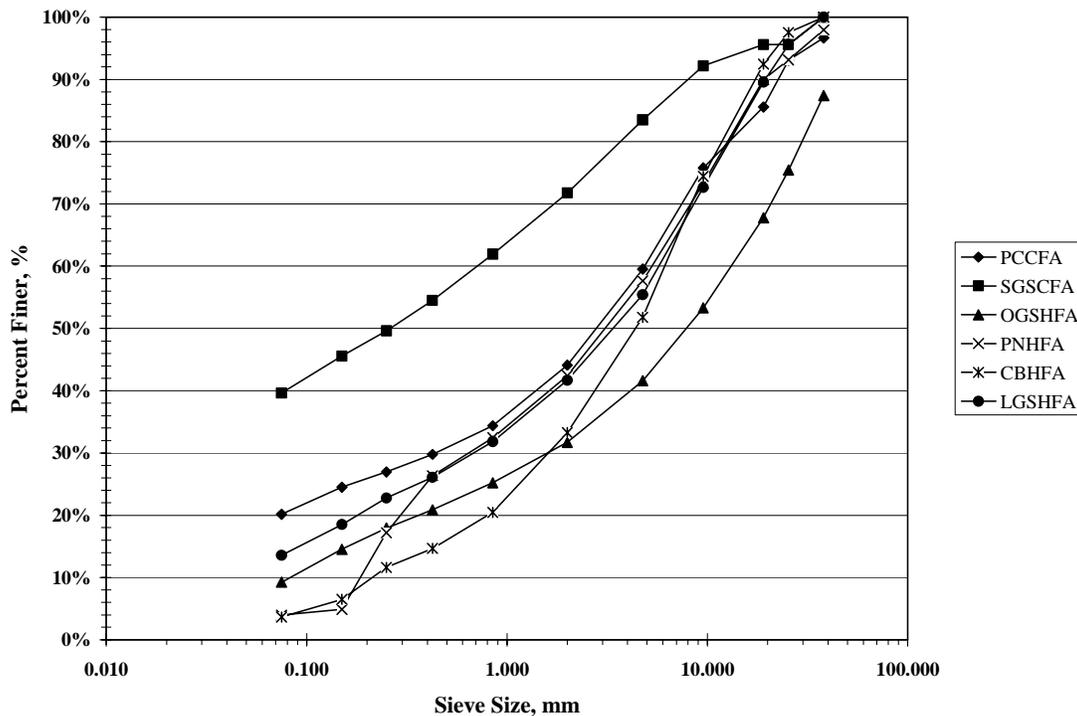


Figure 19. Particle size distribution of hydrated and conditioned fly ash

### X-Ray Analysis

X-ray analysis consisting of XRD and 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. Table 10 shows the XRF results. Minerals identified using XRD on 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, 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 available for cementitious reaction products decrease after fly ash has been hydrated; however, materials are still available for long-term pozzolanic reactions.

Results in Table 11 indicate that some of the CaO in the raw fly ash has been converted to calcite in the HFA and CFA materials. Further, 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 10. XRF results for hydrated and conditioned fly ashes (normalized for LOI)**

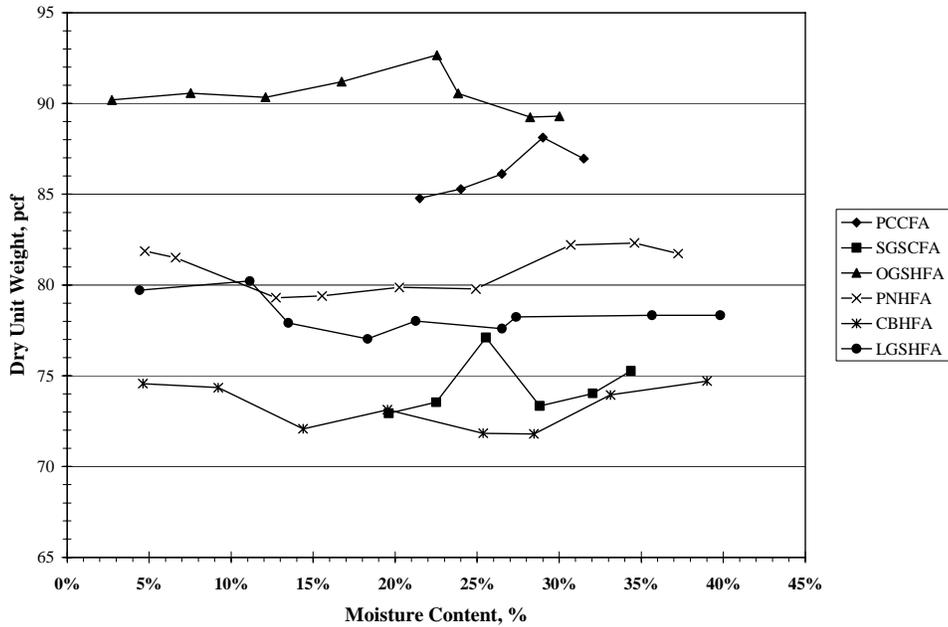
<b>Source</b>	<b>CB3 HFA</b>	<b>LGS HFA</b>	<b>PN HFA</b>	<b>OGS HFA</b>	<b>SGS CFA</b>	<b>PC CFA</b>
Date Sampled	3/23/99	5/20/99	3/23/99	11/6/98	7/5/01	6/15/01
Al <sub>2</sub> O <sub>3</sub>	17.70	19.73	18.42	20.72	17.58	19.51
SiO <sub>2</sub>	33.45	35.42	35.21	35.79	32.41	38.02
Fe <sub>2</sub> O <sub>3</sub>	6.19	5.66	5.91	5.68	7.83	5.88
Sum of Oxides	57.34	60.81	59.54	62.18	57.82	63.41
Na <sub>2</sub> O	1.77	1.82	1.96	2.73	2.54	1.66
MgO	6.19	5.02	5.05	4.44	5.55	4.49
P <sub>2</sub> O <sub>5</sub>	0.95	1.26	1.19	1.45	1.22	1.30
SO <sub>3</sub>	2.48	2.24	2.13	1.64	2.51	2.12
K <sub>2</sub> O	0.35	0.44	0.32	0.49	0.54	0.53
CaO	28.22	25.60	26.97	24.23	27.19	23.82
TiO <sub>2</sub>	1.37	1.55	1.53	1.56	1.47	1.50
Mn <sub>3</sub> O <sub>4</sub>	0.06	0.03	0.03	0.02	—	0.03
SrO	0.47	0.44	0.45	0.43	0.36	0.38
BaO	0.81	0.81	0.82	0.81	0.80	0.77
Total	100	100	100	100	100	100

**Table 11. XRD summary of major minerals in hydrated and conditioned fly ash**

<b>Source</b>	<b>Date Sampled</b>	<b>Notable Minerals</b>
CB3 HFA	3/23/99	Calcite, Tricalcium Aluminate, Quartz, Ettringite
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 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 interpreted optimum moisture contents and maximum dry densities are shown in Table 12. Only one material, the PC CFA, shows a moisture-density relationship similar to typical Proctor curves. Results show that the moisture-density relationships of the HFA and CFA are relatively flat, with only about four pcf difference of dry unit weight over a wide range of moisture contents. Dry unit weights of the materials vary from 72 pcf to 93 pcf over the range of water contents tested. 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 distinct optimum moisture contents.



**Figure 20. Moisture-density relationships of hydrated and conditioned fly ashes**

**Table 12. Optimum moisture content and maximum dry density of hydrated and conditioned fly ash**

Material	Optimum Moisture Content, %	Maximum $\gamma_d$ , 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

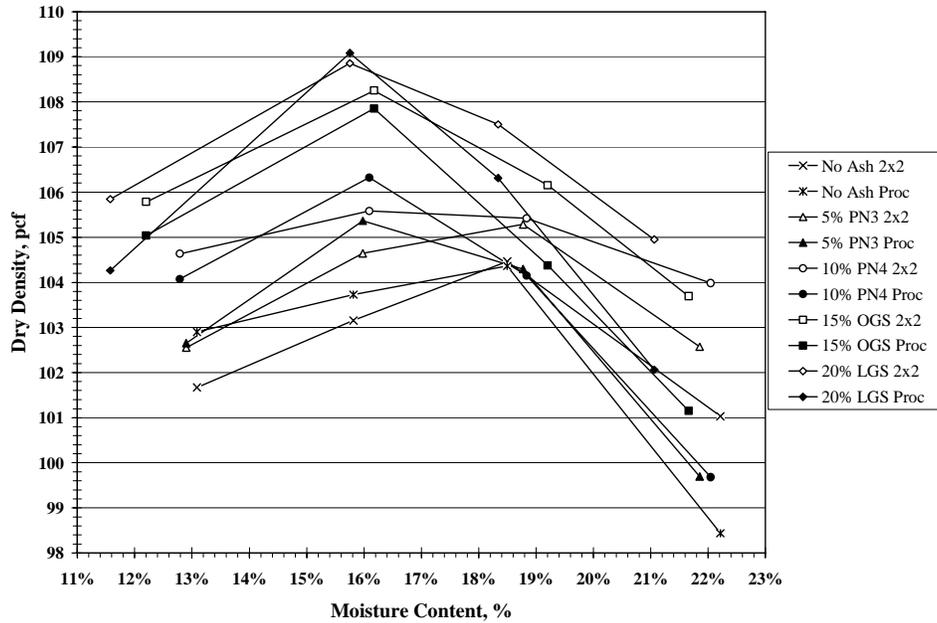
## **COMPACTION CHARACTERISTICS OF SOIL STABILIZED WITH SELF-CEMENTING FLY ASH**

### **Effect of Fly Ash Addition on Soil Moisture-Density Relationships**

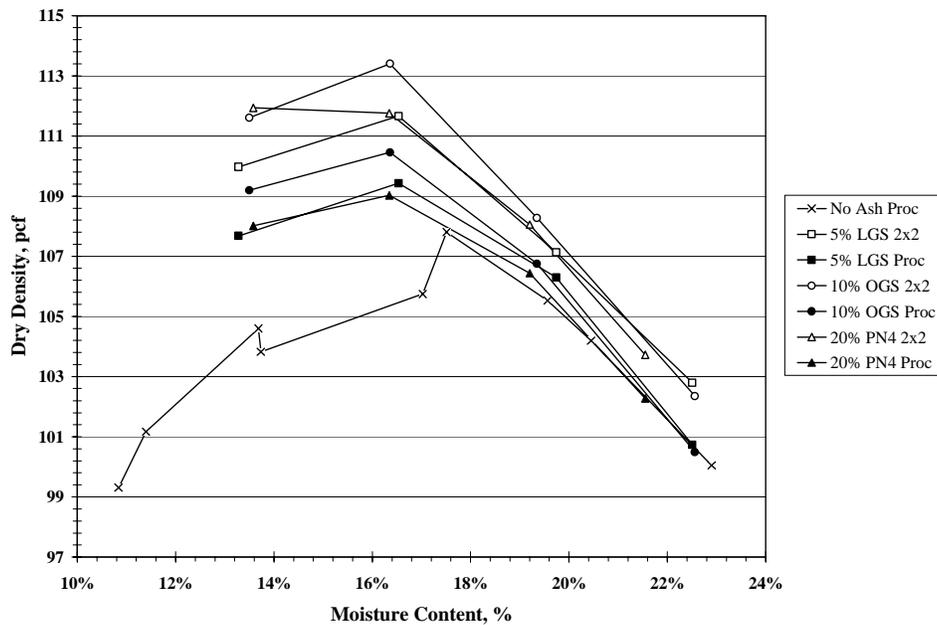
Ferguson and Levenson (1999) showed that fly ash affects the compacted maximum dry density and optimum moisture content of fine-grained soils. To study this effect in Iowa soils and fly ashes, compaction tests were performed with varying amounts of fly ash blended with Turin loess and Le Grand loess. Moisture-density relationships were determined in general accordance with ASTM D698 Method A and the Iowa State University two-inch x two-inch methods (Chu and Davidson 1960). Previous testing of fly ash stabilized soils shows that the two-inch x two-inch method correlates well with the standard Proctor compaction method (White and Bergeson 2000) and has the advantage of more rapid sample preparation. Samples were initially moistened to pre-selected moisture contents and allowed to mellow for 24 hours before adding the fly ash. Fly ash was added at rates of 5%, 10%, 15%, and 20% based on dry weight of soil. After mixing in the fly ash, samples were immediately compacted (i.e., no compaction delay).

Figure 21 shows the results of test performed on the Turin loess. Results between the two-inch x two-inch and standard Proctor methods correlate well. The maximum dry unit weight for the Turin loess with no fly ash is about 104.5 pcf with an optimum moisture content of 18.5%. For the 5% and 10% fly ash-soil mixtures, the two-inch x two-inch method and the standard Proctor method produced maximum dry unit weights of about 105.5 pcf with an optimum moisture content of approximately 19.0%. At 20% fly ash, the dry unit weight increased to about 109 pcf with an optimum water content of about 16%. Dry unit weights and optimum moisture contents for the fly ash-soil mixtures determined from the two-inch x two-inch method are approximately the same as the standard Proctor results. Further, this data set shows that with no compaction delay, fly ash content increases the maximum dry unit weight and decreases the optimum water content. Zia and Fox (2000) found a similar trend and reported that the fly ash particles increase the unit weight by filling void spaces between soil particles.

As shown in Figure 22, the Le Grand loess with no fly ash has a maximum dry unit weight of about 108 pcf with an optimum moisture content of about 17.5%. Comparison of the two-inch x two-inch compaction method with the standard Proctor methods shows that the two-inch x two-inch compaction method produced higher dry unit weights for all fly ash contents, but that the optimum moisture contents are similar across both methods. The 10% fly ash samples produce the highest dry unit weight for both test methods, while the 5% and 20% mixtures are similar. As with the Turin loess, fly ash reduces the optimum moisture content of Le Grand loess.



**Figure 21. Influence of fly ash addition on the moisture-density relationship of Turin loess determined by standard proctor and two-inch x two-inch methods**

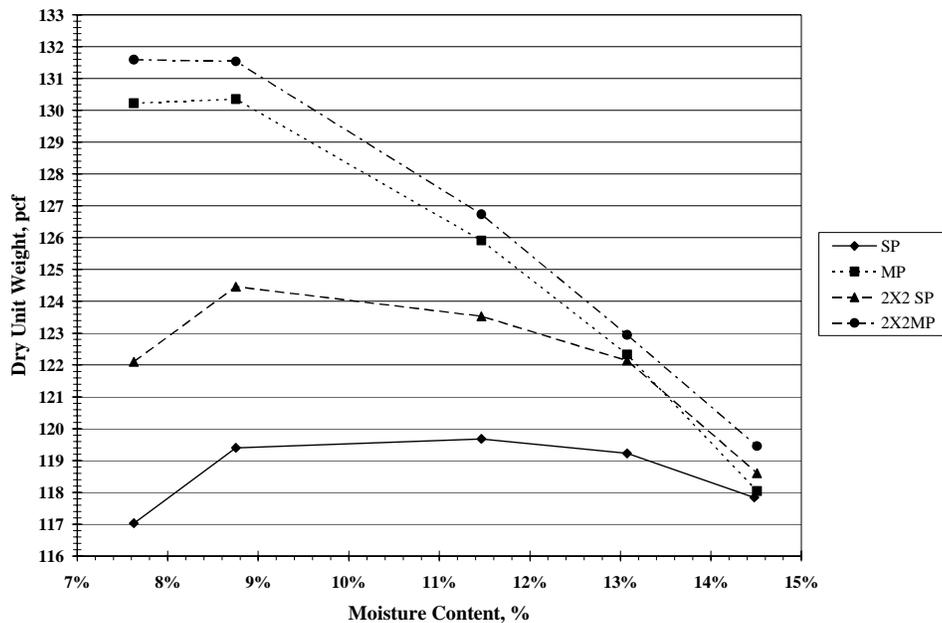


**Figure 22. Influence of fly ash addition on the moisture-density relationship of Le Grand loess determined by standard proctor and two-inch x two-inch methods**

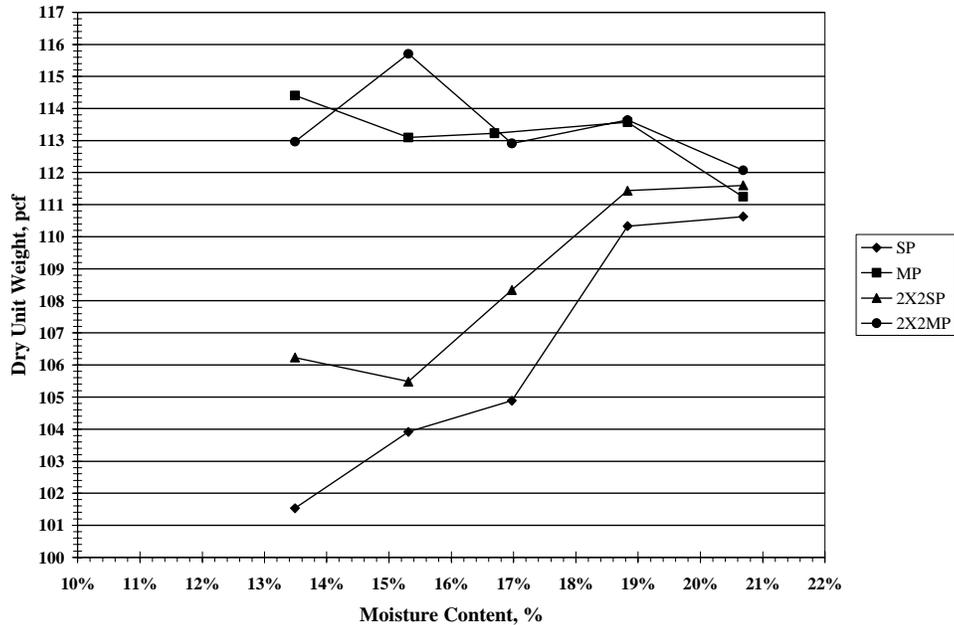
## Influence of Compaction Effort on Moisture-Density Relationships

Fine-grained soils typically show higher maximum densities and lower optimum moisture contents with increased compaction energy. Samples were prepared in accordance with ASTM D698 Method A, ASTM D1557 Method A (Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort [56,000 ft-lb/ft<sup>3</sup>]), and standard and modified two-inch x two-inch methods. Twelve percent (by dry weight) OGS fly ash was added to alluvium and glacial till soils after the soils had mellowed for 24 hours at the selected water contents. Samples were compacted immediately after mixing. Figures 23 and 24 show the till and alluvium test results, respectively.

The results for the glacial till samples show a good correlation between the two compaction methods at modified compaction energy. At the standard compaction energy, the two-inch x two-inch compaction method yields higher dry unit weights. However, at moisture contents wet of optimum, the dry unit weight curves converge. Test results for the alluvium soil show better agreement between the two compaction methods.



**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 relationships of alluvium and 12% OGS ash**

### **Influence of Compaction Delay on Moisture-Density Relationships**

From the literature review, it was learned that the dry unit weight of fly ash-soil mixtures generally decreases with increasing compaction delay time. Two sets of tests were performed to evaluate the influence of compaction delay on dry unit weight. The first test evaluated Turin loess stabilized with 20% fly ash for various fly ashes 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. Three two-inch x two-inch samples were molded at delay times of zero, one-half, one, two, and four hours. Moisture contents were not measured at each delay time, so the results are reported on a total unit weight basis. Figures 25 and 26 show the test results.

Results from Figure 25 show that a significant reduction in total unit weight (7 to 13 pcf) is possible for some fly ashes, while in Figure 26 only a moderate loss (4 pcf) occurs with increasing compaction delay time. There appears to be a link between the fly ash set time (Figures 15 and 16) and the reduction in total unit weight: as the set time increases, the potential for total unit weight reduction increases.

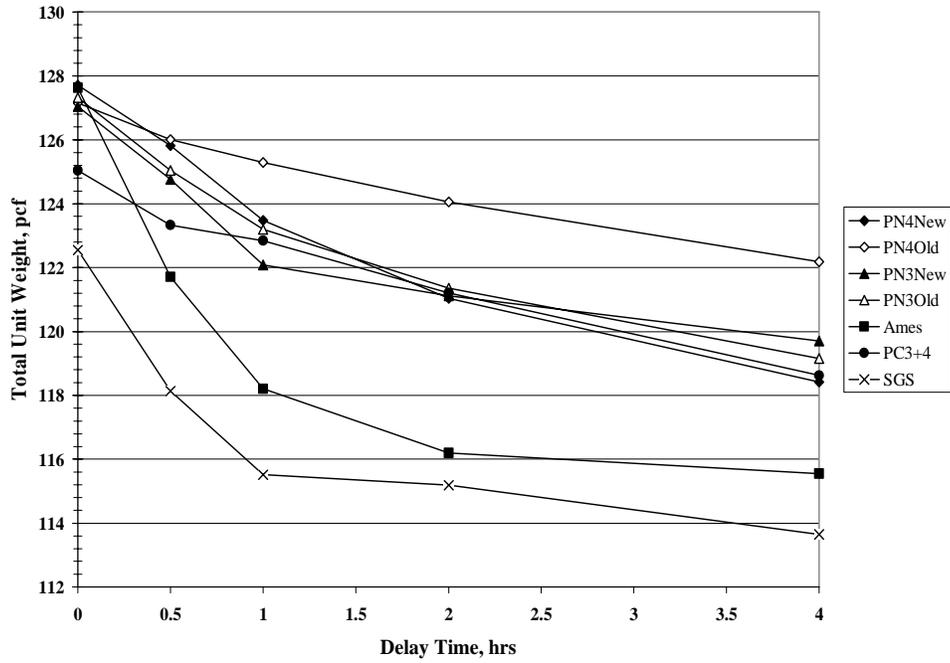


Figure 25. Influence of compaction delay on total unit weight of Turin loess and 20% fly ash

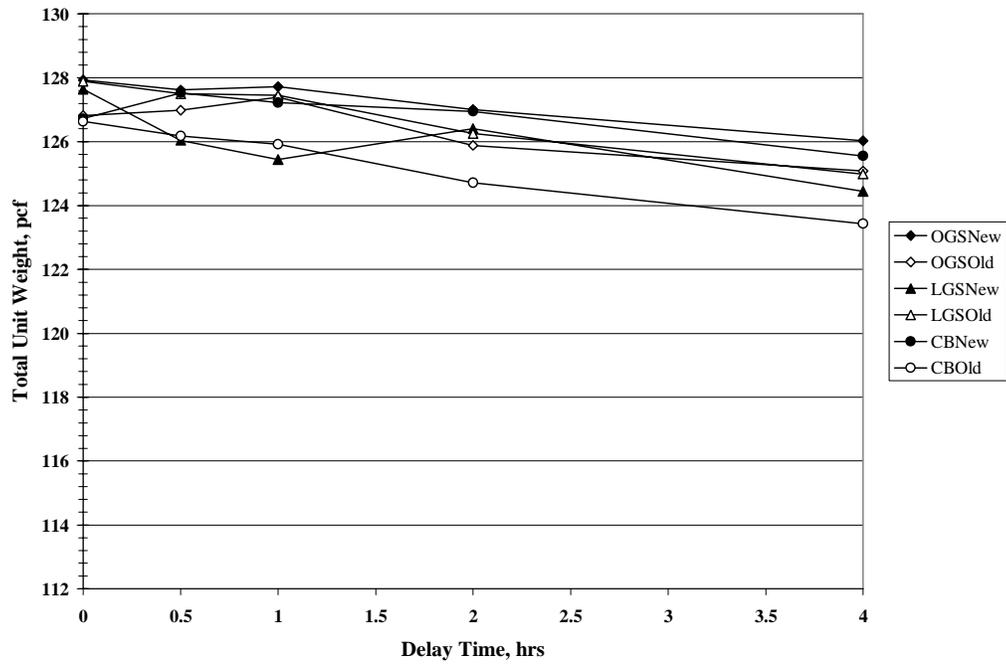
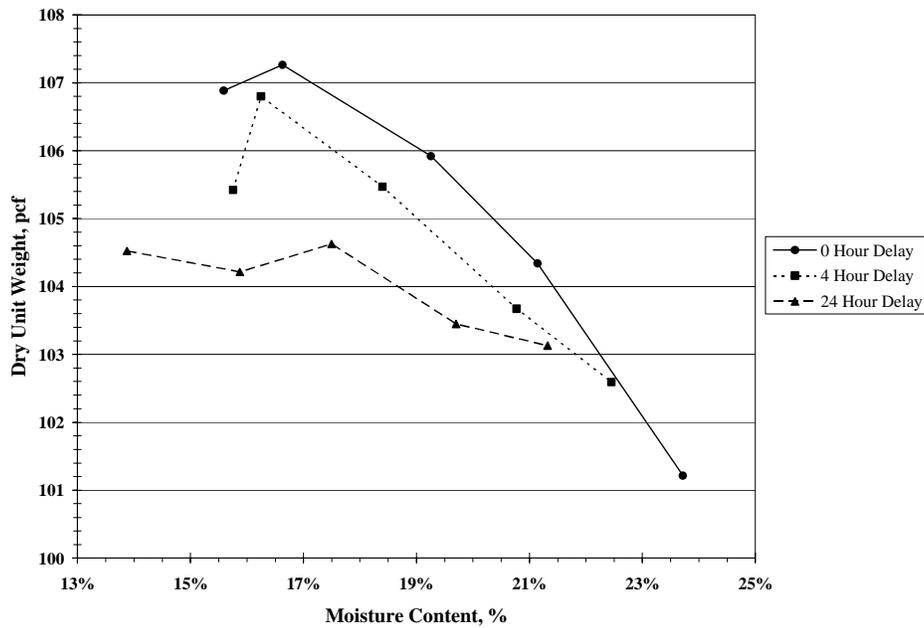


Figure 26. Influence of compaction delay on total unit weight of Turin loess and 20% fly ash

Another series of tests was performed to evaluate density loss due to long-term (24-hour) compaction delay. The Argyle paleosol was blended with 20% PC 3+4 fly ash (by dry weight). Five batches of soil were prepared at various moisture contents to produce a complete moisture-density relationship. Fly ash was added to the soil samples after 24 hours of mellowing at the selected water contents and compaction proceeded at delay times of 0, 4, and 24 hours. Three two-inch x two-inch samples were prepared for each moisture content at each delay time. The average results are presented in Figure 27. The zero-hour delay samples resulted in a typical Proctor curve. However, as delay time increased, the curves began to flatten. At 24 hours there was a 3 pcf reduction in dry unit weight.



**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 applications to increase strength and improve workability. Fine-grained soils stabilized with self-cementing fly ash have been shown to exhibit compressive strengths of 100 psi to 500 psi (Ferguson 1993). The strength of stabilized mixtures depends on a number of factors, including compaction effort, compaction delay, and the percentage of fly ash addition. To illustrate how each of these factors affects strength, this section of the report describes tests conducted to measure strength under the various conditions. Because strength can be measured in several ways, strength comparisons are made using CBR, true UCS ( $L/D=2$ ), two-inch x two-inch UCS ( $L/D=1.0$ ), and Proctor sized UCS ( $L/D=1.15$ ).

### **Influence of Compaction Effort on Strength Gain**

To investigate the influence of compaction effort on strength gain, Proctor-sized and two-inch x two-inch samples were compacted, extruded from their molds, sealed, and cured in an oven at 100° F for 7 days. Both the alluvium and glacial till samples were prepared with 12% OGS fly ash by dry weight. At the end of the seven-day cure period, unconfined compression tests were performed at a loading rate of 0.05 in./min. Samples were not soaked prior to testing. The hypothesis of this experiment was that increased compaction effort (modified Proctor) would produce higher densities, which in turn would increase compressive strengths. Figures 28 and 29 present the test results for the alluvium and glacial till samples, respectively.

As shown, increasing the compaction energy did increase the strength for some samples, but the strength increase was limited to the lower moisture contents evaluated for both soils. Comparison of the standard Proctor and modified Proctor compaction energies shows that as the water content increases, the benefit of increased compaction energy is lost. At the highest water contents, compaction energy appears to have no measurable effect on compressive strength. In general, the two-inch x two-inch samples yielded higher strengths than the Proctor-sized samples.

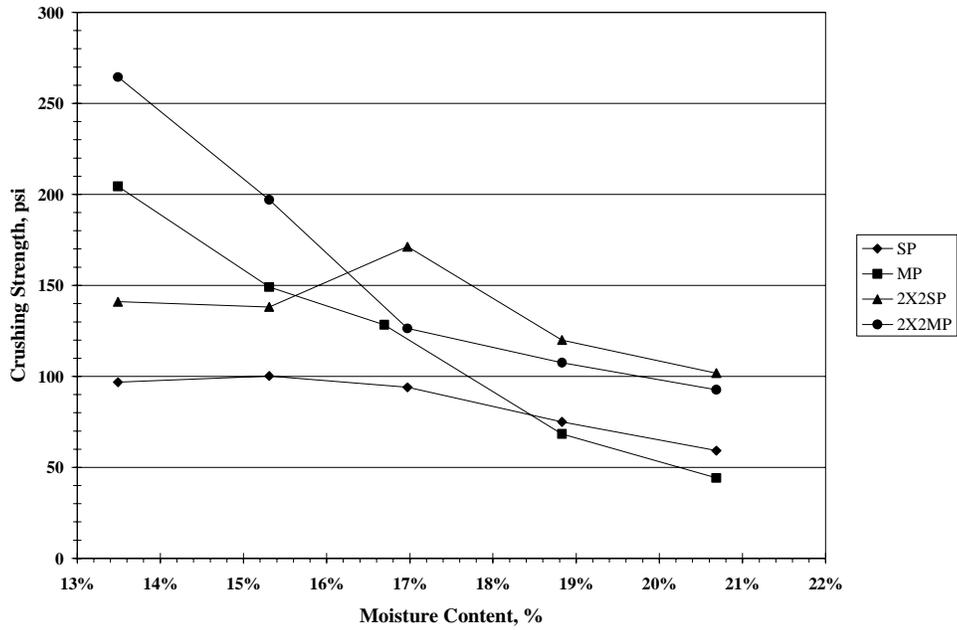


Figure 28. Influence of compaction energy on alluvium and 12% OGS ash

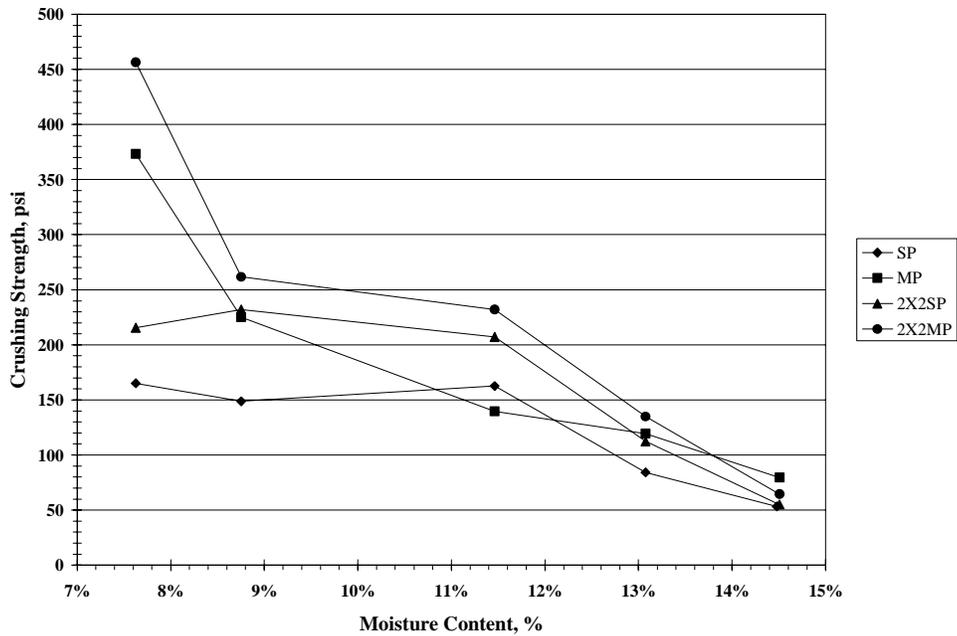


Figure 29. Influence of compaction energy on glacial till and 12% OGS ash

## **Influence of Compaction Delay on Strength Gain**

In the first study, two-inch x two-inch ( $L/D=1.0$ ) samples of Turin loess containing 20% fly ash (by dry weight) were molded at approximately 17.5% moisture content. The delay times were zero, one-half, one, two, and four 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.

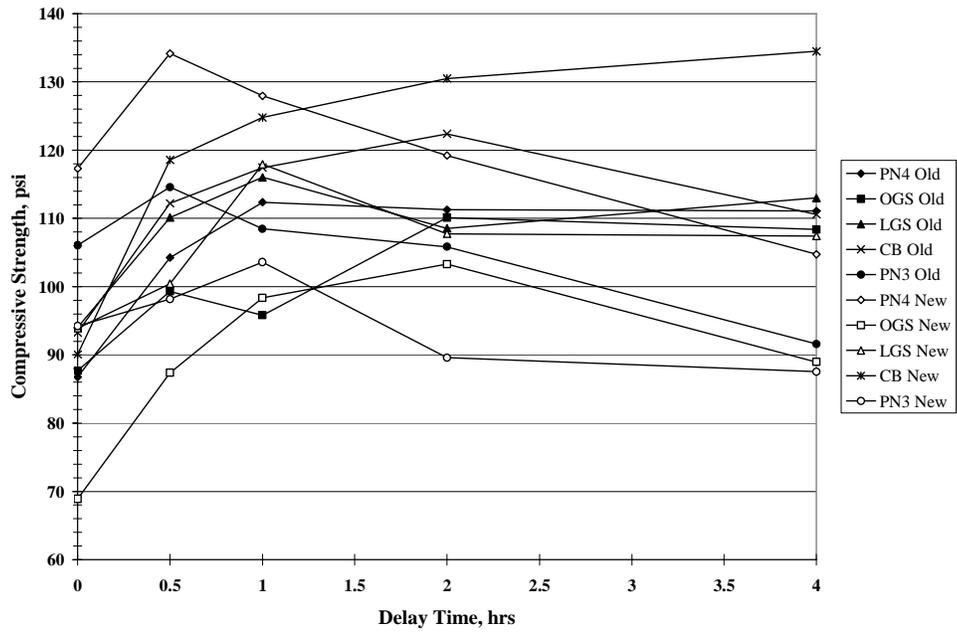
The results do not show an immediate loss of strength for delay times of up to one half-hour. All but one of the ten Class C fly ash samples exhibit a strength gain for one or two hours, and then show a strength decrease. Compressive strength starts to decrease when the mixture is compacted after the fly ash set time. One set of samples made with CB fly ash did not show a strength loss at any delay time. This is likely because the fly ash has a final set time of greater than 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 compressive strength versus compaction delay results for the non-Class C fly ashes used in these tests. The PC3+4 samples do not show much strength loss, even though they had a set time of 18 minutes. The Ames and SGS samples show an immediate strength loss.

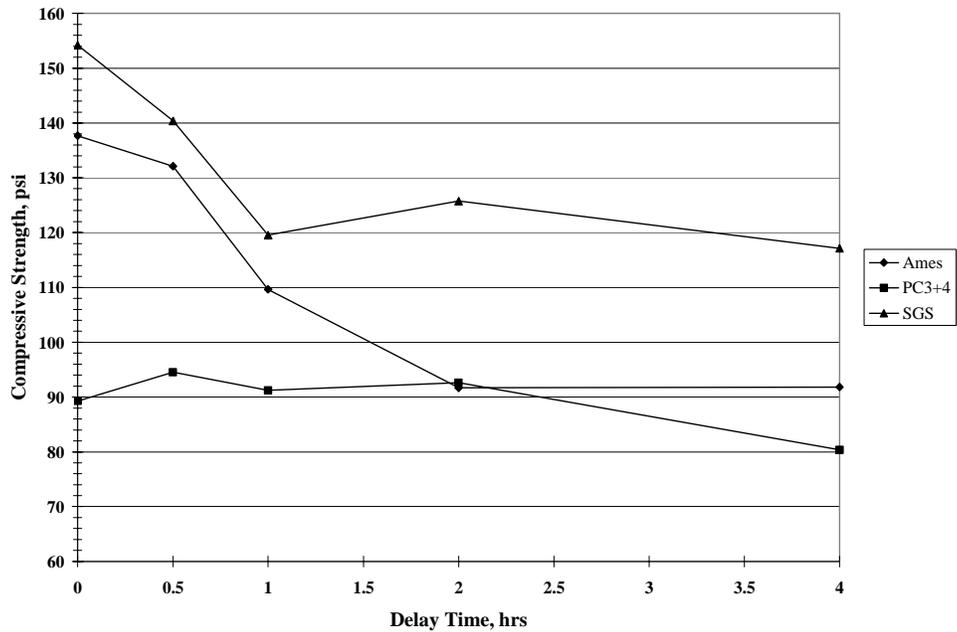
A second set of tests was conducted to measure strength loss with compaction delay. In these tests 20% PC3+4 fly ash (by dry weight) was mixed with paleosol at varying moisture contents. Samples were compacted using the two-inch x two-inch apparatus ( $L/D=1.0$ ) at times of 0, 4, and 24 hours. Three samples were compacted for each moisture content and 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 one hour in water at room temperature. The test results are shown in Figure 32.

For the most part, results show an overall decrease in strength as compaction delay time increased. The exception is noted for the samples at 18.4% moisture at 4 hours, which yielded the highest overall compressive strength for this test set.

All of the 24-hour samples are shown after soaking in Figure 33. The samples at the lowest water content (13.9%) slaked during testing. The second set of samples from the top show some cracking due to swelling, which also contributed to lower strengths. Overall, there is strength decrease at 24 hours compaction delay, compared to the 0- and 4-hour delays.



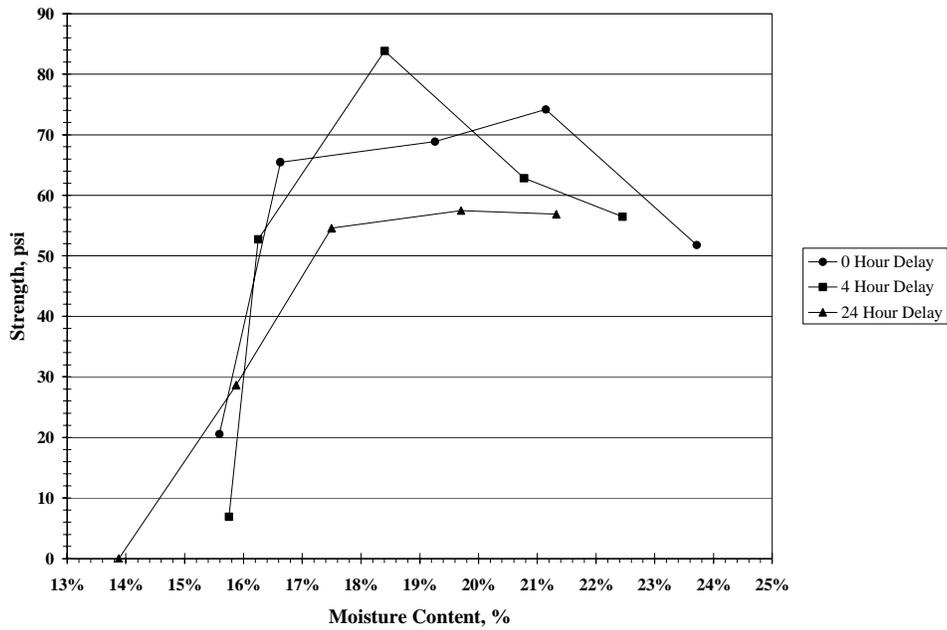
**Figure 30. Influence of compaction delay on strength gain of Class C fly ash and Turin loess**



**Figure 31. Influence of compaction delay on strength gain of non-Class C ash and Turin loess**

**Table 13. Summary of compressive strength results as a function of compaction delay time**

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



**Figure 32. Influence of long-term compaction delay on the strength gain of paleosol and 20% PC3+4 fly ash**



**Figure 33. 24-hour compaction delay samples after soaking (molded moisture content increases from top to bottom)**

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

On August 29, 2002, two-inch x two-inch samples were measured to determine volume change and tested for compressive strength. These samples had been molded in 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. Samples were cured in a humidity room at room temperature. The strength results are presented in Figure 34, with time plotted on a logarithmic scale.

All of the samples, except for the UNI atmospheric fluidized bed combustion (AFBC) residue and PC stoker samples, show an increase in compressive strength throughout the cure period (about 900 days). According to the XRF results, the AFBC residue contains 29%  $\text{SO}_3$  while the PC stoker contains 6%  $\text{SO}_3$ . Due to the high sulfur content in the AFBC, ettringite formed and is believed to have caused volume expansion and subsequent strength loss. Figure 35 shows evidence of ettringite, indicated by the needle-like crystals. The SEM image of the PC3+4 20% sample in Figure 36 also indicates presence of ettringite, but to a lesser extent. Additionally, some of the glassy fly ash spheres are still intact.

The percent 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 began to show signs of cracking and delineation. Also shown in Table 14 is the final percent strength gain (or loss), calculated as a percentage of the seven-day strengths.

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 sulfur is not present to form expansive minerals.

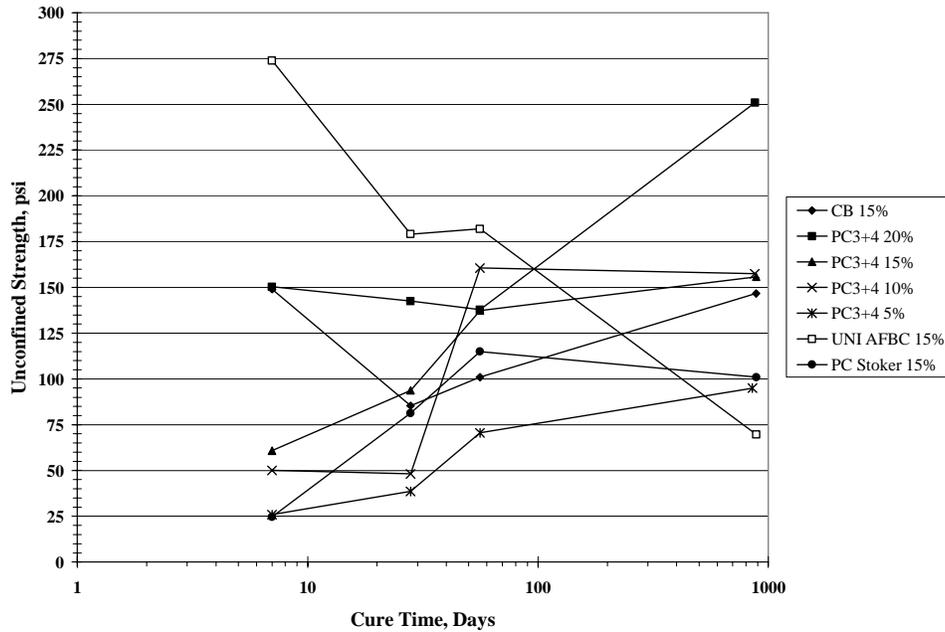


Figure 34. Long-term strength gain of glacial till and fly ash

Table 14. Volume change and strength gain of stabilized glacial till after 900 days

Ash Type	Addition Rate	New Volume, in <sup>3</sup>	Initial Volume, in <sup>3</sup>	Volume Change, in <sup>3</sup>	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

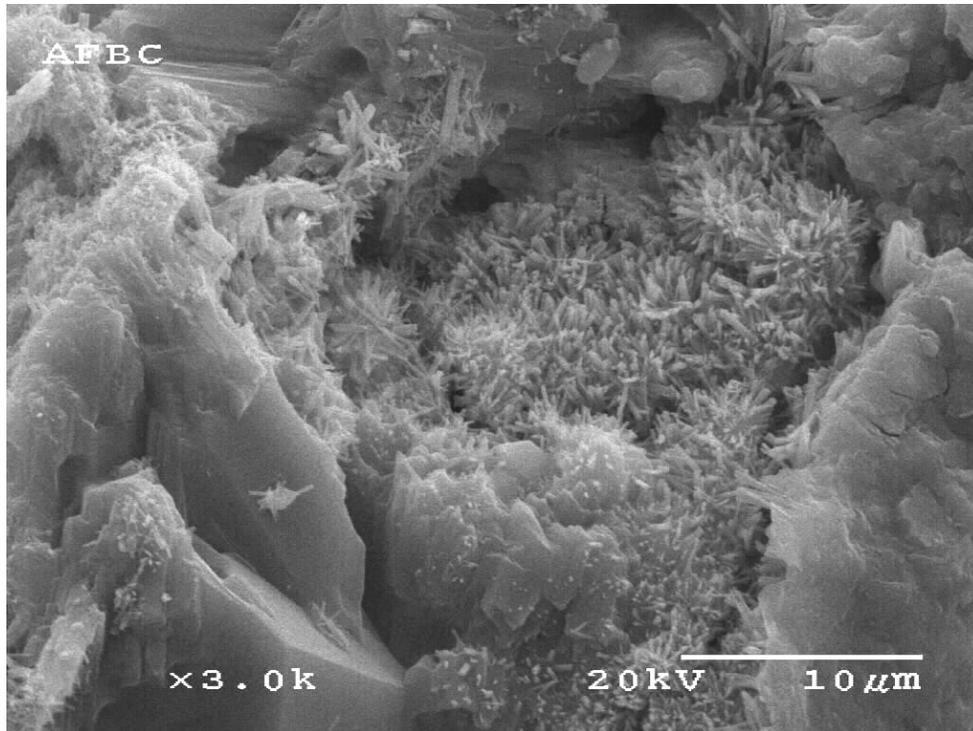


Figure 35. SEM image showing ettringite formation in AFBC-stabilized glacial till

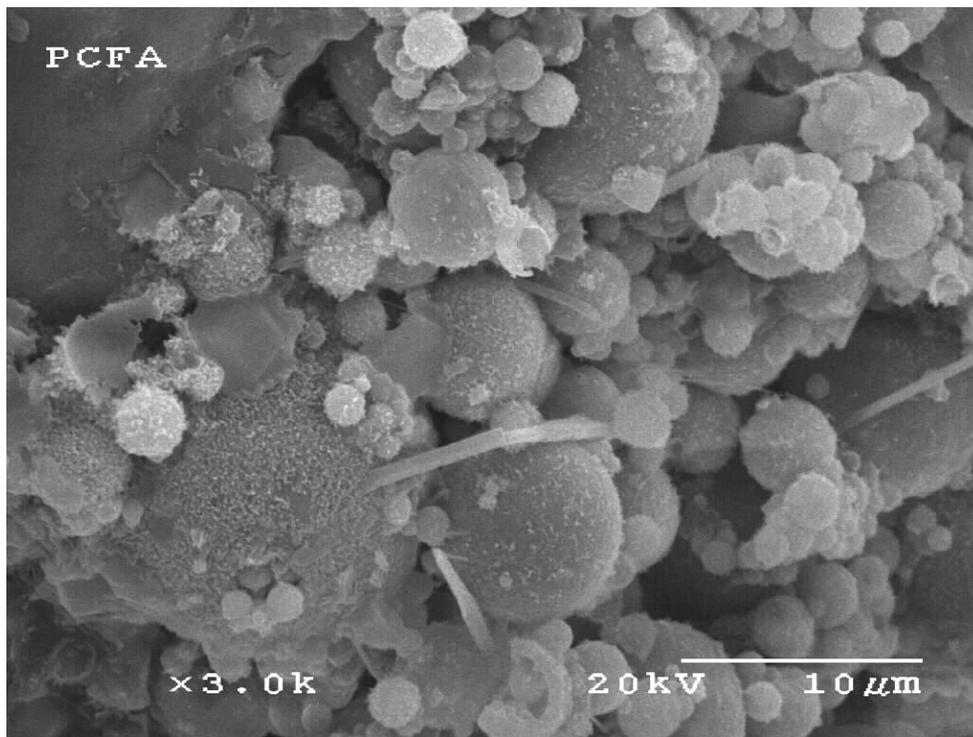


Figure 36. SEM image of glacial till stabilized with 20% PC 3+4 fly ash

## Comparison of Unconfined Compressive Strength Methods for Soil Stabilized with Self-Cementing Fly Ash

Unconfined compressive strength tests can be performed using a number of sample geometries. For practical purposes, this study investigated UCS measurements on three different sample sizes. Measurement of the *true* unconfined compressive strength is considered for samples having a length to diameter ratio of 2.0. To achieve  $L/D = 2.0$  in this study, samples were molded with diameter of 2.8 inches and length of 5.6 inches. Compressive strength was also measured on samples compacted in a standard 4-inch diameter Proctor mold with a height of 4.584 inches ( $L/D=1.15$ ). For Proctor-sized samples, ASTM D1633 (Standard Test Method for Compressive Strength of Molded Soil-Cement Cylinders) was followed. The third set of samples was compacted using the two-inch x two-inch apparatus ( $L/D = 1.0$ ). The Iowa State University two-inch x two-inch method is described by Chu and Davidson (1960). For comparison purposes, the soils and fly ashes were all mixed in the same manner.

Most samples were produced from the Le Grand and Turin loess soils, with some samples using the alluvium and paleosol. True unconfined, Proctor-sized, and two-inch x two-inch 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 with 15% CB fly ash. Proctor and two-inch x two-inch samples of Turin loess with 5% (PN3), 10% (PN4), 15% (OGS), and 20% (LGS) fly ash were also molded. The last set of samples was made with the paleosol and 12% PC3+4 fly ash in true unconfined and two-inch x two-inch sizes.

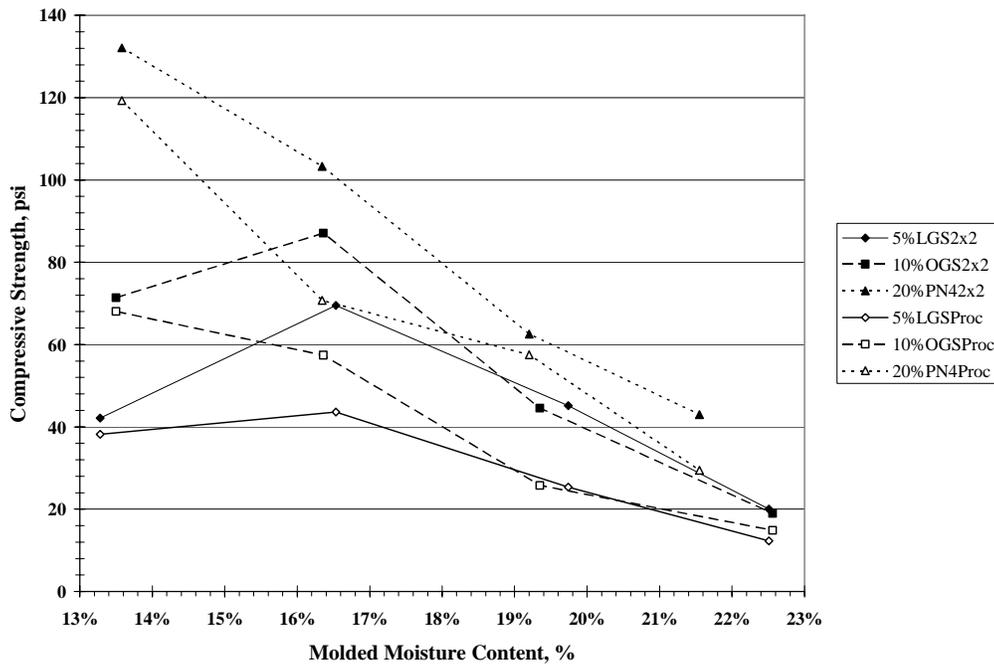
Soils were mixed at a range of pre-selected moisture contents to produce moisture-strength relationships for the various addition rates of fly ash. One Proctor-sized, two two-inch x two-inch cylinders, and two 2.8-in x 5.6-in specimens were molded for each moisture content. All samples were compacted with a delay of less than 20 minutes. After molding, each sample was extruded, sealed in plastic wrap, and cured.

The Proctor and two-inch x two-inch 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 the cure times, the samples were soaked in a water bath at room temperature and tested in unconfined compression at a strain rate of 0.05 in./min. The Proctor-sized and true unconfined samples were soaked for four hours while the two-inch x two-inch 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 correlates to Proctor-sized sample strengths for soil-cement mixtures.

The two-inch x two-inch 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 as moisture content increases. Ferguson (1993) stated that optimum moisture content for strength is typically 0% to 8% below optimum moisture for density. The results of these tests show that higher strengths are achieved at lower water contents. Furthermore, for both soil types it is observed that at high moisture contents, the fly ash addition rate has little influence on strength.

The true UCS results for the Le Grand loess are shown in Figure 39. At 5% and 10% fly ash content, there appears to be an optimum moisture content to achieve maximum strength (~ 17%). At 20% fly ash content, the strength increases with decreasing water content for the range of water contents tested. Soaking appears to have no adverse effects on the 20% fly ash samples.

Results for alluvium with 15% CB fly ash are shown in Figure 40. These samples all exhibit optimum water content for the achieved maximum strength. The paleosol samples shown in Figure 41 exhibit a similar trend. The results also show that the two-inch x two-inch samples generally yield higher strengths than the true UCS samples. Overall, soaking the samples produces lower strength than that of testing without soaking. The soaking was used to simulate saturated field conditions.



**Figure 37. Compressive strength of two-inch x two-inch and Proctor samples of Le Grand loess and fly ash**

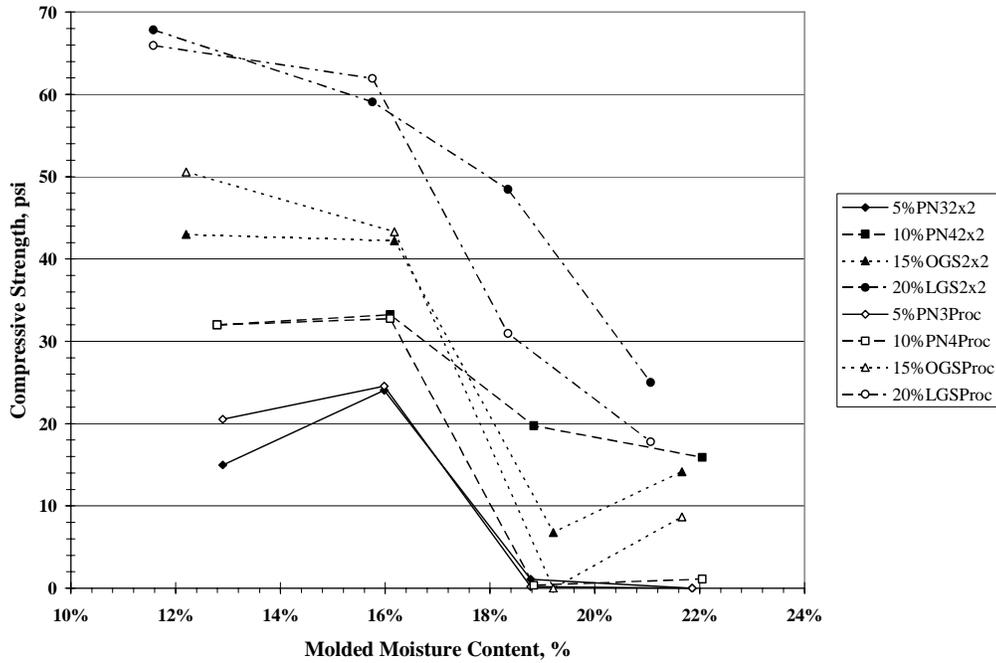


Figure 38. Compressive strength of two-inch x two-inch and Proctor samples of Turin loess and fly ash

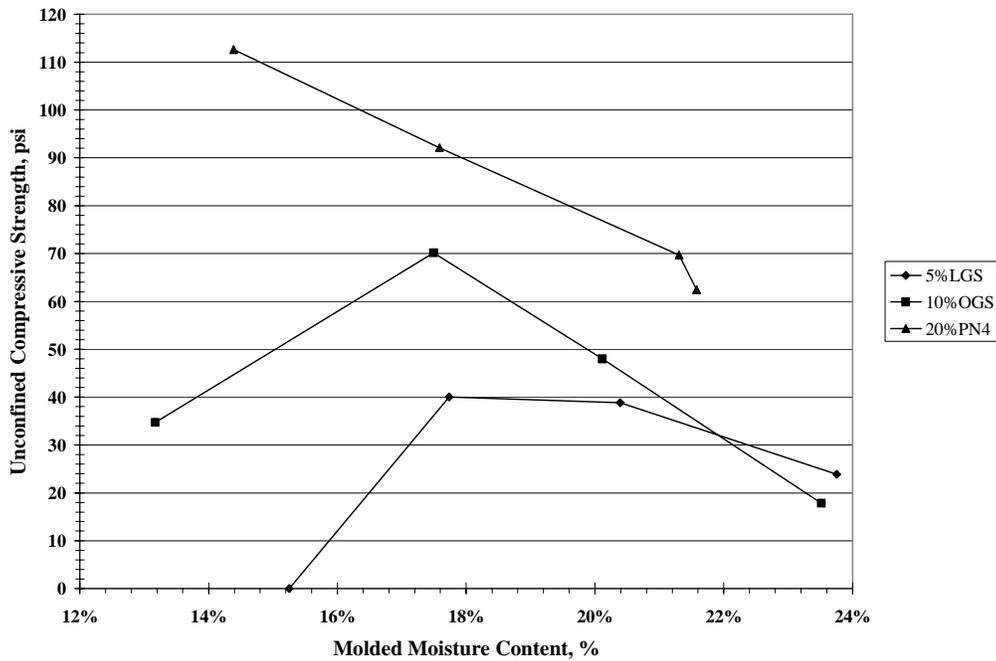
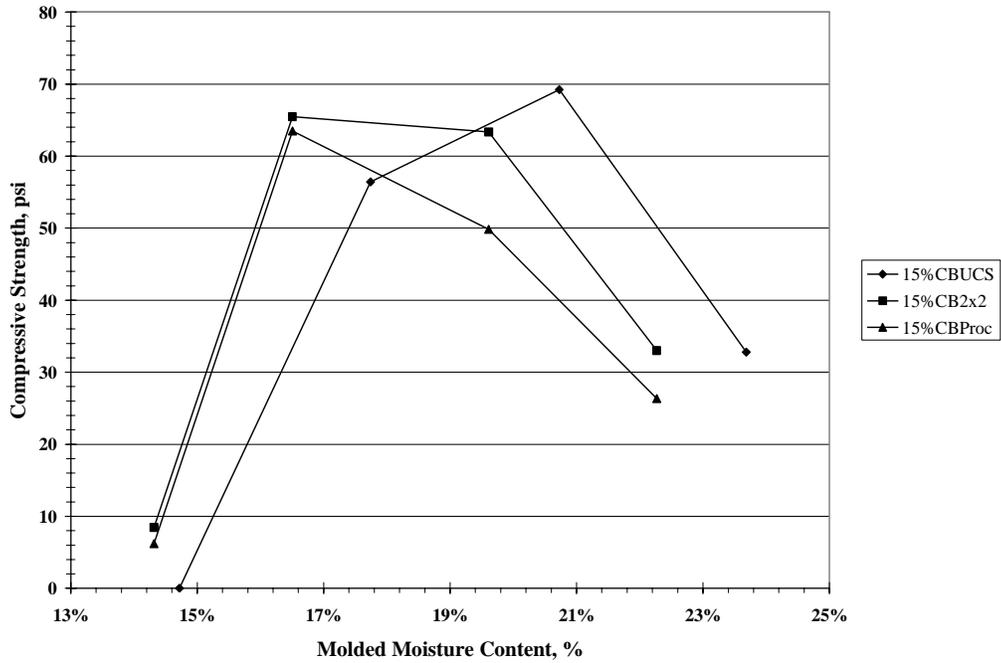
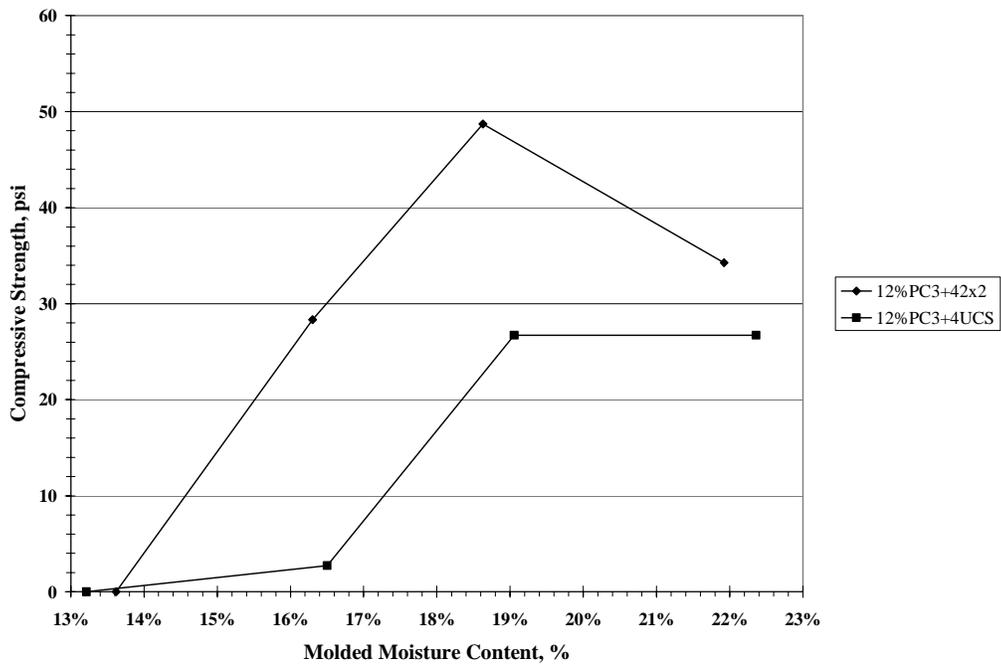


Figure 39. Unconfined compressive strength of Le Grand loess and fly ash

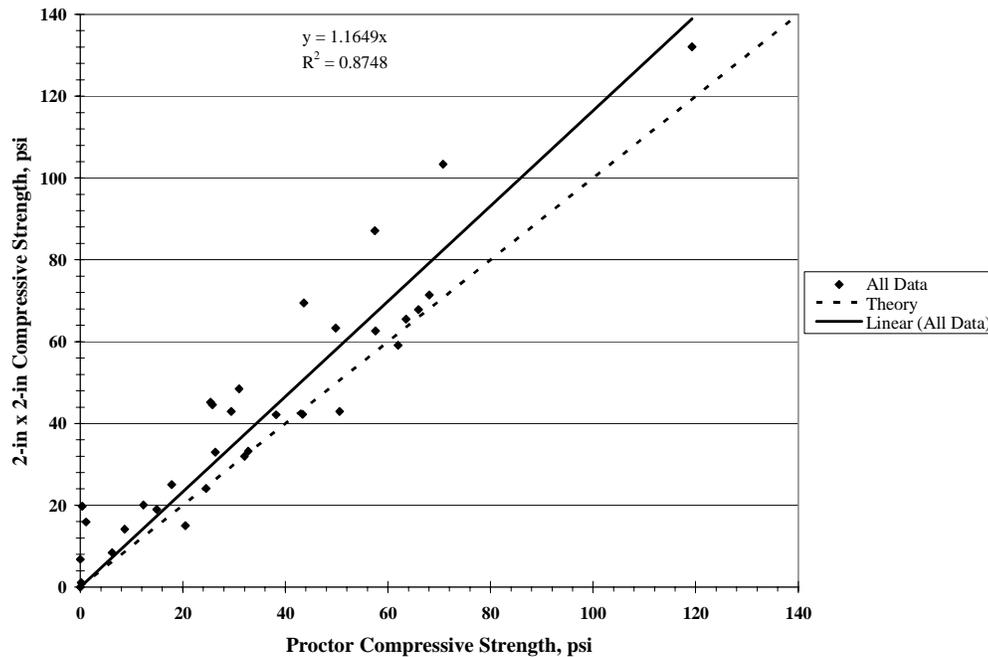


**Figure 40. Strength results of alluvium stabilized with CB fly ash**



**Figure 41. Two-inch x two-inch and UCS strength of paleosol and fly ash**

All of the UCS data is plotted with correlations developed to factor in sample size. Figure 42 shows a regression between the two-inch x two-inch and Proctor-sized samples. This data set produces an  $R^2$  of 0.87 and shows a good correlation between the two sizes of samples. These results indicate that strength determined from the two-inch x two-inch size specimens is about 1.16 times the Proctor-sized specimens.



**Figure 42. Correlation between two-inch x two-inch and Proctor compressive strengths**

Additional correlations of two-inch x two-inch and Proctor with true UCS are shown in Figures 43 and 44, respectively. The two-inch x two-inch samples have an  $R^2$  of 0.70 when plotted against true UCS and are about 1.11 times stronger than true UCS. For the Proctor-sized specimens, the  $R^2$  is 0.60 and the Proctor-sized specimens yielded a lower strength (0.87 times) than the true UCS. There is data scatter in both figures. The best fit equations show that the UCS must be reduced to meet Proctor strengths and increased to meet two-inch x two-inch strengths. ASTM D1633 states that multiplying the UCS of soil-cement by 1.10 gives an approximation of the Proctor compressive strength for the same material. To compare them to ASTM D1633, the UCS strengths were corrected by multiplying by 1.10 (Figures 45 and 46). Figure 45 shows the two-inch x two-inch data and Figure 46 shows the Proctor data. The  $R^2$  values are similar for each set of data, as before, but in this case the corrected UCS must be further reduced to match the Proctor strengths. 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 two-inch x two-inch samples. Overall, a correlation seems to exist between the sample sizes, but more data is needed to predict the true correlations more accurately. These results, although variable, give credence to using any of the three methods in the design of fly-ash soil mixtures.

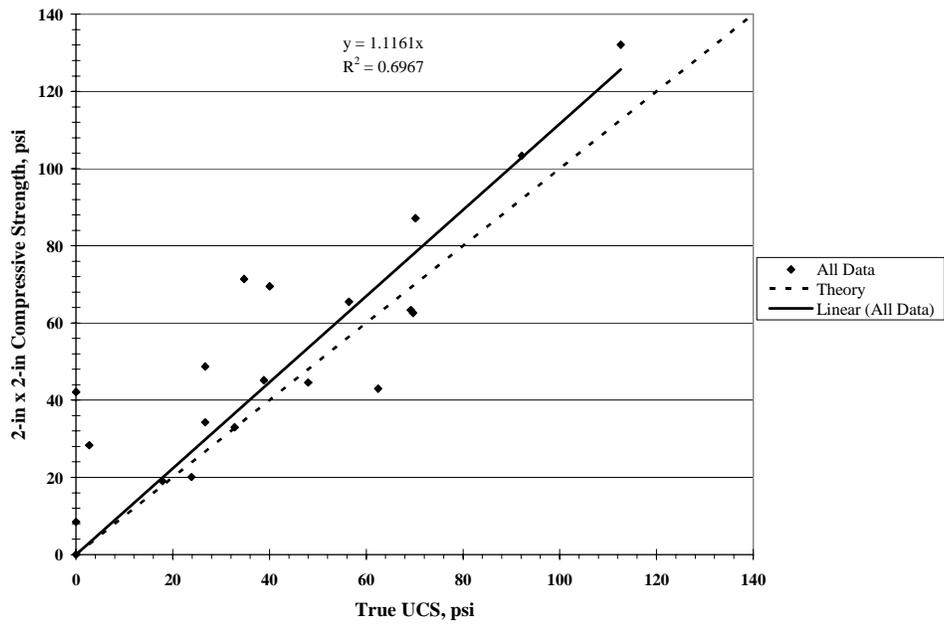


Figure 43. Correlation between two-inch x two-inch compressive strength and true UCS

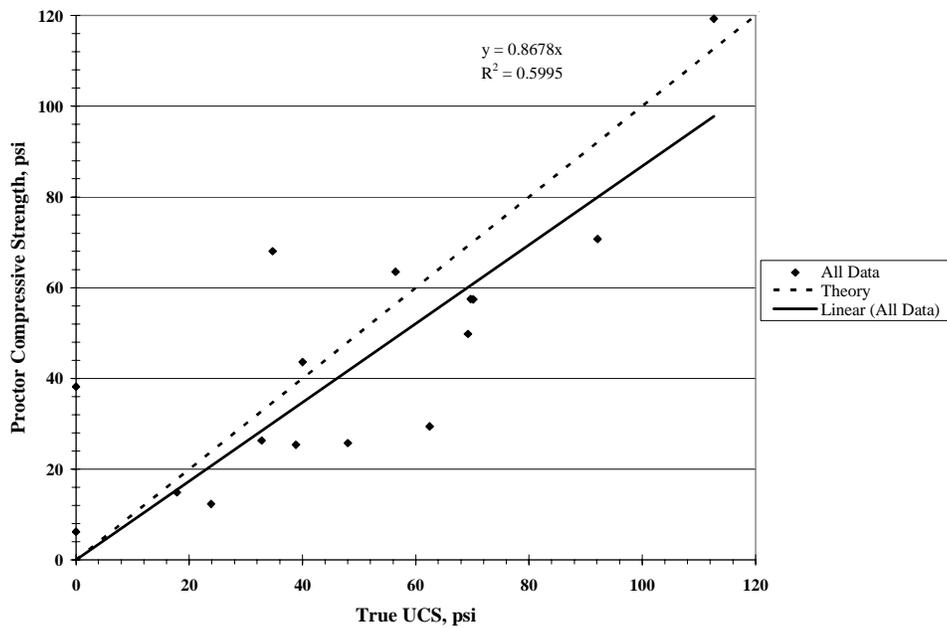
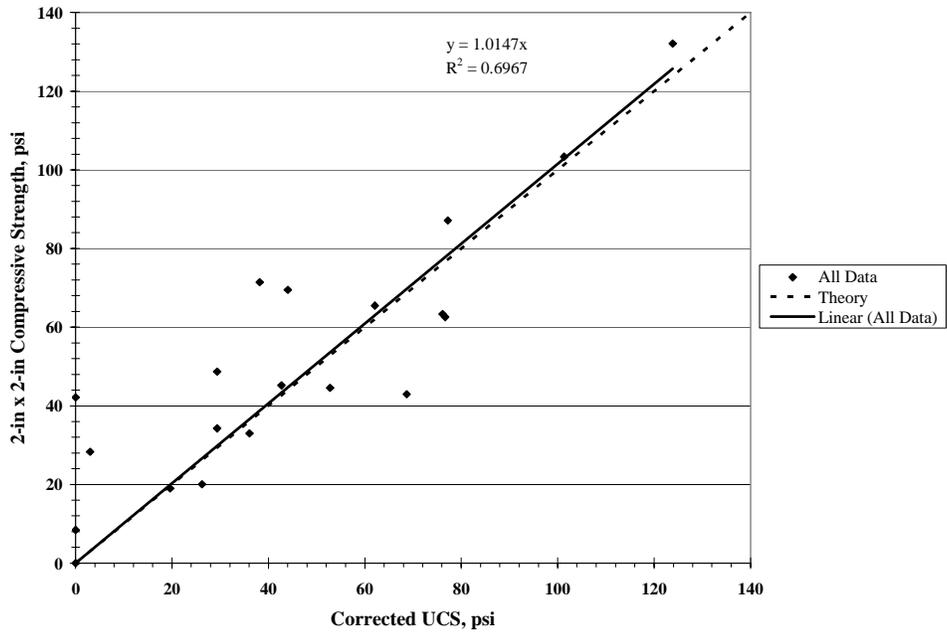
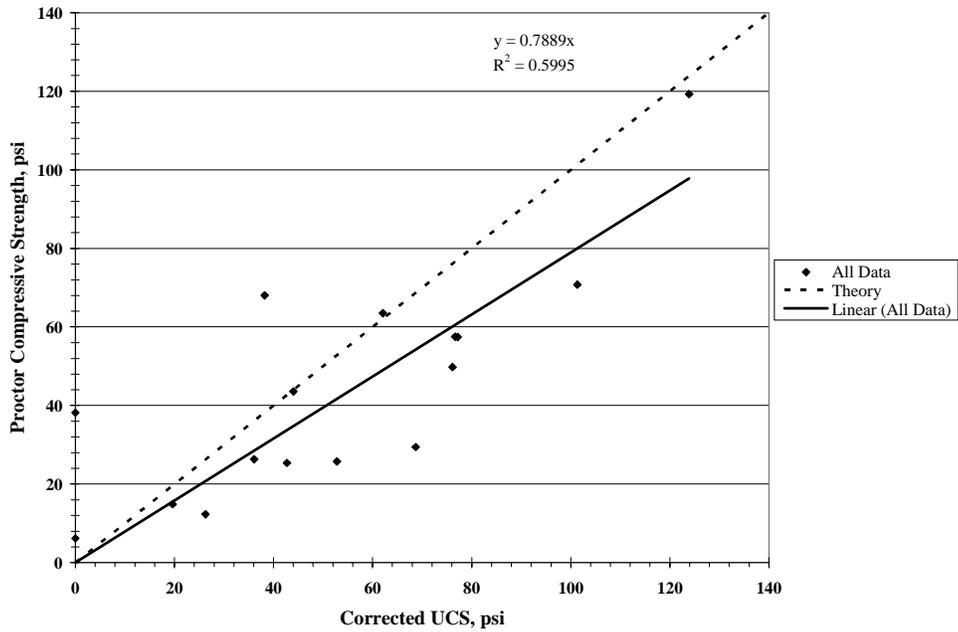


Figure 44. Correlation between Proctor compressive strength and true UCS



**Figure 45. Correlation between two-inch x two-inch compressive strength and corrected UCS**



**Figure 46. Correlation between Proctor compressive strength and corrected UCS**

## California Bearing Ratio for Self-Cementing Fly Ash-Stabilized Soil

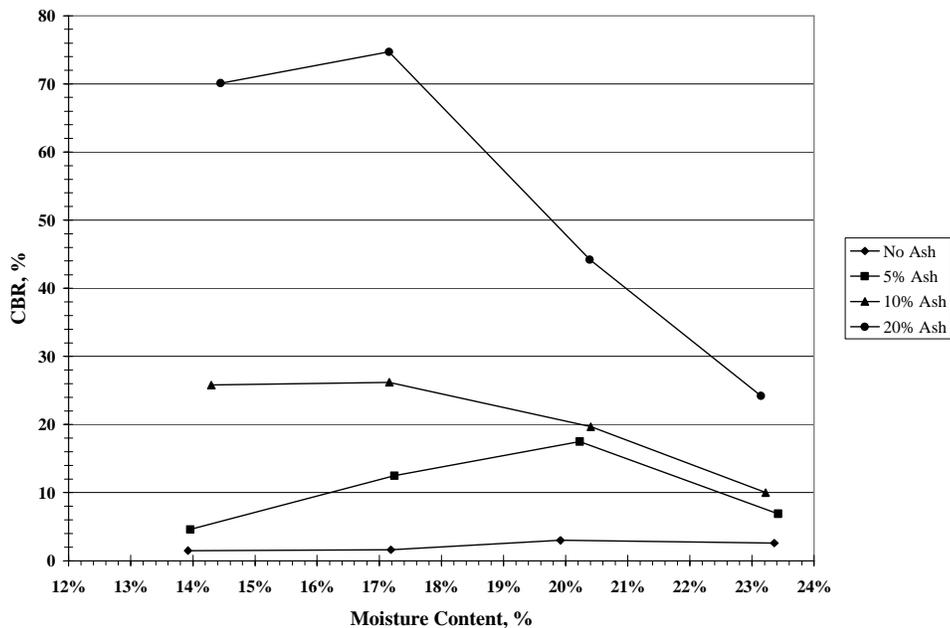
A series of CBR tests were conducted to determine CBR-moisture relationships for soil stabilized with fly ash. A correlation between CBR and the three unconfined compressive strength test methods was attempted. The Le Grand loess was evaluated in this effort. CBR testing was carried out in general compliance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

**Table 15. Summary of swell values and CBR of Le Grand loess and fly ash**

<b>Sample #</b>	<b>Molded Moisture Content, %</b>	<b>Ash Type</b>	<b>Addition Rate, %</b>	<b>Soaked CBR, %</b>	<b>Swell, %</b>
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 largest volume change, as was expected. Also, samples compacted near 14% moisture content 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.

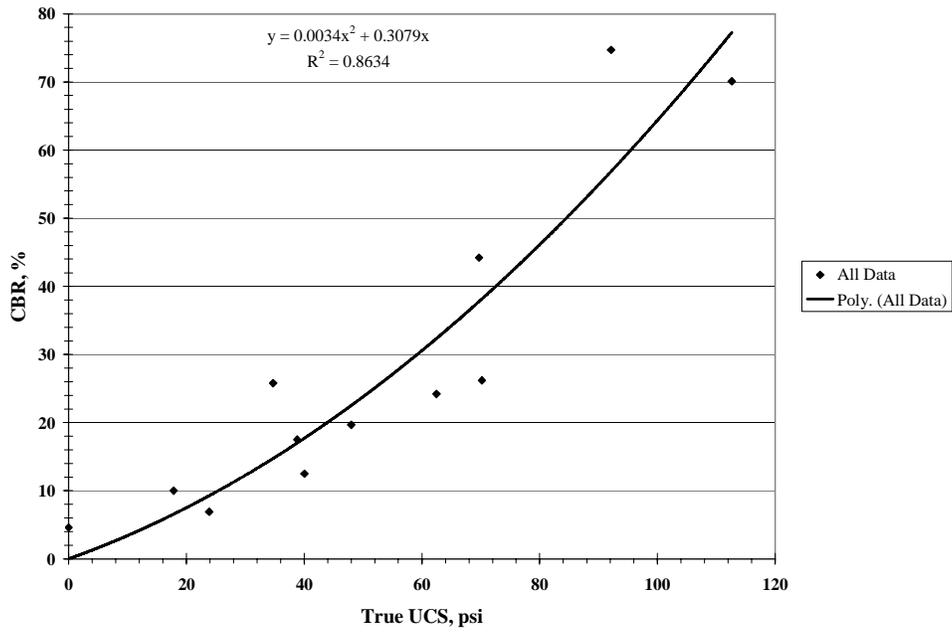
The moisture-CBR curves show the relationship between increasing CBR and increased fly ash content, similar to other moisture-strength relationships. The Le Grand loess with no fly ash exhibited CBR values from 1 to 3%, which are typical for saturated fine-grained soils. At CBR values this low, poor performance and very little support for overlying pavement structures would be expected. With 5% addition of LGS fly ash to the loess, the CBR increases to values that are typical of select soils in the range of 5% to 15%. The CBR-moisture plot for loess and 10% OGS ash shows even higher CBR values at low moisture content, but these values decrease at higher moisture contents. The range of CBR values of 10% to 26% is typical of clayey and poorly graded sands. 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 to prevent samples from weakening after soaking at lower moisture contents. And, as with the other samples, the CBR decreased at higher moisture contents. The 20% ash samples exhibited soaked CBR values near 75%, which are uncommon for fine grained soils. A CBR of 75% is similar to compacted gravels (Rollings and Rollings 1996).



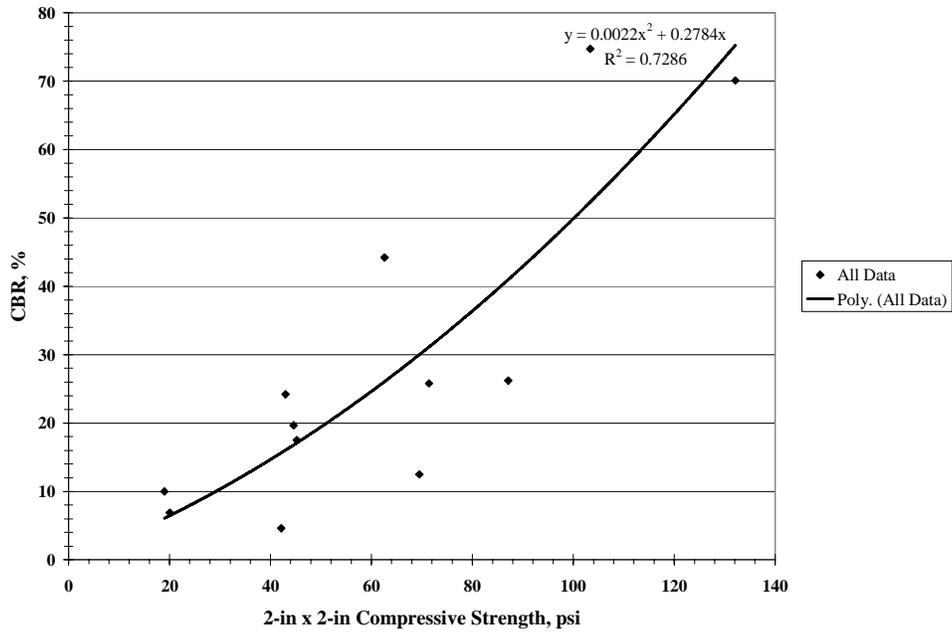
**Figure 47. CBR versus moisture content of Le Grand loess and fly ash**

Correlation plots between CBR and true UCS, two-inch x two-inch UCS, and Proctor-sized UCS are shown in Figures 48, 49, and 50, respectively. As expected, there is a general trend of increasing CBR values with increasing compressive strength for all sample sizes. The relationship is parabolic. The non-linear response could be due to the CBR samples being confined during testing. The UCS correlation had the largest  $R^2$  value, 0.86, while the Proctor-

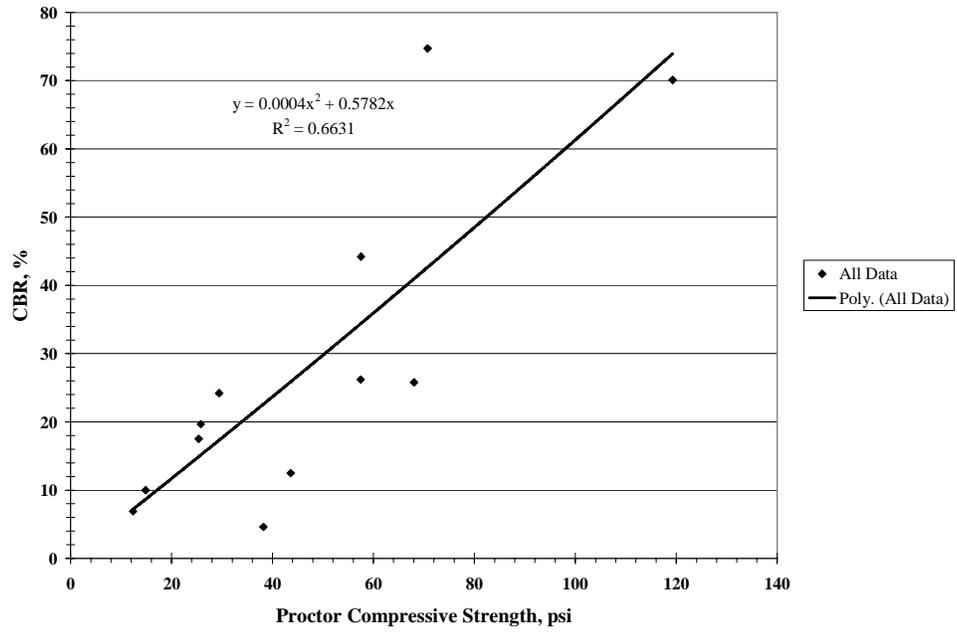
sized samples only provided an  $R^2$  of 0.66. The two-inch x two-inch correlation  $R^2$  is 0.73.



**Figure 48. Correlation between CBR and UCS for Le Grand loess and fly ash**



**Figure 49. Correlation between CBR and two-inch x two-inch 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 DRYING AND SWELL POTENTIAL REDUCTION USING SELF-CEMENTING FLY ASH

This section of the report describes test results that evaluate the effectiveness of using self-cementing fly ash to dry and modify soil engineering properties. Results show that by reducing plasticity characteristics and cementing soil particles together, fly ash can reduce the swell potential of expansive soils, and adding fly ash to soil effectively reduces the water content.

### Self-Cementing Fly Ash as a Drying Agent

In some instances, earthwork and paving operations are delayed due to wet soil conditions. One way to correct this problem is to mix in self-cementing fly ash. Fly ash addition increases the volume of dry material and uses up water to hydrate the fly ash particles, therefore reducing the moisture content and creating a more stable working platform.

OGS, PN4, SGS, and PC3+4 fly ashes were mixed with saturated Turin loess to evaluate the effectiveness of fly ash for drying wet soil conditions. Before adding the fly ash, water was added to increase the soil moisture content to approximately 33%. Each fly ash was then mixed with the soil at 10%, 20%, and 30% addition rates by dry weight. After adding and mixing the fly ash, the moisture content of the mixture was measured and the soil-fly ash mixture was placed in a shallow dish for set time testing. Readings were taken with a pocket penetrometer to measure strength gain as a function of time, as in the fly ash set time test.

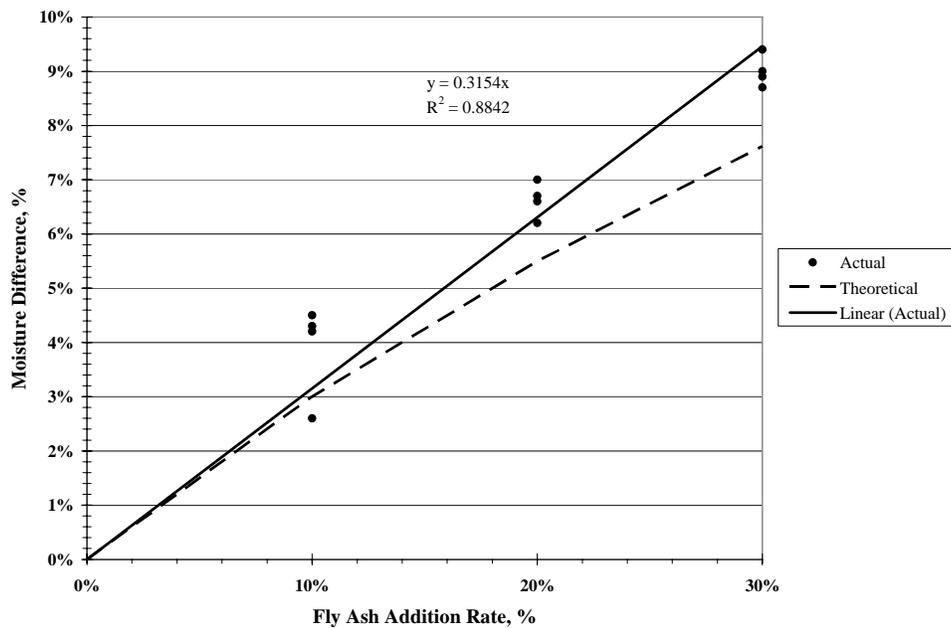
The moisture content change results are shown in Table 16 for all tests. Figure 51 shows the measured and theoretical relationship (shown in Equation 3) for change in moisture content:

$$\Delta w\% = \frac{W_w}{W_s} \left( 1 - \frac{1}{1 + \frac{\%FA}{100}} \right) \quad (3)$$

where  $\Delta w\%$  is the change in moisture content,  $W_w$  is the weight of water,  $W_s$  is the weight of solids, and  $\%FA$  is the percent fly ash based on  $W_s$ . All of the measured results lay above the theoretical line. This suggests that some moisture loss is attributed to hydration reaction products in the fly ash. Thus, it can be determined that fly ash is more effective at drying soil than Equation 3 indicates.

**Table 16. Summary of moisture content measurements for various self-cementing fly ash contents**

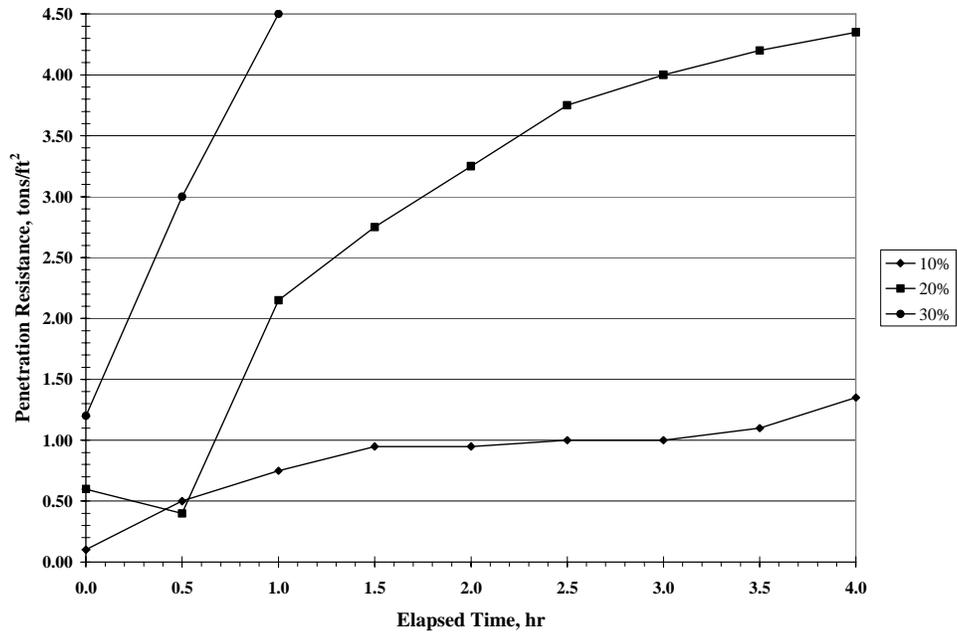
Ash Type	Addition Rate, %	Initial Moisture, %	Final Moisture, %	Moisture Change, %
SGS	10%	32.5%	29.9%	2.6%
SGS	20%	33.8%	27.1%	6.7%
SGS	30%	33.7%	24.8%	8.9%
PC 3+4	10%	33.3%	28.8%	4.5%
PC 3+4	20%	32.7%	25.7%	7.0%
PC 3+4	30%	32.7%	23.7%	9.0%
OGS	10%	33.3%	29.1%	4.2%
OGS	20%	32.3%	25.7%	6.6%
OGS	30%	33.1%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



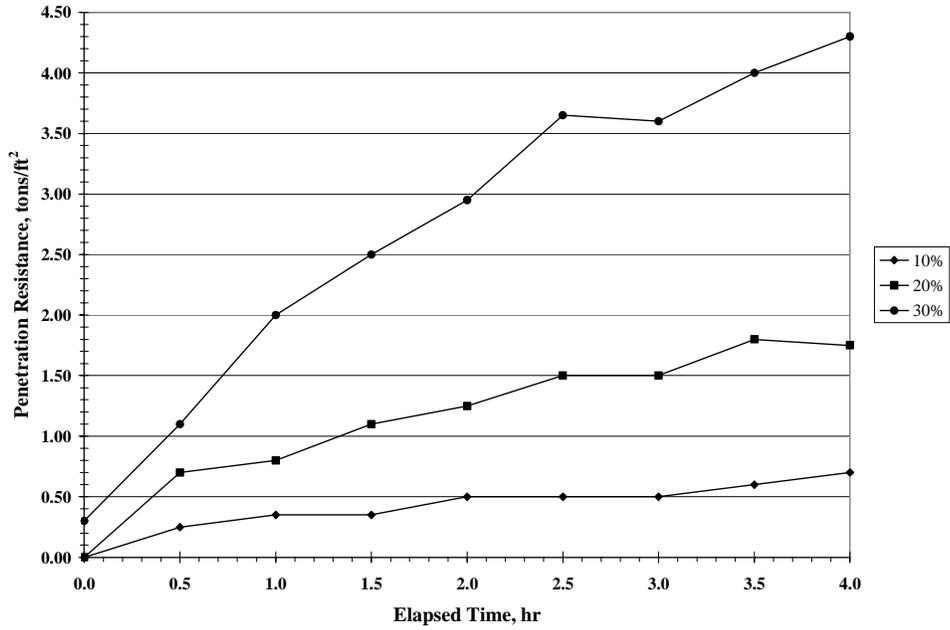
**Figure 51. Soil moisture loss as a function of fly ash content**

The penetration resistance results from the set time tests for the SGS, PC3+4, OGS, and PN4 fly ash-loess mixtures are shown in Figures 52, 53, 54, and 55, respectively. It is observed that all fly ashes provided some strength benefit to the soil. Also, results show the relationship between higher ash addition and increased strength. The SGS fly ash tests show 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

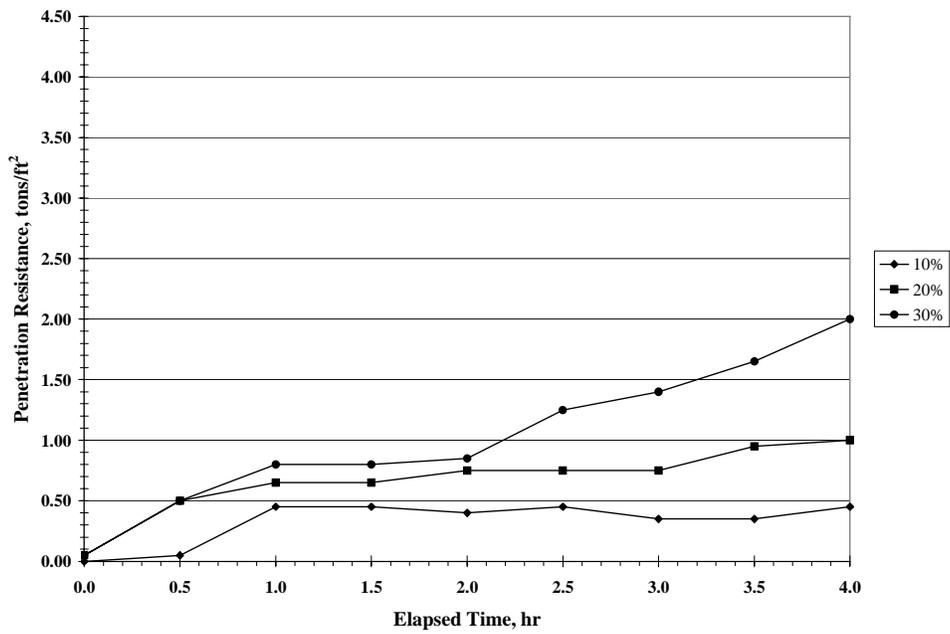
4-hour penetration resistances of 4.35 tsf and 4.00 tsf, respectively. The PC3+4 30% test shows a strength of 4.1 tsf after 4 hours, with a moderate strength gain of 1.75 tsf observed for 20% fly ash addition. The OGS tests show that the soil was drier, but little strength gain occurred. The strength results at 4 hours for the OGS are 0.45 tsf, 1.00 tsf, and 2.00 tsf, for 10%, 20%, and 30% ash addition, respectively. The 10% ash tests had a range of 0.45 tsf to 1.35 tsf, indicating that 10% fly ash addition did not provide much strength improvement.



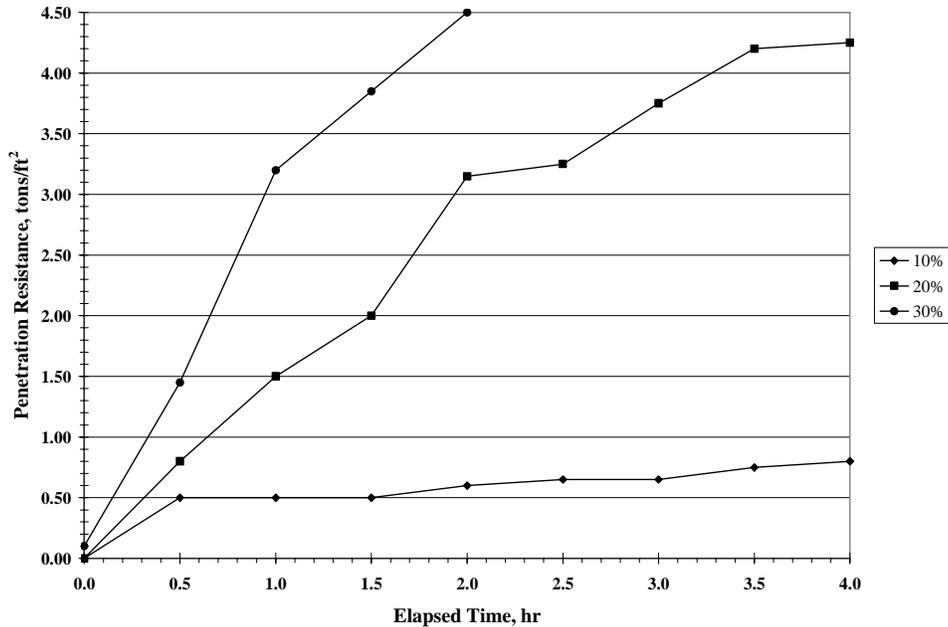
**Figure 52. Strength gain of Turin loess dried with SGS fly ash**



**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 Soil Swell Potential with Addition of Self-Cementing Fly Ash**

Self-cementing fly ash can reduce the swell potential of expansive soils. ASTM D4829 (Standard Test Method for Expansion Index of Soils) was followed for a series of tests. Tests varied from ASTM procedures in that samples were molded in a four-inch diameter Proctor mold, extruded, the ring from the expansion test apparatus was then placed around the molded samples, and the samples were trimmed. This was done to provide more uniform compaction for the samples. Expansion index (EI) is calculated by multiplying the change in sample height by 1000 and dividing this by the original sample height. This test method is intended for soils compacted at a moisture content to achieve 50% saturation. Saturation of 50% is used to simulate a worst case scenario for swell potential (at 100%, saturation swell potential is significantly reduced).

The glacial till, paleosol, and Le Grand loess soils were tested with no fly ash and mixed with 10% and 20% Ames fly ash. The samples were extruded and placed in the expansion test apparatus immediately after molding. All stabilized samples reached final swell within 24 hours of saturation. Most of the swell exhibited by the natural soils was completed within 24 hours, but some swell continued until the tests were stopped. The results are shown in Table 17. Overall, the glacial till shows the lowest swell potential. Swell potential decreases as fly ash content increases for all tests, with the exception of the 10% fly ash-till sample. The EI is low for this sample, but it is slightly higher than that of the natural soil. The variability of the soil could have been a cause of this observation. The paleosol exhibited the largest swell. The paleosol itself has an EI of 84, while 10% and 20% ash-treated samples reduced their EI values to 66 and 62, respectively. EI values of 66 and 62 indicate that the material still has medium swell potential. The Le Grand loess itself exhibited low swell potential, with an EI of 34, and fly ash addition

further lowered the EI to 19 for the 10% fly ash mixture and to 8 for the 20% fly ash mixture. The addition of 20% Ames fly ash removed almost all swell potential of the Le Grand loess. All of the stabilized samples would have had much 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**

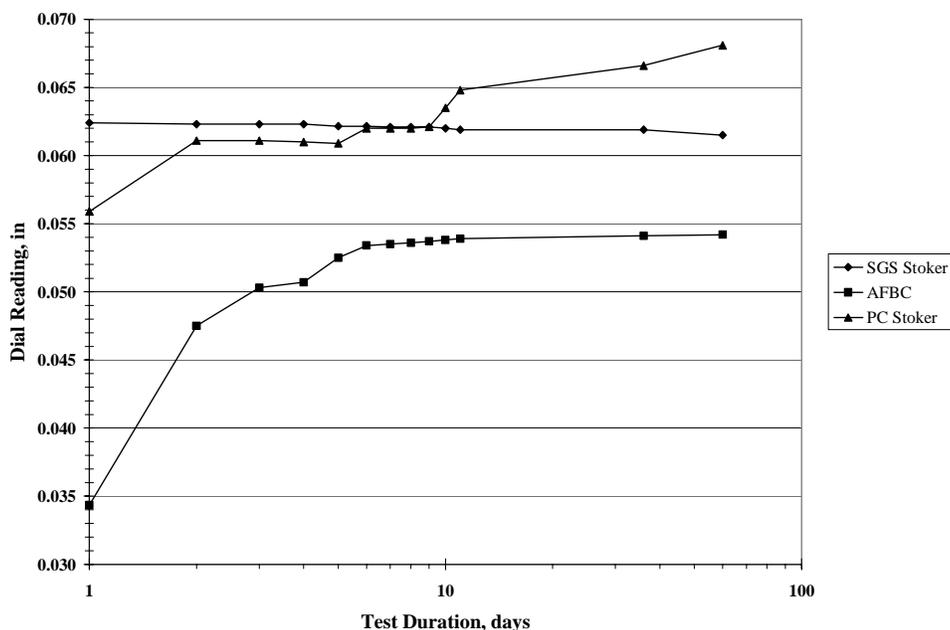
Soil Type	Fly Ash	Expansion Index	Swell Potential
Glacial Till	No Ash	27	Low
	10%	31	Low
	20%	21	Low
Paleosol	No Ash	84	Medium/High
	10%	66	Medium
	20%	62	Medium
Le Grand Loess	No Ash	34	Low
	10%	19	Very Low
	20%	8	Very Low

A limited study was initiated to monitor the effects of sulfur content and potential ettringite formation on EI. SGS fly ash, AFBC residue, and PC1+2 stoker fly ash were in this study. SGS fly ash meets the ASTM C618 requirement of less than 5% sulfur, while the AFBC residue and PC1+2 stoker fly ash 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 expansion index tests. The results reported are after 60 days of testing. Table 18 summarizes the results. 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 continued to swell slowly while the SGS mixture began to shrink. The PC1+2 sample continued to swell at a higher rate than the AFBC sample. It was noted earlier that the AFBC residue has a rapid initial strength gain, which explains why it shows the smallest EI, but ettringite formation decreases the long-term strength of AFBC-stabilized soil. However, it is not known at this time how long it will take for ettringite formation to cause swell problems in this material.

**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

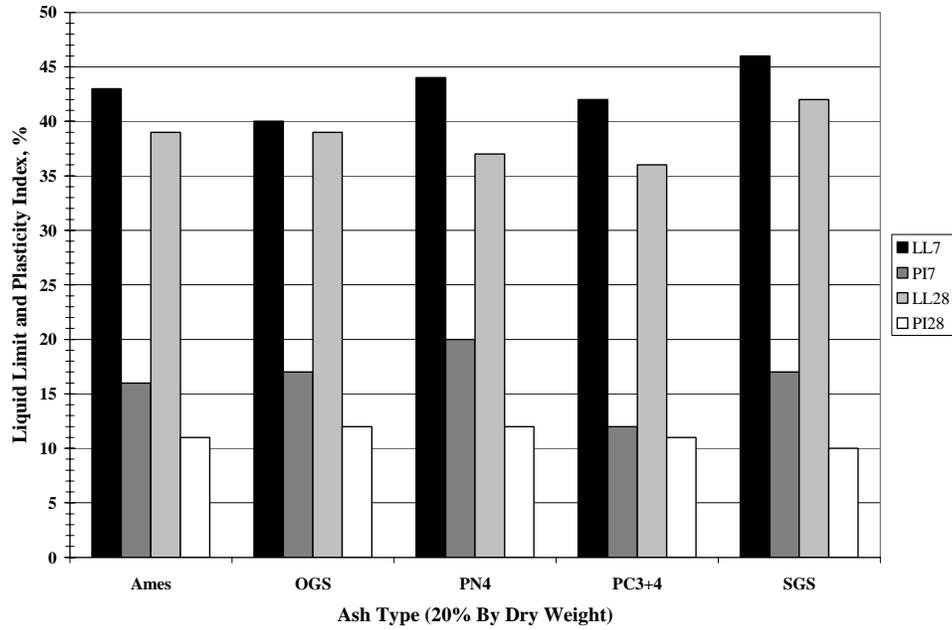


**Figure 56. Swell of paleosol-high sulfur fly ash mixtures**

### 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 (Ferguson and Leverson 1999, Cokca 2001, Parsons 2002, Nalbantoglu and Gucbilmez 2002). Two sets of tests were conducted to monitor the effects of fly ash addition on soil plasticity characteristics. 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 four-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. 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.

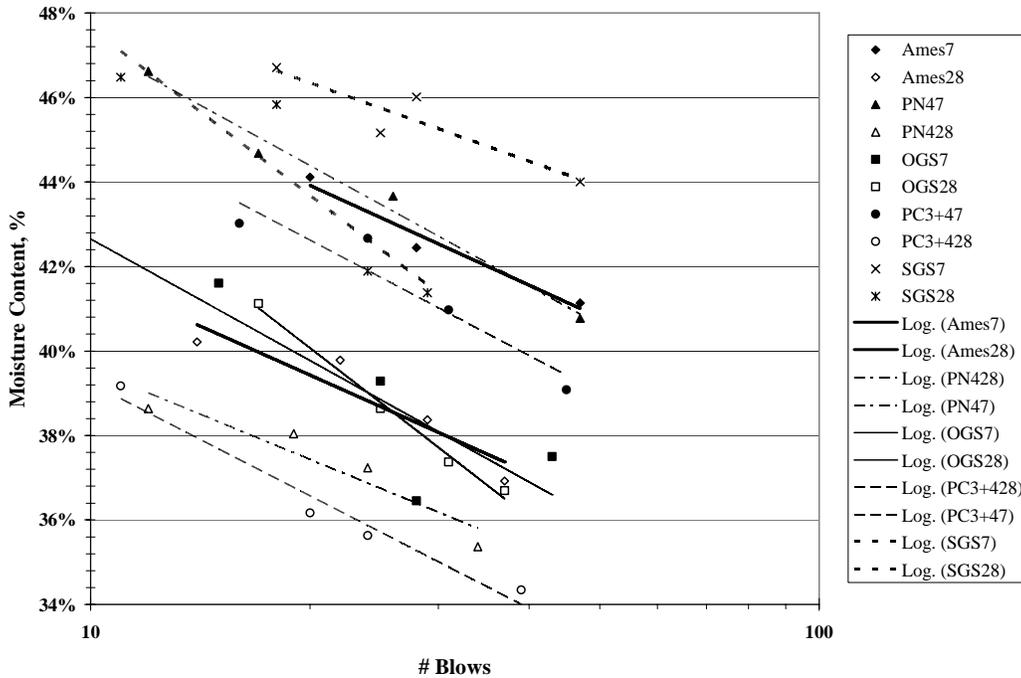
The test results of plasticity as a function of time are shown in Figure 57. The alluvium has a liquid limit of 47% and plasticity index of 25%. After seven days, all of the fly ash treatments reduced the liquid limit to a range of 40% to 46%. Plasticity index was also reduced from 25% to the range of 13% to 20%. Twenty-eight days of curing further reduced the liquid limit and plasticity. The results of this study and the USCS symbols of the stabilized mixtures are provided in Table 19. After seven days, plasticity decreased enough that these materials classified as lean clays. Plasticity was further reduced to the point at which the mixtures behaved as low-plasticity silts to lean clays at 28 days. 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.



**Figure 57. Liquid limit and plasticity index of alluvium with 20% self-cementing fly ash**

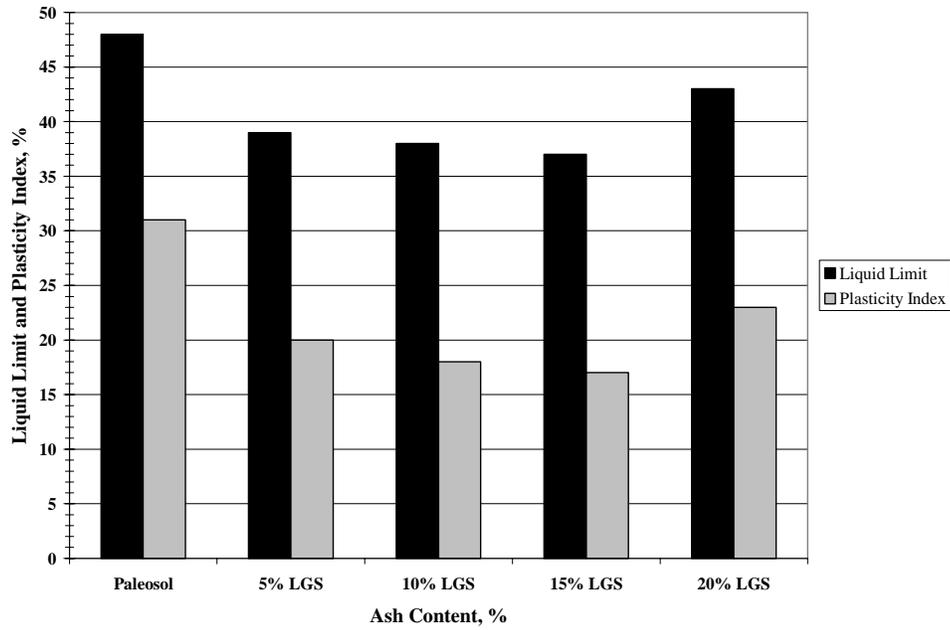
**Table 19. Atterberg limits and USCS symbols of alluvium and 20% various self-cementing fly ashes**

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 on liquid limits and plasticity index 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 soils with fly ash contents of 5%, 10%, and 15%. However, the 20% fly ash-paleosol mixture exhibited higher plasticity than that of the other three addition rates. The resulting USCS symbols are shown in Table 20. The plasticity limit 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 to exhibit the behavior of lean clay (CL).



**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
<b>Cure, days</b>	0	28	28	28	28
<b>LL</b>	48	39	38	37	43
<b>PL</b>	17	19	20	20	20
<b>PI</b>	31	20	18	17	23
<b>USCS</b>	CL-CH	CL	CL	CL	CL

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

## Curing Environment

Ferguson and Levenson (1999) and Vandenbossche and Johnson (1994) recommend that soil-fly ash mixtures be cured at temperatures greater than 40° F in the field. This recommendation was made because the rate of strength gain for self-cementing fly ashes decreases with temperature. The effects of curing environment were evaluated in a series of laboratory experiments to test this concept. Short-term strength gain was evaluated for several fly ash contents. Turin loess and paleosol were both used in this evaluation. Each soil was mixed with enough water to provide a final compacted moisture content of 19% for the soil-fly ash mixtures and mellowed for 24 hours. After 24 hours, fly ash was mixed with the soil at addition rates of 10%, 15%, and 20% (by dry weight). Turin loess was mixed with PN4 fly ash and paleosol was mixed with LGS fly ash. After mixing, samples were compacted immediately (in under 15 minutes) using the two-inch x two-inch (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 unconfined compression testing at a rate of 0.05 in./min. Results are shown in Figures 60 and 61, respectively.

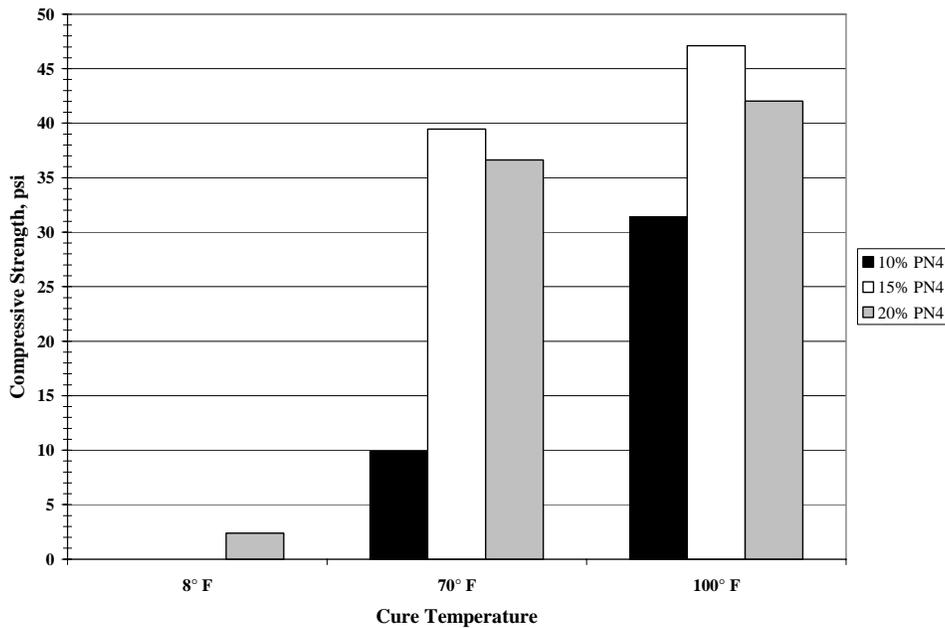
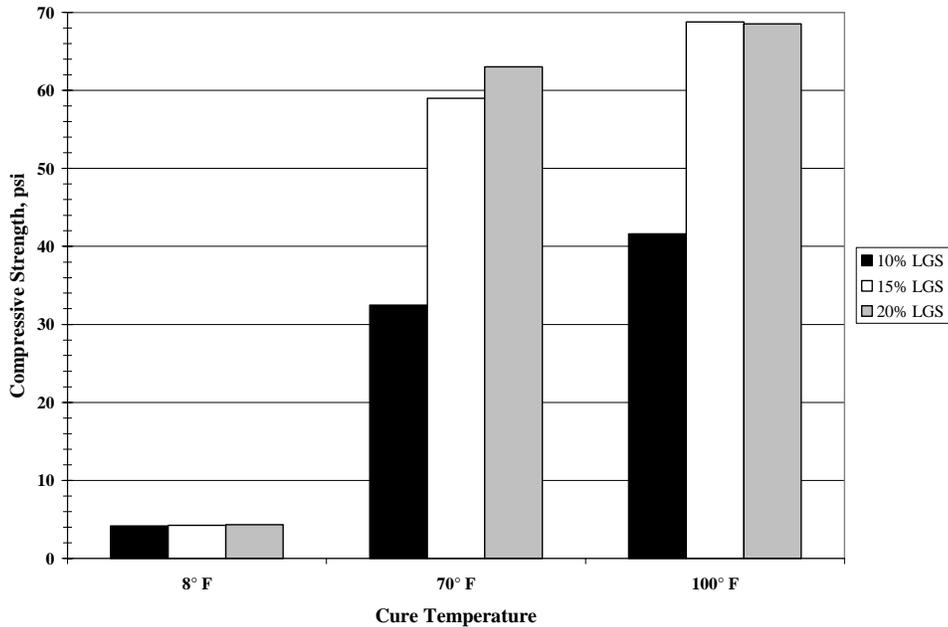


Figure 60. Influence of curing environment on strength gain of Turin loess and PN4 fly ash



**Figure 61. Influence of curing environment on strength gain of paleosol and LGS fly ash**

Results show that as the fly ash content increases, the compressive strength increases. This was the case at least for the 5% and 10% addition rates for the Turin loess. The 20% specimens, however, produced compressive strengths that were actually less (about 5 psi) than the 15% samples for 70° F and 100° F curing temperatures. Increased curing temperature increased the seven-day strength of the stabilized soil. Only one of the three samples cured at 8° F did not slake during soaking.

The paleosol samples also exhibited higher strength gain at increased curing temperatures, as was expected. Samples cured in the freezer had an approximate unconfined compressive strength of four psi. These samples did not slake, but did swell and crack, as shown in Figure 62. This is contrasted by Figures 63 and 64, which show that the paleosol samples cured at 70° and 100° F after soaking, respectively. The samples cured at 70° and 100° F did not exhibit swell or delamination during soaking. These samples also show a trend of increasing strength with increased fly ash content. Observation of 100° F strengths shows that they are only slightly greater than the 70° F strengths.

As shown in Figures 60 and 61, fly ash-stabilized soil does not gain strength below freezing temperatures. A limited study was undertaken to evaluate the potential for autogenous healing. Le Grand loess (20% moisture) and LGS fly ash (18% by dry weight) were used. The samples were prepared in the same manner as previously described. Two sets of samples were cured at 100° F and two sets were cured at 8° F. After seven 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. At the same time, curing environments for the remaining samples were switched: 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 an additional seven 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**

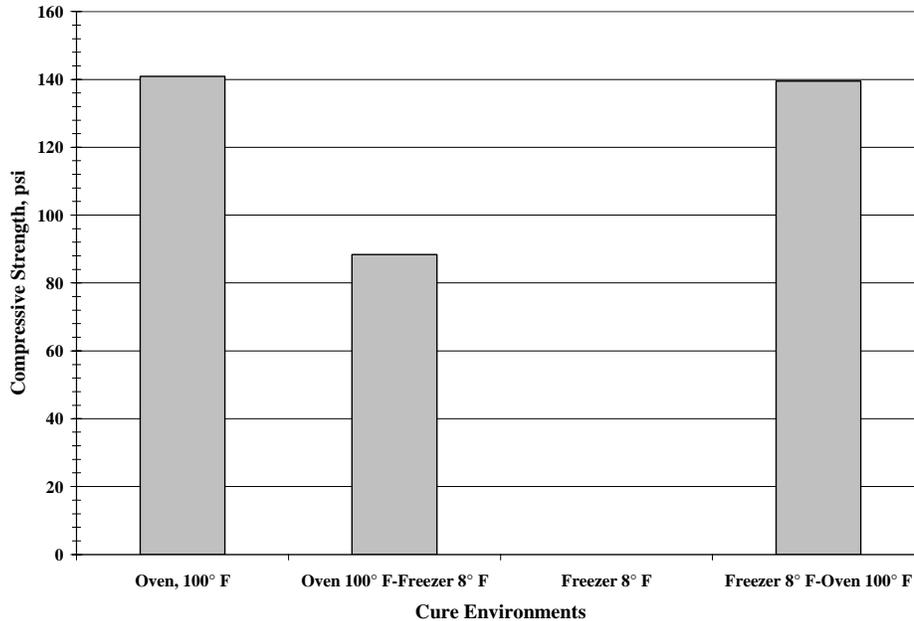
The results for the seven-day samples were as expected. The 8° F samples showed no strength gain while the 100° F samples yielded an average strength of 140 psi. Samples cured initially at 100° F and then transferred to the 8° F environment decreased in strength to 88 psi. A decrease was expected, but not to this degree. Strength gain was exhibited in samples initially cured at 8° F and then transferred to the higher temperature environment. Surprisingly, these samples developed strengths similar to samples cured only 7 days at 100° F. This shows evidence that after thawing, cementitious reactions are reinitiated and strength gain reinitiates. Fly ash-stabilized soil exhibits behavior that leads to the belief that the material could potentially heal after harsh winters.



Figure 63. Paleosol and fly ash cured at 70° F after soaking



Figure 64. Paleosol and fly ash cured at 100° F after soaking

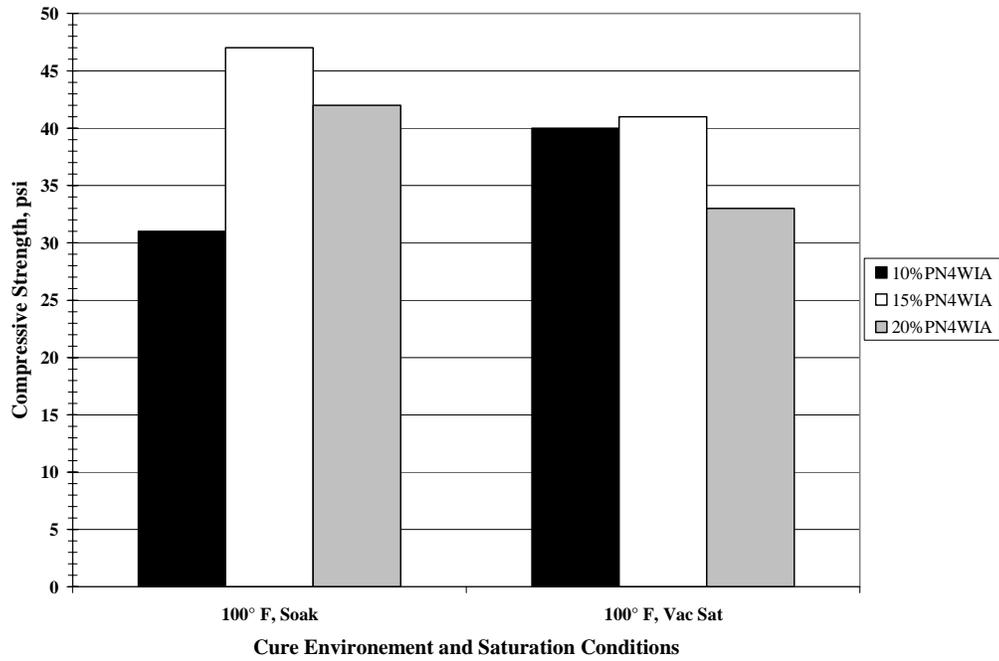


**Figure 65. Influence of cure environment on strength gain of self-cementing fly ash-soil mixtures**

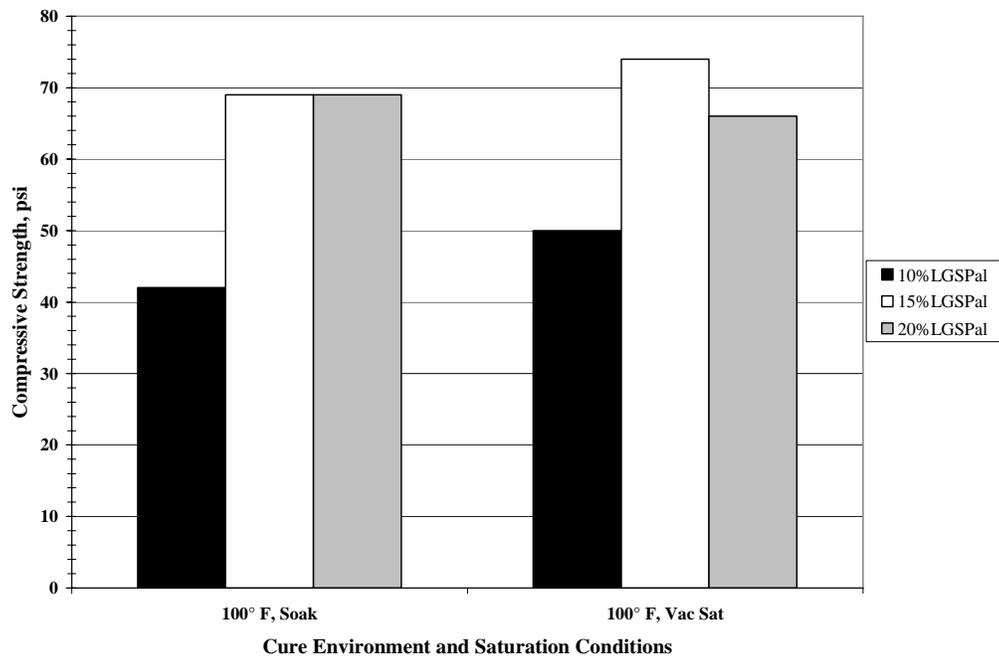
### Freeze-Thaw Durability

No ASTM test methods have been developed to test the freeze-thaw durability of soil stabilized with self-cementing fly ash. Typically, ASTM C593 (Standard Specification for Fly Ash and Other Pozzolans for Use With Lime) and ASTM D560 (Standard Test Methods for Freezing and Thawing Compacted Soil-Cement Mixtures) are 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 described below follows the test procedures of 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 mellow for 24 hours. Fly ash was then mixed with the soil and immediate compaction using the two-inch x two-inch ( $L/D=1.0$ ) 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 while a corresponding sample set was vacuum saturated for one hour. Compression testing at 0.05 in./min was performed after soaking. ASTM C593 (5) states that vacuum saturated samples must exhibit 90% of the strength attained by soaked samples. 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 indicates that the 10% fly ash samples passed the 90% strength difference test, while the 15% and 20% samples did not pass. The vacuum saturated 10% samples actually had higher strength than the soaked samples. Figure 67 shows that the vacuum saturated 10% and 15% samples also exhibit higher strength. 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 that the samples did not exhibit such durability problems as swell, cracking, or slaking.



**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 strengths 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 trapped air bubbles that were forced out of the sample during compression testing, causing excess internal damage and resulting in lower strengths. Although some of the sample sets passed the 90% strength criterion, no sample sets met the minimum ASTM C593 strength requirement of 400 psi for fly ash-lime-soil mixtures. This value would be difficult to attain without large amounts of self-cementing fly ash or the addition of lime. With lime addition, more calcium is available for cementitious reaction products, thus leading to a more rapid strength gain.

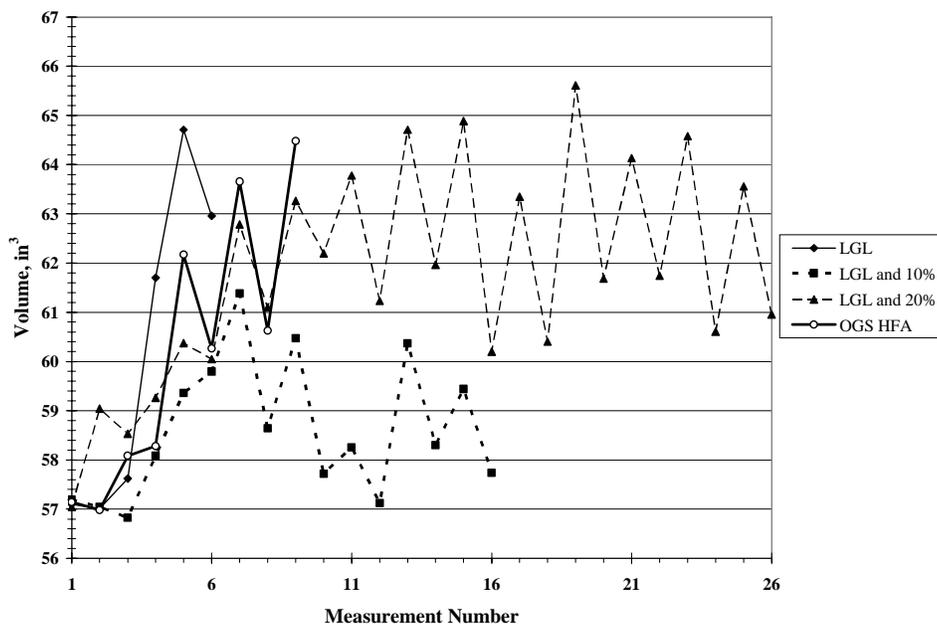
As stated before, ASTM D560 was developed for soil-cement mixtures. Identical specimens are molded, sealed, and cured for 7 days in a 100% humid environment for comparison with the ASTM C593 samples. One sample is used to measure volumetric stability while the other is brushed to measure mass loss. Samples in this study were molded in accordance with ASTM D560. Pre-selected moisture contents were added to the soils, which were allowed to mellow for 24 hours before molding. A set of two samples was molded using a standard four-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, the number 1 samples were measured for volume and the number 2 samples were brushed with a damp sponge. ASTM D560 states that a wire brush must be used to perform the brushing, but this was determined to be too harsh to simulate field conditions, so a less destructive technique, a

sponge, was adopted. Twenty-four hours in the freezer followed by 24 hours in the humidity room constituted one complete cycle.

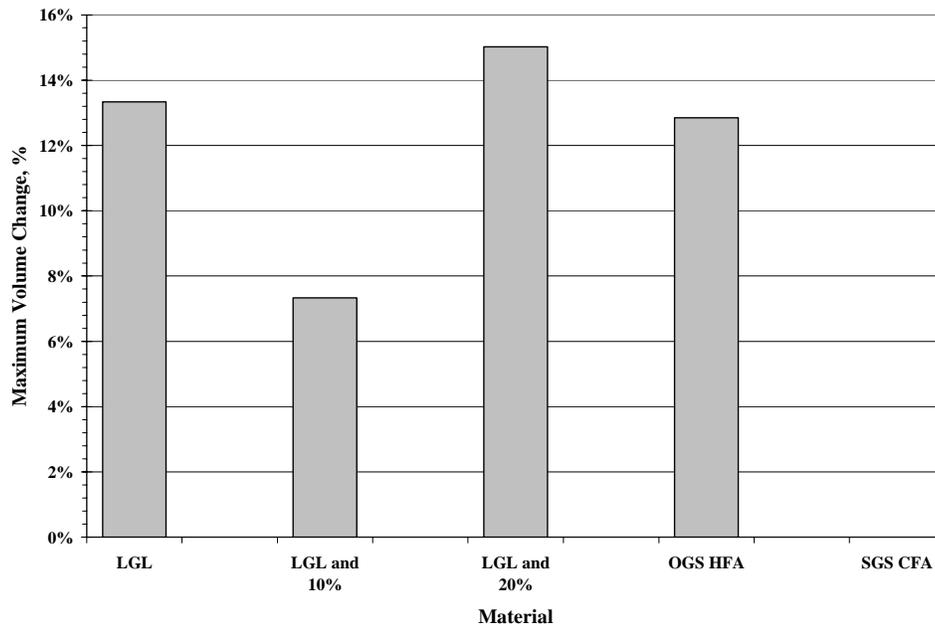
A range of materials was used in this testing. Unstabilized Le Grand loess was molded near optimum moisture content and used as a control. Sets of samples composed of Le Grand loess stabilized with 10% and 20% PN4 fly ash (by dry weight) were also molded. OGS HFA samples and SGS CFA samples were also tested due to durability 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 samples were unstable and fell apart before volume change 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 due to own weight was observed. The Le Grand loess showed a maximum volume increase of nearly eight in.<sup>3</sup> over the initial volume. This sample was lost after the second thaw period. After one cycle, the sample exhibited swelling and began cracking, as shown in Figure 73. The photo in Figure 74 was taken after the second freeze and shows showing a large crack propagating through the sample. The photo in Figure 75 was taken 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 change could be measured after the third thaw cycle. The 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. The 20% fly ash-soil sample exhibited the largest volume change, approximately nine in.<sup>3</sup> (or 15% over the original volume).



**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 the second freeze cycle**

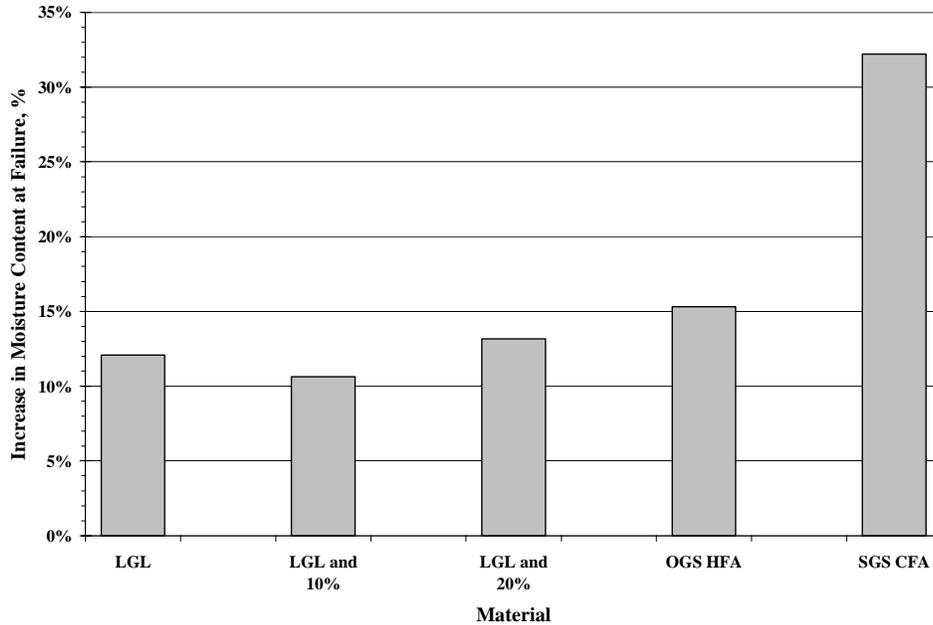


**Figure 75. Le Grand loess after the second freeze-thaw cycle**

Figure 76 shows the increase in the moisture content of the number 1 samples at the time they were discontinued. The SGS CFA exhibited a moisture increase of over 30%, giving the sample a final moisture content of over 50%. No volume change was observed for this sample, even though it absorbed a large amount of water. The OGS HFA absorbed over 15% moisture, which might have been greater if the sample had been subjected to additional 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, as observed in the unstabilized sample.

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

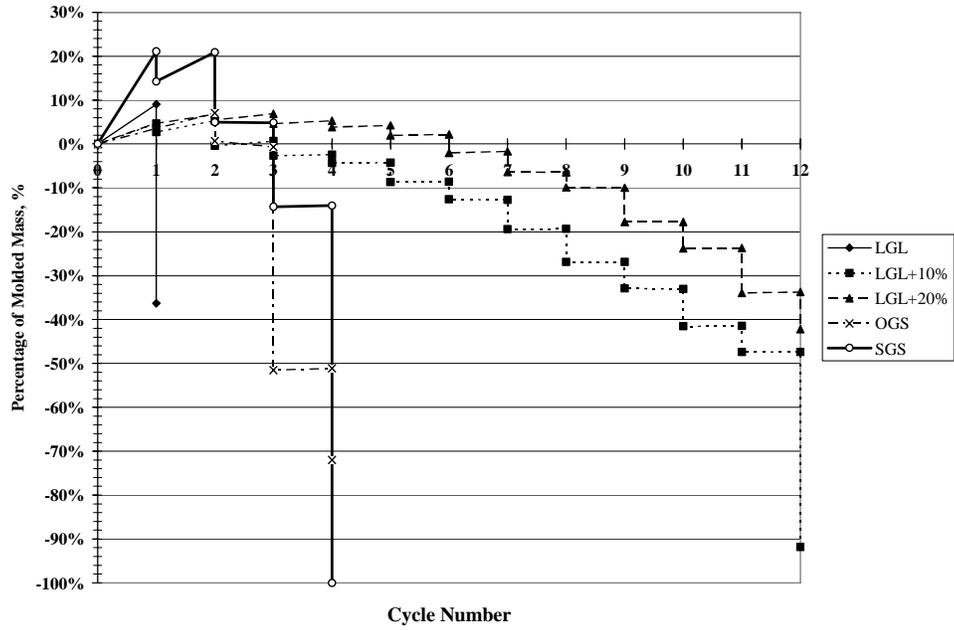
$$\%MoldedMass = \left( \frac{Mass - MoldedMass}{MoldedMass} \right) * 100 \quad (4)$$



**Figure 76. Moisture content increase of ASTM D560 volumetric stability samples**

This equation 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 significantly more 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 is typical of fine-grained materials. The 10% fly ash-soil sample delaminated 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 12 cycles of freeze-thaw and retained 58% of its original mass. 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 materials gain strength from long-term pozzolanic reactions, which were inhibited in the experimental curing environment and testing period. 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 may partly be due to a lack of confinement and inhibited pozzolanic reactions. Also, these materials have an affinity for water absorption, thereby accelerating freeze-thaw deterioration. An activator could increase durability, as determined by Berg (1998). However, activated HFA and CFA have shown good durability in field conditions (White 2002a, White 2002b).



**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 influence the strength gain of pozzolanic materials. In this study, the Proctor-sized samples (L/D=1.15) 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 unconfined compression. Before testing, samples were capped with a high-strength sulfur capping compound to square the ends. 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 generally decreases as 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 typical moisture-density curve. The LGS HFA exhibited the highest strengths. For the most part, moisture content did not have a pronounced effect on the strength gain of the SGS CFA, in which the strength trend appears to be related more to dry density than to molded moisture content. The CB HFA was relatively unreactive and behaved more like aggregate when the samples were extruded. All but one sample fell apart while being extruded.

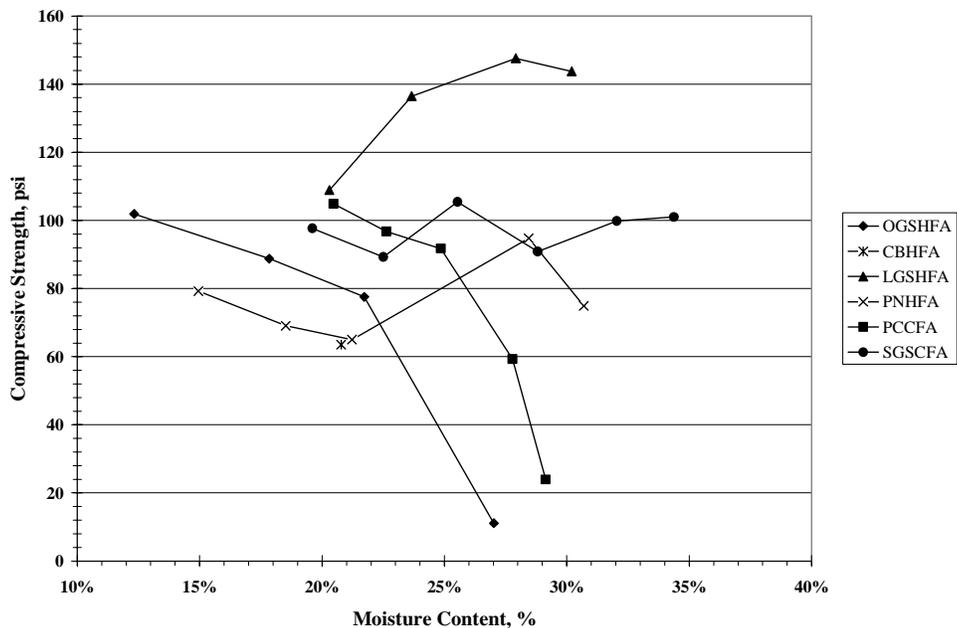
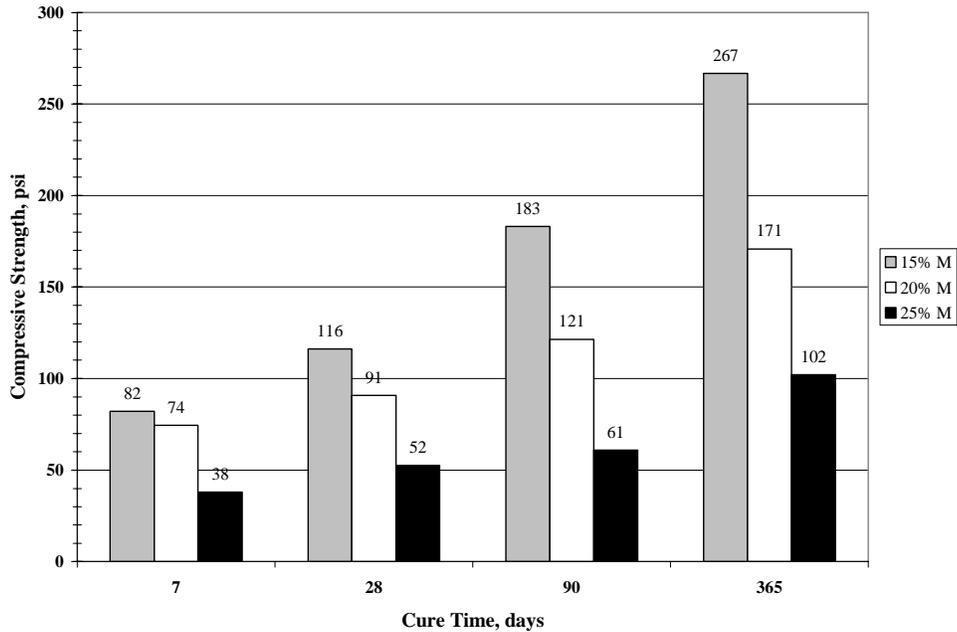


Figure 81. Influence of moisture content on strength gain of HFA and CFA

Another study was undertaken to monitor the 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 four-inch diameter Proctor mold (L/D=1.15), extruded, sealed and allowed to cure in a 100% humid environment. After the specified cure times (7, 28, 90, and 365 days), the samples were capped with sulfur capping compound and tested using compression at a rate of 0.05 in./min.

At each cure time, the samples followed 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. This equates to respective strength gains of 18.7%, 10.8%, and 14.1% per month.



**Figure 82. Long-term strength gain of Ottumwa HFA molded at different moisture contents**

### Influence of Compaction Delay on Strength Gain

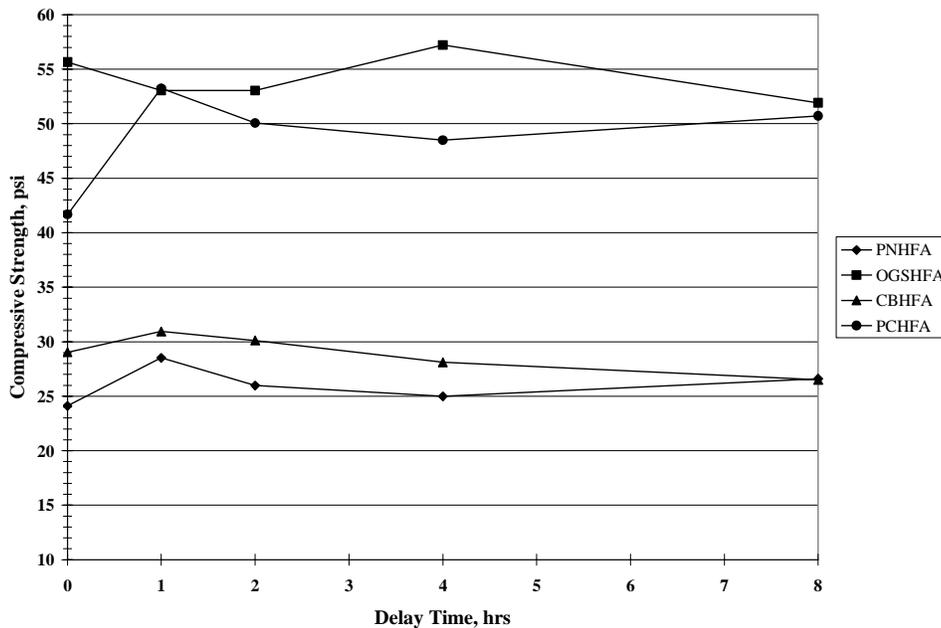
The object of this testing program was to see whether compaction delay has the same effect on the strength gain of HFA and CFA materials as it does with soil and self-cementing fly ash. OGS, PN, and CB HFAs and PC CFA were used in these experiments.

The HFA and CFA were prepared to pass the #4 sieve to allow the samples to be molded with the two-inch x two-inch 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 fly ashes, respectively. Samples were molded at 0, 1, 2, 4, and 8 hours and 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, the trend when comparing strength versus compaction delay time is not significant. The strengths of each material are similar over the range of delay times, with the maximum difference being about 10 psi for the PC CFA. The results are shown in Figure 83.

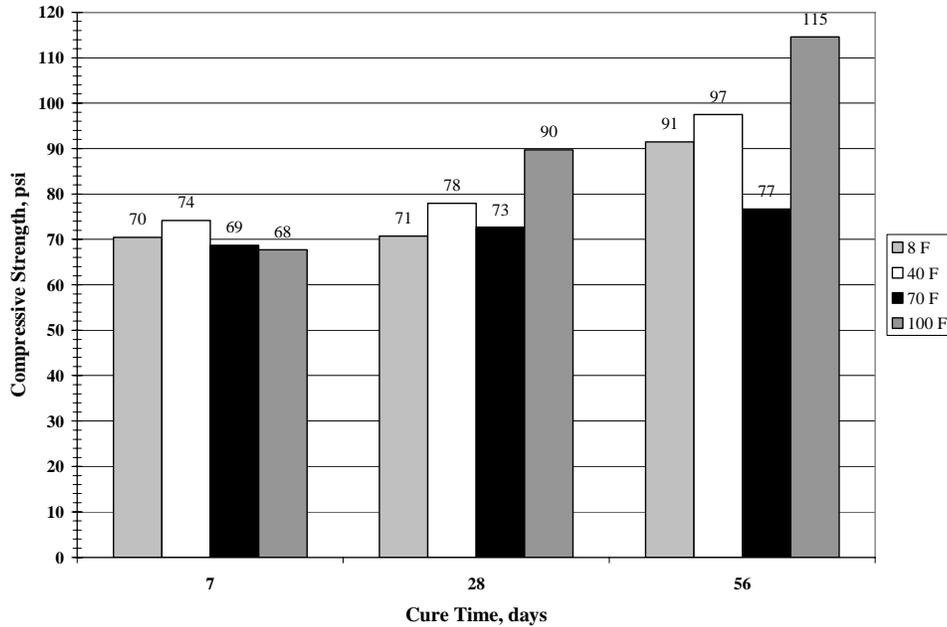
## Influence of Curing Temperature on Strength Gain

This research was used to determine whether curing temperature affected the 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 four-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. A comparison of results is shown in Figure 84.



**Figure 83. Influence of compaction delay on strength gain of HFA and CFA**

At seven days, all samples had approximately the same strength (~ 70 psi). 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, when strength was about 20 psi higher than strengths at 7 and 28 days. Samples cured at 40° F did gain strength, but not until 56 days, when strength increased 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, but this was not the case. The 70° F samples only gained 8 psi between 7 to 56 days. The reasons for this are not completely known. It was anticipated, however, that the 100° F samples would show an accelerated strength gain, and this was the case, with strength gains of 22 psi between 7 and 28 days and 25 psi between 28 and 56 days.



**Figure 84. Influence of curing temperature on strength gain of Ottumwa HFA**

### Shear Strength Parameter Values for Hydrated and Conditioned Fly Ashes

Consolidated drained triaxial compression tests were performed 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 inch x 8 inch cylindrical specimens ( $L/D=2$ ), 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 ( $\phi'$ ) to be calculated. Table 21 is a summary of  $c'$  and  $\phi'$  at the two cure times for each material.

At 7 days, shear strength parameter values for the OGS and PN materials are similar, with  $c'$  values over 10 psi and  $\phi'$  approximately  $32^\circ$ . The PC CFA, containing some raw ash, was slightly more reactive. At 7 days  $c'$  increased to 16 psi and  $\phi'$  to  $41.5^\circ$ . Pozzolanic reactions that occurred during the 28-day cure time should have increased  $c'$  and  $\phi'$  for each material. However, the PN samples tested at 28 days showed the same  $c'$  value of 10 psi, though  $\phi'$  increased to  $34^\circ$ . The PC CFA material tested at 28 days continued to gain strength, producing a  $c'$  value of 18.6 psi and a  $\phi'$  value of  $43.5^\circ$ .

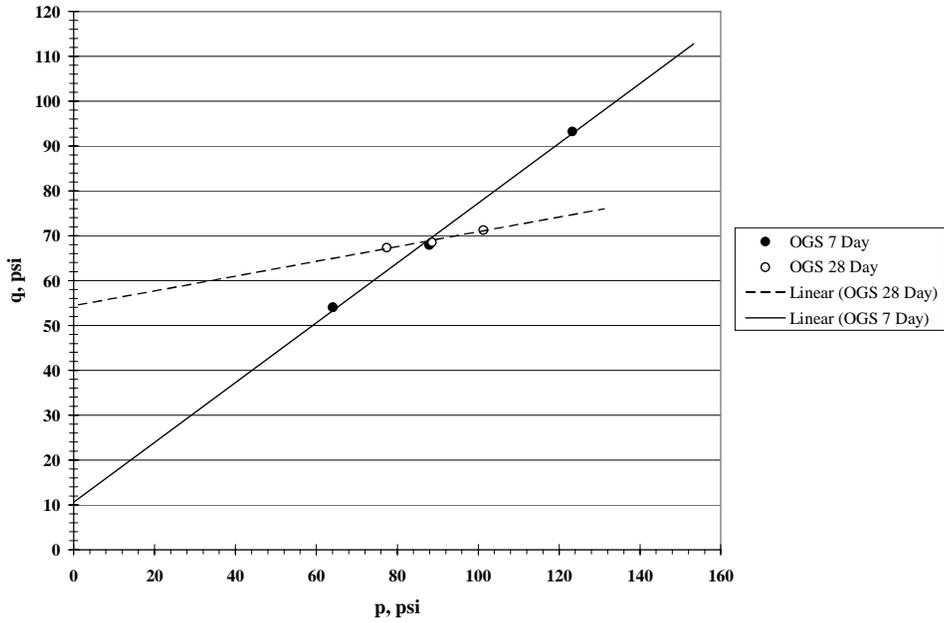


Figure 85. P-Q plot of OGS HFA

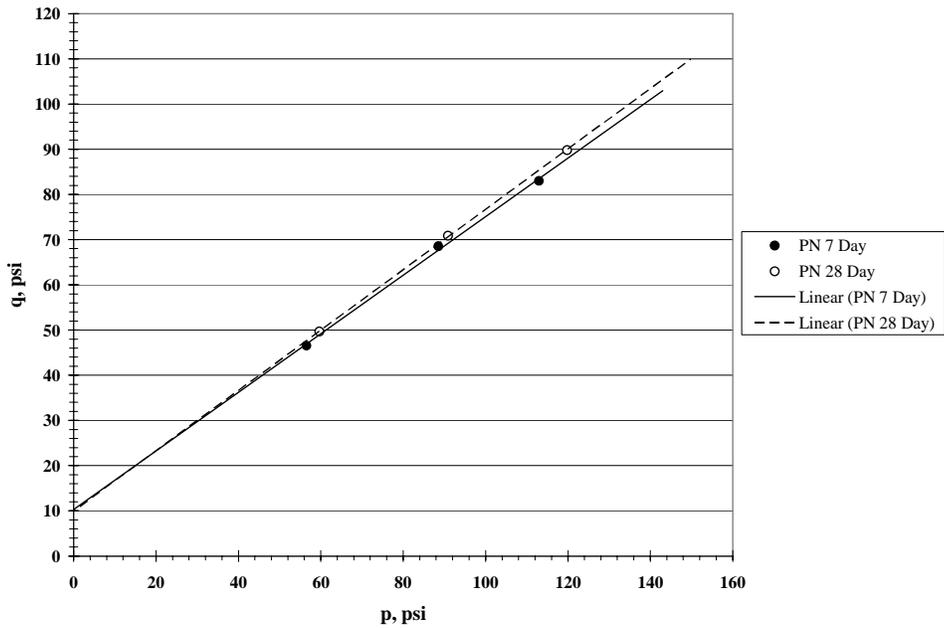
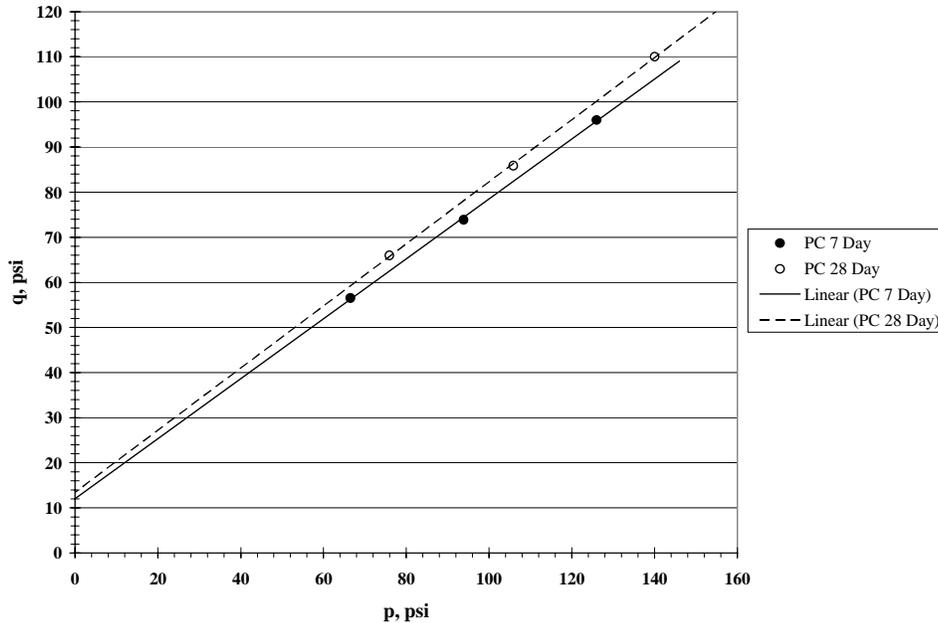


Figure 86. P-Q plot of PN HFA



**Figure 87. P-Q plot of PC CFA**

The OGS HFA exhibited a variable response when tested at 28 days. The cohesion intercept increased to about 54 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 is sample variability.

Overall, the HFA and CFA exhibit shear strength parameter values that make them suitable for use as fill materials in earthwork construction.

**Table 21. Summary of shear strength parameters of HFA and CFA**

Material	7 Day		28 Day	
	c' (psi)	φ' (degrees)	c' (psi)	φ' (degrees)
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**

White and Bergeson (2002) have shown that HFA and CFA materials can be used to increase the strength of soil. The cementitious reactions are not as rapid as raw self-cementing fly ash, but the HFA- and CFA-soil mixtures gain strength due to pozzolanic reactions. To gain the required strength, the HFA and CFA should be mixed with soil at approximately two to three times the rate of raw fly ash (White and Bergeson 2002).

OGS and CB HFA materials and PC CFA were used in this study to evaluate 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 content and the HFA and CFA were added at their original moisture contents. A total of nine two-inch x two-inch samples were molded for each addition rate and allowed to cure in a humidity room at room temperature for 7, 28, and 90 days. Prior to testing, samples were soaked in water for one hour at room temperature 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 zero 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 a project on Highway 63. The results of the OGS testing are shown in Figure 88. The relationship between increased addition rate and higher compressive strengths is observed.

At 7 days, strengths were similar, generally in the range of 30 psi to 41 psi. The 28-day results show that the higher addition rates of 20% and 30% 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 expected for soils stabilized with raw fly ash, soil stabilized with OGS HFA was effective at increasing compressive strength.

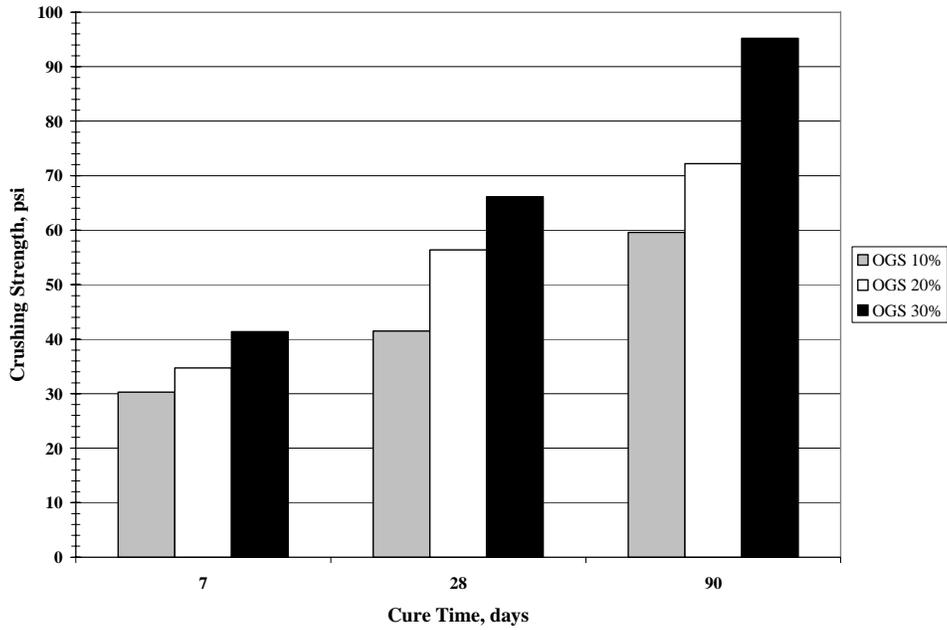
### **Council Bluffs HFA and Soil**

Results of the CB HFA and Central Iowa loess are shown in Figure 89. At seven days, none of the addition rates produced measurable strength, with samples dissolving during the soaking stage of testing. The CB HFA-stabilized samples did show the typical relationship between strength and addition rate at 90 days. The 30% samples exhibited strengths near 50 psi.

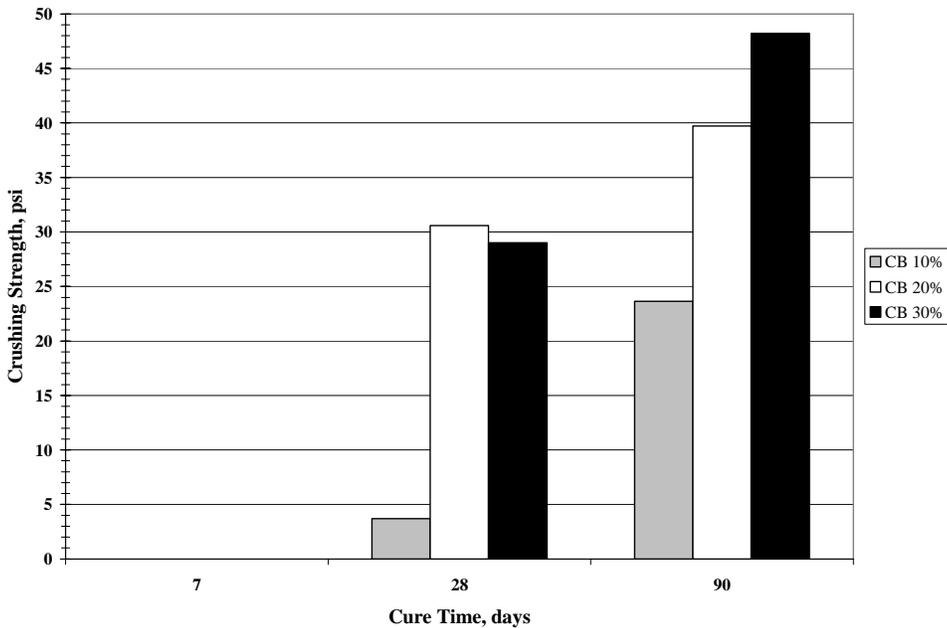
### **Prairie Creek CFA and Soil**

As observed with the CB HFA, the PC CFA-stabilized soil did not show strength gain at seven days, as shown in Figure 90. The samples dissolved during soaking. At 28 days, strengths ranged from 35 psi to 50 psi for the three addition rates. At 90 days, strengths increased to 50 psi, 65 psi,

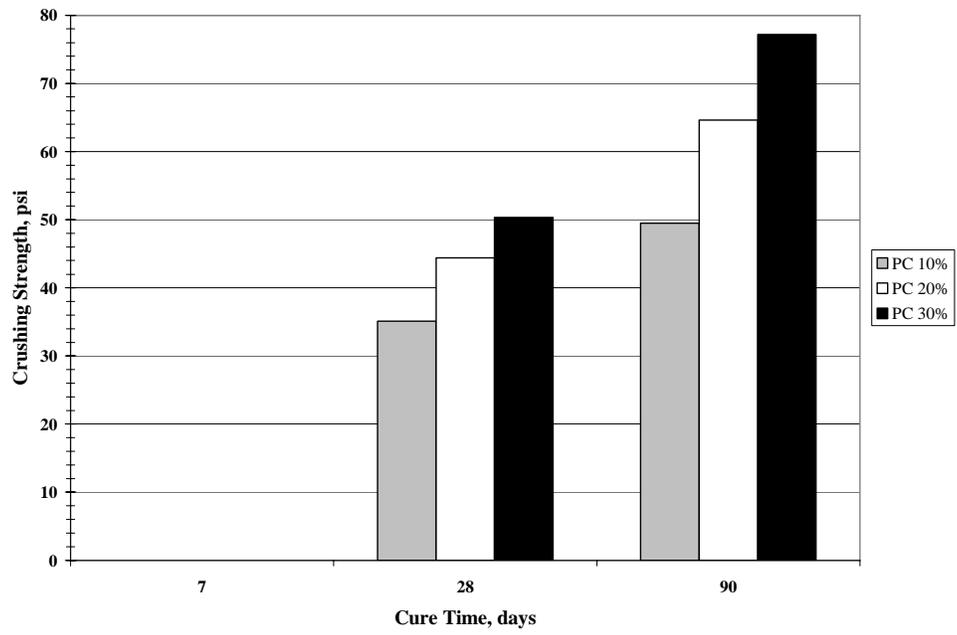
and 77 psi for addition rates of 10%, 20%, and 30%, respectively. During sample preparation, it was observed that the PC CFA was slightly carbonated, which may have reduced strength gain.



**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

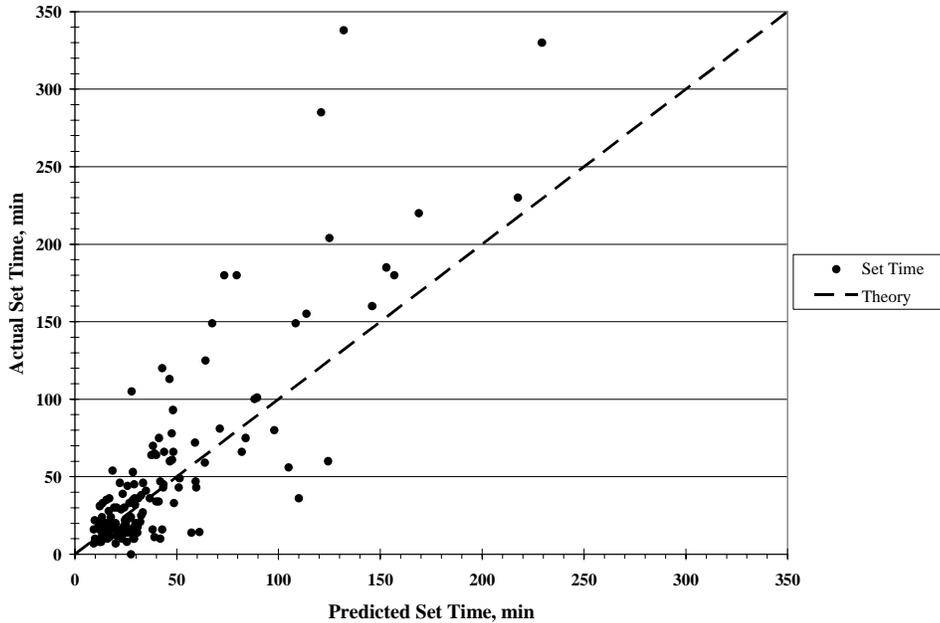
Bergeson and Lapke (1993) theorized that fly ash set time was related to CaO content. It is known that tricalcium aluminate is responsible for the initial hardening of cement, so it was hypothesized that tricalcium aluminate would also influence set time. From 1999 to 2001, the chemical components observed in XRF and the set time of 150 self-cementing fly ash samples from Nebraska power plants were determined by MARL personnel. The samples were collected from power plants that burn Powder River Basin subbituminous coal, similarly to most power plants in Iowa. Statistical analyses were applied to gain an understanding of the chemical components responsible for the initial hardening of fly ash to determine how set could be predicted.

The SAS computer program was used in the analysis. Model selection was based on Mallows'  $C_p$ , which is estimated as the sum of squares error divided by the standard deviation squared of the sample set. The SAS program first computes an initial regression using all input variables. The input with the largest  $Pr$  value is then rejected and the regression is computed again.  $Pr$  is the probability the predicted value will lie about the  $t$ -value for that input in a normal distribution. The  $t$ -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 evaluated by the program. At this time, the researchers can select the  $C_p$  model they would like to use. The  $C_p$  model is selected based on two main criteria: (1) the  $C_p$  must be minimized, and (2) the  $C_p$  should be approximately equal to the number of inputs retained ( $k$ ) plus one. Besides determining a prediction equation,  $Pr$  is used to determine the chemical constituents that have the most effect on the set time. The  $Pr$  that is greater than the absolute value of  $t$  is an indication of this. The smaller this value, the more influence a component has on set time.

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

$$\ln(\text{set time}) = 18.6 + 7.2(\ln \text{Al}_2\text{O}_3) + 3.7(\ln \text{SO}_3) - 13.2(\ln \text{CaO}) + 1.4(\ln \text{MgO}) - 1.3(\ln \text{Na}_2\text{O}) - 1.1(\ln \text{K}_2\text{O}) + 2.4(\ln \text{P}_2\text{O}_5) - 4.0(\ln \text{BaO}) \quad (5)$$

Figure 91 shows the data from the Nebraska fly ashes. If the prediction equation were 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 that the equation provides a reasonable estimate of set time for materials with set times less than 40 minutes, while the relationship tends to underestimate set time for fly ashes with actual set times greater than 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 to be able to compare the Iowa fly ashes to the Nebraska sample set. Analysis of the Iowa fly ashes yielded the following prediction equation:

$$\ln(\text{set time}) = 2.2 + 8.1(\text{SiO}_2) - 10.0(\text{Al}_2\text{O}_3) + 7.2(\text{Fe}_2\text{O}_3) + 22.7(\text{SO}_3) - 5.9(\text{CaO}) + 0.9(\text{MgO}) - 7.1(\text{Na}_2\text{O}) - 106.2(\text{K}_2\text{O}) + 61.9(\text{P}_2\text{O}_5) - 35.2(\text{TiO}_2) \quad (6)$$

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 had more data been available. Set times less than 50 minutes seem to be predicted well, while set times in the middle of the test range are underestimated. No definite trend for predicting high set times is observed because only one data point in this range was available. A general trend is observed, but more data points would aid in distinguishing a more well-defined trend.

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 used 10 of the chemical components. As noted before, CaO and tricalcium aluminate were thought to be the chemical components that have the most influence on set time. The CaO would be reported in CaO, while the tricalcium aluminate would be reflected in both  $\text{Al}_2\text{O}_3$  and CaO. The results for the Nebraska sample set are in good agreement with the previous hypotheses. Both the CaO and  $\text{Al}_2\text{O}_3$  have an influence on set time. Sulfur and sodium also have a great influence on set time. Sulfur acts as a retardant, while sodium is a network modifier in the fly ash, modifying the crystal structure.

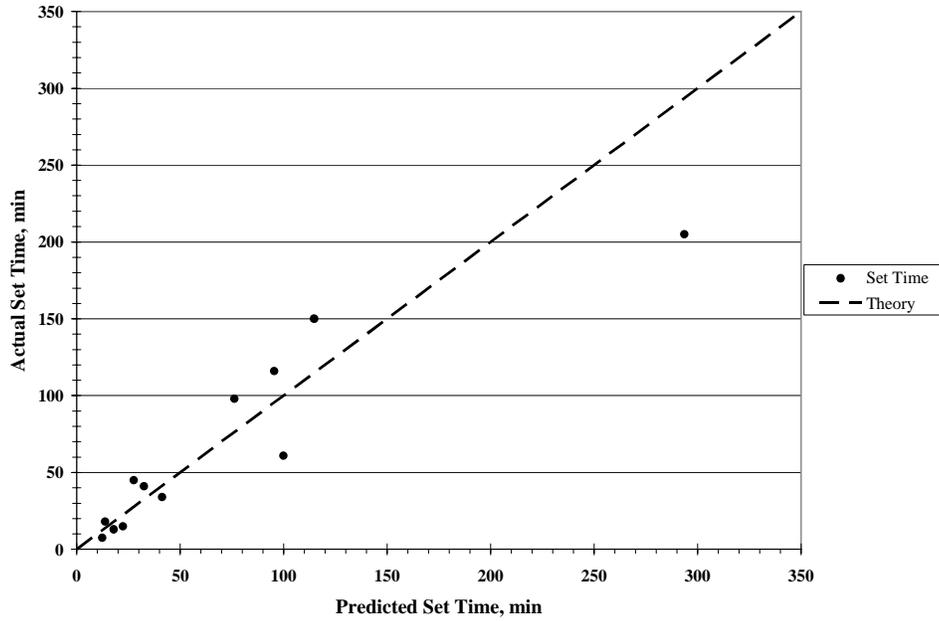


Figure 92. Actual versus predicted set time for Iowa self-cementing fly ashes

Table 22. Probability estimates for 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 <sub>2</sub>	0.0087
Al <sub>2</sub> O <sub>3</sub>	< 0.0001	Na <sub>2</sub> O	0.0119
SO <sub>3</sub>	< 0.0001	SO <sub>3</sub>	0.0134
Na <sub>2</sub> O	< 0.0001	Fe <sub>2</sub> O <sub>3</sub>	0.0138
P <sub>2</sub> O <sub>5</sub>	0.0007	P <sub>2</sub> O <sub>5</sub>	0.0142
K <sub>2</sub> O	0.0290	SiO <sub>2</sub>	0.0151
BaO	0.0292	K <sub>2</sub> O	0.0184
MgO	0.0960	CaO	0.0186
—	—	Al <sub>2</sub> O <sub>3</sub>	0.0196
—	—	MgO	0.1888

## SUMMARY AND CONCLUSIONS

The primary objectives of this research study were as follows:

1. Evaluate the effects of self-cementing fly ash addition on the engineering properties of several Iowa soils
2. Develop construction guidelines and specifications for using self-cementing fly ashes and hydrated or conditioned fly ashes

The main observations and conclusions of this study are as follows:

- Iowa self-cementing fly ashes are effective at stabilizing Iowa soils for earthwork and paving operations.
- With regard to the influence of self-cementing fly ash on density and compaction, test results show that fly ash addition increases the compacted dry density and reduces the optimum moisture content. Dry of optimum moisture content, modified compaction energy further increases the compacted dry density. However, compaction effort does not have a pronounced influence at high moisture contents. For some mixtures, compaction delay decreases the compacted dry density of soil-fly ash mixtures (up to a 13-pcf decrease after a 4-hour delay).
- Compaction energy and compaction delay also influence strength. Unsoaked soil-fly ash mixtures exhibit higher strengths at higher compaction energies and low moisture contents. Increasing compaction effort at high moisture contents shows little benefit for increasing compressive strength. In most instances, compaction delay negatively influences strength gain in soil-fly ash mixtures. However, strength gain potential tends to continue until the fly ash has reached its final set time.
- Soaking laboratory samples before strength testing is recommended to evaluate samples in a saturated condition. Samples compacted dry of optimum moisture content tend to slake when saturated. This overly dry condition should be avoided in the field. However, high fly ash addition rates (~ 20%) can strengthen soil-fly ash mixtures against slaking.
- Strength gain in soil-fly ash mixtures is time-dependent. Rapid strength gain in soil-fly ash mixtures occurs during the first 7 to 28 days of curing, and a less pronounced increase continues with time due to long-term pozzolanic reactions.
- Sulfur contents in the fly ash (greater than about 5%) can cause formation of ettringite in soil-fly ash mixtures, which severely reduces strength and durability. Sources of sulfur in the soil and mixing water can also potentially cause problems with expansive mineral formation.
- Various laboratory sample compaction techniques, such as true unconfined sample ( $L/D = 2$ ), Procter-sized ( $L/D = 1.15$ ), and two-inch x two-inch ( $L/D = 1$ ), were evaluated for a

wide range of soil-fly ash mixtures. Sample strength measurements from Iowa State University two-inch x two-inch samples can be multiplied by 0.86 to correlate with compressive strengths of standard four-inch diameter Proctor-sized samples and multiplied by 0.90 to correlate with true UCS measurements. Proctor-sized samples multiplied by 1.15 provide an indication of true UCS, although this data needs further evaluation.

- Fly ash addition increases the CBR of natural soils, and in the case of 20% fly ash addition, the CBR can be increased up to values that simulate compacted gravel (~75%). The compressive strengths of samples with  $L/D=2$ ,  $L/D=1.15$ , and  $L/D=1$  exhibit a parabolic correlation to the CBR for corresponding mixture properties.
- Fly ash effectively dries wet soils and provides an initial rapid strength gain, which is useful during construction in wet, unstable ground conditions. Fly ash also decreases the swell potential of expansive soils by replacing some of the volume held by expansive clay minerals and by cementing the soil particles together.
- Soil-fly ash mixtures cured below freezing temperatures and then soaked in water are highly susceptible to slaking and strength loss. Compressive strength increases as fly ash content and curing temperature increase. Samples cured below freezing and subsequently cured at 100° F show strength gains similar to that of samples initially cured at 100° F.
- Paleosol stabilized with fly ash exhibits increased freeze-thaw durability when tested in accordance with ASTM C593. Fly ash also increases the resistance of Le Grand loess to mass loss due to freeze-thaw, when tested in accordance with ASTM D560. HFA and CFA did not perform well when tested in accordance with ASTM D560. This is believed to be because the test method does not allow pozzolanic reactions to occur and does not allow confinement of the samples.
- Moisture-strength relationships of HFA and CFA do not show a clear trend, but most materials exhibit a strength decrease with increased moisture content. HFA shows a continued strength gain over long cure times (365 days). Compaction delay does not have a pronounced negative effect on HFA and CFA strengths. The results of this study suggest that HFA shows a strength increase with increasing cure temperature. Triaxial tests indicate that PN HFA has shear strength parameter values of  $c'=10$  psi and  $\phi'=32^\circ$  after curing 28 days. PC CFA has approximately  $c'=18.6$  psi and  $\phi'=43.5^\circ$  after curing 28 days. OGS HFA exhibited variable results.
- Soil strength can be increased with HFA and CFA addition, but at higher addition rates and not as effectively as strength gains observed for self-cementing fly ash.
- Due to fly ash variability, set times greater than 40 minutes were not able to be predicted accurately from a statistical model. A statistical regression, however, shows that CaO, Al<sub>2</sub>O<sub>3</sub>, SO<sub>3</sub>, and Na<sub>2</sub>O influence the set time characteristics of self-cementing fly ash.

Based on the results of this study, three proposed specifications (Appendices C, D and E) were

developed for using self-cementing fly ash, HFA, and CFA. The specifications describe laboratory evaluation, field placement, moisture conditioning, compaction, quality control testing procedures, and basis of payment. These specifications are based on Ferguson and Levenson (1999).

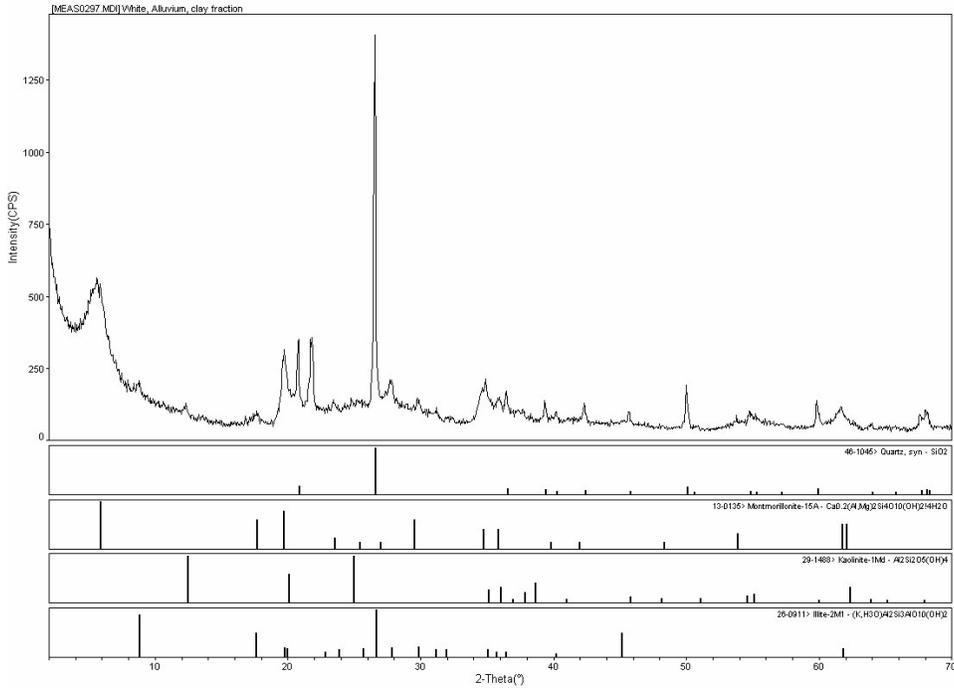
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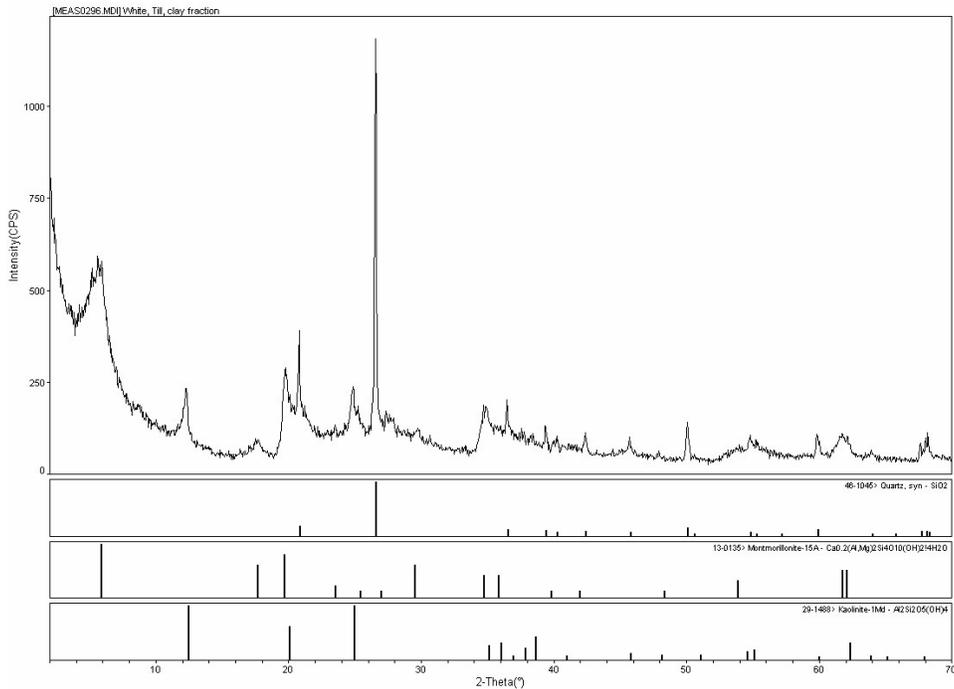
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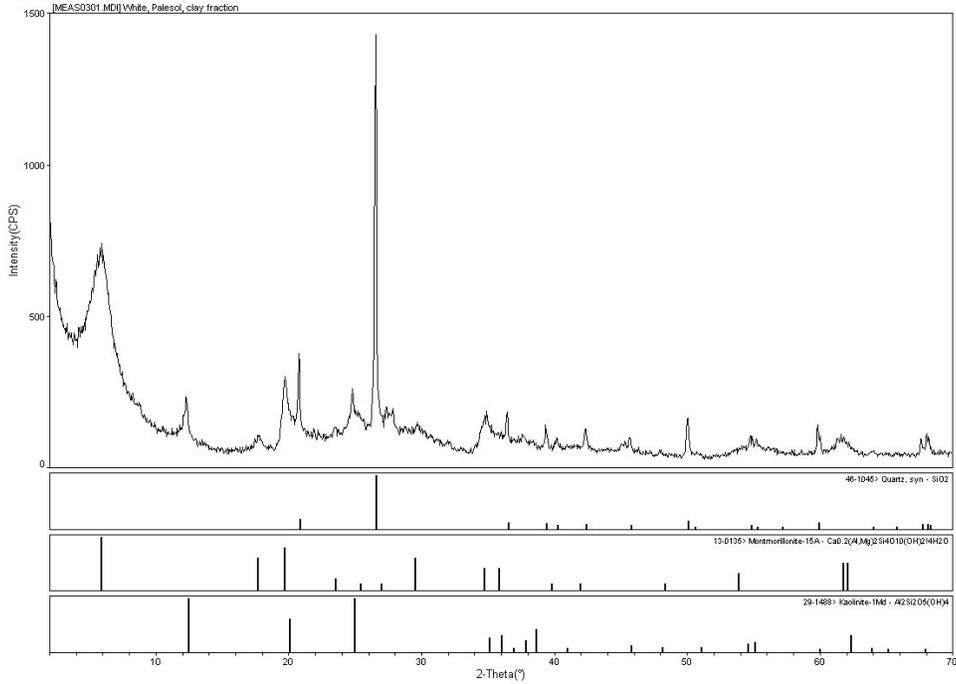
**APPENDIX A: X-RAY DIFFRACTOGRAMS FOR FLY ASHES AND HYDRATED AND  
CONDITIONED FLY ASHES**



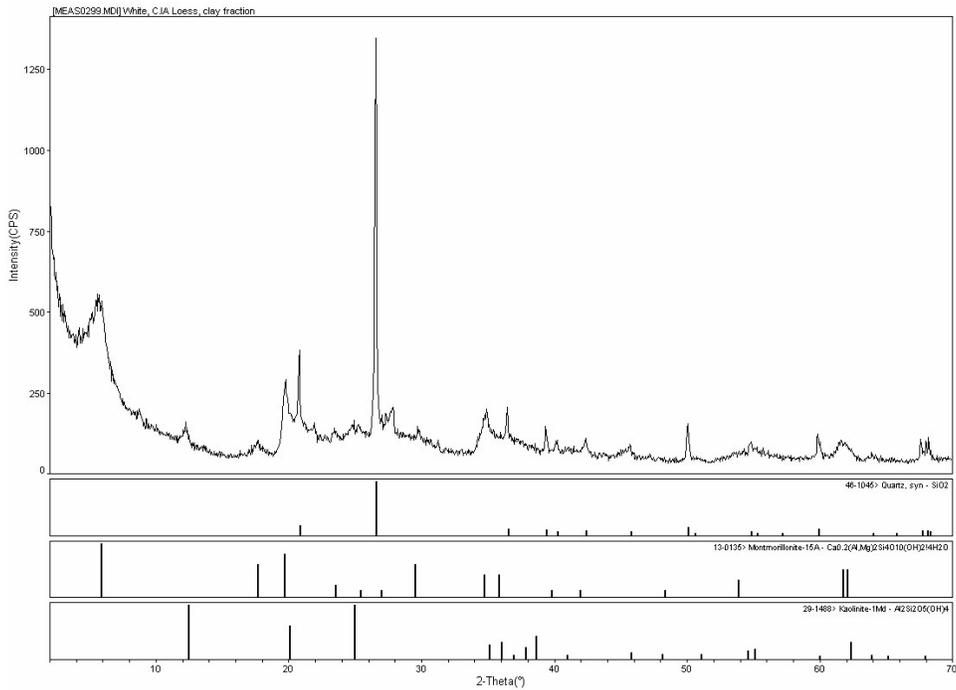
**Figure A.1. X-Ray diffractogram for Neola alluvium clay fraction (<2 microns)**



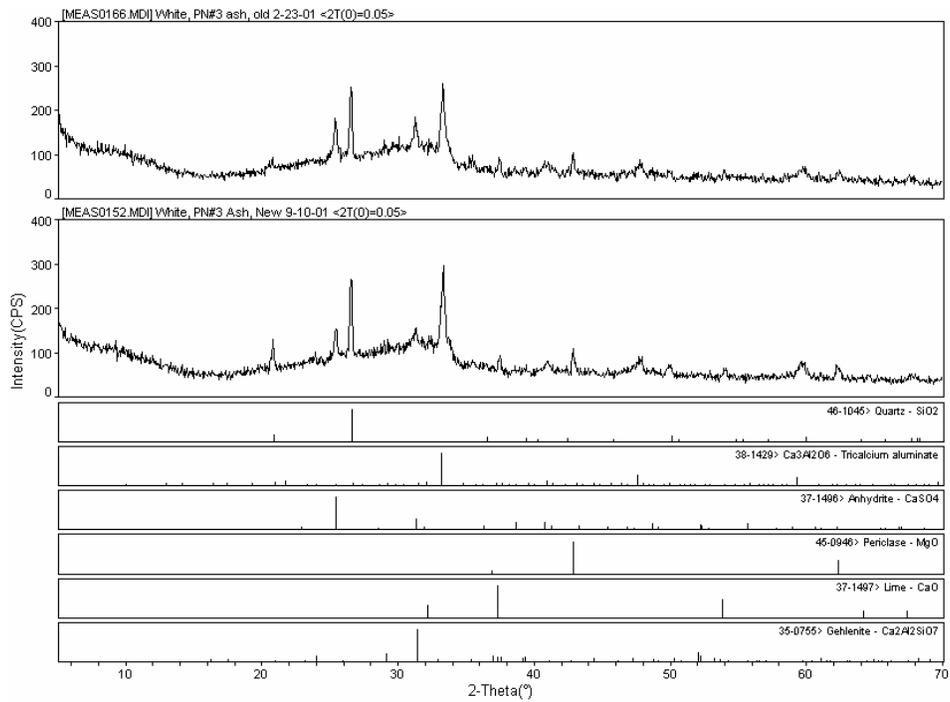
**Figure A.2. X-Ray diffractogram for Cedar Rapids till clay fraction (<2 microns)**



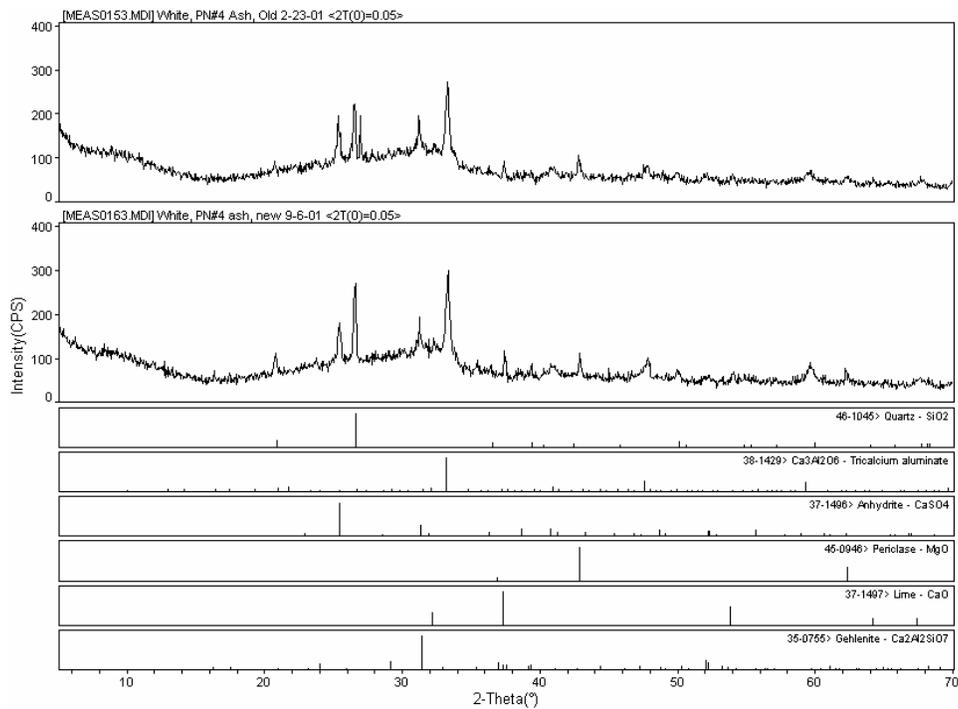
**Figure A.3. X-Ray diffractogram for Argyle paleosol clay fraction (<2 microns)**



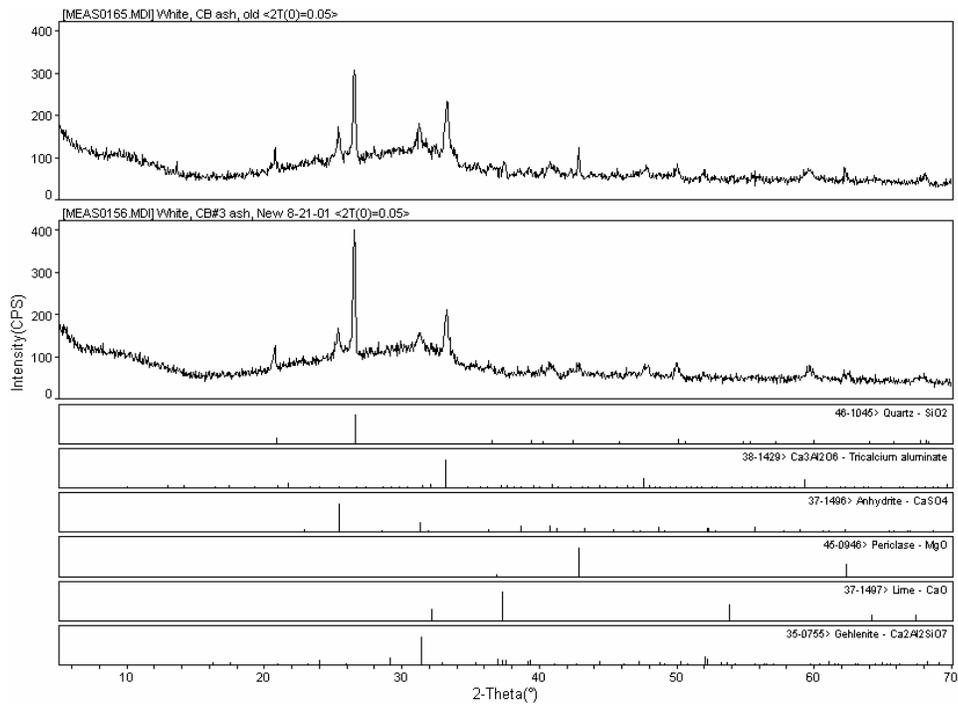
**Figure A.4. X-Ray diffractogram for Le Grand loess clay fraction (<2 microns)**



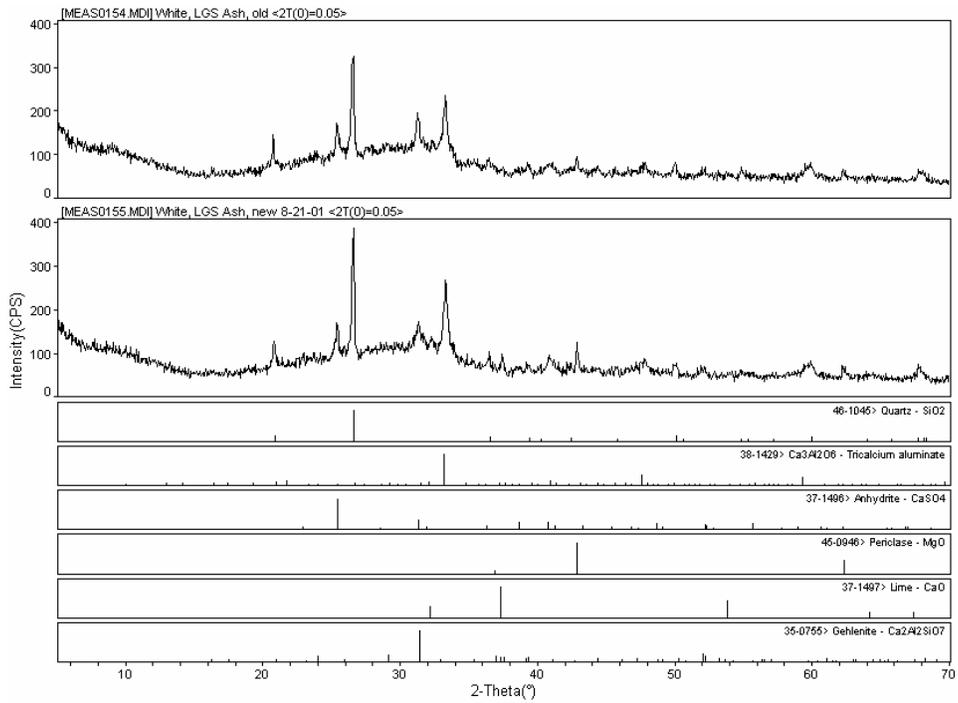
**Figure A.5. X-Ray diffractograms for PN3 old and PN3 new fly ash**



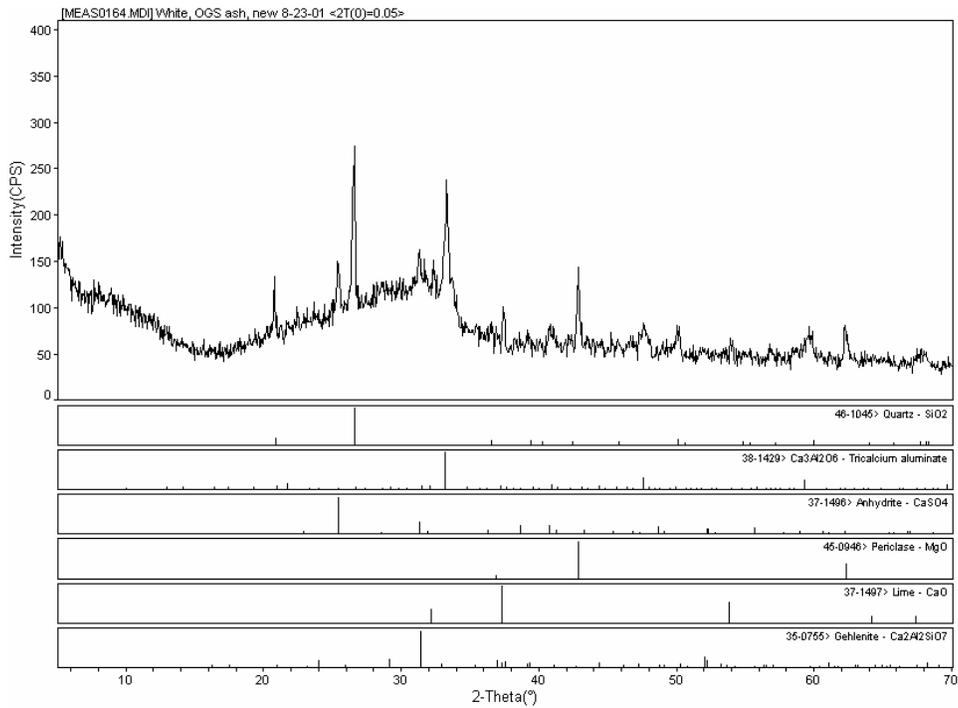
**Figure A.6. X-Ray diffractograms for PN4 old and PN4 new fly ash**



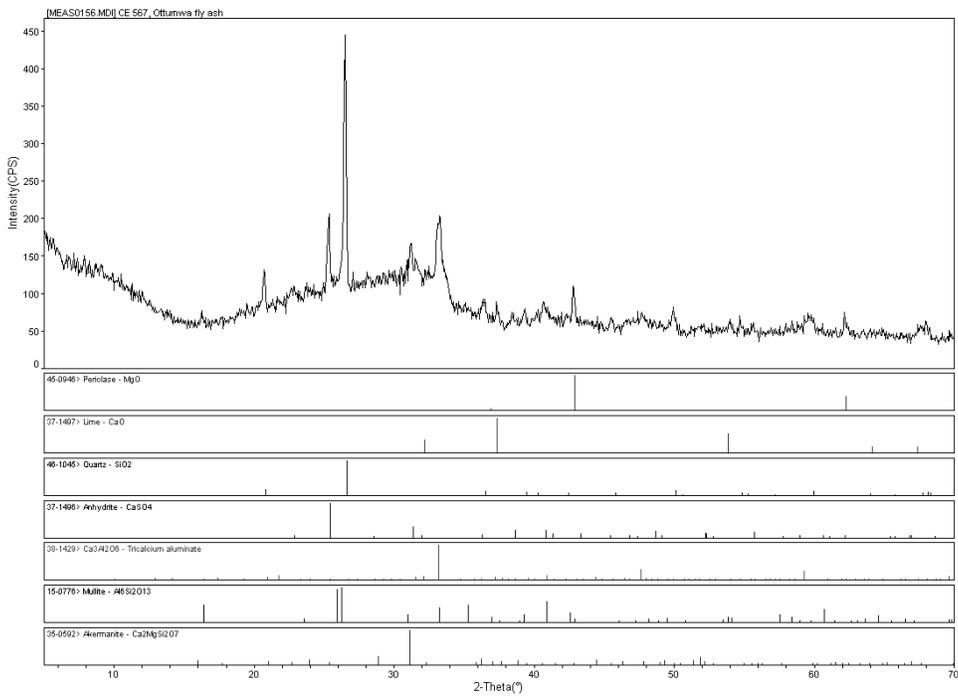
**Figure A.7. X-Ray diffractograms for CB old and CB new fly ash**



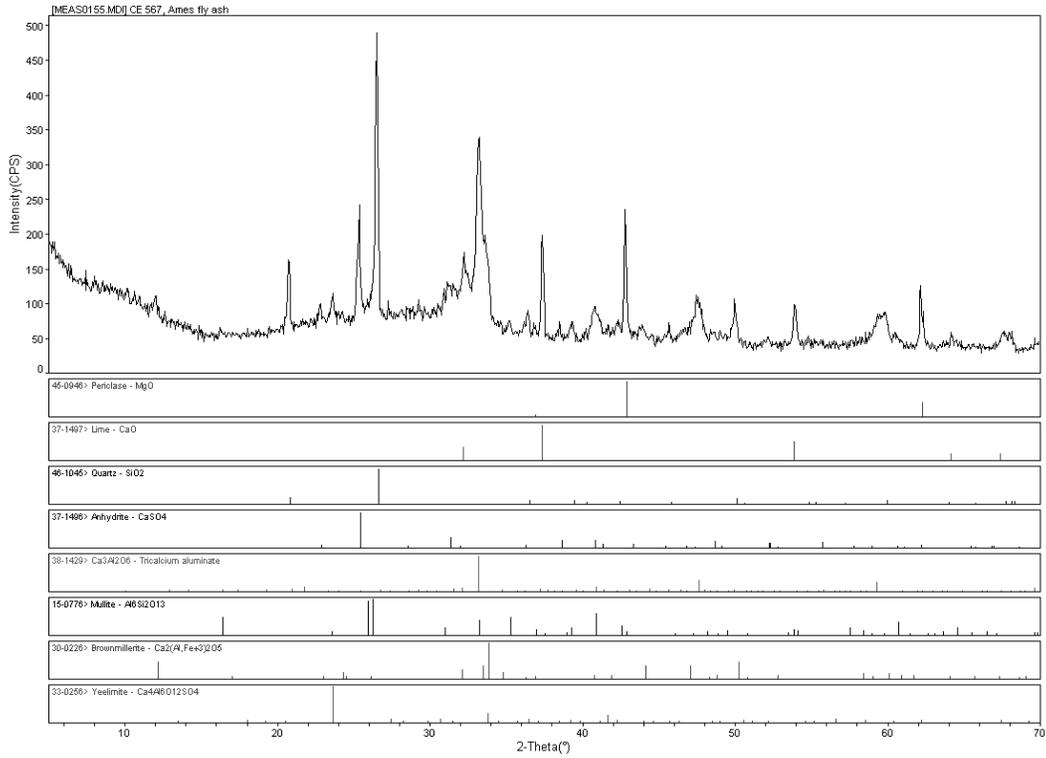
**Figure A.8. X-Ray diffractograms for LGS new and LGS old fly ash**



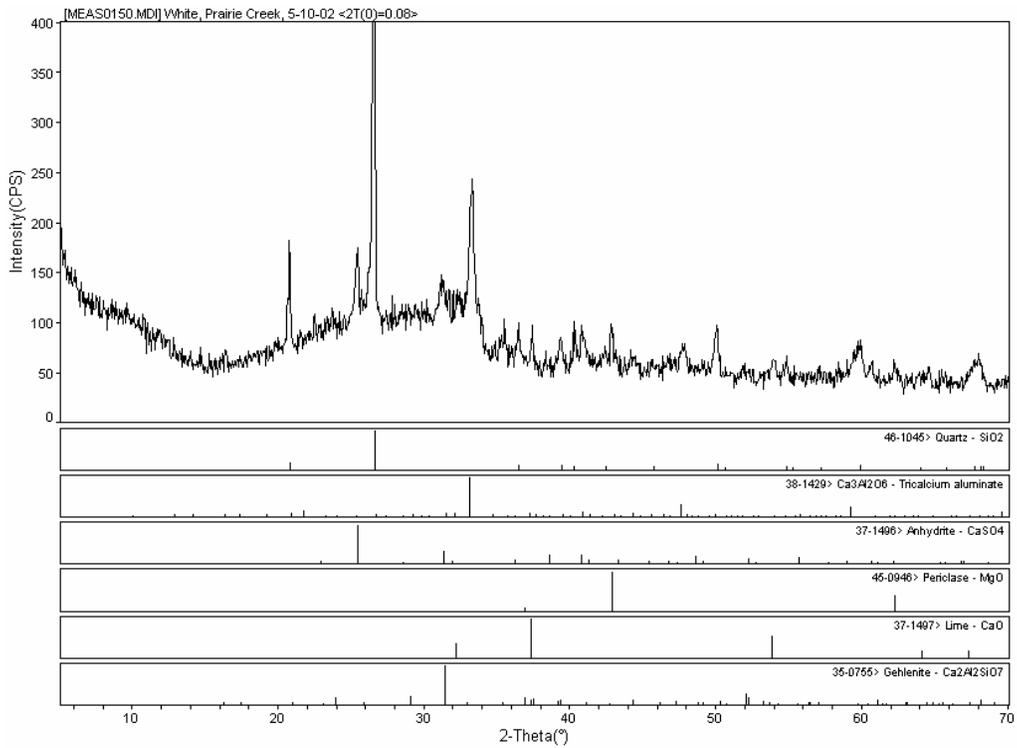
**Figure A.9. X-Ray diffractogram for OGS new fly ash**



**Figure A.10. X-Ray diffractogram for OGS old fly ash**



**Figure A.11. X-Ray diffractogram for Ames fly ash**



**Figure A.12. X-Ray diffractogram for PC3+4 fly ash**

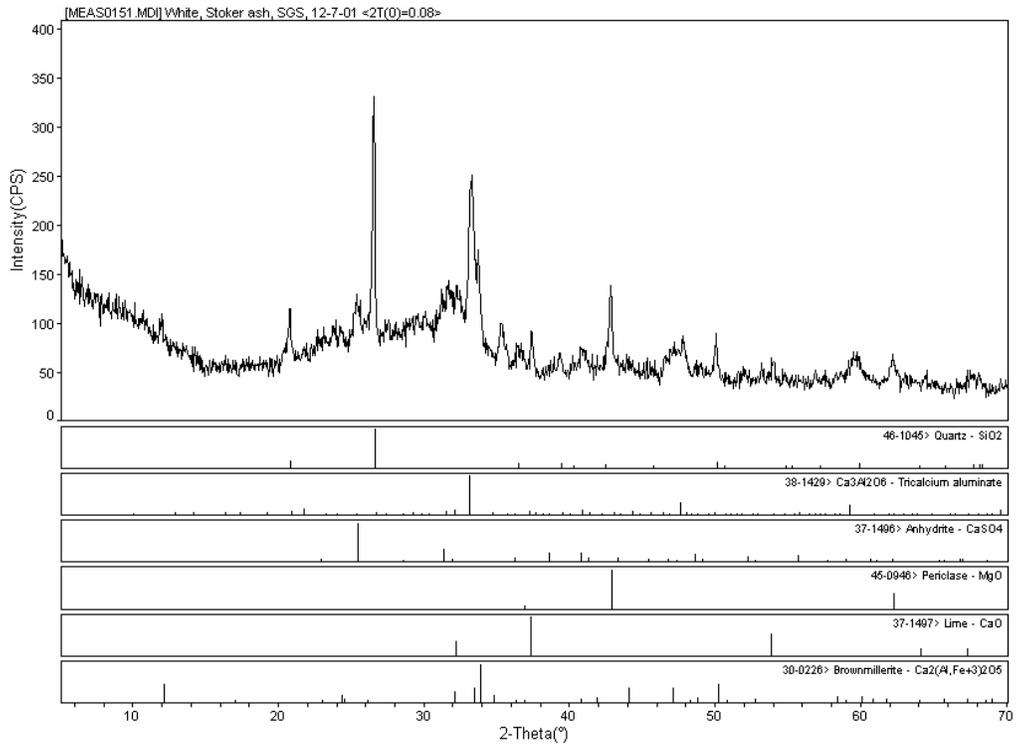


Figure A.13. X-Ray diffractogram for SGS fly ash

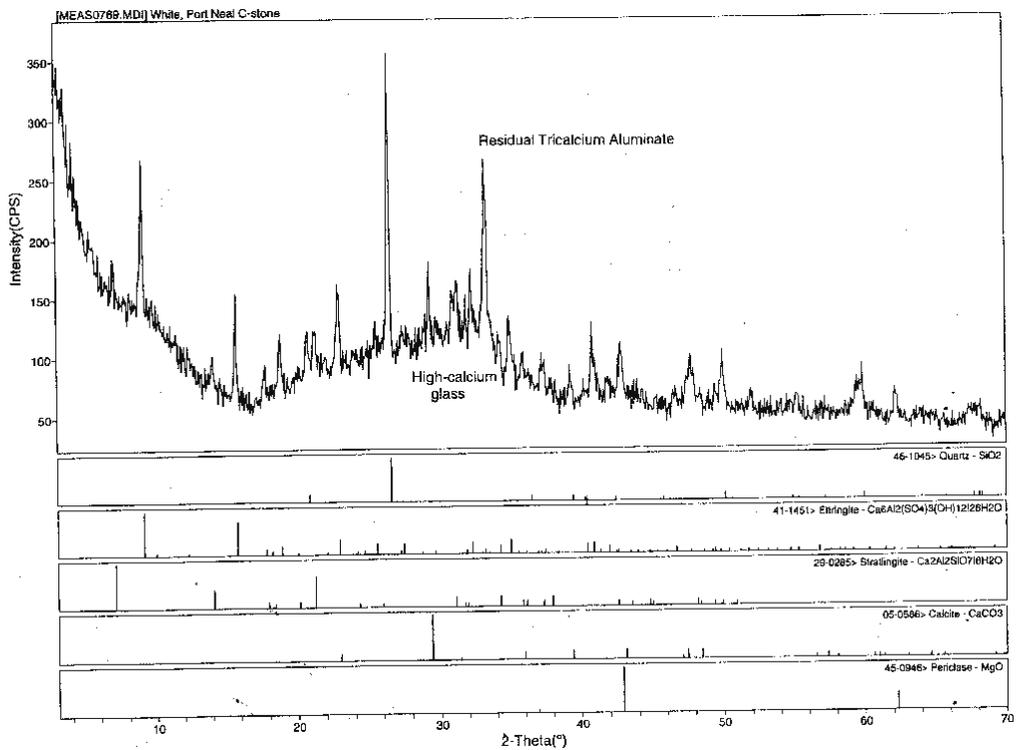


Figure A.14. X-Ray diffractogram for PN hydrated fly ash

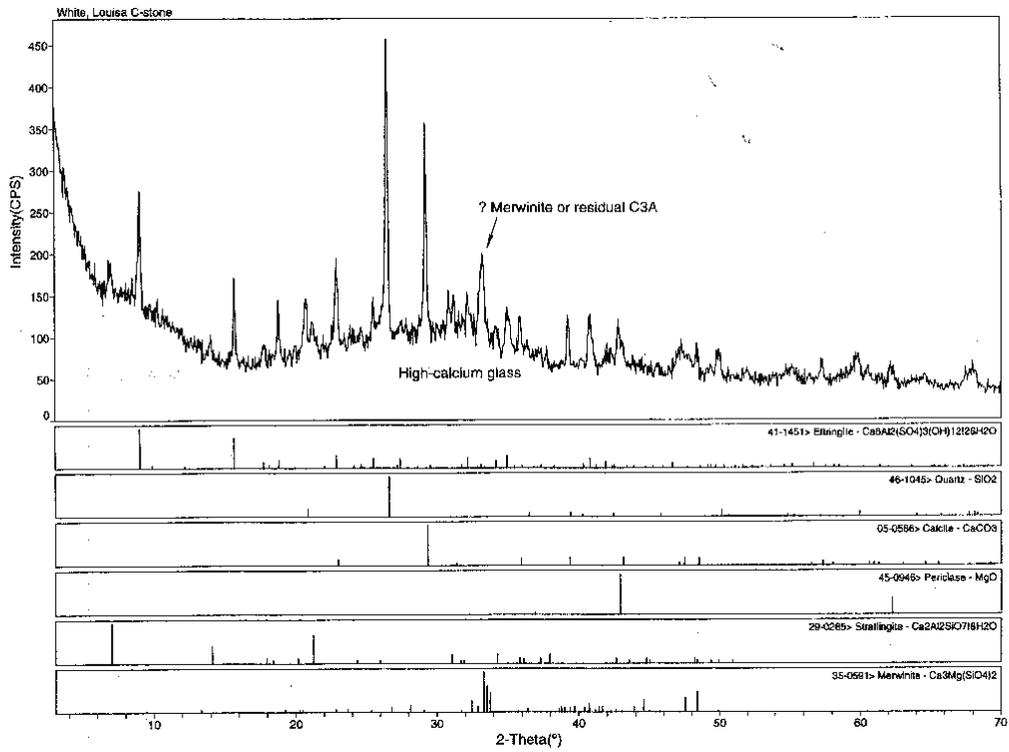


Figure A.15. X-Ray diffractogram for LGS hydrated fly ash

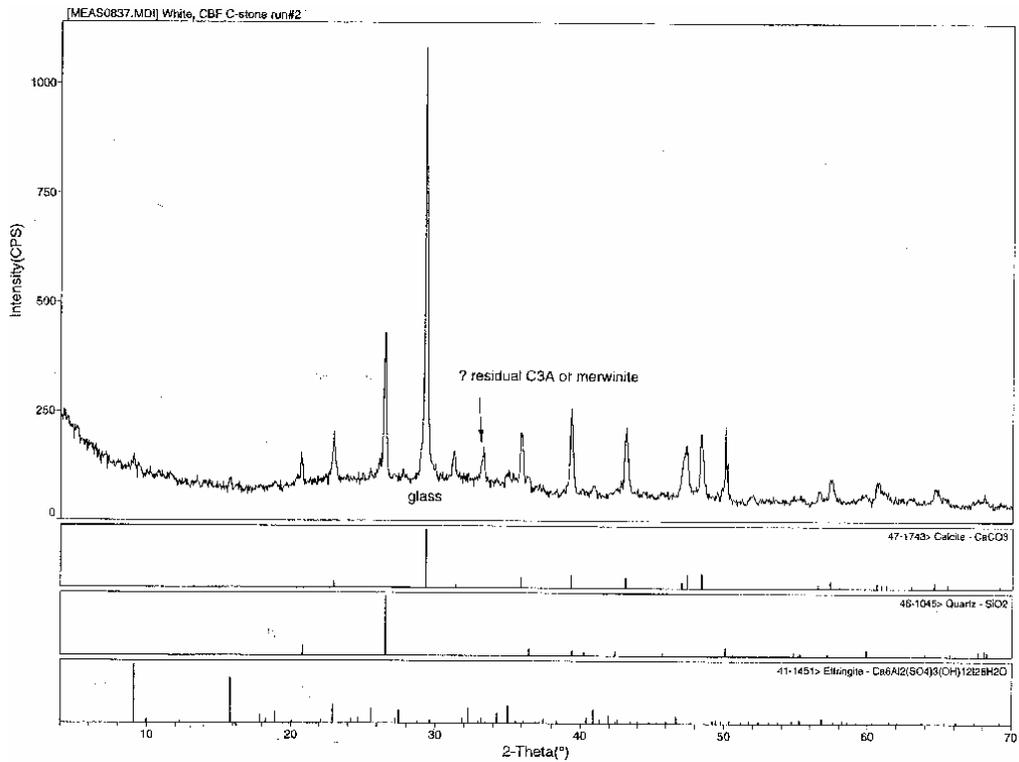
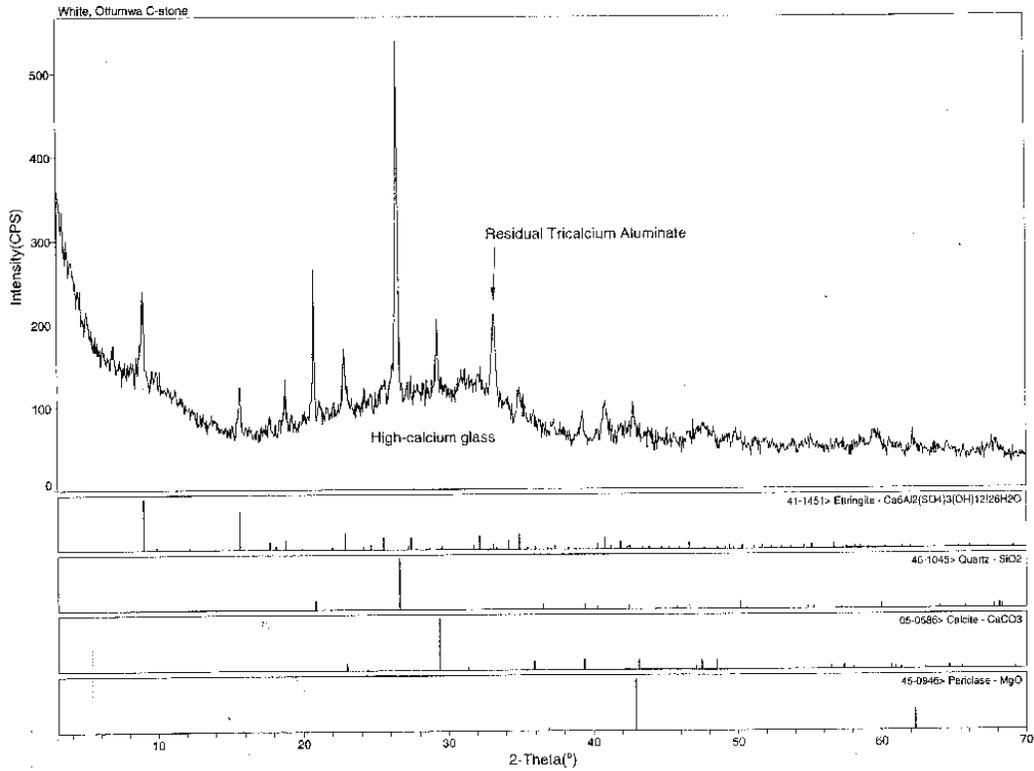
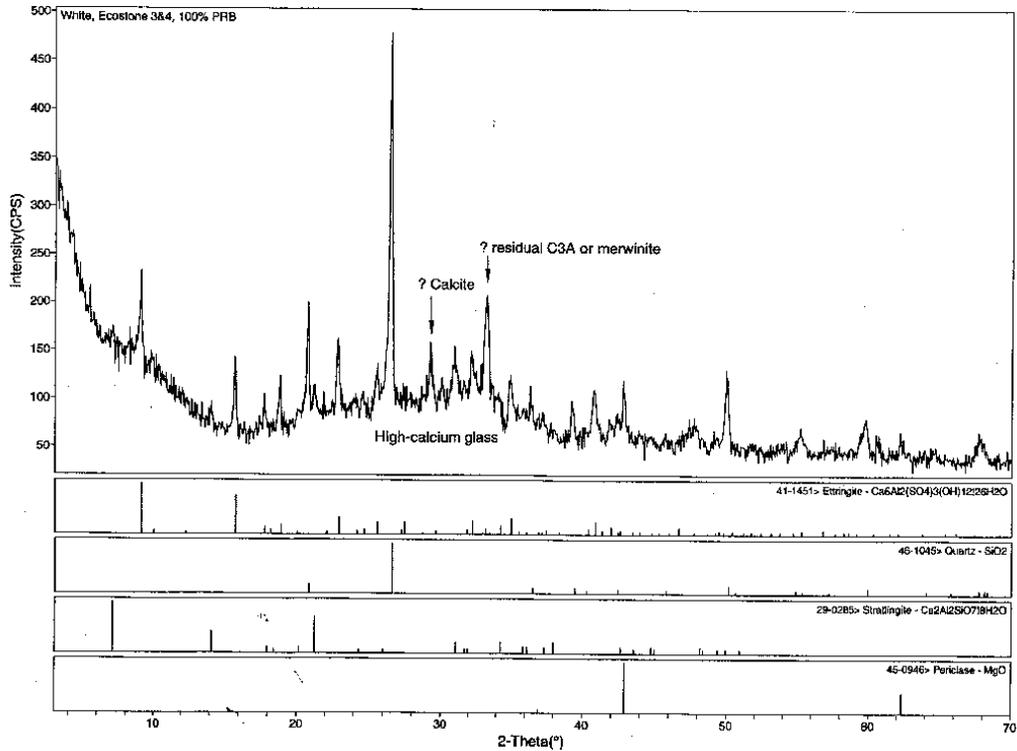


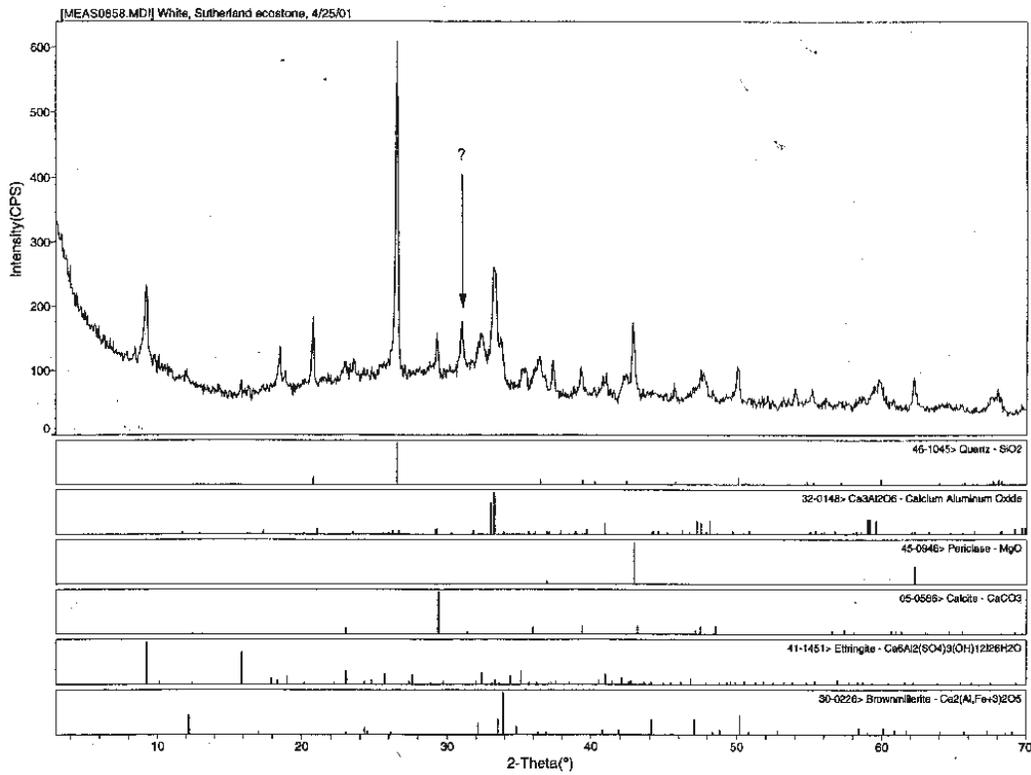
Figure A.16. X-Ray diffractogram for CB hydrated fly ash



**Figure A.17. X-Ray diffractogram for OGS hydrated fly ash**



**Figure A.18. X-Ray diffractogram for PC conditioned fly ash**



**Figure A.19. X-Ray diffractogram for SGS conditioned fly ash**

## **APPENDIX B: LABORATORY STRESS-STRAIN DATA**

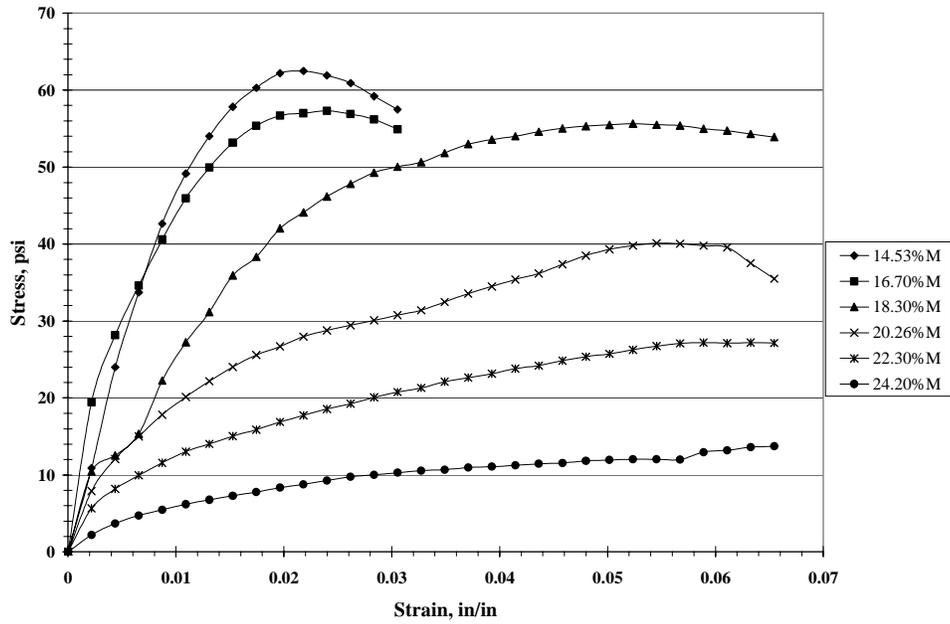


Figure B.1. Stress-strain response of Neola alluvium at various moisture contents

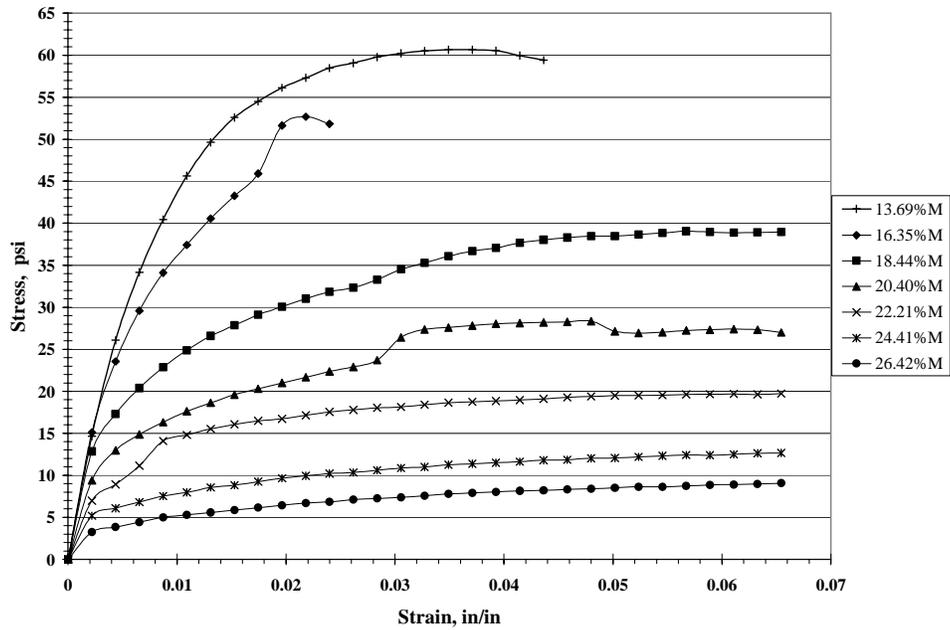


Figure B.2. Stress-strain response of Argyle paleosol at various moisture contents

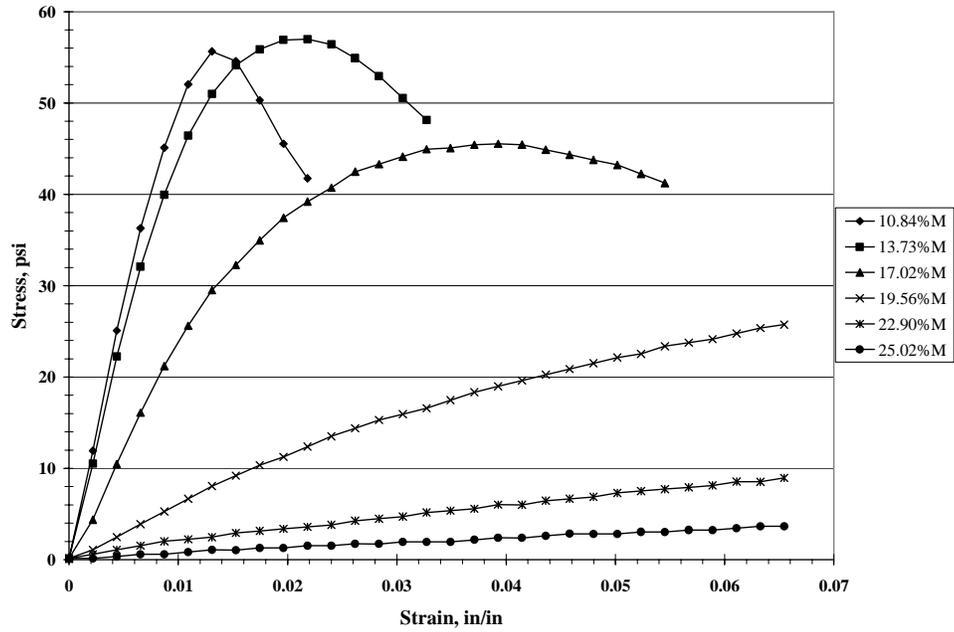


Figure B.3. Stress-strain response of Le Grand loess at various moisture contents

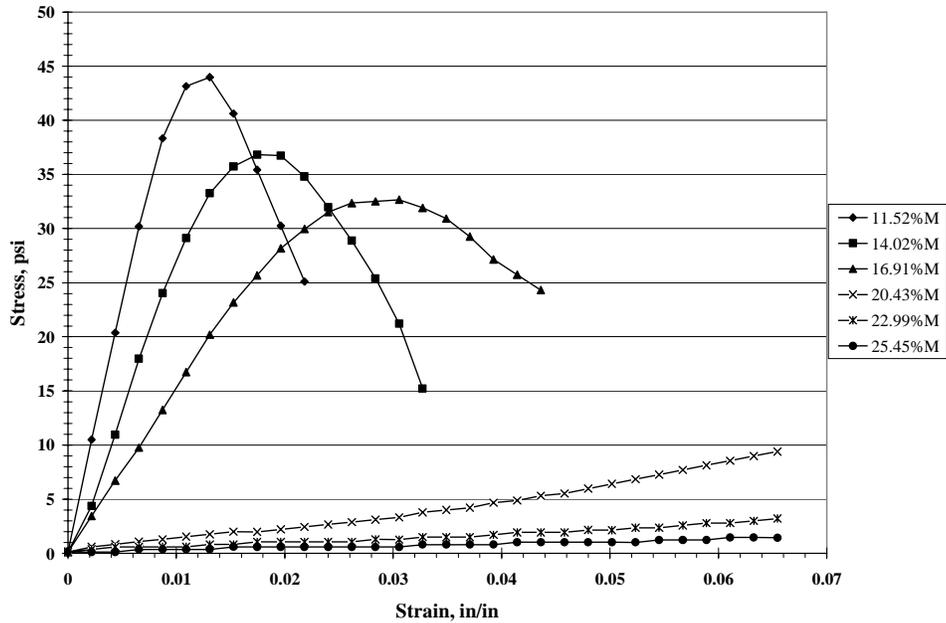


Figure B.4. Stress-strain response of Turin loess at various moisture contents

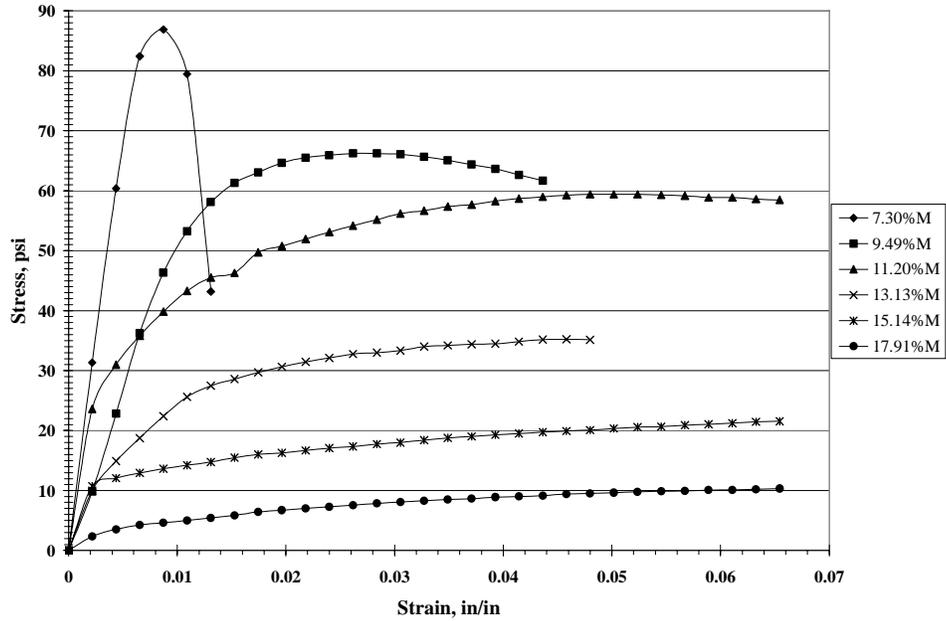


Figure B.5. Stress-strain response of Cedar Rapids glacial till at various moisture contents

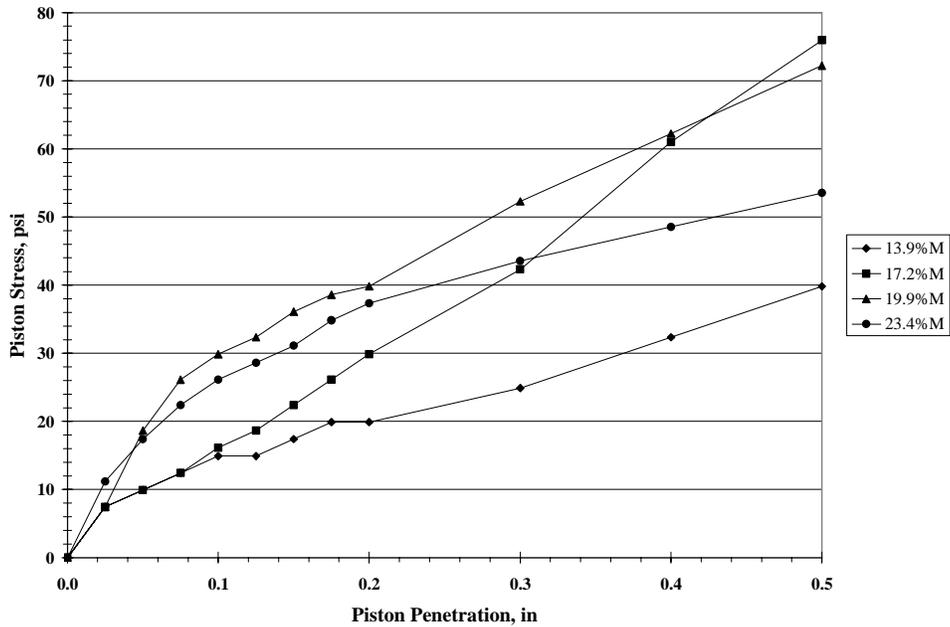


Figure B.6. CBR stress-penetration data for Le Grand loess at various moisture contents

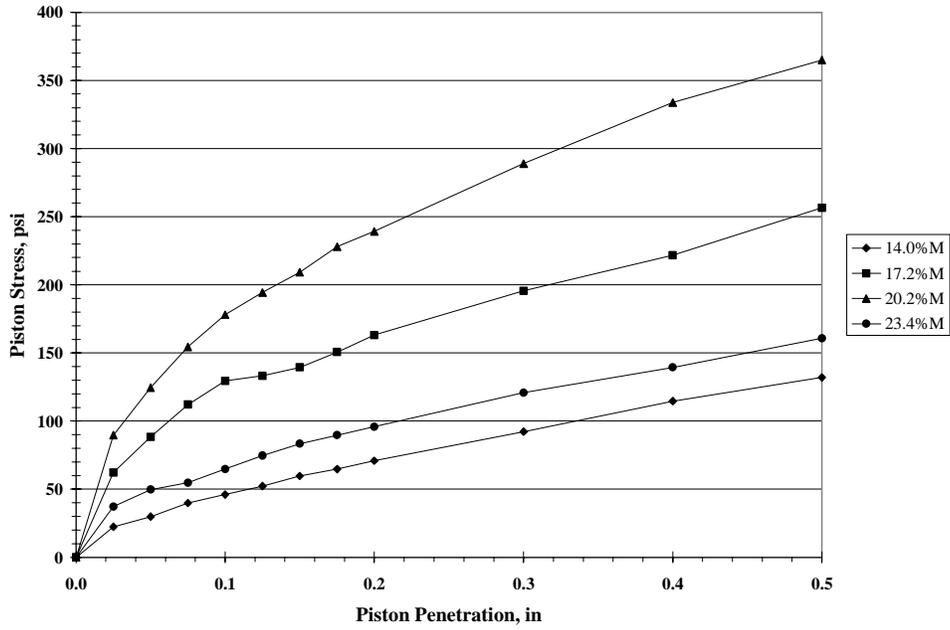


Figure B.7. CBR stress-penetration data for Le Grand loess and 5% LGS fly ash

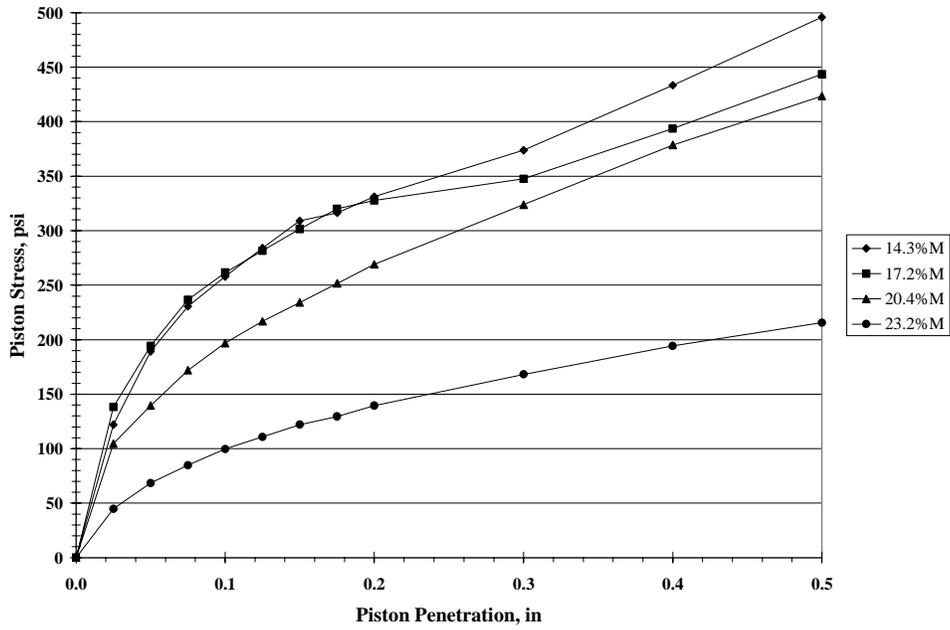
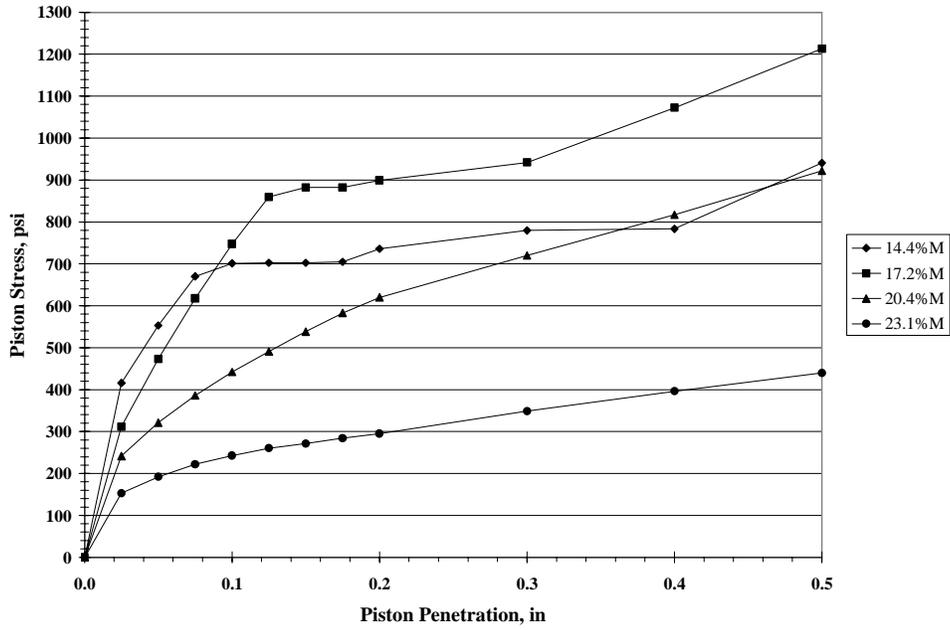


Figure B.8. CBR stress-penetration data for Le Grand loess and 10% OGS fly ash



**Figure B.9. CBR stress-penetration data for Le Grand loess and 20% PN4 fly ash**

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

**PROPOSED SPECIFICATION:  
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 2,000 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  $\pm 2\%$  (based on dry weight) of 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)	≥ 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, recompact, 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 two-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 four-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 four-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 four-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 testing 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  $\pm 2\%$  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 the following: Dynamic Cone Penetrometer, Clegg Hammer, and GeoGauge™, 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 C.1. Allowable differences for laboratory and field test measurements**

<b>Property</b>	<b>Reporting of Results</b>	<b>Acceptable Range for QC/QA Test Comparisons</b>
Field Moisture Content	0.1 % (based on dry weight)	± 1.0 %
Field Density Tests for Compaction	1 lb/ft <sup>3</sup> (20 kg/m <sup>3</sup> )	5 lb/ft <sup>3</sup> (80 kg/m <sup>3</sup> )
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 <sup>3</sup> (20 kg/m <sup>3</sup> )	5 lb/ft <sup>3</sup> (80 kg/m <sup>3</sup> )
Standard Proctor Laboratory “Maximum” Unconfined Compressive Strength (based on strength)	5 lb/in <sup>2</sup> (30 kPa)	10 lb/in <sup>2</sup> (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 (2,000 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 hour 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 increases. Moisture-density relationship can be plotted as shown in Figure C1. Influence of compaction delay time on dry unit weight is shown in Figure C2.

### **8.2 Moisture-Strength Relationship**

Samples prepared for determination for “Moisture-Density Relationship” may be cured and 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 humidity, 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 for four hours before testing. Iowa State University 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.<sup>2</sup> (345 kPa) to 100 lb/in.<sup>2</sup> (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

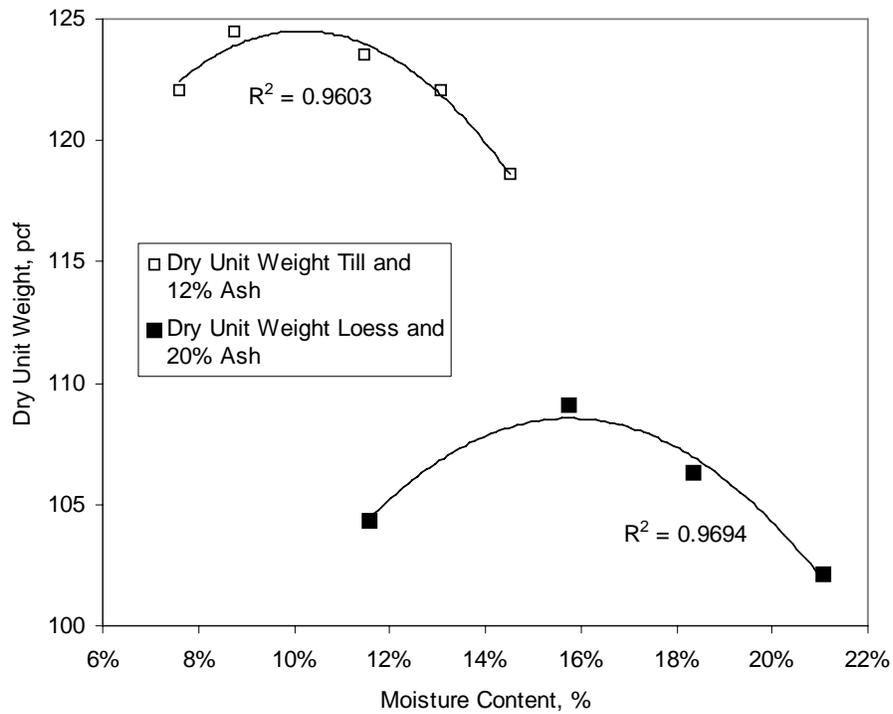
relationships, as shown in Figure C3. Typically, "optimum" moisture content for strength varies from -5% to +5% of that determined for density.

#### 8.4 Example Fly Ash Addition Rate Design Calculations

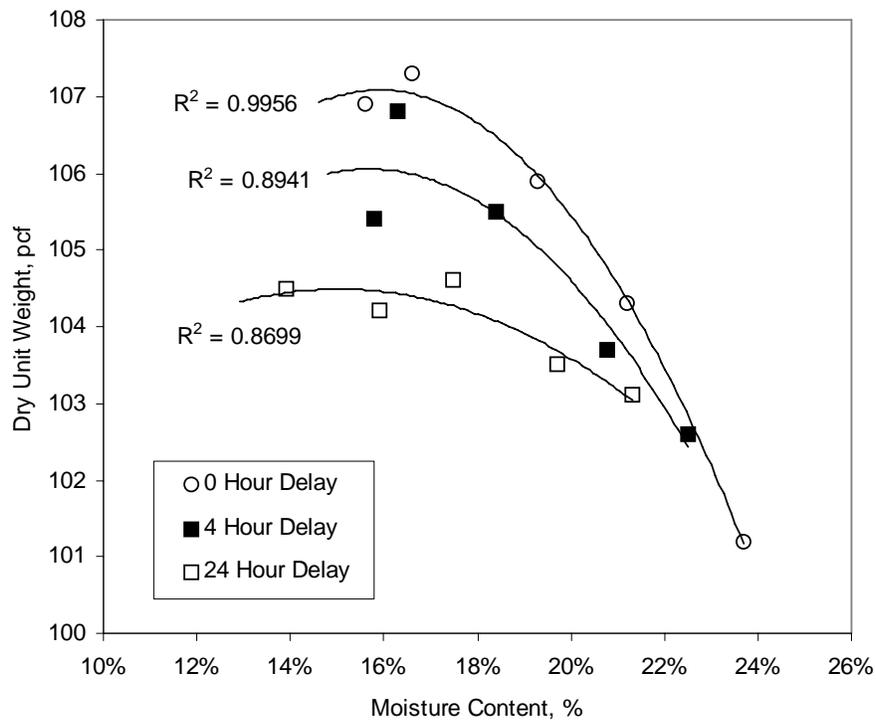
Specified Fly Ash Content	11% (by dry weight of subgrade soil) Note: Typically specify 1% greater than lab optimum
Standard Proctor Dry Unit Weight of Subgrade Soil	114 lb/ft <sup>3</sup>
Depth of Stabilized Section	12 inches
Weight of Fly Ash	22 tons/truck load
Rate of Fly Ash Distribution	$(114 \text{ lb/ft}^3)(11\%)(1\text{ft}) = 12.54 \text{ lb/ft}^2$
Area to be Covered by Truck Load of Ash	$(22 \text{ tons} \times 2,000 \text{ lb})/12.54 \text{ lb/ft}^2 = 3,509 \text{ ft}^2$
Length to Spread for 8 ft-Wide Section	$3,509 \text{ ft}^2/8\text{ft} = 439 \text{ ft}$

## 9. REFERENCES

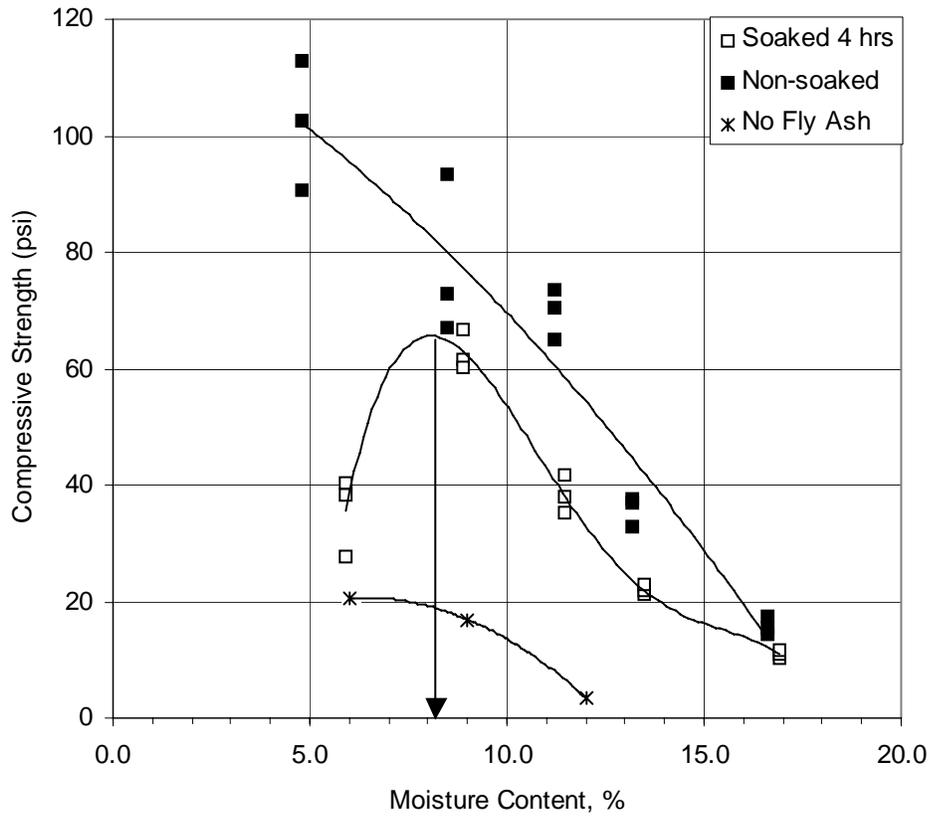
Chu, T.Y. and Davidson, D.T. (1955). "Some Laboratory Tests for the Evaluation of Stabilized Soils." Iowa Engineering Experiment Station, Project 283-S, Iowa Highway Research Board.



**Figure C.1. Typical moisture-density relationships of stabilized soil**



**Figure C.2. Influence of compaction delay on dry unit weight**



**Figure C.3. Typical moisture-strength relationships for soaked and unsoaked fly ash modified subgrade. (For soaked samples, optimum moisture content is approximately 8.2%)**

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

**SUGGESTED SPECIFICATION  
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<sup>3</sup> 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 than 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<sub>2</sub>) + aluminum oxide (Al<sub>2</sub>O<sub>3</sub>) + iron oxide (Fe<sub>2</sub>O<sub>3</sub>) ≥ 50.0 and ≤ 70.0 percent
- Sulfur trioxide (SO<sub>3</sub>) ≤ 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 D1, 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 sheepfoot may be used if shown to meet the required relative compaction level. The fill shall be compacted to  $\geq 90$  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 four 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 two-inch layer of well-graded crushed limestone on the completed course
- (f) Build the section two inches high and trim prior to paving

A minimum of three 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 (2,000 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 D1 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°C 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 D.1. Analytical chemical composition of a typical HFA expressed on an LOI-free basis**

Constituent	As-received (mass %)	LOI-free basis (mass %)
Na <sub>2</sub>	1.18	1.76
MgO	4.12	6.16
Al <sub>2</sub> O <sub>3</sub>	11.78	17.62
SiO <sub>2</sub>	22.26	33.29
P <sub>2</sub> O <sub>5</sub>	0.63	0.94
SO <sub>3</sub>	1.65	2.47
K <sub>2</sub> O	0.23	0.34
CaO	18.78	28.08
TiO <sub>2</sub>	0.91	1.36
Mn <sub>3</sub> O <sub>4</sub>	0.04	0.06
Fe <sub>2</sub> O <sub>3</sub>	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 five moisture contents evaluated. Results can be plotted as shown in Figure D1.

### 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

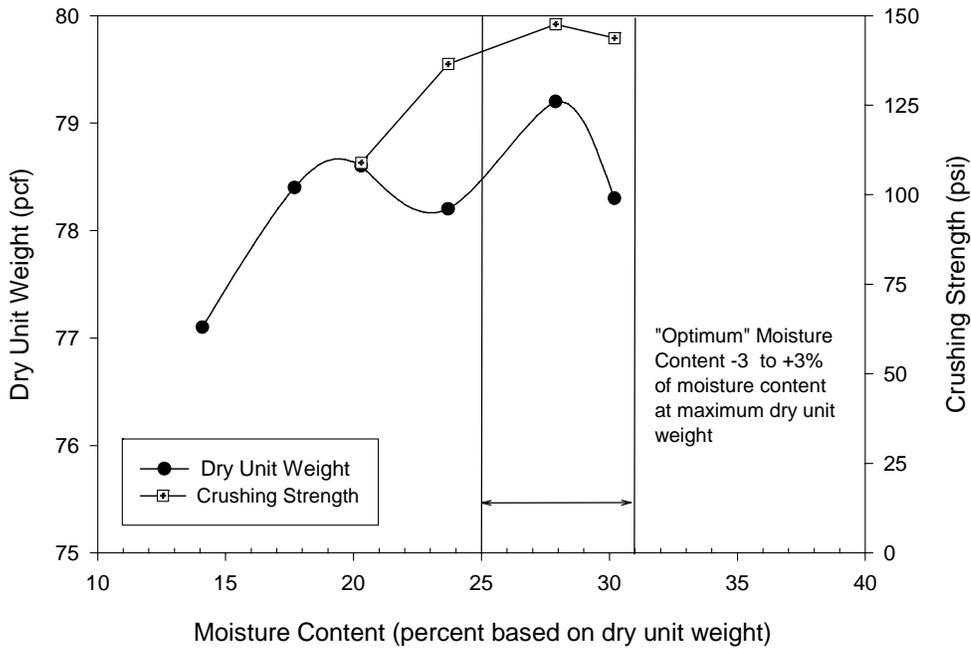
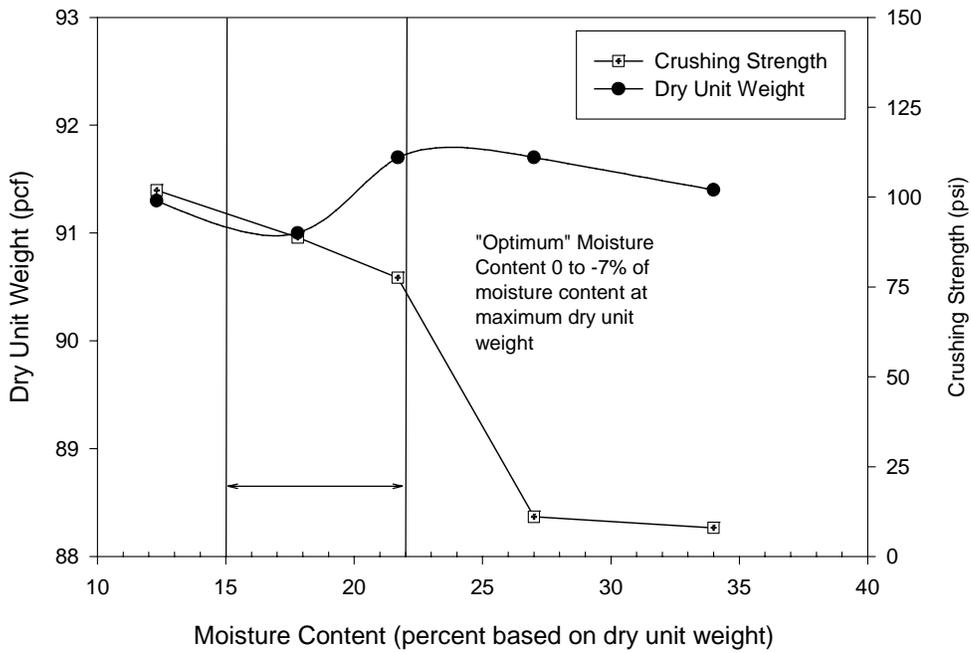
strength testing shall be compacted in a Proctor mold of the split type. Once compacted, the sample should be removed from the mold, weighed, and measured for height. To cure, the specimen is wrapped in plastic wrap and aluminum foil, labeled, sealed in an air-free plastic zip-lock bag, and then stored 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% of the moisture content at the maximum dry unit weight. Sample laboratory test results are shown in Figure D1.



**Figure D.1. Typical 28-day moisture-density and moisture-strength relationships for HFA materials**

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

**SUGGESTED SPECIFICATION  
USE OF RECLAIMED CONDITIONED 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**

**2.1 Conditioned 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<sup>3</sup> 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 than 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<sub>2</sub>) + aluminum oxide (Al<sub>2</sub>O<sub>3</sub>) + iron oxide (Fe<sub>2</sub>O<sub>3</sub>) = 50.0 to 70.0 percent
- Sulfur trioxide (SO<sub>3</sub>) ≤ 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 E1, 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  $\geq 90$  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 four 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 two-inch layer of well-graded crushed limestone on the completed course
- (i) Build the section two inches high and trim prior to paving

A minimum of three 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 (2,000 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 E1 shows a typical bulk analytical chemical analysis of a CFA sample via X-ray fluorescence. The assay is expressed on an as-received basis. This is done because the drying processes (105°C 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.1. Analytical chemical composition of a typical CFA expressed on an LOI-free basis**

<b>Constituent</b>	<b>As-received (mass %)</b>	<b>LOI-free basis (mass %)</b>
Na <sub>2</sub>	1.85	2.55
MgO	4.04	5.58
Al <sub>2</sub> O <sub>3</sub>	12.8	17.7
SiO <sub>2</sub>	23.6	32.6
P <sub>2</sub> O <sub>5</sub>	0.89	1.23
SO <sub>3</sub>	1.83	2.53
K <sub>2</sub> O	0.39	0.54
CaO	19.8	27.4
TiO <sub>2</sub>	1.07	1.47
Fe <sub>2</sub> O <sub>3</sub>	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 five moisture contents evaluated. Results can be plotted as shown in Figure E1.

### 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

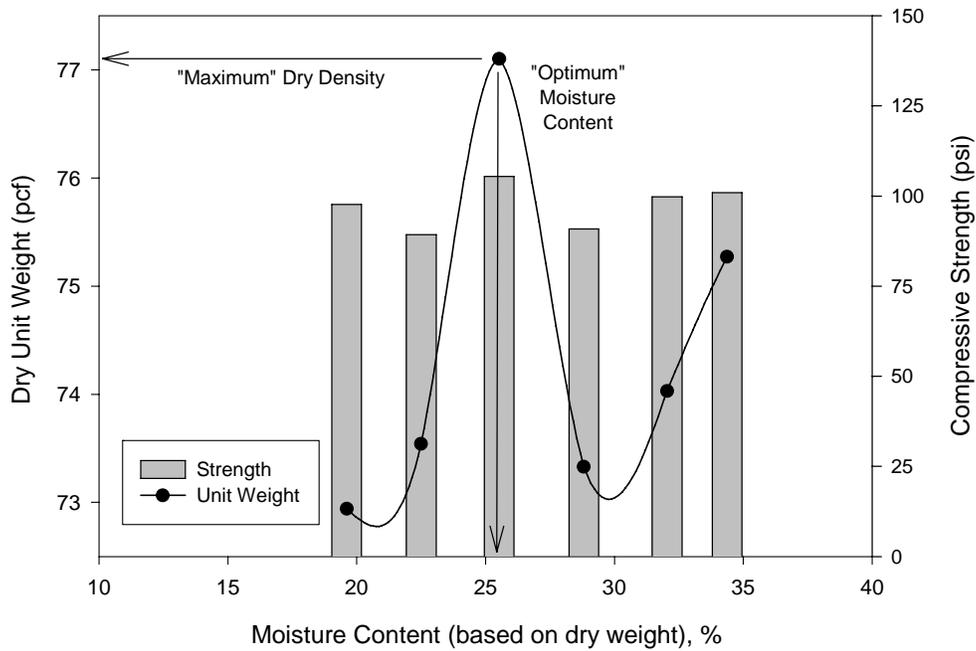
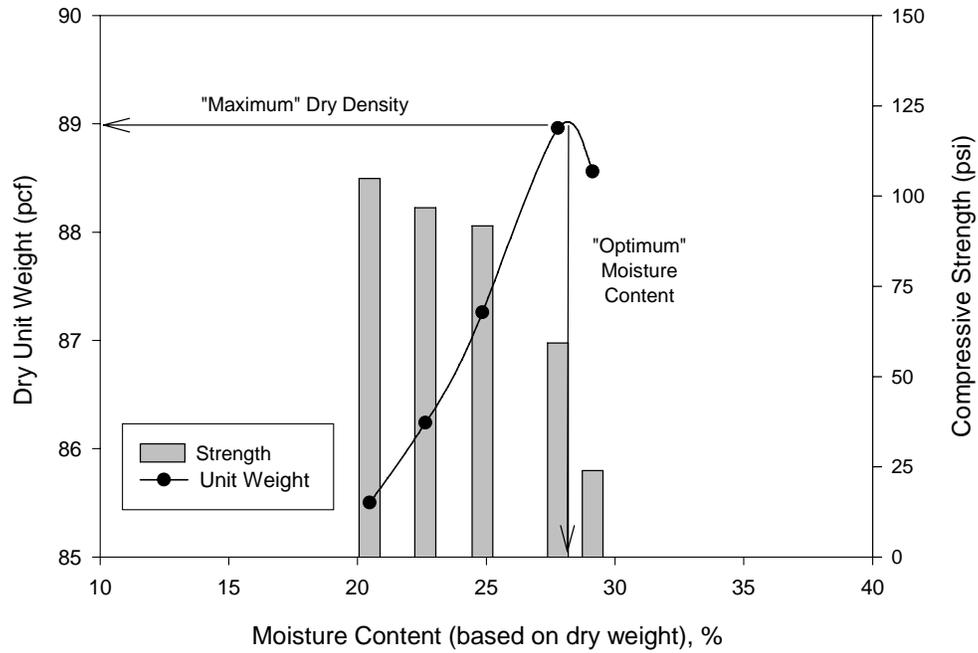
specimen is wrapped in plastic wrap and aluminum foil, labeled, and sealed in an air-free plastic zip-lock bag. Extra sample material should be prepared for each group of samples in order to evaluate the moisture content at time of compaction.

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.

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**Figure E.1. Typical moisture-density and moisture-strength relationships for CFA materials**