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Subgrade Stabilization Alternatives to Lime and Cement

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16. Abstract

This project involved four distinct research activities, (1) the influence of temperature on lime-stabilized soils, (2) the influence of temperature on cement-stabilized soils (3) temperature modeling of stabilized subgrade and (4) use of calcium chloride to accelerate strength gain of cement-stabilized soils.

Significant conclusions from the lime-focused research include that the minimum lime content of a soil increases as temperature decreases. Increased curing duration leads to decreases in pore fluid pH. However, this reduction in pH is less at lower temperatures, indicating that little reactivity occurs below 50°F. Exposure to freeze-thaw cycles or initial curing at 35°F resulted in significant reductions in strength gain for a given curing duration. However, once the freeze-thaw cycles or temperature reduction was removed, strength gain resumed at approximately the same rate. Overall, these results suggest that current specifications may be modified to allow lime stabilization to proceed in cooler temperatures, provided a corresponding increase in curing time and/or thermal protection is provided prior to loading.

The soil cement data indicate that curing soil-cement at lower temperatures will result in lower strengths. For example, the 7 day strength for samples cured at 25°F was less than the strength of samples cured at 50°F or 70°F by a factor ranging from 2-6. Likewise, the 7 day strength for samples cured at 35°F was less than the strength of samples cured at 50°F or 70°F by approximately 20-25%. Additionally, on the basis of 15 repeat tests for 3 and 7 day curing periods, for three different soils, results indicate that the mean strength at 3 days is 84-93% of that for 7 days, in support of a potential change in current subgrade evaluation practice predicated on the longer duration.

Specifications for stabilization work have often been based on air temperature measurements, however the performance of lime or cement treated soil is expected to be more closely related to the in situ temperature. This research has found that the thermal diffusivity of both lime and cement-stabilized subgrades varies from 3.8×10^{-7} m^2/s (2.14 in^2/hr) to 9.8×10^{-7} m^2/s (5.46 in^2/hr). These data were incorporated into a model that relates air and soil temperatures. A computer application was developed to use the model to make predictions of subgrade temperatures and cured strength.

A window of efficacy was observed for Buncombe, Guilford and Johnston county soil cement mixes, with ideal ranges at 50°F curing conditions of 0.25% – 1.0%, 0.25%-0.75% and 1.25%-1.75%, respectively (percentages reflect mass of $CaCl_2$ per mass of cement). However these optimum ranges vanished or changed to levels untested at 35°F curing conditions. A field trial was conducted with $CaCl_2$ doses of 2.3% and 8.3%, and both of these dosage levels resulted in weakening of the material, as evaluated by in situ dynamic cone penetration tests and unconfined compression testing of field-mixed samples. Laboratory mixed samples of the same material resulted in strength increases. $CaCl_2$ modification increases the electrical conductivity and dielectric value of soil-cement mixtures which might imply increased susceptibility to longer term moisture-induced weakening at high dosage levels. The cost of $CaCl_2$ modification at effective doses is likely to be less than 10% of cement costs. The overall body of research presented in this report suggests that $CaCl_2$ modification of soil-cement is not a mature enough approach to serve as a method for mitigating the effects of low temperatures on strength gain. Additional data are required to probe the sensitivity of temperature, mixing method and soil type.

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Summary

This project involved four distinct research activities, (1) the influence of temperature on lime-stabilized soils, (2) the influence of temperature on cement-stabilized soils (3) temperature modeling of stabilized subgrade and (4) use of calcium chloride to accelerate strength gain of cement-stabilized soils. The report is therefore organized into four chapters and appropriate suggestions for implementation are located within each chapter.

Significant conclusions from the lime-focused research include that the minimum lime content of a soil increases as temperature decreases. Increased curing duration leads to decreases in pore fluid pH. However, this reduction in pH is less at lower temperatures, indicating that little reactivity occurs below 50°F. Exposure to freeze-thaw cycles or initial curing at 35°F resulted in significant reductions in strength gain for a given curing duration. However, once the freeze-thaw cycles or temperature reduction was removed, strength gain resumed at approximately the same rate. Overall, these results suggest that current specifications may be modified to allow lime stabilization to proceed in cooler temperatures, provided a corresponding increase in curing time and/or thermal protection is provided prior to loading.

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CHAPTER 1: Lime-Stabilized Subgrade Soils

Chapter Summary

Lime stabilization is one of the most practical and cost effective techniques of subgrade stabilization in pavement design. However, an undesirable characteristic is the presumed inability of lime-stabilized soils to achieve necessary strengths at colder temperatures, possibly resulting in premature pavement failures. The goal of this research was to more clearly define this relationship between soil-lime reactions and temperature through pH, conductivity, and unconfined compression testing. Significant conclusions from this research include that the minimum lime content of a soil increases as temperature decreases. Increased curing duration leads to decreases in pore fluid pH. However, this reduction in pH is less at lower temperatures, indicating that little reactivity occurs below 50°F. Short-term curing durations show no significant gain in unconfined compression strength, while long-term curing durations lead to significant strength gains, which suggests that pozzolanic reactions do not begin to take place until after 7 days of curing. Long-term curing durations also show the trend of increasing unconfined compressive strength with increasing temperature. However, lower temperatures resulted in less strength gain. For example, 7-day sample strengths at 35ºF increased by only 10% with a 56-day cure, while those samples cured at 70ºF nearly doubled when curing duration was increased from 7 to 56 days. Exposure to freeze-thaw cycles or initial curing at 35°F resulted in significant reductions in strength gain for a given curing duration. However, once the freeze-thaw cycles or temperature reduction was removed, strength gain resumed at approximately the same rate. Overall, these results suggest that current specifications may be modified to allow lime stabilization to proceed in cooler temperatures, provided a corresponding increase in curing time and/or thermal protection is provided prior to loading.

1.1 Introduction

Lime stabilization is a widely utilized practice in North Carolina for subgrade stabilization in pavement design. Not only is it effective in strength increases in subgrade, but it also a cost efficient option in both initial and later maintenance costs. Other alternatives to soil stabilization through chemical additives, such as lime and cement, are recompacting the soil subgrade, hauling in aggregate for stabilization, undercutting and hauling in select fill, or placing geosynthetics. Recompaction of subgrade soil may not always meet the required strengths, while aggregate and geosynthetic stabilization is typically more costly. Lime stabilization solves both of these problems because the addition of lime to high plasticity, fine-grained soils normally yields the necessary strengths for subgrade while being a more cost effective alternative. Lime is also effective in reducing moisture susceptibility. However, an undesirable attribute of lime stabilization is its presumed inability to perform under colder curing temperatures. According to Section 501-3 of the North Carolina Department of Transportation Standard Specifications for Roads and Structures, lime stabilization construction cannot take place when the air temperature is below 45ºF or in the period from November 1 to March 15 (NCDOT 2006). This research evaluates the suitability of this specification and concentrates on the performance of lime-stabilized soils under varying curing temperatures with particular focus on relatively low curing temperatures.

The principal purpose of this research was to evaluate the effects of curing temperature on lime stabilization of three North Carolina soils. This investigation initiated with soil characterization testing, including Atterberg limits and grain-size analysis. It continued with exploration of reactivity of lime-soil interactions and minimum lime content through both pH and conductivity testing. These tests were performed as a function of temperature to provide insight on seasonal response. Once lime-soil reactivity trends and minimum lime content were established, the next phase of laboratory testing was unconfined compression testing of lime-soil mixtures under various curing temperatures and curing durations. Curing temperatures included 70°F, 50°F, 35°F, 25°F, freeze-thaw cycles, and "spring thaws" while curing duration included a short-term durations of 1, 3, and 7 days as well as long-term durations of 28 and 56 days. Overall, this research is intended to provide a better understanding of lime-soil interactions based on curing temperature.

1.2 Literature Review

Lime stabilization is a practice that has been used for ages. It was first utilized when the Romans built the Appian Way over 2,000 years ago (Kezdi 1979). It has since been used throughout the United States for transportation purposes of improving soil subgrade beneath roadways. Lime stabilization has proven to be a simple, cost effective technique for subgrade strengthening. The ideal candidate for lime stabilization is a fine-grained, high plasticity clayey soil. Because of the mineralogy of clay, it reacts with the addition of lime, which results in both short-term reactions through soil modification and long-term reactions through soil stabilization. The process of chemical reactions that take place with the addition of lime to clayey soils alters many physical characteristics of the clay particles.

1.2.1 Lime Modification

Lime modification is the more immediate effect of the addition of lime to clayey soils. This initial addition of lime results in cation exchange and flocculation. Because all forms of lime contain calcium, free aqueous calcium ions (Ca^{++}) are released into the soil-lime mixture. These calcium ions are then able to exchange with prevailing ions of lower electronegativity (e.g., sodium, potassium) surrounding clay particles. The affinity for cations follows the following series: $Na^{+} < K^{+} < Ca^{++} < Mg^{++}$ (Grim 1953).

The secondary reaction is the flocculation and agglomeration of clay particles with lime, which results in larger particles and some initial added strength. This occurs with the change in clay particle alignments. An unstabilized clay soil has particles that are aligned in a parallel structure with face-to-face and edge-to-edge interfaces. When the clay particles begin to the move, the alignment is disrupted, and there are more edge-to-face interfaces resulting in an agglomeration of clay particles into larger particles, as shown in Figure 1-1 (Prusinski and Bhattacharja 1999).

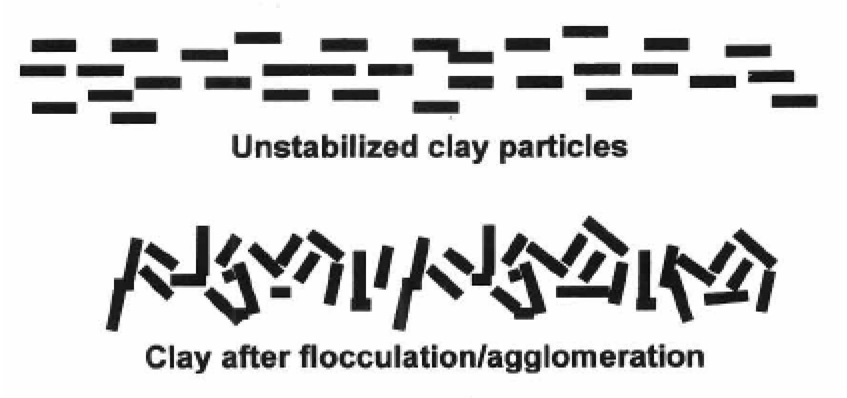

Figure 1-1: Flocculation and Agglomeration (Prusinski and Bhattacharja, 1999)

Throughout these chemical reactions, the original physical characteristics of the clay are altered. Some of these immediate effects are a decrease in plasticity index, increase in plastic limit, increase or decrease in liquid limit depending on the cation present, increase in grain-size, decreases in maximum dry density, increase in optimum moisture content, decrease in volume change, decrease in swell potential, increase in soil suction, decrease in permeability, increase in pH, and a slight increase in compressive strength (Diamond and Kinter 1965). These effects of lime addition to soil will be discussed later in more depth.

1.2.2 Lime Stabilization

Lime stabilization is a more long-term effect, which results in more significant strength gain due to pozzolanic reaction. A pozzolan can be defined as "a siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value but will, in finely divided form and in the presences of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties" (Gillot 1968).
As reactions continue to occur in a soil-lime mixture, silica and alumina found in soils reacts on the surfaces of the clay particles to produce calcium silicates and calcium aluminates. These chemical reactions that occur between clayey soils and lime are displayed below:

Initial dissociation of hydrated lime:

$$Ca[OH]_2 \rightarrow Ca^{++} + 2[OH^-]$$

Reaction products of soil-lime interaction:

$$Ca^{++} + 2[OH^-] + SiO_2 \text{ (clay silica)} \rightarrow \text{CSH (calcium silicate hydrates)}$$

$$Ca^{++} + 2[OH^-] + Al_2O_3 \text{ (clay alumina)} \rightarrow \text{CAH (calcium aluminate hydrates)}$$

To elaborate on the chemical reactions, the addition of a hydrated lime ($Ca(OH)_2$) results in a separation of the calcium ion from the hydroxide ion leaving both ions free floating in this mixture. The silica and alumina that are naturally present in clayey soils then react with the free calcium ion to form calcium silicate hyrdrates or calcium aluminate hydrates. The silica and alumina can come from potential sources, such as clay minerals, quartz, feldspars, or micas (TRB 1987). These clay silicates and clay aluminates bond or gel to the clay particles together to further strengthen the soil. This reaction is dependent on time in the fact that the longer a specimen is allowed to cure, the more the clay reacts with the lime, and the higher the strength will be. This reaction may take place for weeks to months depending on the soil composition. Thompson claims that some field data show that this strength increase continues to occur for up to 10 years or even more (Thompson, 1975).

1.2.3 Lime Type

In the process of lime stabilization, there are several different types of stabilizing agents that can be utilized. The stabilizing agent of lime can also be added at various dosages to achieve either lime modification or lime stabilization. In general, there are primarily three different types of lime that can be added to soil to achieve stabilization. The two most commonly used types of lime are quicklime and hydrated lime. Quicklime (CaO) is produced by burning limestone ($CaCO_3$). Once the calcium carbonate in limestone is burnt, carbon dioxide is released leaving quicklime. Hydrated lime, which can also be called slaked lime, is produced by the exothermic reaction of adding water to quicklime to produce $Ca(OH)_2$. When quicklime is used for lime stabilization, this chemical reaction occurs first, because of the water available in the mixture. However, if hydrated lime is used, then this reaction has already occurred, and proceeding chemical reactions can take place. The last type of lime that can be used for lime stabilization is dolomitic lime. Dolimitic lime is a combination of calcium carbonate ($Ca(CO_3)$) and magnesium carbonate ($Mg(CO_3)$). It is not particularly reactive and the least commonly used type for lime stabilization.

However, there have been tests done to determine the effects of each specific lime type. It has been shown that more strength development occurs in soil-lime mixtures with quicklime rather than slaked or hydrated lime (Gillot 1968). Also, in a study of 31 fine-grained Illinoian soils, Thompson found that plasticity reduction occurred in all soil types with the addition of lime (Thompson 1967). Three different types of lime were mixed with each soil, and it was found that high calcium hydrated lime was more effective than monohydrated dolomitic lime.

1.2.4 Lime Content

With the addition of lime to soil, there is a minimum lime content necessary to achieve the effects of lime modification or lime stabilization. Also, there is typically a higher optimum lime content to attain maximum unconfined compressive strengths. Both of these are primarily dependent on soil composition. There have been several general recommendations and test methods proposed to reach the minimum required and optimum lime content.

There are primarily two methods for achieving minimum lime content. Numerous unconfined compression tests have been done to determine a minimum threshold of strength that must be attained under specific curing conditions. Considerable work has been conducted to differentiate reactive soils from nonreactive soils by applying curing conditions of 28 days and 73ºF. Those soil-lime mixtures with a strength greater than 50 psi were classified as reactive and candidates for lime stabilization (Thompson 1966). According to the KTC, a lime-soil mixture cured for 2 days at 120ºF must attain an unconfined compressive strength of 100 psi or at least 50 psi greater than the unconfined compressive strength of the untreated soil (KTC 1996). If the minimum strength is not met, then the lime dosage is increased in an attempt to reach this minimum threshold. North Carolina Department of Transportation uses a similar accelerated curing condition with a minimum unconfined compressive strength of 50 psi (NCDOT 2006). Khattab et al. (2007) developed an empirical equation from unconfined compressive strength and clay content to achieve the minimum lime content. This equation is:

$$L_m = [\text{Clay Content } (< 2\mu\text{m})] / 35 + 1.25$$

Most previous work determined minimum lime content by performing numerous unconfined compressive tests at varying lime contents. However, more recently pH testing has come into practice to determine minimum lime content. Similarly, Eades and Grim (1966) developed a test method where soil is mixed with varying lime dosages to reach a minimum pH of 12.4. The effects of lime on pH will be discussed in more detail in Section 1.2.7.

Throughout research of lime stabilization, effect of lime content has always been a key concern. However, due to the variability in soil composition, no universal recommendations for optimum can be made. Nonetheless there have been many general suggestions made based upon soil type. Unconfined compression tests were used to establish an optimum lime content. This was performed by mixing the same soil with several dosages of lime (ie. 1%, 3%, 5%, 7%). The soil-lime mixtures were then tested for unconfined compressive strength, and this strength was plotted as a function of lime content. The maximum unconfined compressive strength resulted in optimum lime content.

Some of the earliest research began with the investigation into different types of clay. Because there are various types of clay, research was done to determine the optimum lime content for three representative clay minerals: kaolinite, illite, and montmorillonite. The optimum lime content for kaolinite was approximately 4% to 6%, while the optimum lime content was 8% and above for various illite and montmorillonite soils (Grim 1962). This is due to the fact that kaolinite begins crystalline formation with small lime dosages as the lime attacks the kaolinite crystal edges. Illite and montmorillonite require more lime content because the interlayers between particles must first be saturated with calcium before significant strength development proceeds, and perhaps because of the higher specific surface area. Ingles later did work to establish minimum lime contents for both lime modification and lime stabilization.

It is recommended that 1 to 3% hydrated lime by weight be added to silty clays, heavy clays, and very heavy clays to achieve modification, while lime contents greater than 3% and up to 8% are proposed to accomplish stabilization needed to reach higher compressive strengths (Ingles, 1973). The unconfined compressive strength of a soil-lime mixture typically increases with lime content until about 8% followed by a decrease in strength with additional lime content (Ingles, 1973). Work done on Jordanian clays revealed that an optimum lime content of 6% resulted in maximum compressive strength (El-Rawi and Al-Samadi 1995). In a study by the Kentucky Transportation Cabinet (KTC), 766 samples were tested to find the optimum lime content. The soil-lime mixtures were cured for 2 days at 120ºF at varying lime contents with the optimum lime content being 5% and a mean unconfined compressive strength of 115 psi (KTC 1996). Overall, between 3% and 8% lime content by weight of soil was required to achieve lime stabilization and maximum compressive strength.

1.2.5 Workability and Plasticity

In some instances, the goal of lime stabilization is not to increase strength, but rather to reduce plasticity and make the on-site soil more workable. On the whole, the addition of lime to a clay soil, which typically has a high plasticity index, results in a reduction in this high plasticity. This reduction in plasticity gives the soil-lime mixture a more friable texture making the soil more amenable to movement and manipulation with field equipment. As noted, an ideal candidate for lime stabilization typically has a high plasticity. According to various studies, a clayey soil with a plasticity index of at least 10% is ideal for lime stabilization (Gillot, 1968). To distinguish a soil as a candidate for lime stabilization, the KTC uses soils with plasticity indices greater than 20% and a percentage passing the #200 sieve greater than 35% (KTC 1996). This criterion is in turn based on FHWA-1P-80-2 (FHWA 1980). North Carolina Department of Transportation practice is to consider lime or cement for soils with plasticity indexes between 10 and 20 and primarily lime for soils with plasticity indexes greater than 25 (Wainaina 2010).

Gillot (1968) investigated plasticity among different clay types, and his results found that plasticity is most affected in montmorrilonite, somewhat affected in illite, and least affected in kaolinite. To demonstrate this effect of plasticity reduction, previous work (Rogers and Glendinning 2000) was performed on a London clay comprised of primarily illite and kaolinite resulting in plasticity index of about 32% with no lime. The plasticity index peaked at 45% approximately 24 hours later at a lime content of 1%. The plasticity index was also measured at 2%, 3%, and 7%, and it gradually decreased to 20% at 7% lime content.

Table 1-1 delves into further works on plasticity reduction. Examples of plasticity reduction are in decreasing order of initial soil only plasticity with the amount of lime content treatment and corresponding soil-lime plasticity. Throughout these examples, almost all soil-lime mixtures had a plasticity reduction. Those soils with higher plasticity indices typically encountered larger plasticity reductions. Lastly, increased lime content characteristically results in increased plasticity reduction until the soil-lime mixture is considered nonplastic (NP).

Table 1-1: Plasticity Reduction from Previous Research

Soil Description	USCS	AASHTO	Initial Plasticity (soil only)	% Lime	Final Plasticity	Source
Bentonite clay (FoCa)	-----	-----	70.0%	4.0%	12.0%	Khattab, et al (2007)
Clay	-----	A-7-6(20)	43.0%	3.0%	25.0%	Chistensen (1969)
Clay	-----	A-7-6(20)	43.0%	5.0%	17.0%	Chistensen (1969)
Cisne B	-----	A-7-6(20)	39.0%	3.0%	NP	Transportation Research Board (1987)
Irbid clay	CH-MH	-----	39.0%	3.0%	10.5%	Tuncer and Basma (1991)
Irbid clay	CH-MH	-----	39.0%	6.0%	7.3%	Tuncer and Basma (1991)
Irbid clay	CH-MH	-----	39.0%	9.0%	5.4%	Tuncer and Basma

Soil Description	USCS	AASHTO	Initial Plasticity	% Lime	Final Plasticity	Source (1991)
Piasa Silt Loam	-----	A-7-6(19)	35.7%	3.0%	11.4%	Thompson (1967)
Clay	-----	A-7-6(18)	34.0%	3.0%	18.0%	Chistensen (1969)
Clay	-----	A-7-6(18)	34.0%	5.0%	10.0%	Chistensen (1969)
Cowden B	-----	A-7-6(19)	33.0%	3.0%	7.0%	Transportation Research Board (1987)
Clay	-----	A-7-6(19)	33.0%	3.0%	9.0%	Chistensen (1969)
Cowden B	-----	A-7-6(19)	33.0%	5.0%	NP	Transportation Research Board (1987)
Clay	-----	A-7-6(19)	33.0%	5.0%	6.0%	Chistensen (1969)
Clay	-----	A-7-6(19)	32.0%	3.0%	19.0%	Chistensen (1969)
Clay	-----	A-7-6(19)	32.0%	5.0%	15.0%	Chistensen (1969)
Cowden Silt Loam	-----	A-7-6(19)	31.4%	3.0%	13.7%	Thompson (1967)
Drummer B	-----	A-7-6(19)	31.0%	3.0%	10.0%	Transportation Research Board (1987)
Drummer B	-----	A-7-6(19)	31.0%	5.0%	NP	Transportation Research Board (1987)
Drummer Silty Clay Loam	-----	A-7-6(19)	30.8%	3.0%	10.4%	Thompson (1967)
Clay	-----	A-7-6(20)	30.0%	3.0%	10.0%	Chistensen (1969)
Clay	-----	A-7-6(20)	30.0%	5.0%	10.0%	Chistensen (1969)
Bryce B	-----	A-7-6(18)	29.0%	3.0%	21.0%	Transportation Research Board (1987)
Huey B	-----	A-7-6(17)	29.0%	3.0%	9.0%	Transportation Research Board (1987)
Huey B	-----	A-7-6(17)	29.0%	5.0%	NP	Transportation Research Board (1987)
Huey Silt Loam	-----	A-7-6(17)	28.9%	3.0%	8.9%	Thompson (1967)
Bryce Silty Clay	-----	A-7-6(18)	28.8%	3.0%	21.4%	Thompson (1967)
Elliot Silt Loam	-----	A-7-6(18)	28.4%	3.0%	19.1%	Thompson (1967)
Elliot B	-----	A-7-6(18)	28.0%	3.0%	19.0%	Transportation Research Board (1987)
Elliot B	-----	A-7-6(18)	28.0%	5.0%	NP	Transportation Research Board (1987)
Clay	-----	A-7-6(17)	27.0%	3.0%	7.0%	Chistensen (1969)
Clay	-----	A-7-6(17)	27.0%	5.0%	5.0%	Chistensen (1969)
Wisconsin Clay Till	-----	A-7-6(17)	26.9%	3.0%	13.7%	Thompson (1967)
Wisconsin Clay Till	-----	A-7-6(17)	26.9%	5.0%	11.0%	Thompson (1967)
Wisconsin Clay Till	-----	A-7-6(17)	26.9%	7.0%	14.8%	Thompson (1967)
Huey Silt Loam	-----	A-6(15)	26.4%	3.0%	9.2%	Thompson (1967)
Sable B	-----	A-7-6(16)	24.0%	3.0%	NP	Transportation Research Board (1987)

Soil Description	USCS	AASHTO	Initial Plasticity	% Lime	Final Plasticity	Source
Clay	-----	A-7-6(14)	22.0%	3.0%	9.0%	Chistensen (1969)
Clay	-----	A-7-6(14)	22.0%	5.0%	4.0%	Chistensen (1969)
Accretion Gley	-----	A-6(12)	21.9%	3.0%	8.2%	Thompson (1967)
Clay loam	-----	A-6(9)	20.0%	3.0%	8.0%	Chistensen (1969)
Clay loam	-----	A-6(9)	20.0%	5.0%	4.0%	Chistensen (1969)
Lateric soil	-----	-----	17.8%	2.0%	13.7%	Ola (1977)
Lateric soil	-----	-----	17.8%	4.0%	9.2%	Ola (1977)
Lateric soil	-----	-----	17.8%	6.0%	6.2%	Ola (1977)
Lateric soil	-----	-----	17.8%	8.0%	2.0%	Ola (1977)
Lateric soil	-----	-----	17.8%	10.0%	1.3%	Ola (1977)
Hosmer B	-----	A-7-6(11)	17.0%	3.0%	NP	Transportation Research Board (1987)
Clay	-----	A-6(8)	17.0%	3.0%	2.0%	Chistensen (1969)
Clay	-----	A-6(9)	17.0%	3.0%	NP	Chistensen (1969)
Clay	-----	A-6(8)	17.0%	5.0%	1.0%	Chistensen (1969)
Clay	-----	A-6(9)	17.0%	5.0%	NP	Chistensen (1969)
Clay loam	-----	A-6(6)	15.0%	3.0%	5.0%	Chistensen (1969)
Clay loam	-----	A-6(6)	15.0%	5.0%	3.0%	Chistensen (1969)
Clay	CL	A-6a	13.8%	5.0%	8.2%	Chou, et al (2004)
Clay	CL	A-6a	13.4%	5.0%	9.0%	Chou, et al (2004)
Clay	CL	A-6a	13.4%	5.0%	10.3%	Chou, et al (2004)
Illinoian Till	-----	A-6(6)	11.7%	3.0%	4.8%	Thompson (1967)
AASHO Road Test	-----	A-6(18)	11.0%	3.0%	6.0%	Transportation Research Board (1987)
Illinoian Till	-----	A-6(6)	11.0%	3.0%	5.9%	Thompson (1967)
AASHO Road Test	-----	A-6(18)	11.0%	5.0%	5.0%	Transportation Research Board (1987)
Ottowa	-----	A-6(8)	10.8%	3.0%	5.6%	Thompson (1967)
Ottowa	-----	A-6(8)	10.8%	5.0%	4.6%	Thompson (1967)
Silty clay	CL-ML	A-4a	9.0%	5.0%	9.8%	Chou, et al (2004)
Silty clay	CL-ML	A-4a	7.9%	5.0%	2.2%	Chou, et al (2004)

1.2.6 Volume and Moisture Effects

Several physical transformations take place with respect to volume and moisture. Lime not only acts to make soil less plastic and more workable. It is also used in cases where there is excess moisture in the soil making it unworkable, where lime is added to act as a dewatering agent that absorbs this excess water. This characteristic of lime is also useful for field purposes other than typical strength gain. Moreover, lime addition to soil results in swell reduction, and it affects the moisture-density relationship by resulting in a decreased maximum dry density and an increased optimum moisture content.

Some work has also been done to investigate appropriate moisture contents for mixing of soil and lime. In most cases, the soil-lime mixtures should be mixed at optimum moisture content. Mixing at moisture contents dry of optimum results in decreased strengths due to a lack of water for chemical reactions to continue with time. In a recent study by Geiman (2005), the effects of mixing soft clay-lime mixtures at moisture contents higher than the optimum moisture content were investigated. A NoVa (Northern Virginia) clay that classified as CL and A-6 according to USCS and AASHTO soil classification systems was mixed with 5% hydrated lime and cured for 28 days at 70ºF. The average maximum unconfined compressive strength decreased by approximately 26% when mixing at 1.2 times the optimum moisture content. A Staunton clay that classified as a CH and A-7-5 was mixed and cured at the same conditions resulting in a 29% decrease in average maximum unconfined compressive strength. A Lynchburg silty sand that classified as a SM and A-2-4 was mixed and cured at the same conditions resulting in a 54% decrease in average maximum unconfined compressive strength (Geiman 2005).

1.2.7 Physicochemical Characteristics (pH and Conductivity)

Much of the earlier lime stabilization work involved numerous unconfined compression tests to establish a sufficient lime content to achieve lime stabilization. Eades and Grim (1966) were looking for a quick method to determine this minimum lime requirement, and they became the pioneers of testing for pH in soil-lime mixtures. They developed a test method by which the pH of the soil-lime mixture after one hour should be 12.4 to achieve minimum lime requirement. This value was based on hundreds of tests on many different soil types where both compressive strength tests and pH tests were performed. The soils were cured at a constant temperature at varying curing durations. It was found that the longer curing durations resulted in higher compressive strengths and lower pH readings (Eades and Grim 1966).

Curing duration and curing temperature are two variables that have been investigated since the inception of pH testing of soil-lime mixtures. The work of Sabry and Parcher (1979) investigated the effect of curing duration on pH. The soil-lime mixtures were compacted and cured at room temperature for varying curing durations. The pH results according to curing duration can be seen in Table 1-2.

Table 1-2: pH Reduction Due to Curing Duration (Sabry and Parcher 1979)

Soil Type	% Lime	pH after Curing Duration				
		0 days	2 days	10 days	30 days	90 days
Camargo clay	8%	12.37	12.37	11.49	11.40	11.11
Burleson soil	6%	12.43	12.63	12.12	11.79	11.36
Union City clay	5%	12.50	12.84	12.72	12.59	11.69
Summit soil	7%	12.44	12.63	12.61	12.09	11.64

It can be seen that there is an initial rise in pH during the first two days of curing. However, the pH tends to decrease over time, which indicates a decrease in the amount of free lime available for reaction (Sabry and Parcher 1979). Others claim that the increased curing duration leads to more time for pozzolanic reaction to occur, which results in a pH reduction (TRB 1987). In later work by Rogers and Glendinning (2000), and others, the English China Clay and Wyoming Bentonite were tested in pH and conductivity at varying durations. Standard samples were prepared by compacting soil-lime (quicklime) mixtures in plastic molds and curing them at approximately 50ºF. The samples were batch tested at 7, 175, and 301 days for both pH and conductivity. There is a slight reduction in pH over time as hydroxide ions continue to react in crystallization processes (Boardman et al. 2001). There is a more significant reduction in conductivity, which is due to a reduction in the calcium ion concentration over time.

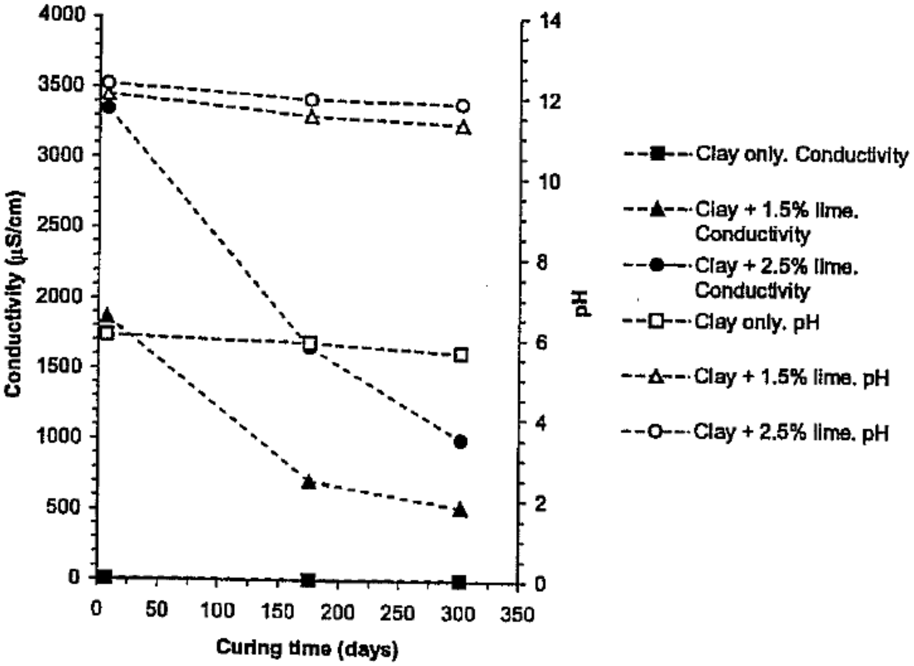

Figure 1-2: pH and Conductivity with Curing Duration (Boardman et al., 2001)

Rogers also performed similar work to evaluate the effects of temperature on pH values of soil-lime mixtures. It was found that the lime requirement for stabilization for an English China Clay consisting primarily of kaolinite and a Wyoming Bentonite increased with increasing temperatures. For example, at a temperature of approximately 35ºF the lime requirement for stabilization was 1% lime, while this lime requirement increased to 6% at a temperature of approximately 73ºF (Rogers and Glendinning 2000).

1.2.8 Compressive Strength

Unconfined compression tests have long been the chief test performed on soil-lime mixtures. From the reactions that occur in a soil-lime mixture, there is an initial strength gain, which is followed by the more obvious long-term pozzolanic strength gain. The amount of strength gain that is encountered under specific curing conditions dictates that soil's suitability as a candidate for lime stabilization. There are various factors that significantly impact the amount of strength gain that is experienced in a soil-lime mixture including curing temperature, curing duration, moisture content, and lime content. Unconfined compressive strength values can be seen at the end of this section with corresponding soil type, lime content, curing duration, and curing temperature in Table 1-5. The unconfined compressive strength values are arranged in increasing order up to a value of 100 psi.

1.2.8.1 Curing Temperature Influence

Previous research and testing has indicated that increasing curing temperature of a soil-lime mixture results in increased unconfined compressive strength. Various works have been done investigating a correlation between accelerated curing conditions and field curing conditions. Other research has been done exploring the effects of freeze-thaw cycles on unconfined compressive strength. Some of the other work studies the effects of curing at different (but constant) curing temperatures.

In an attempt to relate accelerated curing conditions to field curing conditions, a study was performed in conjunction with CalTrans testing 12 different soil types throughout California for unconfined compressive strength. The accelerated curing conditions consist of 7 days of curing at 110ºF, while the other soil-lime mixtures were cured for 28, 180, and 360 days at 72ºF. From this testing it was concluded that soil-lime mixture cured under accelerated conditions is a reasonable comparison to two to three months of field curing at 72ºF (Alexander 1978).
As noted, several state Departments of Transportation require a minimum unconfined compressive strength under this accelerated curing condition. However, by curing soil-lime mixtures at excessively high temperatures, it has been established that these temperatures are not indicative of field curing conditions, and they may result in overestimated compressive strengths for lime stabilized soils in the field. Some have recommended using a maximum temperature of 105ºF (TRB 1987, Little 1999).

Work was done by George et al. (1992) to explore temperature influence on soil-lime mixtures. Unconfined compression tests were performed on a clayey silt and sandy silty clay at 7 days and 21 days and higher temperatures of 20 ºC, 35 ºC, and 50ºC. At 7 days of curing the clayey silt at optimum lime content, the 50ºC samples had about 6 and 12 times the strength of the 35ºC and 20ºC samples. At 21 days of curing the clayey silt and optimum lime content, the 50ºC samples had about 3 and 7 times the strength of the 35ºC and 20ºC samples. The sandy silty clay had significantly lower strengths. At 7 days of curing the sandy silty clay and optimum lime content, the 50ºC samples had about 3 and 4 times the strength of the 35ºC and 20ºC samples. At 21 days of curing the sandy silty clay and optimum lime content, the 50ºC samples had about 4 and 6 times the strength of the 35ºC and 20ºC samples. Although this work investigates the upper end of the temperature scale, it further affirms that unconfined compressive strength increases with increasing curing temperature (George et al. 1992).

While much work has been done at the upper end of the temperature spectrum, a significant amount of research has been done at the lower end of this spectrum through freeze-thaw testing. Freeze-thaw experienced by lime-soil mixtures is also a concern for lime stabilization. In many accounts, the freezing stage results in an overall strength reduction leading to failures.
Dempsey and Thompson (1963) investigated the effects of exposing soil-lime mixtures to control temperatures after freeze-thaw cycles. With an initial two day cure at 120ºF followed by a 24-

hour soaking period, the soil-lime mixtures were exposed to 9 freeze-thaw cycles, which consisted of 16 hours of curing at 22ºF followed by 8 hours at room temperature. Following these freeze-thaw cycles, the samples were cured for another two days at 120ºF, and then exposed to another nine freeze-thaw cycles. Samples were tested after each curing condition. The results can be seen in the Table 1-3. From these strengths under various curing conditions, it can be seen that the first 9 freeze-thaw cycles reduced the strength. However, the additional two-day cure at 120ºF resulted in strengths approximately 1.5 times the strength after the initial two-day cure. The second set of freeze-thaw cycles also had less impact on the strength.

Table 1-3: Effect of Freeze-Thaw Cycles (Thompson and Dempsey, 1963)

Lime-Soil Mixture	Curing Conditions	UCS (psi)
Illinoian till, 3% lime	2 days at 120F + 24 hour soak	341
	+ 9 F-T cycles	280
	+ 2 days at 120F	481
	+ 9 F-T cycles	466
Bryce B, 5% lime	2 days at 120F + 24 hour soak	178
	+ 9 F-T cycles	137
	+ 2 days at 120F	298
	+ 9 F-T cycles	196

Dempsey and Thompson (1968) performed freeze-thaw testing on four different Illinois soil types to observe its effects. One freeze-thaw cycle consisted of a short time period of freezing temperatures around 22ºF to a longer time period of thawing temperatures around 77ºF. Samples experienced 0, 3, 6, 9, or 12 freeze-thaw cycles after a 2-day and 4-day initial quick cure at 120ºF. From the results of the unconfined compression tests, linear regression graphs were developed and it was found that the 2-day initial cure decreased at a rate of 9.4 psi/cycle while the 4-day initial cure decreased at a rate of 18.5 psi/cycle revealing that a higher initial strength resulted in greater degradation and strength loss due to freeze-thaw cycles. O'Flaherty and Andrews (1968) investigated the influence of freeze-thaw effects on various clay types, and their work revealed that lime-soil mixtures that consist of clayey soils that are predominantly kaolinite are less resistant to frost action than lime treated clayey soils that are predominantly montmorillonite (O'Flaherty and Andrews, 1968).

Dempsey and Thompson (1972) performed additional freeze-thaw testing with a more field related thaw condition to observe its effects. One freeze-thaw cycle consisted of a short time period of freezing temperatures ranging from 18 to 25ºF to a longer time period of thawing temperatures ranging from 35 to 45ºF. This freeze-thaw cycle was different from previous research because the thawing period was typically a quick two-day cure. However, the lower temperatures are more indicative of the type of thaw cycle that would be experienced in the field. Samples experienced 0, 3, or 6 freeze-thaw cycles. From the results of the unconfined compression tests, those samples experiencing no freeze-thaw cycles typically had strengths about 4 times the strength of those samples experiencing 6 freeze-thaw cycles (Dempsey and Thompson 1972).

Although most of the previous work done indicates that an increased number of freeze-thaw cycles results in decreased compressive strength, work done by Esmer and others contradicts this observation. Unconfined compressive tests were performed on three different Virginia clay soils after being exposed to different number of freeze-thaw cycles throughout curing. It was revealed that two of the three soils exhibited higher compressive strengths after five freeze-thaw cycles than samples tested after one and three freeze-thaw cycles. It was proposed that this was a result of excess lime that was not used in initial reactions making lime-soil mixtures self-healing (Esmer et al. 1969). This may also have been a result of the curing duration, where a sample tested after five freeze-thaw cycles may have had significantly more time to cure than those samples tested after one and three freeze-thaw cycles.

Much work has been done relating to temperature. However, this work has been done at higher temperatures to accelerate the curing process or to investigate the effects of freeze-thaw cycles on soil-lime mixtures. Not much work has been done to investigate the influence of curing temperatures above freezing and below typical room temperature (70°F). The temperature at which lime stabilization ceases to take place is an issue that has always been debated. According to Thompson, temperatures below 55ºF to 60ºF tend to inhibit soil-lime reactions, while temperatures greater than this expedite reactions (Thompson 1975).

According to NCDOT Standards, Section 501-3, lime stabilization construction cannot take place when the air temperature is below 45ºF or in the period from November 1 to March 15 (NCDOT 2006). Some of the only research done in this area was performed in a project sponsored by the KTC core samples of lime-soil mixtures were collected in four different counties during October construction of lime stabilization. Because October is the cutoff date for lime stabilization construction, the effects of colder temperatures could be observed. Although the samples were cured at varying durations, temperatures were monitored, and it was known whether the lime-soil mixture cured above or below 50ºF. The mean unconfined compressive strength for each curing condition was compared. Those strengths for samples cured below 50ºF were then compared to the statewide mean unconfined compressive strength for a specific curing condition. The results of these tests can be seen in Table 1-4 below.

Table 1-4: Effects of Temperatures Below 50ºF (KTC 1996)

	Curing Below 50ºF		Curing Above 50ºF			Curing Above 50ºF (Statewide Mean Value)		
County	Q_u (psi)	Age (days)	Q_u (psi)	Age (days)	Strength Difference (Loss)	Q_u (psi)	Age (days)	Strength Difference (Loss)
Logan	72	14	153	9	53%	116	12	38%
Mercer	62	9	113	10	45%	116	12	47%
Warren	90	25	131	16	31%	116	12	22%
Caldwell	96	12	N/A	N/A	N/A	116	12	17%

It can be seen that the average unconfined compressive strength for all cores cured below 50ºF had lower than the minimum of 100 psi required by the Kentucky Transportation Cabinet. There was a significant decrease in unconfined compressive strength from those samples cured above

50ºF to those below 50ºF. This is even the case in Mercer county, where the samples cured above 50ºF had longer curing durations than those samples cured below 50ºF.
Bell (1996) investigated the effects of temperature as well in his 1996 study. Lime-soil mixtures were cured for 7 days at temperatures from approximately 50ºF to 122ºF. The results of these unconfined compression tests can be seen in Figure 1-3.

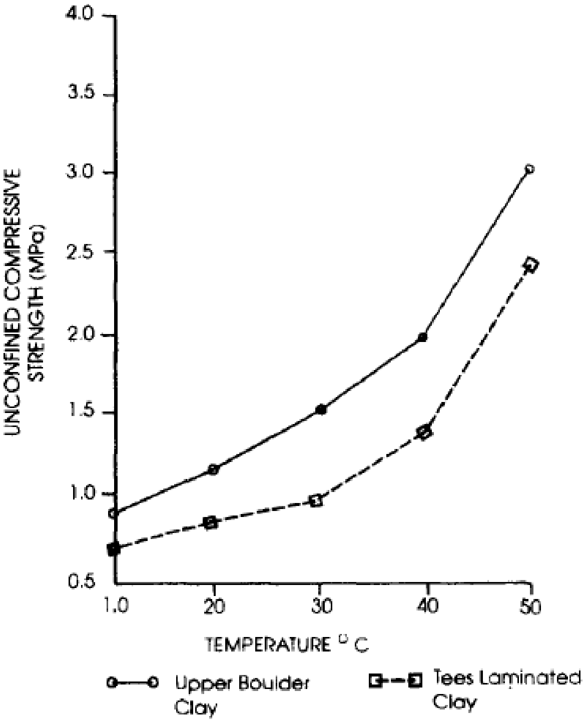

Figure 1-3: Effect of Temperature on Unconfined Compressive Strength (Bell, 1996)

It can be seen that unconfined compressive strength increases with increasing temperature with a significant strength gain after 30ºC or 86ºF.

1.2.8.2 Curing Duration Influence

Similar to curing temperature, increased curing duration also results in increase unconfined compressive strength. This is a fact that has been well-documented through multitudes of tests. Unconfined compressive strength values can be seen at the end of this section with soil type, lime content, curing duration, and curing temperature in Table 1-5.
Other significant research that has been done in terms of curing duration is establishing when the initial onset of pozzolanic reaction, which leads to the more significant strength gains in lime-soil mixtures. Studies on red tropical soils indicated that strengths gains experienced in the first seven days of curing were a result of hydration and an increase in crystallinity instead of lime reactions (TRB 1987). This was also supported by the pH and conductivity testing, which revealed that no pozzolanic reactions occur until at least after 7 days of curing (Boardman et al. 2001).

Some of Thompson's other work included a study of 31 various Illinoian soils, samples that were cured at 73ºF for both 28 and 56 days. Overall, the lime-soil mixtures cured at 56 days had a considerably higher compressive strength than the samples cured for 28 days (Thompson 1967). Unconfined compressive strength values can be seen in Table 1-5.

Table 1-5: Unconfined Compressive Strength Values from Previous Research

Soil Description	USCS	AASHTO	Curing Duration (days)	Curing Temp. (°F)	% Lime	UC Strength (psi)	Source
Eutaw	-----	-----	7	75F	6%	19	Lockett and Moore (1981)
Elliot Silt Loam	-----	A-7-5(12)	28	73F	3%	21	Thompson (1967)
Eutaw	-----	-----	14	75F	6%	25	Lockett and Moore (1981)
Drummer Silty Clay Loam	-----	A-7-6(14)	28	73F	3%	29	Thompson (1967)
Leeper	-----	-----	7	75F	6%	30	Lockett and Moore (1981)
Lynchburg Silty Sand	SM	A-2-4	3	70F	3%	30	Geiman (2005)
Lynchburg Silty Sand	SM	A-2-4	7	70F	3%	31	Geiman (2005)
Lynchburg Silty Sand	SM	A-2-4	14	70F	3%	32	Geiman (2005)
Drummer Silty Clay Loam	-----	A-7-6(14)	28	73F	7%	32	Thompson (1967)
Tama Silt Loam	-----	A-7-5(20)	28	73F	3%	32	Thompson (1967)
China clay	-----	-----	7	70F	3%	33	Rogers and Lee (1994)
Elliot Silt Loam	-----	A-7-5(12)	28	73F	7%	33	Thompson (1967)
Lynchburg Silty Sand	SM	A-2-4	28	70F	3%	34	Geiman (2005)
Roger Mills gray clay*	-----	-----	7	80F	6%	35	Drake et al (1972)
Eutaw	-----	-----	28	75F	6%	35	Lockett and Moore (1981)
Hosmer Silt Loam	-----	A-4(8)	28	73F	3%	35	Thompson (1967)
Piasa Silt Loam	-----	A-6(10)	28	73F	7%	35	Thompson (1967)
Fayette Silt Loam	-----	A-7-5(9)	28	73F	3%	37	Thompson (1967)
Roger Mills gray clay*	-----	-----	14	80F	6%	38	Drake et al (1972)
Kipling	-----	-----	7	75F	6%	38	Lockett and Moore

Soil Description	**USCS**	**AASHTO**	**Curing Duration**	**Curing Temp.**	**% Lime**	**UC Strength**	**Source**
							(1981)
Elliot Silt Loam	-----	A-7-5(12)	28	73F	5%	38	Thompson (1967)
Vaiden	-----	-----	14	75F	6%	39	Lockett and Moore (1981)
Miami Silt Loam	-----	A-4(8)	28	73F	3%	39	Thompson (1967)
Roger Mills gray clay*	-----	-----	21	80F	6%	40	Drake et al (1972)
Tama Silt Loam	-----	A-7-5(20)	28	73F	7%	40	Thompson (1967)
Roger Mills gray clay*	-----	-----	28	80F	6%	41	Drake, et al (1972)
Leeper	-----	-----	14	75F	6%	41	Lockett and Moore (1981)
Sumter	-----	-----	7	75F	6%	41	Lockett and Moore (1981)
Vaiden	-----	-----	7	75F	6%	41	Lockett and Moore (1981)
Hosmer Silt Loam	-----	A-4(8)	28	73F	7%	41	Thompson (1967)
Tama Silt Loam	-----	A-7-5(20)	28	73F	5%	41	Thompson (1967)
Cowden Silt Loam	-----	A-4(8)	28	73F	3%	42	Thompson (1967)
Bryce Silty Clay	-----	A-7-5(17)	28	73F	3%	43	Thompson (1967)
Clayey silt	-----	-----	7	68F	6%	44	George et al (1992)
Boswell	-----	-----	7	75F	6%	44	Lockett and Moore (1981)
Roger Mills gray clay*	-----	-----	7	80F	11%	45	Drake, et al (1972)
Cowden Silt Loam	-----	A-4(8)	28	73F	7%	45	Thompson (1967)
Hosmer Silt Loam	-----	A-4(8)	28	73F	5%	45	Thompson (1967)
Miami Silt Loam	-----	A-4(8)	28	73F	7%	45	Thompson (1967)
Permian red clay*	-----	-----	7	80F	8%	46	Drake et al (1972)
Demopolis	-----	-----	14	75F	6%	46	Lockett and Moore (1981)
Sumter	-----	-----	14	75F	6%	46	Lockett and Moore (1981)
Susquehanna	-----	-----	14	75F	6%	46	Lockett and Moore (1981)
Fayette Silt Loam	-----	A-7-5(9)	28	73F	5%	46	Thompson (1967)
Kipling	-----	-----	14	75F	6%	47	Lockett and Moore (1981)
NoVa Clay	CL	A-6	3	70F	3%	47	Geiman (2005)

Soil Description	**USCS**	**AASHTO**	**Curing Duration**	**Curing Temp.**	**% Lime**	**UC Strength**	**Source**
Cowden Silt Loam	-----	A-4(8)	28	73F	5%	47	Thompson (1967)
Permian red clay*	-----	-----	7	80F	4%	48	Drake et al (1972)
NoVa Clay	CL	A-6	7	70F	3%	48	Geiman (2005)
Roger Mills gray clay*	-----	-----	14	80F	11%	49	Drake et al (1972)
Lynchburg Silty Sand	SM	A-2-4	3	70F	5%	49	Geiman (2005)
Drummer Silty Clay Loam	-----	A-7-6(14)	28	73F	5%	49	Thompson (1967)
Fayette Silt Loam	-----	A-7-5(9)	28	73F	7%	49	Thompson (1967)
Roger Mills gray clay*	-----	-----	21	80F	11%	50	Drake et al (1972)
NoVa Clay	CL	A-6	14	70F	3%	50	Geiman (2005)
Lynchburg Silty Sand	SM	A-2-4	7	70F	5%	50	Geiman (2005)
Leached Peorian Loess	-----	A-4(8)	28	73F	3%	50	Thompson (1967)
NoVa Clay	CL	A-6	3	70F	5%	51	Geiman (2005)
Lynchburg Silty Sand	SM	A-2-4	14	70F	5%	51	Geiman (2005)
Sandy Lean Clay	CL	A-6	28	~70F	3%	52	Geiman (2005)
Roger Mills gray clay*	-----	-----	28	80F	11%	52	Drake et al (1972)
NoVa Clay	CL	A-6	28	70F	3%	52	Geiman (2005)
Lynchburg Silty Sand	SM	A-2-4	28	70F	5%	52	Geiman (2005)
Bryce Silty Clay	-----	A-7-5(17)	28	73F	7%	53	Thompson (1967)
Miami Silt Loam	-----	A-4(8)	28	73F	5%	53	Thompson (1967)
Leached Peorian Loess	-----	A-4(8)	28	73F	7%	53	Thompson (1967)
Piasa Silt Loam	-----	A-6(10)	28	73F	3%	53	Thompson (1967)
Irbid clay	CH-MH	-----	4	72F	3%	54	Tuncer and Basma (1991)
Na'ur	CH	A-7-6(29)	7	77F	4%	55	El-Rawi and Al-Samadi (1995)
Oktibbeha	-----	-----	7	75F	6%	55	Lockett and Moore (1981)
NoVa Clay	CL	A-6	7	70F	5%	55	Geiman (2005)
Leached Peorian Loess	-----	A-4(8)	56	73F	5%	55	Thompson (1967)
Permian red clay*	-----	-----	14	80F	4%	56	Drake et al (1972)
Leeper	-----	-----	28	75F	6%	56	Lockett and Moore

Soil Description	USCS	AASHTO	Curing Duration	Curing Temp.	% Lime	UC Strength	Source
							(1981)
Staunton Clay	CH	A-7-5	3	70F	3%	56	Geiman (2005)
Piasa Silt Loam	-----	A-6(10)	28	73F	5%	56	Thompson (1967)
Irbid clay	CH-MH	-----	21	72F	3%	56	Tuncer and Basma (1991)
Leached Peorian Loess	-----	A-4(8)	28	73F	5%	57	Thompson (1967)
Staunton Clay	CH	A-7-5	7	70F	3%	58	Geiman (2005)
Bryce Silty Clay	-----	A-7-5(17)	28	73F	5%	58	Thompson (1967)
Permian red clay*	-----	-----	21	80F	4%	59	Drake et al (1972)
NoVa Clay	CL	A-6	14	70F	5%	59	Geiman (2005)
Beaumont clay	CH	-----	7	72F	4%	60	Kennedy, et al (1987)
Irbid clay	CH-MH	-----	14	72F	3%	60	Tuncer and Basma (1991)
Permian red clay*	-----	-----	28	80F	4%	61	Drake et al (1972)
Permian red clay*	-----	-----	14	80F	8%	61	Drake et al (1972)
Staunton Clay	CH	A-7-5	14	70F	3%	61	Geiman (2005)
Sandy Lean Clay	CL	A-6	28	~70F	5%	62	Geiman (2005)
Demopolis	-----	-----	28	75F	6%	62	Lockett and Moore (1981)
Kipling	-----	-----	28	75F	6%	62	Lockett and Moore (1981)
Wilcox	-----	-----	7	75F	6%	62	Lockett and Moore (1981)
Fat Clay	CH	A-7-5	28	~70F	3%	63	Geiman (2005)
Staunton Clay	CH	A-7-5	28	70F	3%	63	Geiman (2005)
Leached Peorian Loess	-----	A-4(8)	56	73F	3%	63	Thompson (1967)
Clay loam	-----	A-6(6)	7	~70F	3%	64	Chistensen (1969)
Irbid clay	CH-MH	-----	28	72F	3%	65	Tuncer and Basma (1991)
Vaiden	-----	-----	28	75F	6%	65	Lockett and Moore (1981)
Staunton Clay	CH	A-7-5	3	70F	5%	65	Geiman (2005)
Permian red clay*	-----	-----	21	80F	8%	66	Drake et al (1972)
Boswell	-----	-----	28	75F	6%	66	Lockett and Moore (1981)
Clayey silt	-----	-----	7	68F	3%	67	George et al (1992)
Staunton Clay	CH	A-7-5	7	70F	5%	67	Geiman (2005)
Staunton Clay	CH	A-7-5	14	70F	5%	69	Geiman (2005)

Soil Description	USCS	AASHTO	Curing Duration	Curing Temp.	% Lime	UC Strength	Source
Fat Clay	CH	A-7-5	28	~70F	5%	70	Geiman (2005)
Beaumont clay	CH	-----	14	72F	4%	70	Kennedy, et al (1987)
Permian red clay*	-----	-----	28	80F	8%	70	Drake et al (1972)
Staunton Clay	CH	A-7-5	28	70F	5%	70	Geiman (2005)
Miami Silt Loam	-----	A-6(9)	28	73F	3%	70	Thompson (1967)
Miami Silt Loam	-----	A-6(9)	56	73F	7%	70	Thompson (1967)
Susquehanna	-----	-----	28	75F	6%	71	Lockett and Moore (1981)
Tama Silt Loam	-----	A-7-5(20)	28	73F	3%	71	Thompson (1967)
Fat Clay	CH	A-7-5	28	~70F	9%	75	Geiman (2005)
Beaumont clay	CH	-----	7	72F	7%	75	Kennedy, et al (1987)
Fat Clay	CH	A-7-5(10)	7	70F	4%	75	Frempong (1995)
Clay	-----	A-7-6(20)	7	~70F	3%	76	Chistensen (1969)
Clay	-----	A-7-6(18)	7	~70F	3%	78	Chistensen (1969)
Wilcox	-----	-----	14	75F	6%	78	Lockett and Moore (1981)
Clay	-----	A-6(8)	7	~70F	3%	80	Chistensen (1969)
Houston	-----	-----	7	75F	6%	80	Lockett and Moore (1981)
Miami Silt Loam	-----	A-6(9)	56	73F	5%	80	Thompson (1967)
Sumter	-----	-----	28	75F	6%	81	Lockett and Moore (1981)
Cowden Silt Loam	-----	A-7-6(19)	28	73F	3%	81	Thompson (1967)
Tama Silt Loam	-----	A-7-6(20)	56	73F	7%	82	Thompson (1967)
Clay	-----	A-7-5(13)	7	110F	3%	84	Alexander (1978)
Clay	-----	A-7-5(13)	28	72F	3%	84	Alexander (1978)
Tama Silt Loam	-----	A-7-5(20)	28	73F	7%	85	Thompson (1967)
Silt	ML	A-7-6(5)	7	70F	4%	86	Frempong (1995)
Wilcox	-----	-----	28	75F	6%	86	Lockett and Moore (1981)
Leached Peorian Loess	-----	A-4(8)	56	73F	7%	86	Thompson (1967)
Zeizia	CL	A-6(13)	7	77F	4%	87	El-Rawi and Al-Samadi (1995)
Oktibbeha	-----	-----	14	75F	6%	87	Lockett and Moore (1981)
Tama Silt Loam	-----	A-7-6(20)	56	73F	5%	87	Thompson (1967)
Clay loam	-----	A-6(9)	7	~70F	3%	89	Chistensen (1969)

Soil Description	USCS	AASHTO	Curing Duration	Curing Temp.	% Lime	UC Strength	Source
Elastic Silt	MH	A-7-6(4)	7	70F	4%	89	Frempong (1995)
Elliot Silt Loam	-----	A-7-6(18)	28	73F	3%	89	Thompson (1967)
Fayette Silt Loam	-----	A-7-5(17)	56	73F	7%	91	Thompson (1967)
Beaumont clay	CH	-----	14	72F	7%	92	Kennedy et al (1987)
Hosmer Silt Loam	-----	A-7-6(13)	28	73F	3%	92	Thompson (1967)
Fayette Silt Loam	-----	A-7-5(17)	56	73F	5%	92	Thompson (1967)
Calcareous Peorian Loess	-----	A-4(8)	28	73F	5%	94	Thompson (1967)
Hosmer Silt Loam	-----	A-7-6(11)	56	73F	3%	94	Thompson (1967)
Clay	-----	A-7-6(18)	28	~70F	5%	95	Chistensen (1969)
Hosmer Silt Loam	-----	A-7-6(11)	28	73F	7%	95	Thompson (1967)
Miami Silt Loam	-----	A-6(9)	28	73F	5%	96	Thompson (1967)
Calcareous Peorian Loess	-----	A-4(8)	28	73F	7%	96	Thompson (1967)
Clay	-----	A-7-6(18)	7	~70F	3%	97	Chistensen (1969)
Fayette Silt Loam	-----	A-7-5(17)	56	73F	3%	97	Thompson (1967)
Clay	-----	A-7-6(20)	7	~70F	3%	98	Chistensen (1969)
Calcareous Peorian Loess	-----	A-4(8)	28	73F	3%	98	Thompson (1967)
Tama Silt Loam	-----	A-7-5(20)	28	73F	5%	98	Thompson (1967)
Hosmer Silt Loam	-----	A-7-6(11)	56	73F	5%	100	Thompson (1967)

Similarly, through correspondence with NCDOT, data were gathered establishing average unconfined compressive strengths based on soil type and percent lime addition at a 2-day cure at 120ºF. These data are summarized in Table 1-6, and seems similar to those results from accelerated curing for Chatham, Cherokee, and Iredell samples.

Table 1-6: Unconfined Compressive Strength Values from NCDOT (2-day cure at 120ºF)

Soil Classification	Location	Strength	% Lime Added
A-5(7)	Wake	16.4 psi	3
A-7-5(26)	Chatham	37.1 psi	3
A-7-6(24)	Chatham	43.2 psi	3
A-6(8)	Chatham	45.1 psi	3
A-7-5(24)	Chatham	46.0 psi	3
A-6(4)	Cabarrus	47.6 psi	3

Soil Classification	Location	Strength	% Lime Added
A-7-6(10)	Mecklenburg	48.6 psi	3
A-7-6(1)	Stanly	50.5 psi	3
A-7-5(17)	Mecklenburg	55.2 psi	3
A-2-6(0)	Wilson	56.5 psi	3
A-6(14)	Lee	56.5 psi	3
A-7-6(16)	Mecklenburg	57.9 psi	3
A-6(12)	Stanly	58.3 psi	3
A-2-6(0)	Wilson	58.5 psi	3
A-5(7)	Lincoln	58.8 psi	3
A-7-6(20)	Lee	58.8 psi	3
A-6(5)	Mecklenburg	59.0 psi	4
A-6(2)	Mecklenburg	64.2 psi	4
A-5(3)	Wake	64.8 psi	3
A-2-6(1)	Wilson	65.9 psi	3
A-2-6(0)	Wilson	67.7 psi	3
A-6(4)	Stanly	68.2 psi	3
A-6(9)	Stanly	69.2 psi	3
A-2-6(1)	Wilson	69.9 psi	3
A-2-6(1)	Wilson	70.9 psi	3
A-5(1)	Wake	74.0 psi	3
A-6(9)	Wake	78.9 psi	4
A-7-5(12)	Lincoln	82.3 psi	4
A-6(10)	Stanly	83.2 psi	3
A-6(7)	Wake	83.3 psi	4
A-6(3)	Wake	87.8 psi	3
A-5(4)	Lincoln	89.9 psi	3
A-7-6(4)	Wake	90.6 psi	3
A-7-5(17)	Lincoln	94.6 psi	4
A-6(5)	Wake	95.9 psi	3
A-7-6(8)	Richmond	96.0 psi	3
A-7-6(14)	Lincoln	97.5 psi	4
A-5(1)	Lincoln	97.6 psi	3
A-7-6(9)	Rowan	97.6 psi	3
A-6(4)	Richmond	98.4 psi	3
A-6(5)	Wake	98.6 psi	4
A-7-6(17)	Wake	101.3 psi	3
A-6(1)	Richmond	101.6 psi	3
A-7-5(14)	Gaston	102.5 psi	4
A-7-5(14)	Lincoln	104.7 psi	4

Soil Classification	Location	Strength	% Lime Added
A-7-6(2)	Forsyth	106.9 psi	3
A-7-5(17)	Mecklenburg	107.9 psi	3
A-7-6(13)	Rowan	108.8 psi	3
A-5(5)	Guilford	109.3 psi	3
A-7-6(9)	Richmond	110.1 psi	3
A-7-5(32)	Rowan	111.3 psi	3
A-7-6(5)	Wake	111.8 psi	3
A-7-6(11)	Chatham	114.6 psi	3
A-6(2)	Wayne	134.6 psi	3
A-7-6(10)	Rowan	138.2 psi	3
A-2-6(1)	Wayne	142.8 psi	3
A-7-6(13)	Wake	143.2 psi	4
A-6(4)	Chatham	143.7 psi	3
A-7-6(10)	Wake	147.3 psi	4
A-6(2)	Wayne	154.7 psi	3
A-6(6)	Chatham	162.2 psi	3

1.2.9 Construction and Cost

On the whole, lime stabilization has often been a more cost effective option than using a similar alternative, such as a stone base. However, strength is always the determining factor because there is so much variability in strength due to soil type, so the ideal soil type must be used. Some of the average cost values of the years are summarized below. However, these costs are also variable depending upon location. According to 1995 average bid prices in the state of Kentucky, the cost of construction of lime stabilization was $0.36 per square yard per inch of depth (KTC 1996). This unit cost was upheld by the average values from 2000 from the KTC which yielded a cost of implementation of $0.3525 per square yard per inch of depth. It was also suggested that soil stabilization was ultimately more cost effective in various studies throughout Kentucky (Hopkins et al. 2002). Some more recent work found that the cost of hydrated lime itself is approximately $150 per ton while the cost to implement lime subgrade stabilization is $2.00 per square yard at a 6-inch depth ($0.33 per square yard per inch of depth) and $3.50 per square yard at a 12-inch depth ($0.29 per square yard per inch of depth) (Geiman 2005). Based on 2005 average bid prices for the state of North Carolina, some of the more local cost estimates for subgrade stabilization were $14.69 per cubic yard for lime stabilization, $18.01 per cubic yard for cement stabilization, and $42.19 per cubic yard for undercut and replacement with select fill (NCDOT 2006). This cost comes out to be approximately $0.41 per square yard per inch of depth, which is somewhat higher than previous estimates, but still more cost effective than both cement stabilization and undercut and fill replacement.

1.3 Materials and Methods

The procedures followed for this research were developed in consultation with NCDOT (see Appendix 1-1) as well as recommendations from AASHTO, ASTM and the open literature.

1.3.1 Materials

The only materials necessary to begin testing, other than the laboratory equipment, were the soil samples as well as the lime being used for testing.

1.3.1.1 Soil

To begin the process of research, soil first had to be obtained. Because this is a statewide project, soil was obtained from three different regions of North Carolina. North Carolina is typically broken up into the mountain, piedmont, and coastal regions. However, the coastal region was not considered for this project because temperature rarely drops as low as it does in the mountain and piedmont regions and the soils are typically less amenable to lime stabilization. Also, most of the coastal soil is coarse-grained soils, which are not typical candidates for lime stabilization. The piedmont region was broken up into the Eastern and Western Piedmont regions. Soil was obtained from Cherokee, Iredell, and Chatham counties. The locations of these three soils can be seen in Figure 1-4, while more details are provided in Table 1-7. Eight 55-gallon drums were obtained from each location. Each drum was labeled, securely tightened and closed until needed. The collection of drums is shown in Figure 1-5.

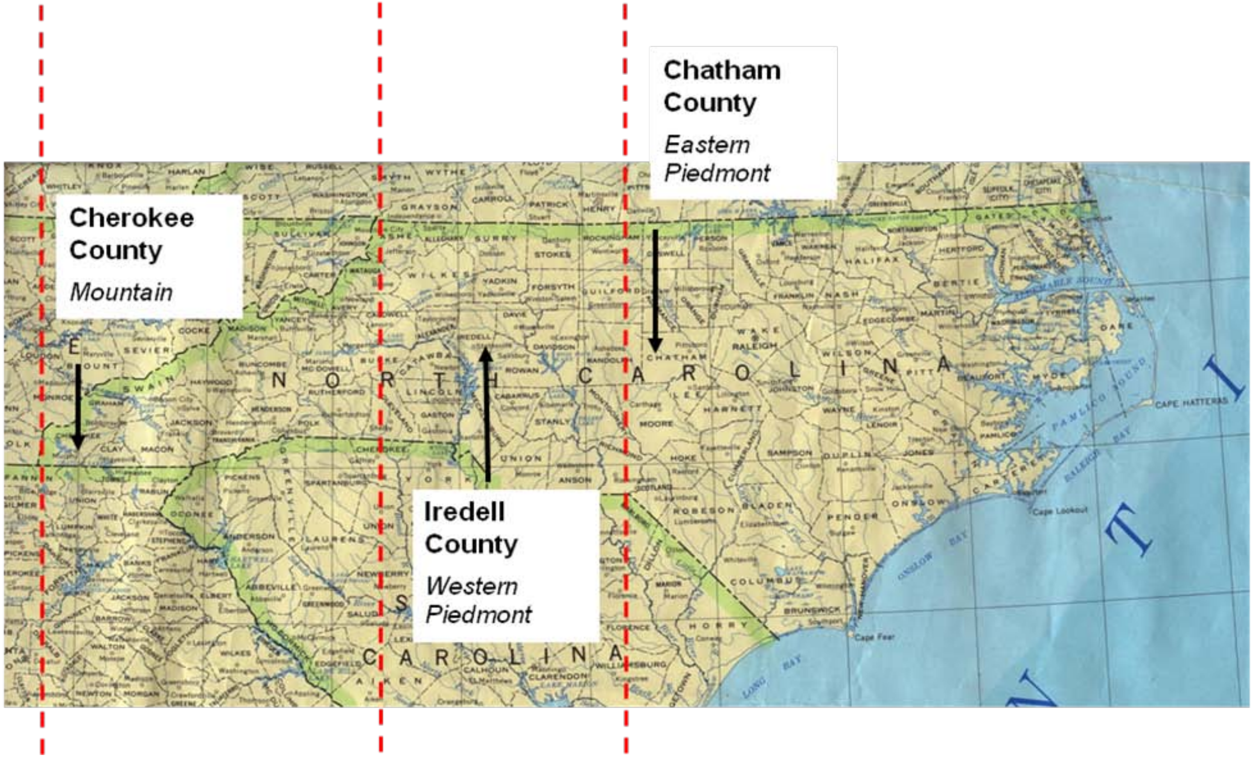

Figure 1-4: Soil Locations

Figure 1-5: Soil Stockpile in 55 gallon drums

Table 1-7. Soil collected for lime stabilization research

No.	Dates Collected	Region	County	Location	AASHTO Soil Classification	Sampling
1	7/14/06	Western Piedmont	Iredell	U.S. 70, near Statesville	A-7-5	
2	10/4/06	Eastern Piedmont	Chatham	U.S. 421, near Siler City	A-7-6	
3	12/14/06	Mountain	Cherokee	U.S. 64, near Murphy	A-4	

1.3.1.2 Lime

Magnolia brand type N chemical hydrated lime, or calcium hydroxide, was obtained from Southern Lime in Alabama (through retail stores in Raleigh, NC) and used as the primary stabilizer. To preserve the lime from carbonation, each bag of lime was double bagged with black plastic trash bags that were securely tied.

1.3.2 Methods

Basic classification tests were performed by both UNC Charlotte and NCDOT Materials and Test Unit to characterize the collected soils. Physicochemical tests were performed to resolve details of the reactivity of lime with the various soil types.

1.3.2.1 Grain-Size Analysis

To begin the investigation into the three different soil samples chosen for lime stabilization, some preliminary tests were run for soil classification purposes. Soil is classified by either the American Association of State Highway and Transportation Officials (AASHTO) soil classification system or the Unified Soil Classifcation System (USCS). Both of these soil classification systems are classified according to the results of sieve analysis testing according to ASTM D422-63 as well as Atterberg limit testing according to ASTM D 4318. The sieve analysis test was first run to differentiate between the granular and silt-clay materials. The representative soil sample was passed through a series of sieves with progressively smaller opening until a minimum of the No. 200 (0.075 mm) sieve. The percent soil passing and percent soil retained on each sieve was measured, and the percent passing was graphed against the grain size diameter. According to the AASHTO classification, a soil with less than 35% passing the No. 200 sieve would be classified as a granular material while a soil with more than 35% passing the No. 200 sieve would be classified as a silt-clay material. According to USCS soil classification, those soils with less than 50% passing the No. 200 sieve would be classified as a coarse-grained material while a soil with more than 50% passing the No. 200 sieve would be classified as a fine-grained soil.

1.3.2.2 Atterberg Limits

To further distinguish between soil types, the Atterberg limit tests are performed. The liquid limit and plasticity index are the two determining factors. The liquid limit and plastic limit tests were performed in accordance with ASTM D 4318. Both tests were performed by passing a representative soil sample through the 425 μm (No. 40) sieve. The liquid limit test was performed by using a bowl-shaped device. The soil was mixed with water to achieve an almost liquid state. This soil is then placed in the bowl and divided with a grooving tool. The bowl is lifted and dropped until there is a ½” closure between the two sides. This is performed several times, and the moisture content at which the number of drops to cause this ½” closure is exactly 25 is the corresponding liquid limit. The plastic limit is performed by wetting the sieved soil sample and rolling it into ⅛” threads of soil until they begin to crack. A representative sample of these rolled threads is collected, and the moisture content is the corresponding plastic limit. From these two limit states of the soil, the plasticity index can be determined by subtracting the plastic limit from the liquid limit. The liquid limit and plasticity index were then used to classify each soil type.

1.3.2.3 pH Testing

The Eades and Grim test was conducted according to ASTM D 6276 to determine minimum lime content and to evaluate its sensitivity to temperature. According to the Eades and Grim test, the minimum lime content of a soil is reached when the pH of the soil, lime, and water mixture with 25 grams of soil passed through the 425 µm (No. 40) sieve, a certain percentage of lime, and 100 grams of distilled water reaches 12.4. The lime dosages range from no lime to 2 grams of lime mixed with just distilled water. In between, the lime dosage is incrementally increased until the pH of this mixture reaches 12.4. The pH readings are taken after 60 minutes of shaking the mixtures for 30 seconds at 10 minute intervals. Between pH readings, the probe was washed with distilled water to eliminate contamination, and the probe was wiped with Kimwipes™ brand tissues. A standard pH meter and probe was used as shown in Figure 1-6.

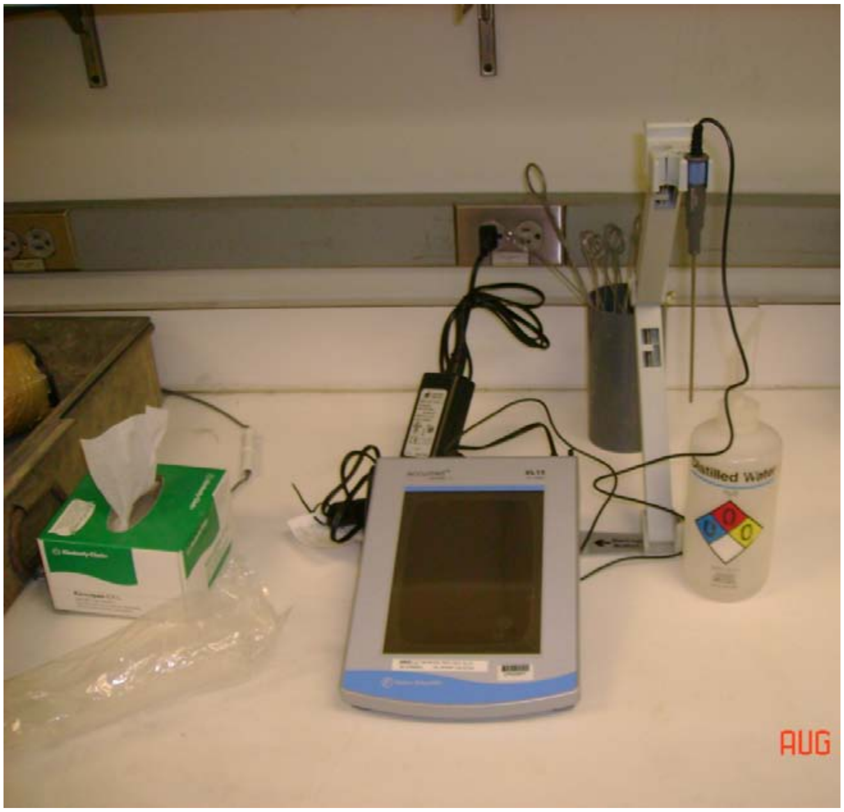

Figure 1-6: pH Meter

To evaluate the impact of curing temperature and curing duration on the pH and reactivity of the soil/lime mixture, this test was performed at 120°F, 70°F, 50°F, and 35°F for each different soil type. To cure samples at specific temperatures, a NESLAB model RTE 110 curing chamber was used, as shown in Figure 1-7. The soil/lime/water mixtures were sealed in hard plastic containers with screw caps, and allowed to sit in the water inside of the curing chamber. The water circulated through the chamber maintaining a constant designated temperature.

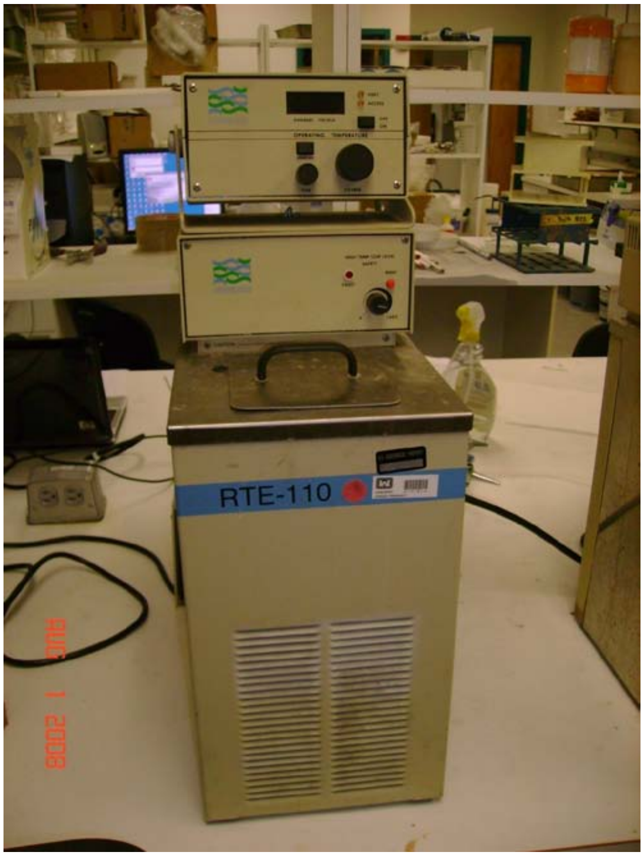

Figure 1-7: Curing Chamber for pH and Conductivity Testing

An alternative test method was also introduced for pH testing. Sample preparation was performed according to ASTM D 2166, and is described in more detail in Section 1.3.2.5. The samples were mixed at optimum lime and water content, and then compacted in three layers at standard compaction energy. The samples were cured for specific curing durations at relative 100% humidity in curing rooms at specific curing temperatures. Following a given curing duration, the sample was removed, and a 25 gram sample was removed. The 25 gram sample was then crushed and pulverized and mixed with 100 grams of distilled water. After this method of sample preparation, the samples were tested for pH using the same test method according to ASTM D 6276.

1.3.2.4 Conductivity Testing

Conductivity readings were taken following pH readings as a bulk measure of solution activity, i.e., conductivity readings reflect the summation of all dissolved ions in solution whereas pH simply measures the availability of the Hydrogen ion. Conductivity readings were taken with a Corning conductivity meter 411, as shown in Figure 1-8. Between conductivity readings, the probe was washed with distilled water to eliminate contamination, and the probe was wiped with Kimwipes™ brand tissues.

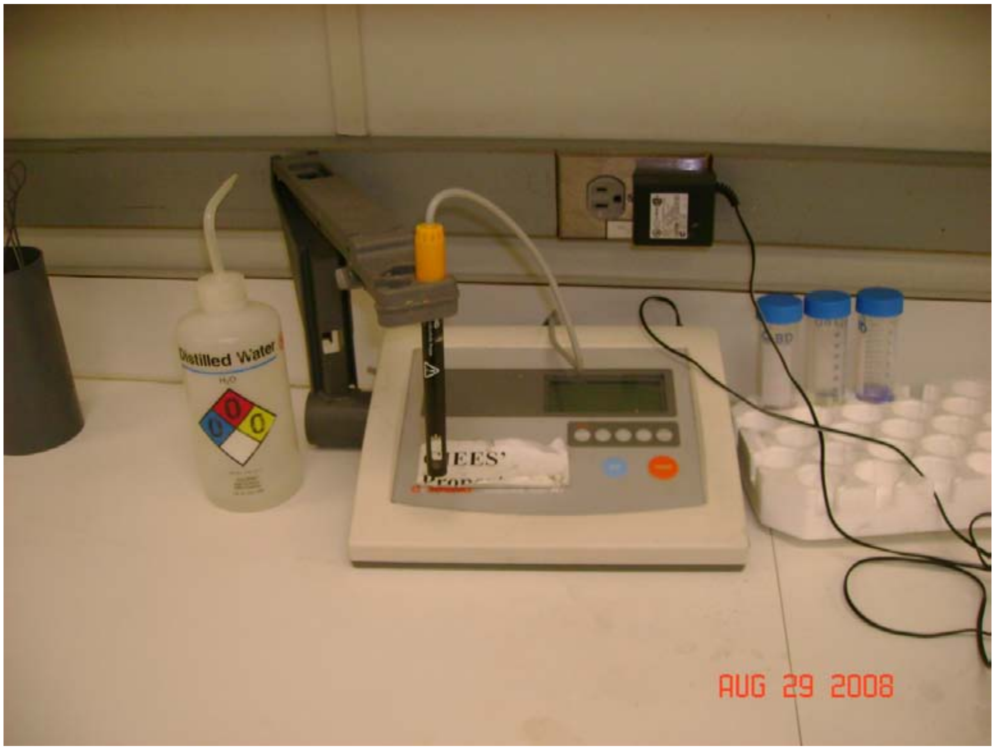

Figure 1-8: Conductivity Meter

1.3.2.5 Moisture-Density Relationships and Unconfined Compression (UC) Testing

Once the appropriate lime percentages were known (or in this case specified on the basis of typical NCDOT lime dosages, 4 - 6%, by weight), compaction testing was performed on each soil type to determine the optimum moisture content (OMC) and maximum dry unit weight for each soil/lime mixture. The compaction tests were performed according to ASTM D698. This is performed by mixing the soil and lime at various moisture contents beginning with a lower moisture content and increasing the moisture content until the dry unit weight increases, peaks, and begins to descend. However, these optimum moisture contents were increased by 1-3% to account for the hydration of the lime during the curing process.

Because the effect of temperature on unconfined compressive strength was the main concern for this project, this process for making samples differed somewhat from standard procedures. All of the materials necessary to make each sample were pre-cooled at the temperature at which curing was to occur. This was done to prevent any additional strength that may have been gained during the time-lag in temperature. This additional strength could be gained by mixing and compacting the materials at a higher temperature, which requires some amount of time until the sample has dropped to the specified curing temperature. These few hours of the sample being exposed to higher temperatures can have a considerable effect on the unconfined compressive strength, especially the samples that are cured for short-term durations. Figure 1-9 illustrates the time-lag that occurs between mixing temperature and curing temperature when all sample ingredients are not pre-cooled. It can be seen that just over 20 hours passes until the specimen is exposed to a constant curing temperature of 25°F. This amount of time at a temperature above the required curing temperature could have aided in increased strengths, especially 1 or 3-day samples. The soil, lime, and water are all separately pre-cooled in the appropriate curing chamber for approximately 24 hours. This was done by weighing out enough soil, lime, and water to prepare two samples. Each ingredient was then placed in a gallon plastic bag, and allowed to pre-cool.

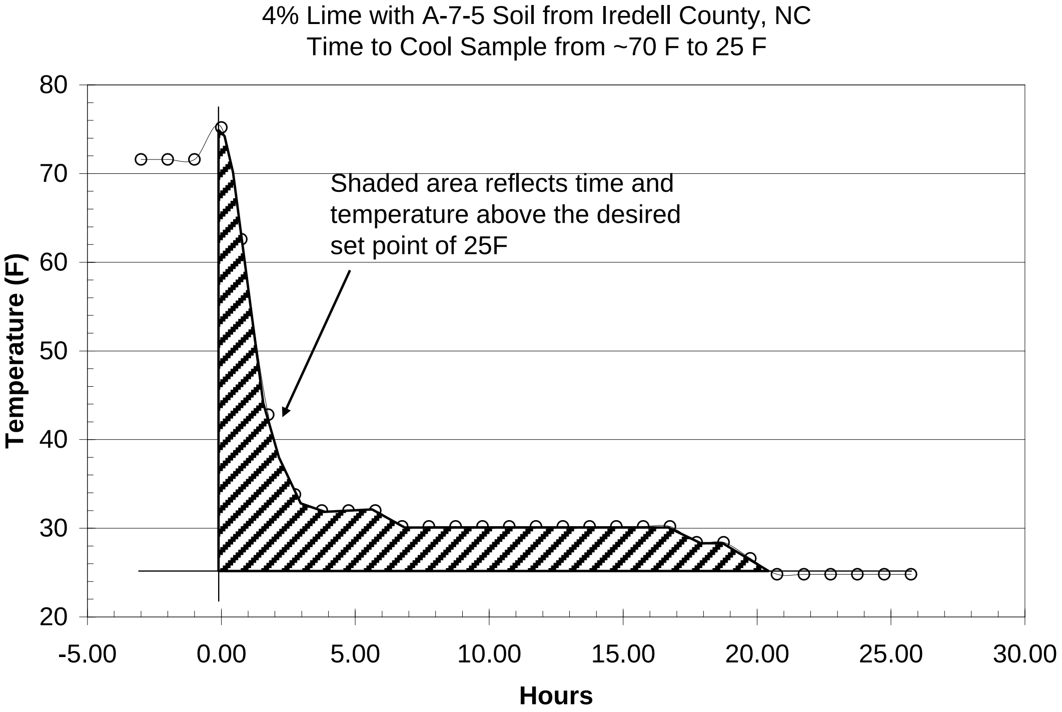

Figure 1-9: Time-Lag without Pre-cooling

All samples were prepared at ±1% of the optimum moisture content. This tended to be a somewhat difficult task because it was initially assumed that the air dried moisture content was approximately 0%. However, it is evident that as soil air dries, the humidity in the air is also being absorbed into the soil, which is called a hygroscopic moisture content. Although this moisture content may be very minimal, it is significant enough to affect the moisture content at compaction. After pre-cooling, the initial moisture content of the “dry” soil is taken, and subsequently subtracted from the optimum moisture content to determine the amount of water to add to the mixture to achieve the target moisture content.

The lime is subsequently dry mixed by hand until the lime is thoroughly mixed into the soil. Enough water is then added to fall in a ±1% range of the optimum moisture content of that particular soil-lime mixture. Water is progressively added at increments and hand mixed to ensure a consistent mix of the soil, lime, and water. The moisture content of the wet soil, lime, and water mixture is also taken before compaction to make certain that the mixture is at optimum moisture content.

Once the soil, lime, and water were well mixed and ready to be compacted, the Proctor mold was first lubricated with Molytex EP-2 Texaco brand grease to ensure that the sample could be easily extruded from the mold. According to ASTM D2166, the sample is compacted at three equal layers with 25 blows per layer at a hammer energy of 5.5 lb hammer falling 12 inches with scarification between each layer. The collar is removed so that the shavings can be taken off of the top of the mold. The moisture content was then taken from these shavings to determine moisture content at compaction. The sample was then removed from the mold by extrusion. Each sample is then placed a sealed plastic bag with a damp sponge to ensure near 100% humidity and prevent moisture loss in the sample during the curing process. This was measured with a relative humidity sensor and shown to be reliable, i.e., maintaining a relative humidity in excess of 95%. A typical bagged sample is shown in Figure 1-10.

Figure 1-10: Curing Samples in Ziploc® Bag with Sponge

The curing process has the most significant effect on the unconfined compressive strength of each sample. This can be attributed to the curing temperature as well as the curing duration to which the sample is exposed. These were the two main variables throughout the project. The samples were cured for both short-term duration, such as 1, 3, and 7 days, as well as long-term durations, such as 28 and 56 days. The constant curing temperatures used throughout testing were 70°F, 50°F, 35°F, and 25°F. Those samples cured at 25°F were thawed for approximately four hours prior to unconfined compression test to reduce any additional strength gains from a frozen specimen. This thaw period was determined from temperature monitoring with temperature probes on test samples. Samples cured at 70°F were generally placed in a 100% humidity (misting) curing room. Separate chambers were used to control the temperature at 50°F, 35°F, and 25°F. A picture of the typical chamber setup is shown in Figure 1-11.

Figure 1-11: Curing Chamber

The effects of freeze-thaw cycles were also investigated. Short-term freeze-thaw testing was initially investigated by pre-cooling and curing the soil-lime mixture at 50ºF for 24 hours, followed by curing at 25ºF for 12 hours, and additional curing at 50ºF until either 3 or 7 days of total curing duration.

Long-term freeze-thaw testing was also investigated. Soil-lime mixtures were exposed to either one or five freeze-thaw cycles. Those soil-lime mixtures exposed to one freeze-thaw cycle experienced an initial 24 hours of curing at 70ºF, followed by 12 hours of curing at 25ºF, and additional curing at 70ºF until either 28 or 56 days of total curing duration. Those soil-lime mixtures exposed to five freeze-thaw cycles experienced an initial 24 hours of curing at 70ºF, followed by 24 hours of curing at 25ºF. This cycle was repeated five times, and the soil-lime mixture was then cured at 70ºF until either 28 or 56 days of total curing duration.
In an attempt to simulate a warming period from winter to spring, samples were exposed to an initial 28 days of curing at 35ºF followed by either 28 days or 56 days of curing at 70ºF for a total of 56 and 84 days of curing.

All samples cured at 70ºF were placed in a 100% humidity room in open plastic bags. However, it was ensured that excess moisture did not condense into the bag. All samples cured at 50ºF, 35ºF, and 25ºF were placed in curing chambers in sealed plastic bags with a wet sponge to ensure 100% humidity, as noted above.

After the samples have cured for their specified duration, an unconfined compression test is performed on the sample to determine the unconfined compressive strength. An unconfined compression test simply provides the unconsolidated, undrained strength of a cohesive soil.

Although it does not directly simulate field conditions like a triaxial test by providing lateral confining pressures, the unconfined compression test is more commonly used to determine lime-stabilized strengths. It is a reasonable indicator of strength and also more comparable to previous work. The UC tests were conducted in accordance to ASTM D2166 with some modifications made by NCDOT. To begin the UC test, the sample is placed on the base of the load machine, as shown in Figure 1-12. A platen was placed on top of the sample to evenly distribute the load. Once the base is raised and the load cell is touching the top platen, the test begins. An axial load is applied at a 0.05 inches/minute strain rate until the sample can no longer withstand increasing load. The axial load and deformation of the sample is monitored and recorded every six seconds throughout testing to provide an accurate stress versus strain curve to illustrate the failure of the sample. Failure generally manifests as longitudinal cracks along the specimen, as shown in Figure 1-13. The maximum axial load that the sample can withstand divided by the area of the top of the sample is the unconfined compressive strength. After the sample has failed, a post-test moisture content was also taken from the top, middle, and bottom sections of the sample to ensure that excess moisture is not lost or gained during the curing process.

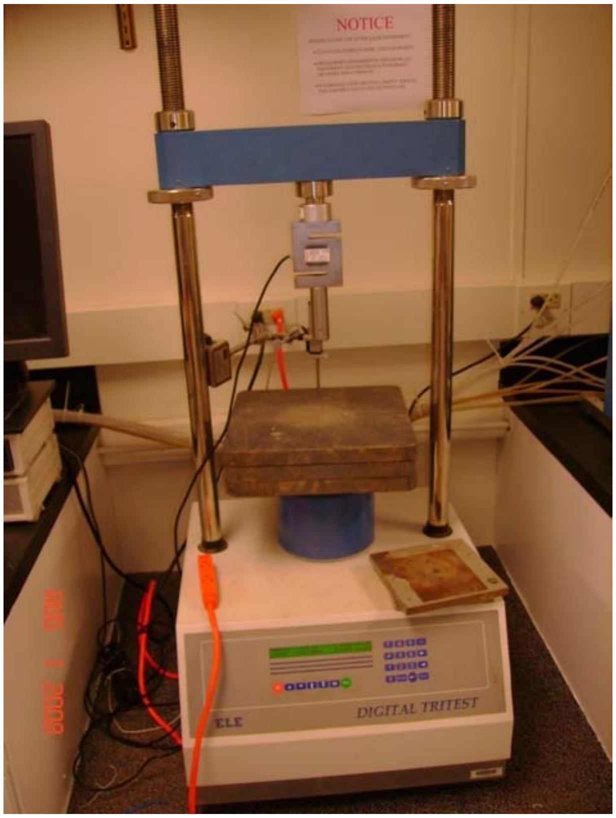

Figure 1-12: Unconfined Compression Testing Apparatus

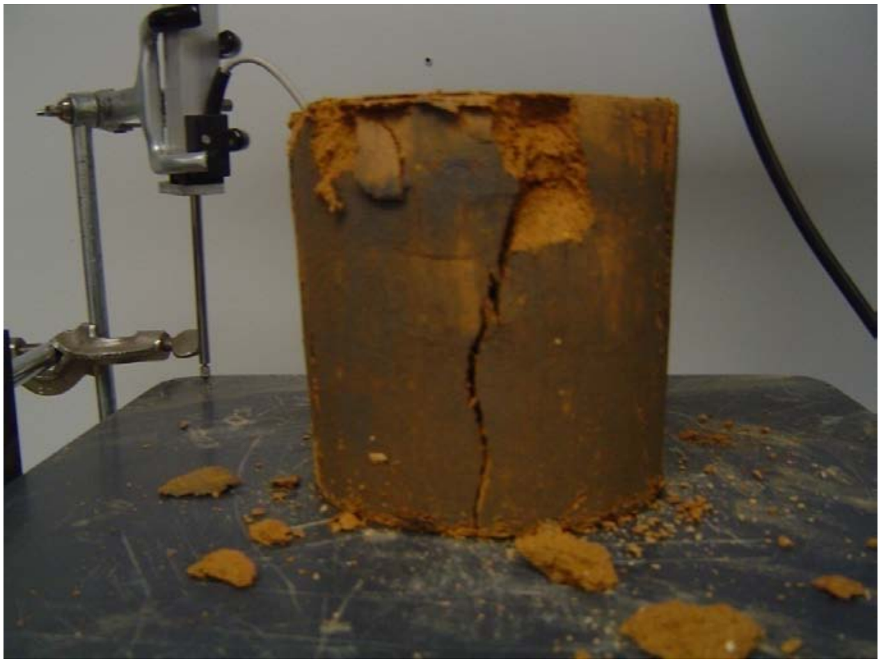

Figure 1-13: Typical Sample Failure

1.4 Results

1.4.1 Soil Characterization

Soil characterization tests were performed by the NCDOT Materials and Test Unit, and in accordance with ASTM standards as outlined in previous "Materials and Methods" section. The NCDOT results of these tests are summarized in Table 1-8 below.

Table 1-8: Soil Characterization Test Results

Sample	**Atterberg Limits**			**% Fines**
	LL	**PL**	**PI**	
Chatham	42	24	18	52%
Cherokee	39	31	8	51%
Iredell	53	37	16	68%

Grain-size analyses were also performed by NCDOT on each soil type. Grain-size distribution curves for each soil type can be seen in Figure 1-14 below.

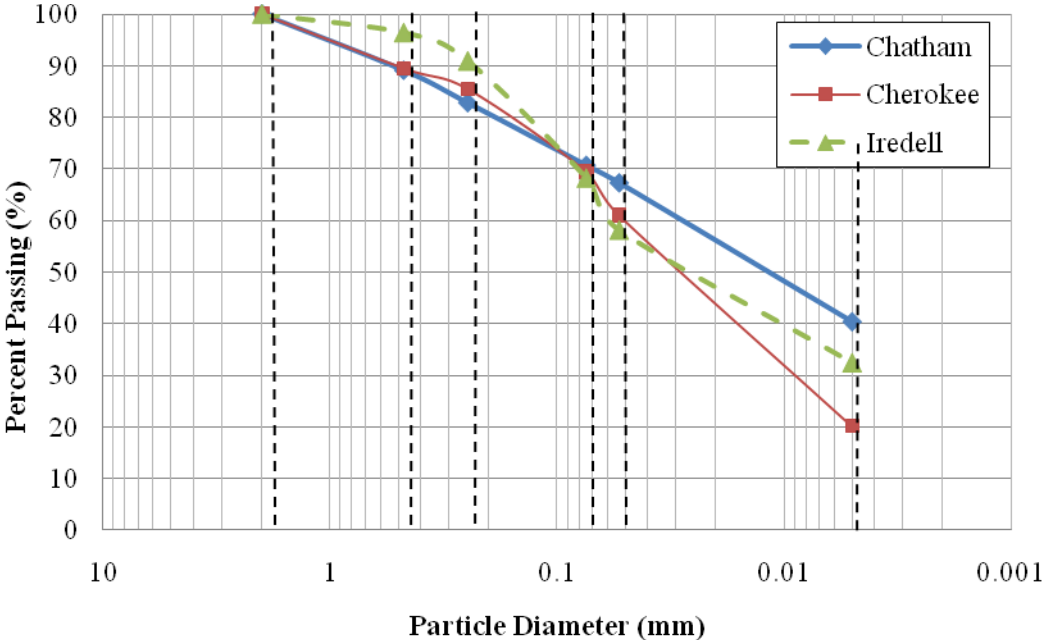

Figure 1-14: Grain-Size Distribution Curves

Based on the results of soil characterization tests and grain-size analysis performed on each soil type, the Chatham county soil classifies as a A-7-6(7) according to AASHTO soil classification system and a CL or lean clay according to USCS soil classification system. The Cherokee county soil classifies as a A-4(2) according to AASHTO soil classification system and a ML or a silt according to USCS soil classification system. The Iredell county soil classifies as a A-7-5(12) according to AASHTO soil classification system and a MH or an elastic silt according to USCS soil classification system.

1.4.2 Moisture-Density Relationships

Moisture-density tests were performed on each soil type to establish maximum dry density and optimum moisture content with soil only and soil plus the appropriate lime content. Figures 1-15 through 1-17 illustrate these moisture-density relationships.

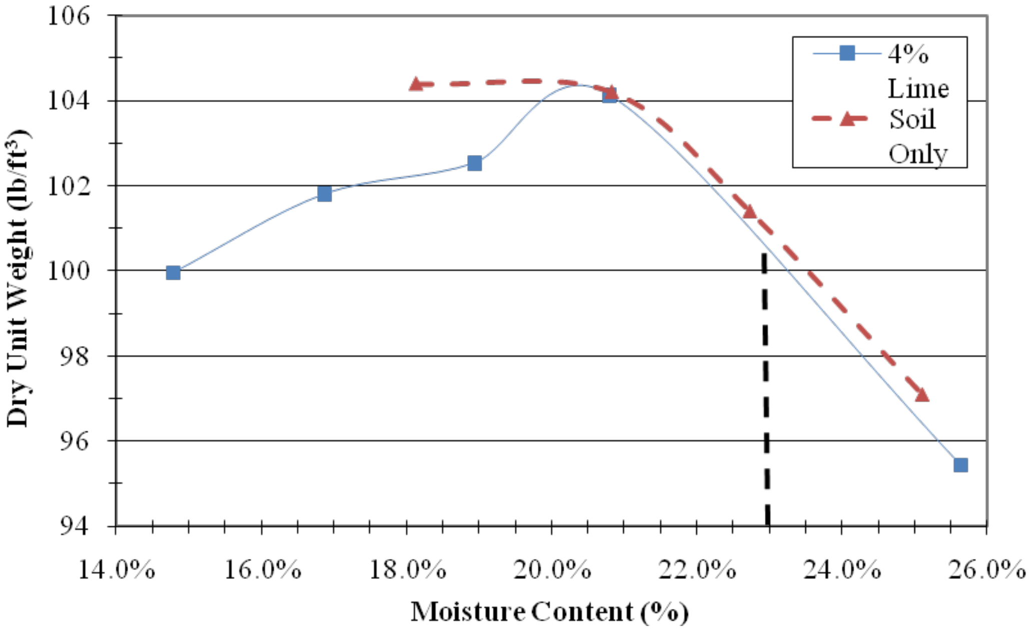

Figure 1-15: Chatham – Moisture-Density

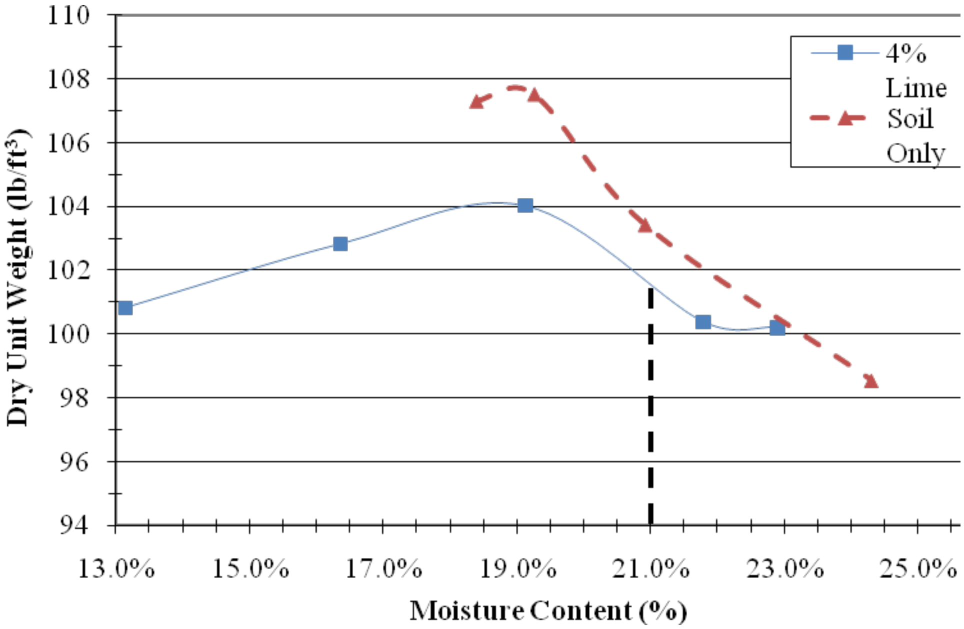

Figure 1-16: Cherokee – Moisture-Density

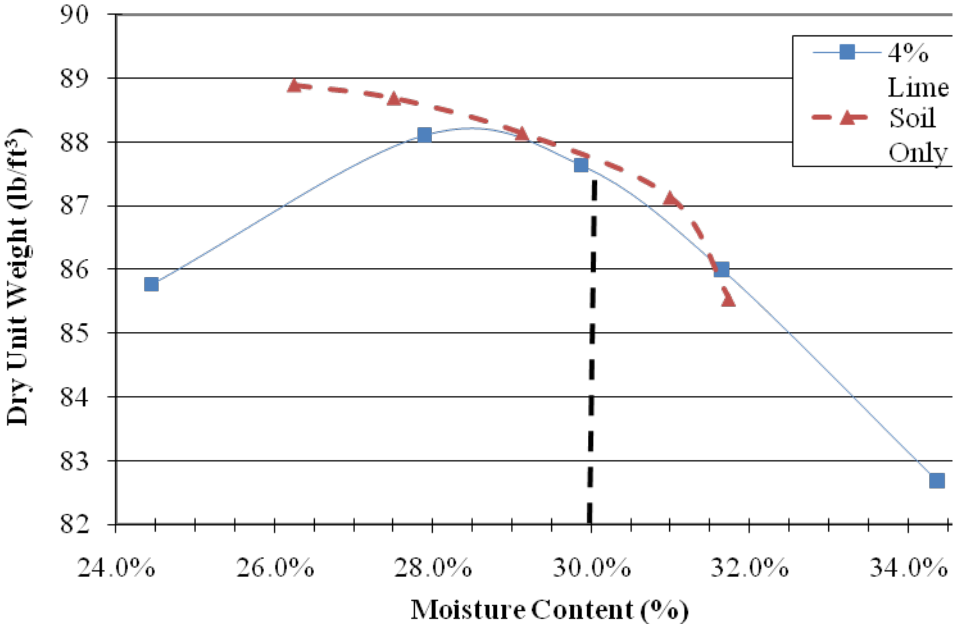

Figure 1-17: Iredell – Moisture-Density

Molding water contents used for unconfined compression specimens varied between 2 and 3% wet of optimum, which is recommended when performing lime stabilization on clayey soils (Mateos and Davidson 1963; Ozier and Moore 1977). The molding water contents used are 23% for Chatham soils, 21% for Cherokee soils, and 30% for Iredell soils. These are displayed on Figures 15 through 17 as vertical dashed lines.

1.4.3 Physicochemical Testing

Physicochemical test results were tested in regards to curing temperature and curing duration. Initial testing for pH and conductivity was investigated with regards to both lime content and curing temperature. Additional pH and conductivity testing was investigated with regards to lime content and curing temperature as well as curing duration. Curing duration pH and conductivity testing was evaluated using two different sample preparation methods, which are discussed in further detail in the "Methods and Materials" section.

1.4.3.1 pH Test Results vs. Temperature

Testing for pH was performed in according with ASTM standards as outlined in the previous "Materials and Methods" section. However, pH testing was performed at varying temperatures to observe the effects of temperature on lime reactivity. The pH values were corrected for temperature effects on the glass electrode. At 25°C (77°F) there is no need for temperature adjustment. However, temperatures above this standard temperature underestimate the pH, and the value should be increased to adjust for temperature. Temperatures below this standard temperature overestimate the pH, and the value should be decreased to adjust for temperature. The correction was made according to:

$$pH_{corrected} = pH_{uncorrected} + \{ (0.03) [(Temp-25^{o}C) / (10\,^{o}C)] (pH_{uncorrected} - 7) \}$$

The results of pH testing at varying temperatures can be seen for Chatham, Cherokee, and Iredell counties in Figures 1-18 through 1-20 below.

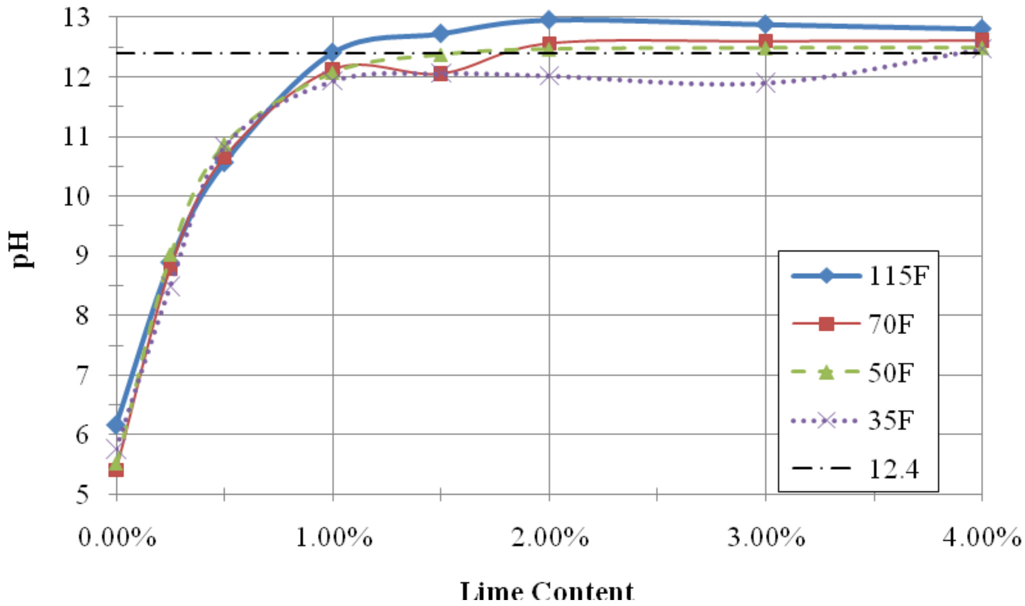

Figure 1-18: Chatham County – pH Results vs. Temperature

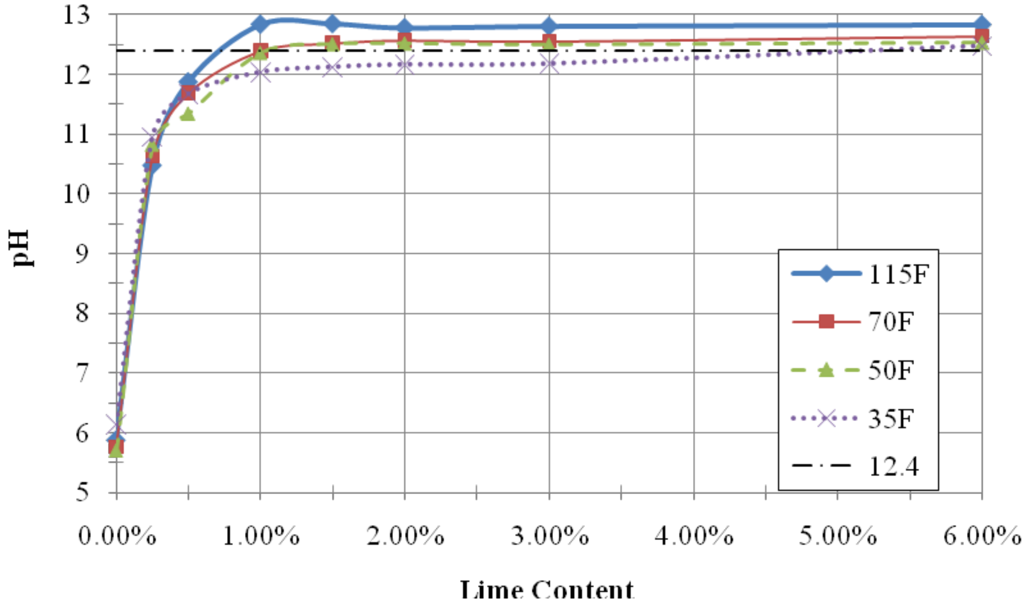

Figure 1-19: Cherokee County – pH Results vs. Temperature

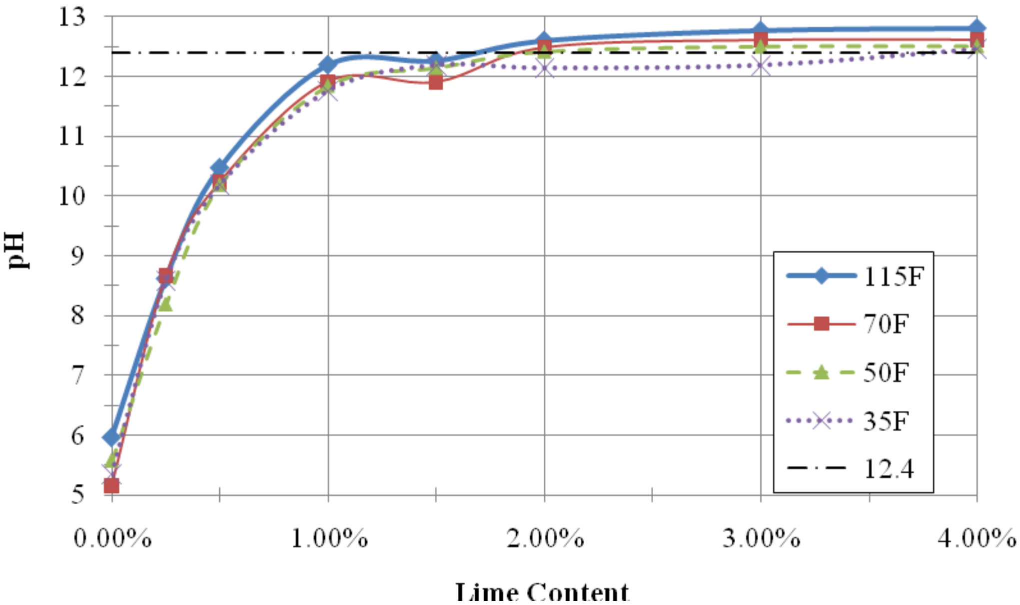

Figure 1-20: Iredell County – pH Results vs. Temperature

1.4.3.2 Conductivity Test Results vs. Temperature

Conductivity readings were taken for the same samples used in pH testing. Similar to pH readings, conductivity readings must also be corrected for temperature when readings are not taken at 25ºC. The equation used to correct conductivity readings for temperature is given as:

$$\text{Conductivity}_{\text{corrected}} = \text{Conductivity}_{\text{uncorrected}}[1+0.021(\text{Temperature} - 25^{o}\text{C})]$$

The results of conductivity testing at varying temperatures for Chatham, Cherokee, and Iredell counties can be seen below in Figure 1-21 through 1-23 below.

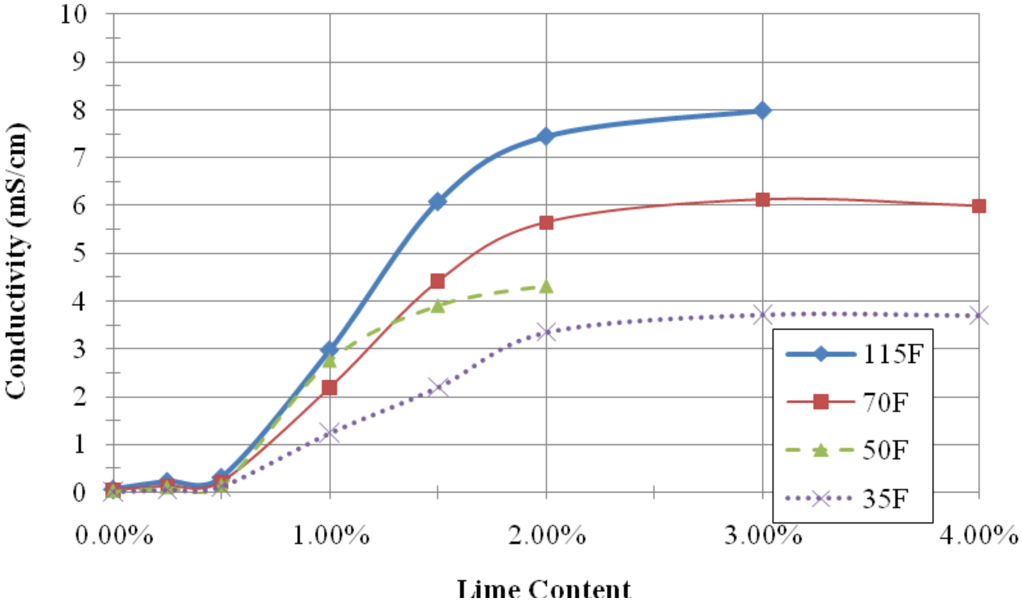

Figure 1-21: Chatham County – Conductivity Results vs. Temperature

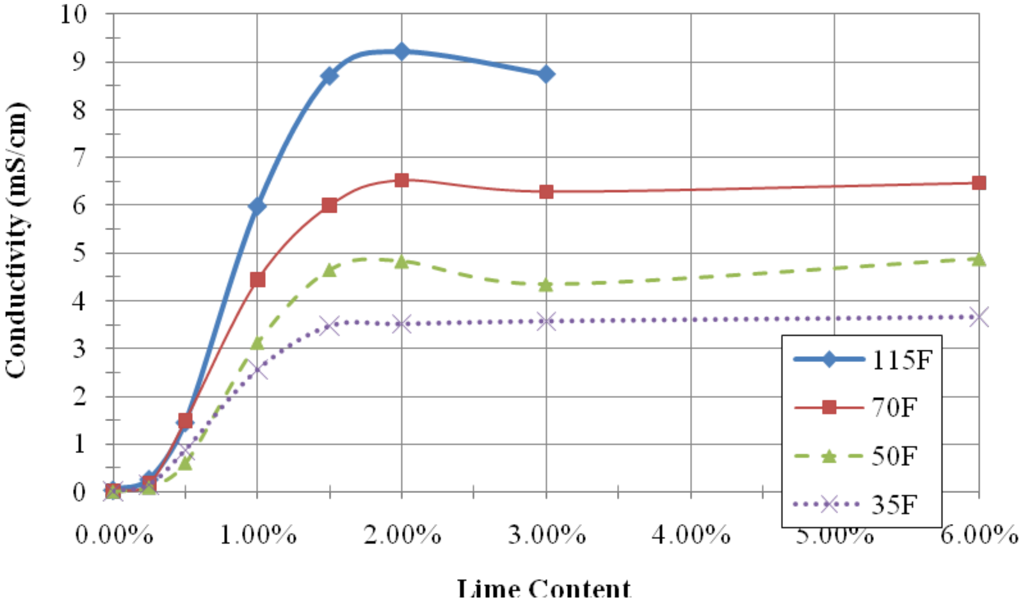

Figure 1-22: Cherokee County – Conductivity Results vs. Temperature

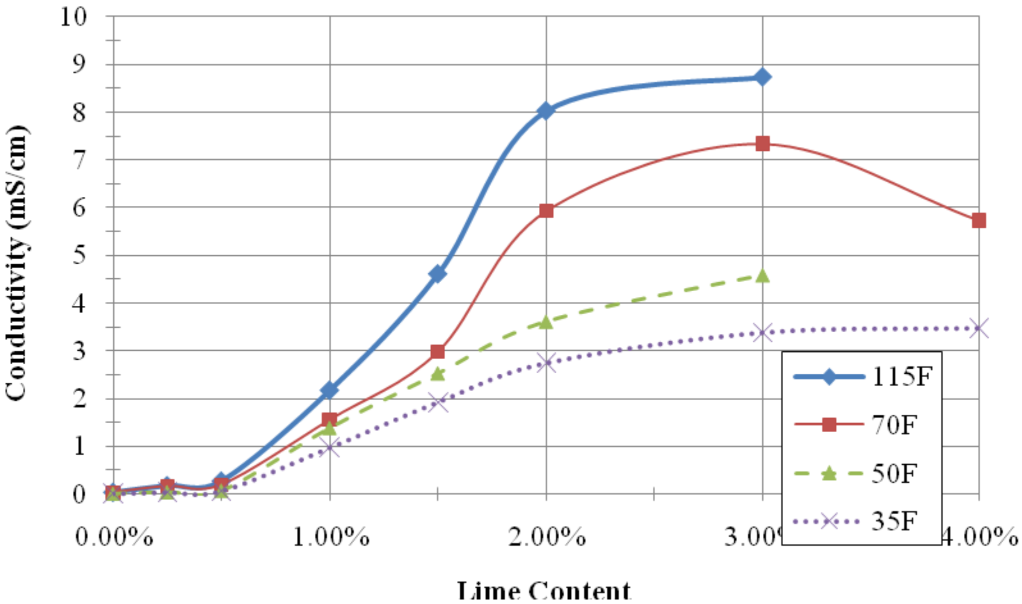

Figure 1-23: Iredell County – Conductivity Results vs. Temperature

Similar to pH results, conductivity readings also increase with increasing lime content and increasing temperature.

1.4.3.3 pH Test Results vs. Duration (Eades and Grim Method)

During initial pH testing against duration, samples were prepared similarly to pH testing against temperature. The soil, lime, and water were mixed and exposed to the one hour of shaking as described in the "Methods and Materials" section. All samples were mixed with optimum lime content, which was 4% lime for Chatham and Iredell counties, and 6% lime for Cherokee county. Samples were then cured in the curing chambers used to cure unconfined compression samples. After specific curing durations, the samples were exposed to another hour of shaking. Samples were cured at the initial 1 hour, 3 hours, 6 hours, 12 hours, 1 day, 3 days, 7 days, 14 days, and 28 days before pH readings were taken.

The results for pH test results with varying curing durations for Chatham, Cherokee, and Iredell counties can be seen in Figures 1-24 through 1-26 below.

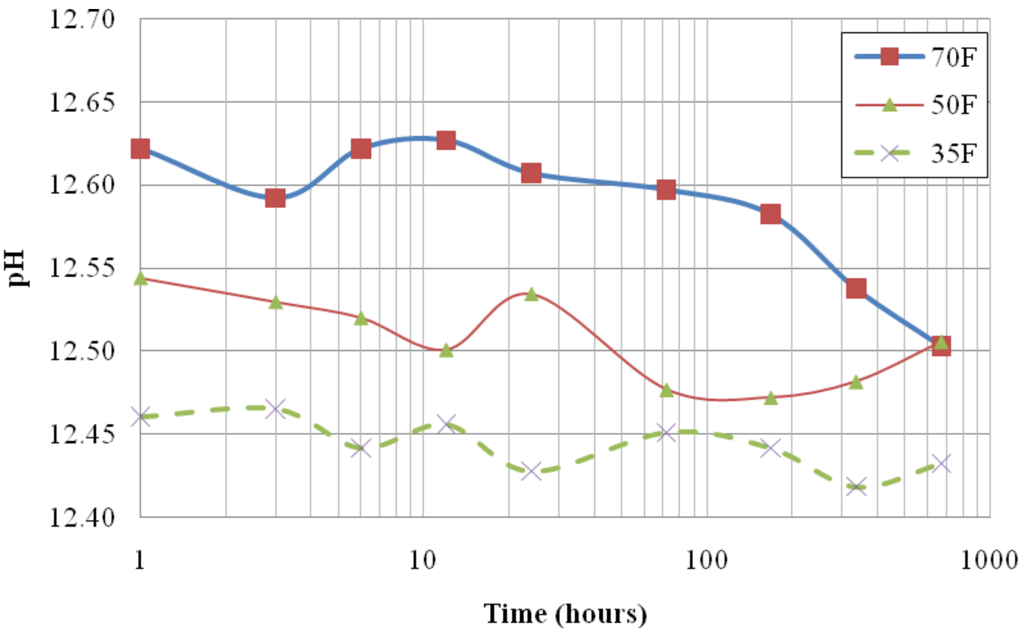

Figure 1-24: Chatham County (4% Lime) – pH Results vs. Duration

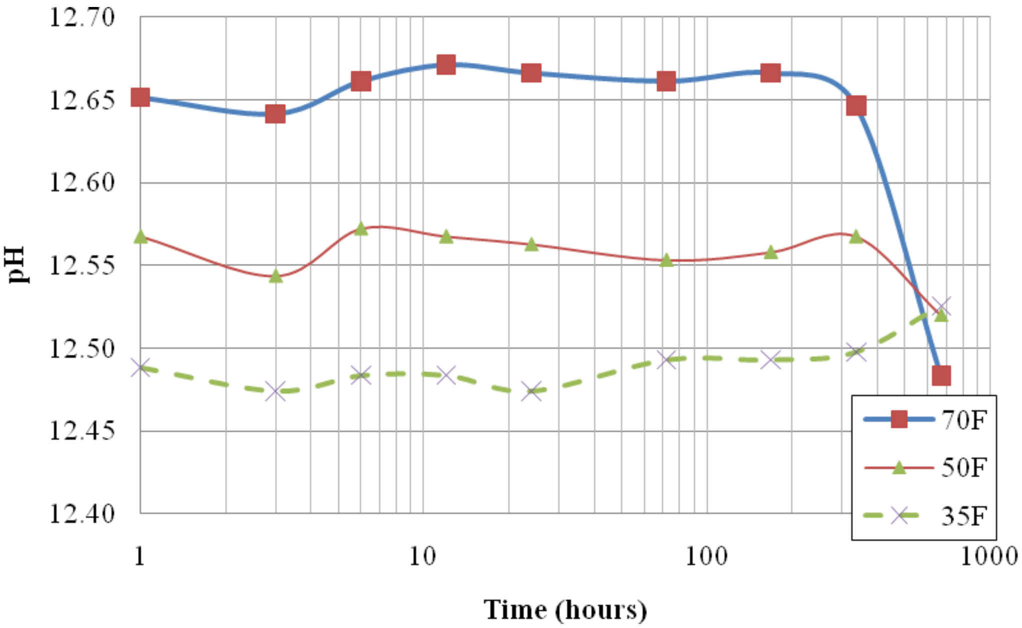

Figure 1-25: Cherokee County (6% Lime) – pH Results vs. Duration

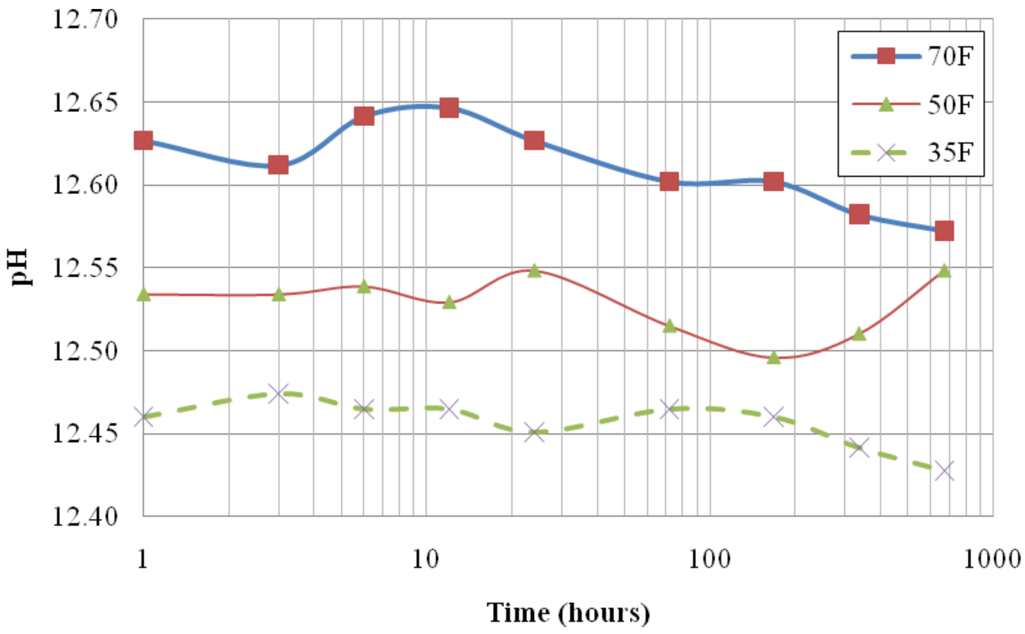

Figure 1-26: Iredell County (4% Lime) – pH Results vs. Duration

On the whole, there is a pH drop between the initial and final pH readings. This pH decrease is due to the fact that the lime dissociates in the water leaving hydroxide ions (OH^-). Over time, the aluminates and silicates in the soil react with these free hydroxide ions to form hydrates. The longer the soil and lime are allowed to react, the less hydroxide ions in solution, and the lower the pH of the solution. For those samples that were cured at 70ºF, especially Chatham and Cherokee counties, there is a sharp pH drop after 7 days of curing. Those samples cured at 50ºF and 35ºF do not demonstrate significant pH drops throughout 28 days of curing. This may be a result of the fact that pozzolanic reaction is ceasing to take place resulting in minimal pH drop over this time period.

1.4.3.4 Conductivity Test Results vs. Duration (Eades and Grim Method)

Conductivity readings were taken subsequently to pH readings using the same soil, lime, and water mixtures. The conductivity test results against duration for Chatham, Cherokee, and Iredell counties can be seen below in Figures 1-27 through 1-29 below.

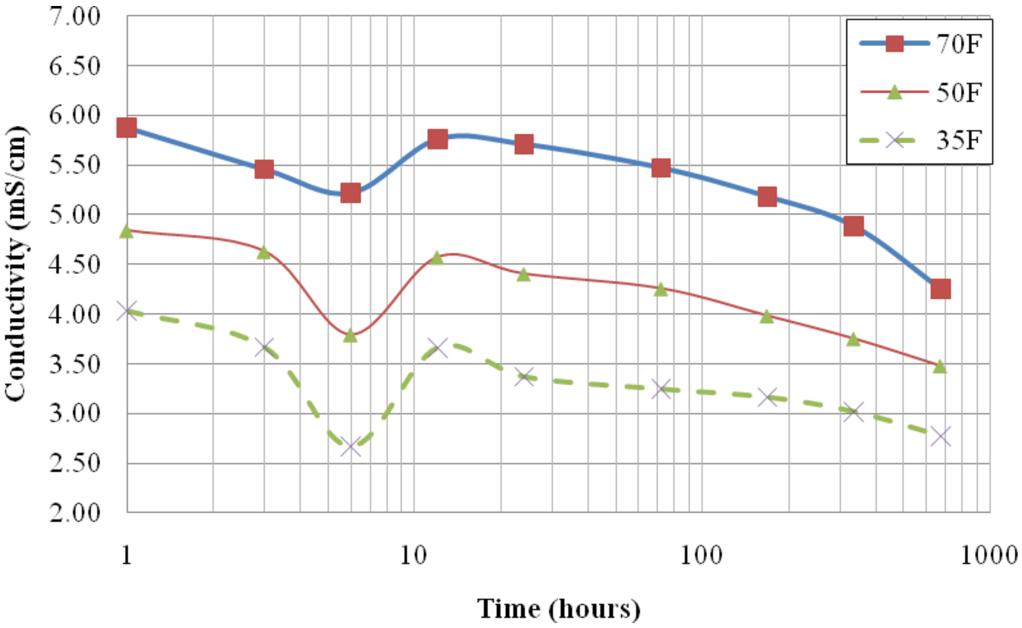

Figure 1-27: Chatham County (4% Lime) – Conductivity Results vs. Duration

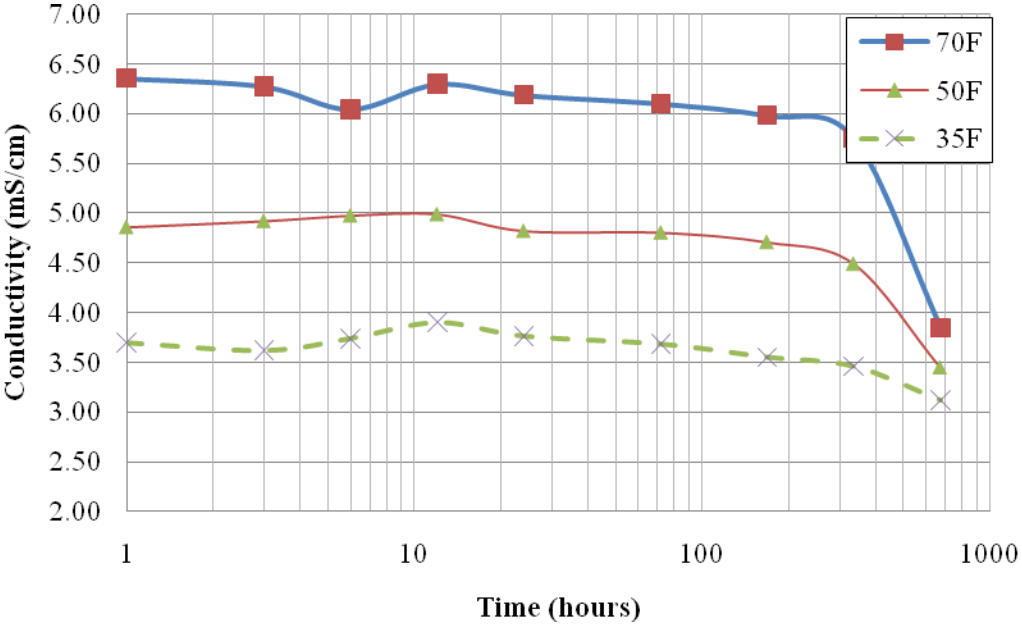

Figure 1-28: Cherokee County (6% Lime) – Conductivity Results vs. Duration

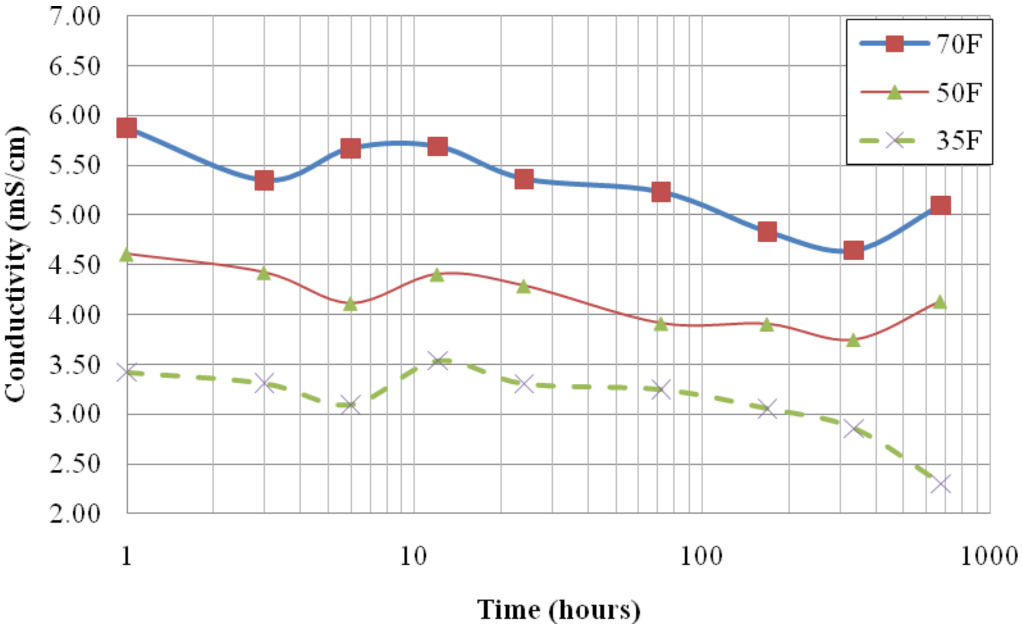

Figure 1-29: Iredell County (4% Lime) – Conductivity Results vs. Duration

1.4.3.5 pH Test Results vs. Duration (Alternative Method)

Since little research had been done before evaluating the effects of curing duration on pH, and there was no drastic decrease in pH over a 28 day curing period using the Eades and Grim method, an alternative method was proposed to derive more insight into lime reactivity. Samples were prepared similarly to unconfined compression test samples as described in the “Methods and Materials” section. After the sample had cured, 25 grams of the soil-lime mixture was chipped off of the sample. The 25 gram sample was finely pulverized and passed through the No. 40 sieve. The soil-lime sample and water were mixed at a 1:2.5 ratio, so 25 grams of soil-lime sample was mixed with 62.5 grams of water. All samples were mechanically shaken at 200 rpms for an hour prior to pH testing. This alternative method (Rao and Shivananda 2005) was used because it is a more realistic approach of how soil and lime actually react in the field. The results for pH test results using the alternative method with varying curing durations for Chatham, Cherokee, and Iredell counties can be seen in Figures 1-30 through 1-32 below.

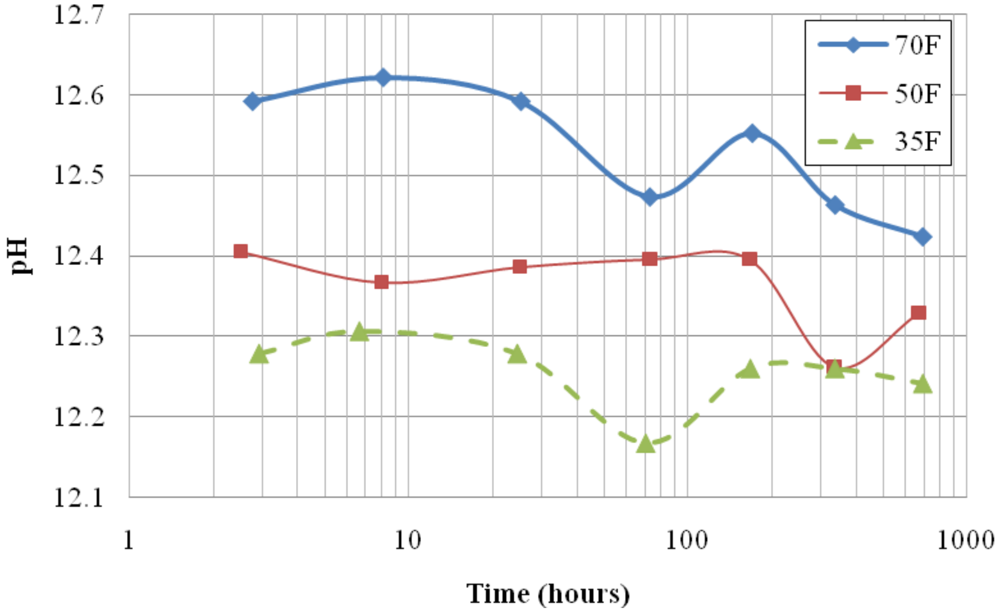

Figure 1-30: Chatham County (4% Lime) – pH Results vs. Duration (Alternative)

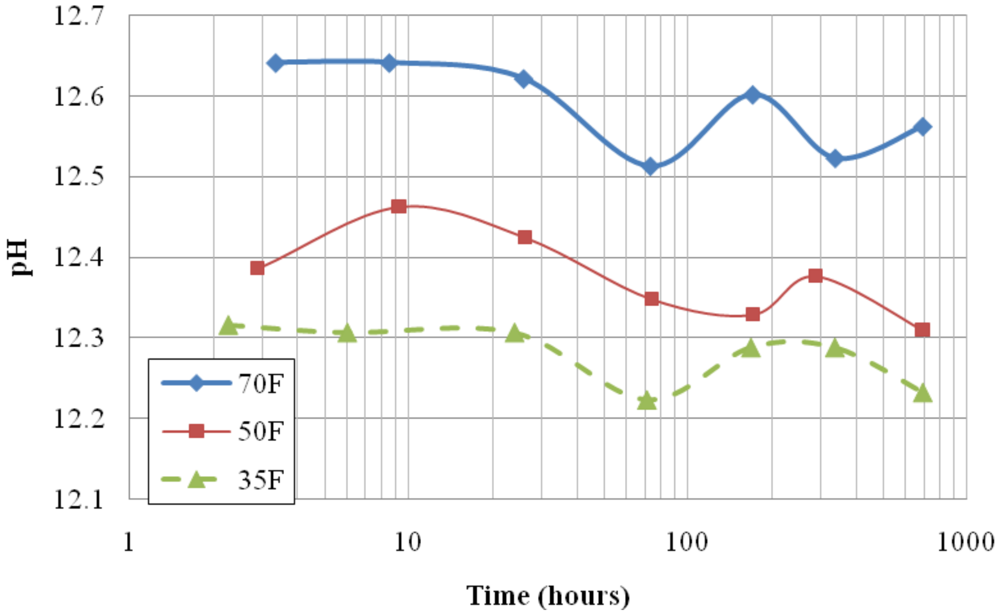

Figure 1-31: Cherokee County (6% Lime) – pH Results vs. Duration (Alternative)

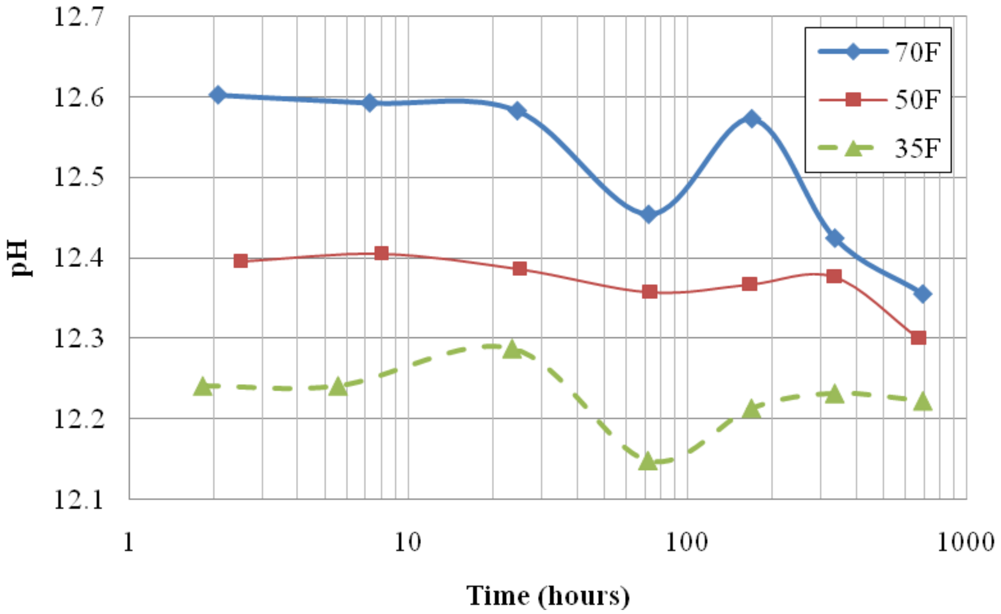

Figure 1-32: Iredell County (4% Lime) – pH Results vs. Duration (Alternative)

From the figures, it can be seen that there is a distinct pH drop during the 72 hour reading or 3 days. This drop is present in both samples that cured at 70ºF and 35ºF. Overall, the alternative method demonstrates similar trends to the Eades and Grim method. The samples cured at 70ºF resulted in higher pH decreases than those samples cured at 50ºF and 35ºF.

1.4.3.6 Conductivity Test Results vs. Duration (Alternative)

Conductivity readings were taken subsequently to pH readings using the same soil, lime, and water mixtures. The conductivity test results against duration for Chatham, Cherokee, and Iredell counties can be seen below in Figures 1-33 through 1-35 below.

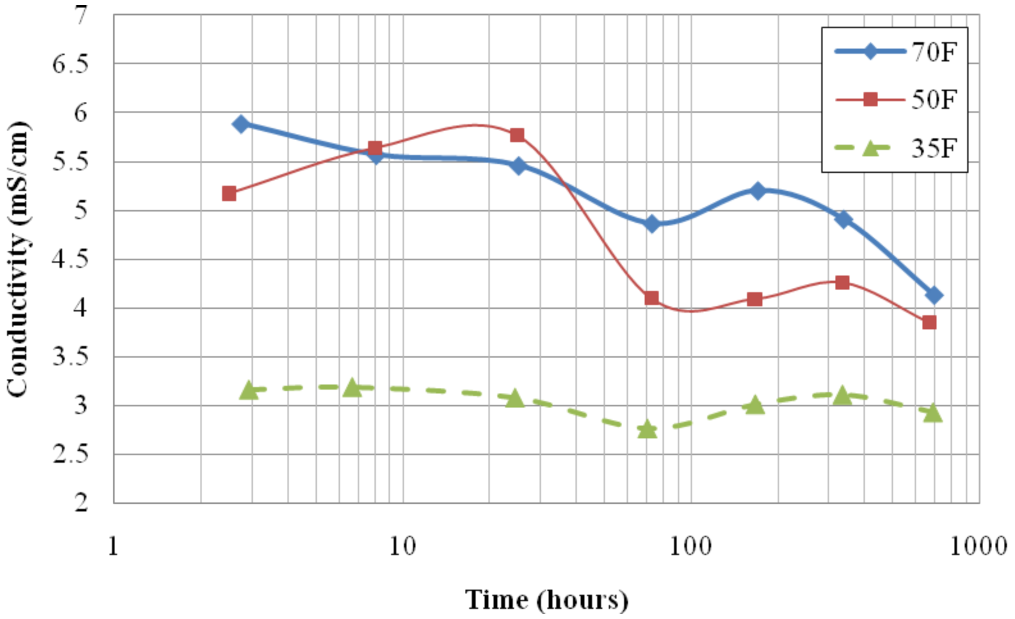

Figure 1-33: Chatham County (4% Lime) – Conductivity Results vs. Duration (Alternative)

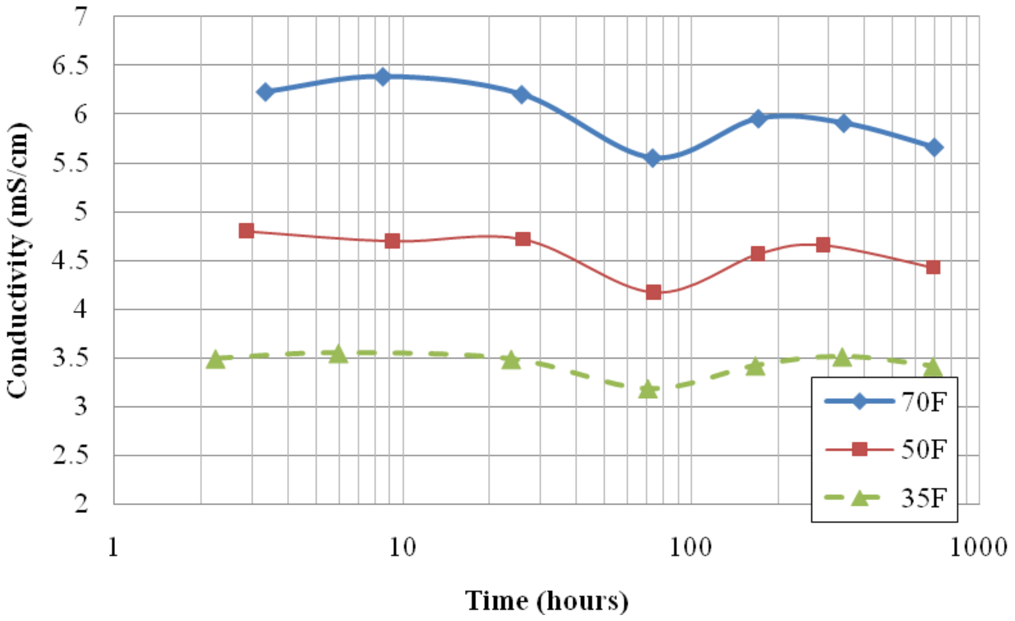

Figure 1-34: Cherokee County (6% Lime) – Conductivity Results vs. Duration (Alternative)

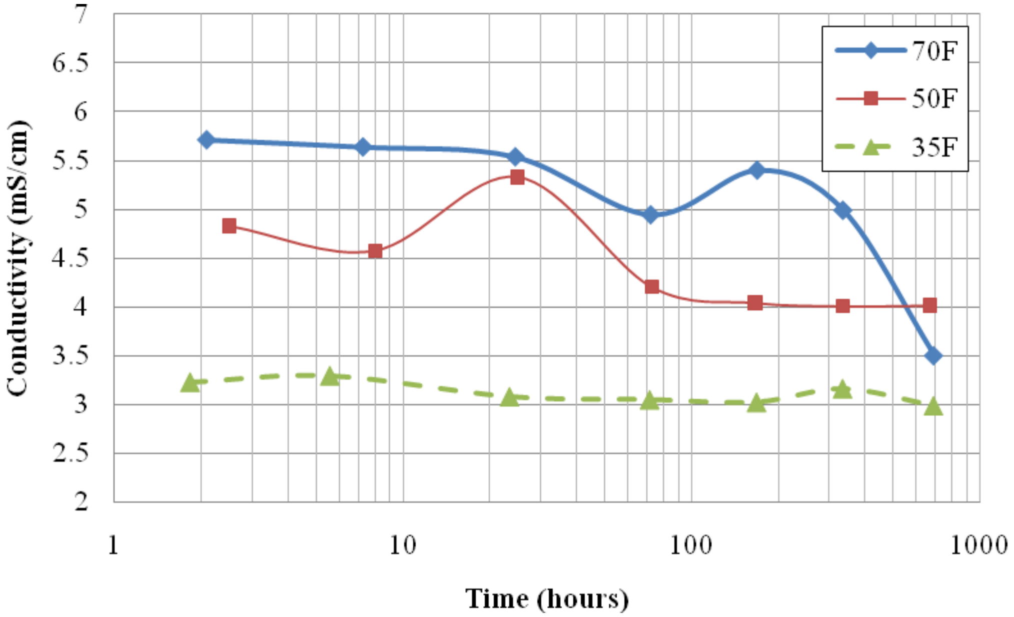

Figure 1-35: Iredell County (4% Lime) – Conductivity Results vs. Duration (Alternative)

1.4.4 Unconfined Compression (UC) Tests

The majority of testing throughout the project was unconfined compression tests to establish an average unconfined compressive strength under each specific curing condition. After establishing an optimum lime content and optimum water content for each soil type, unconfined compression soil-lime mixtures were prepared for several different curing conditions.

1.4.4.1 Constant Temperature

With the purpose being to investigate the performance of soil-lime mixtures under cold weather temperature, the most significant variable throughout testing was temperature. Soil-lime mixtures were cured under constant durations of 1, 3, 7, 28, and 56 days at temperatures of 70ºF, 50ºF, 35ºF, and 25ºF. Samples were also exposed to freeze-thaw cycles as described in Section 1.4.4.3. With a specific curing temperature and curing duration, the effects of strength could be evaluated against both variables.

This section investigates the effects of curing temperature on unconfined compressive strength. Figures 1-36 through 1-40 illustrate the effect of constant curing durations of 1, 3, 7, 28, and 56 days at varying temperatures of 70ºF, 50ºF, 35ºF, and 25ºF on unconfined compressive strength.

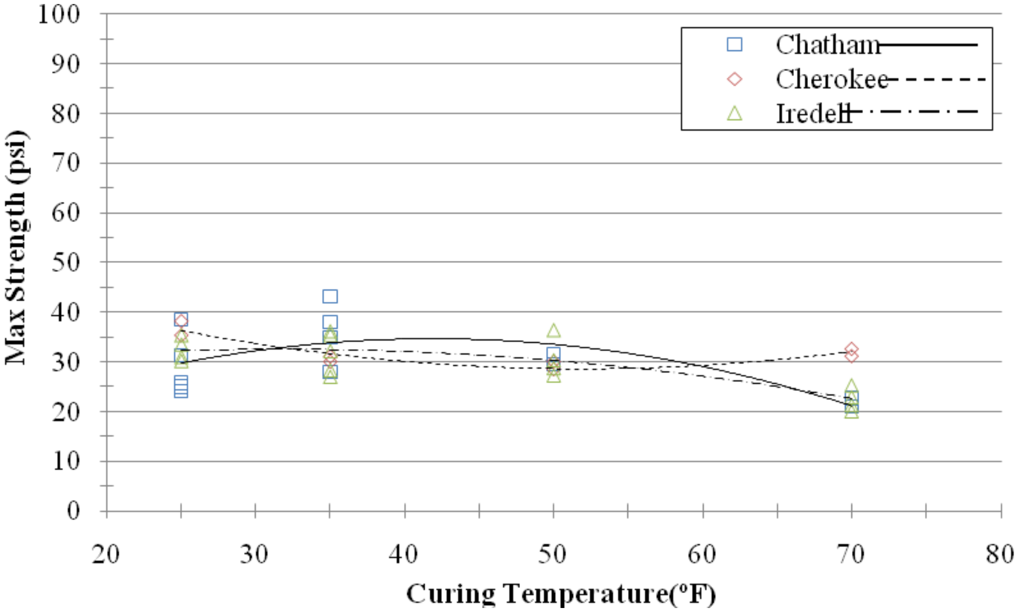

Figure 1-36: UC Strength vs. Curing Temperature (1 Day Curing)

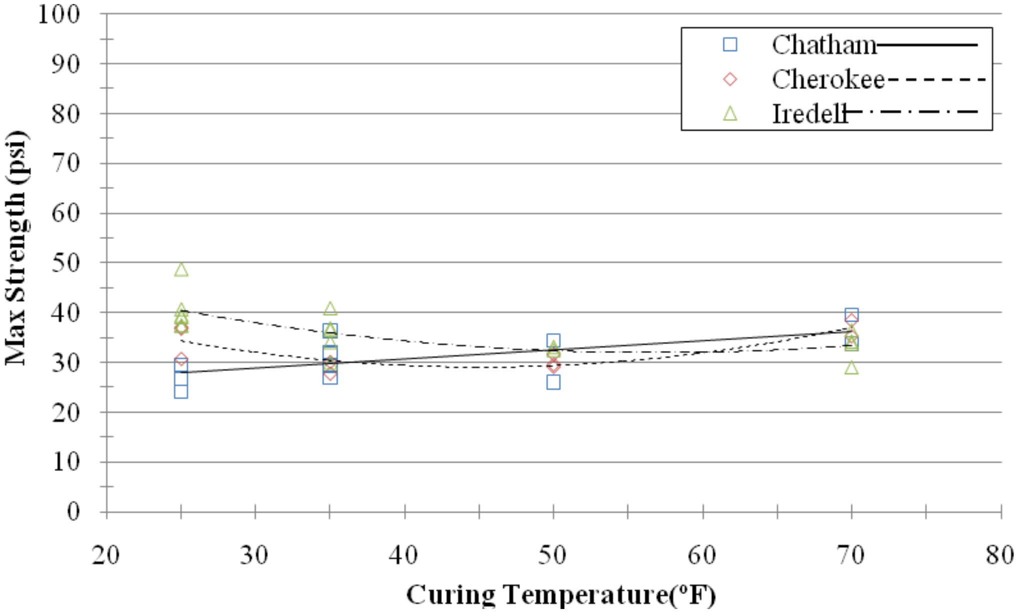

Figure 1-37: UC Strength vs. Curing Temperature (3 Day Curing)

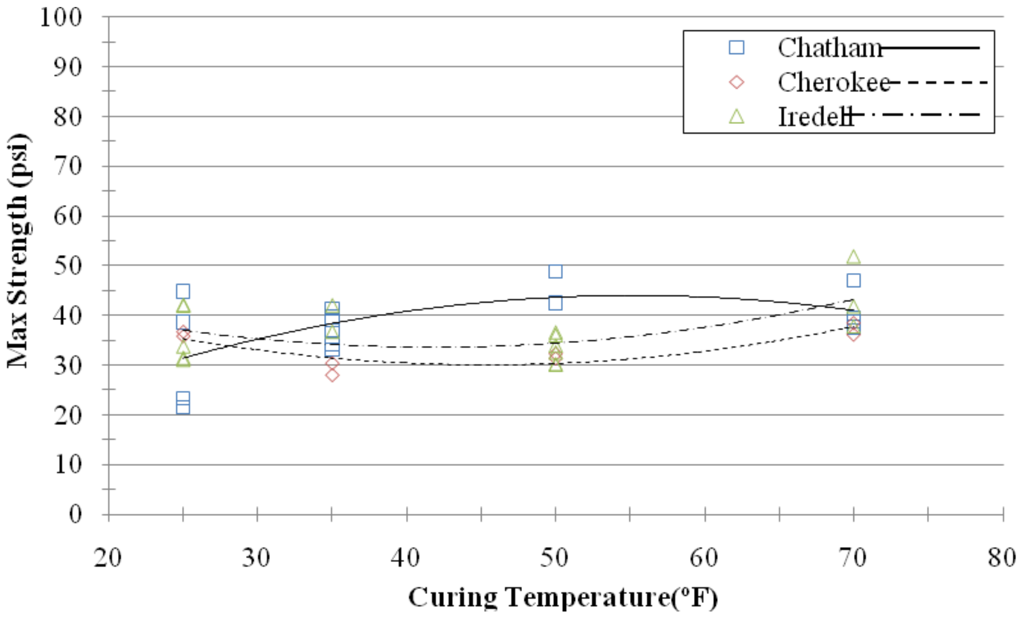

Figure 1-38: UC Strength vs. Curing Temperature (7 Day Curing)

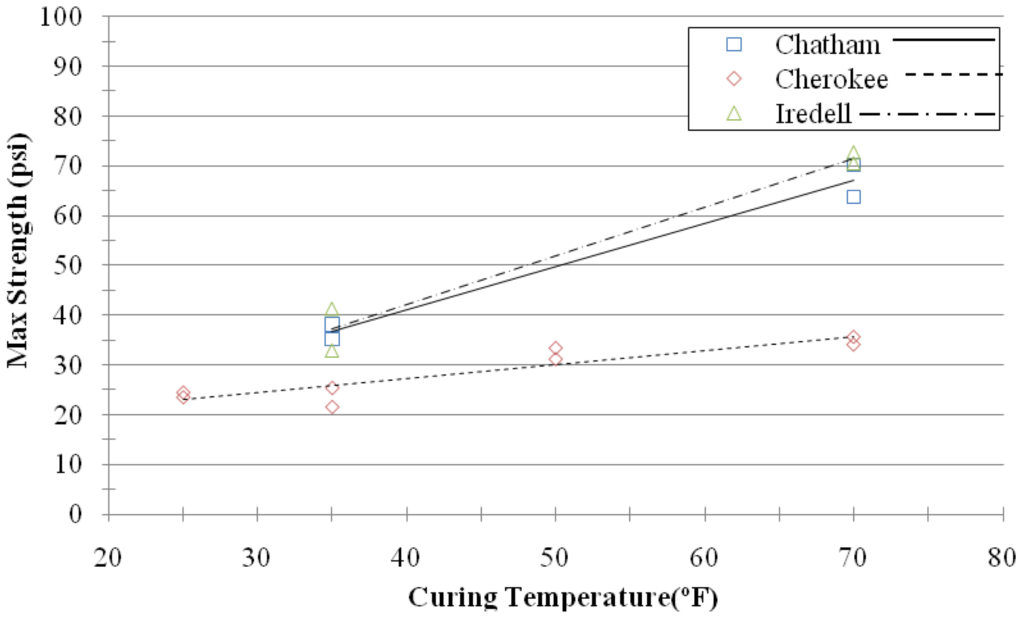

Figure 1-39: UC Strength vs. Curing Temperature (28 Day Curing)

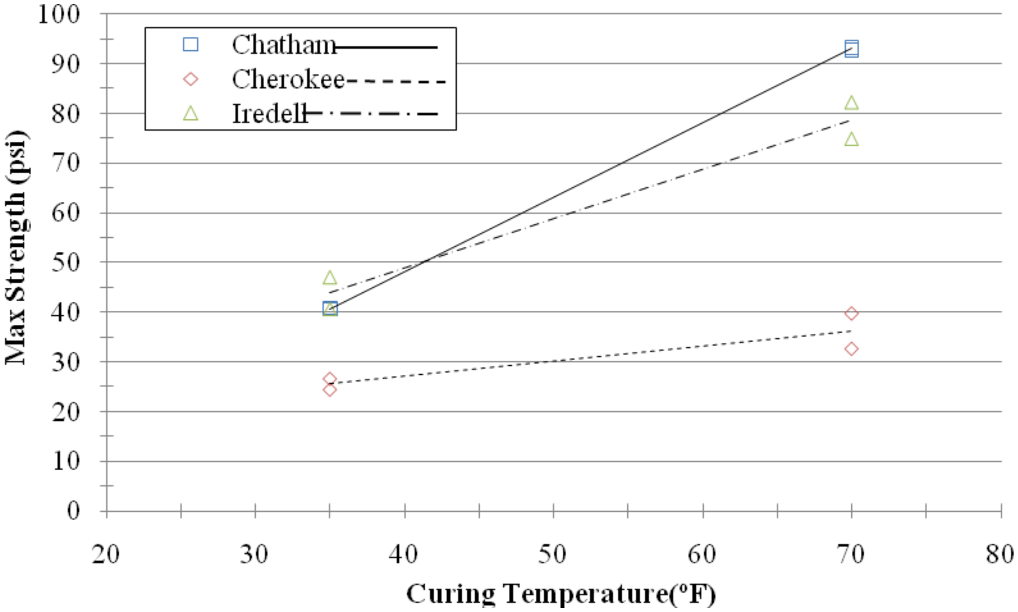

Figure 1-40: UC Strength vs. Curing Temperature (56 Day Curing)

1.4.4.2 Constant Duration

This section investigates the effects of curing duration on unconfined compressive strength. Figures 1-41 through 1-44 illustrate the effect of constant curing temperatures of 70ºF, 50ºF, 35ºF, and 25ºF and varying short-term curing durations of 1, 3, and 7 days on unconfined compressive strength.

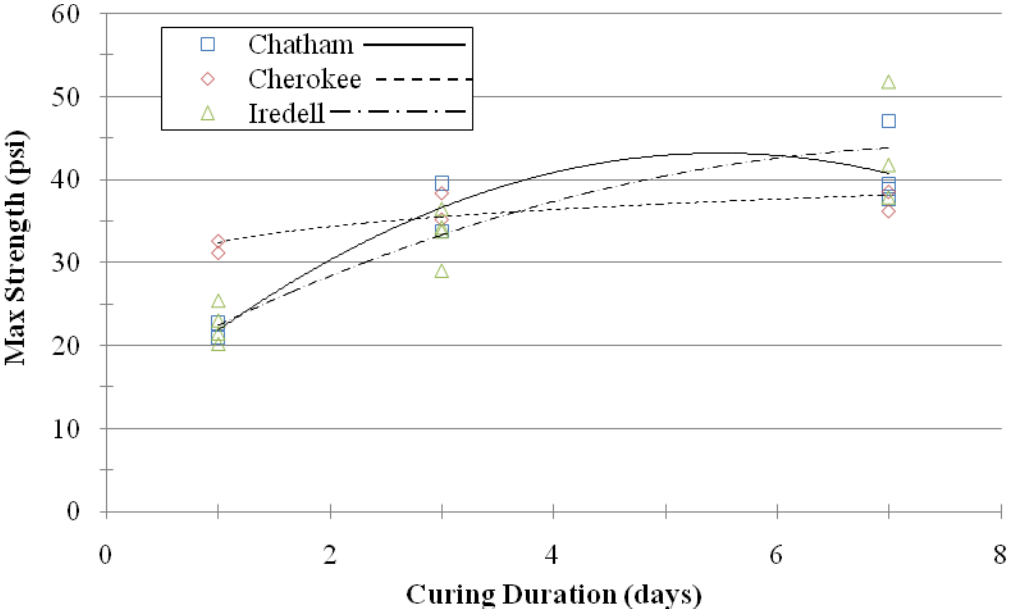

Figure 1-41: UC Strength (Short-Term) vs. Curing Duration (70ºF)

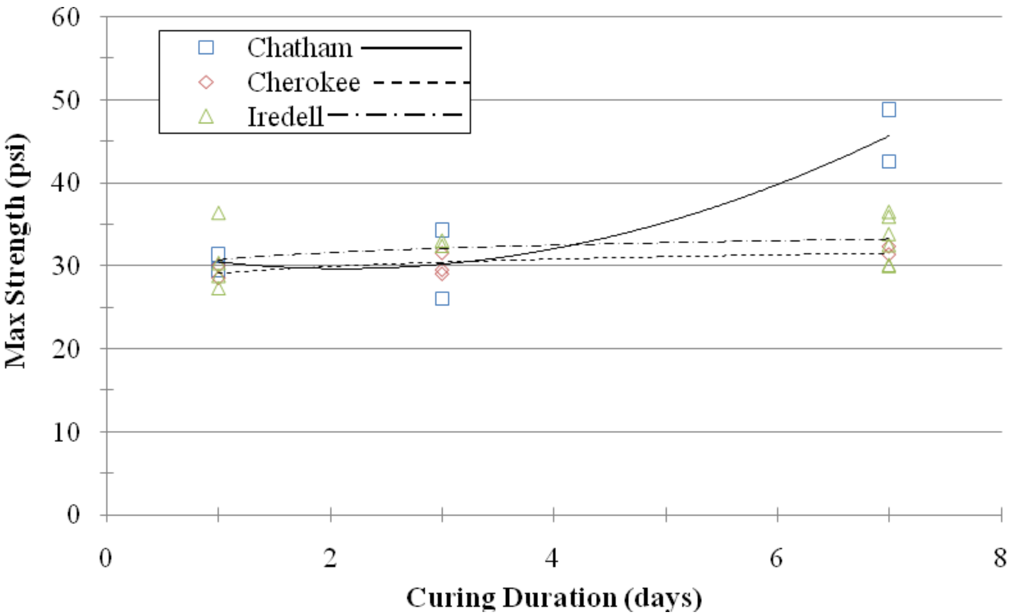

Figure 1-42: UC Strength (Short-Term) vs. Curing Duration (50ºF)

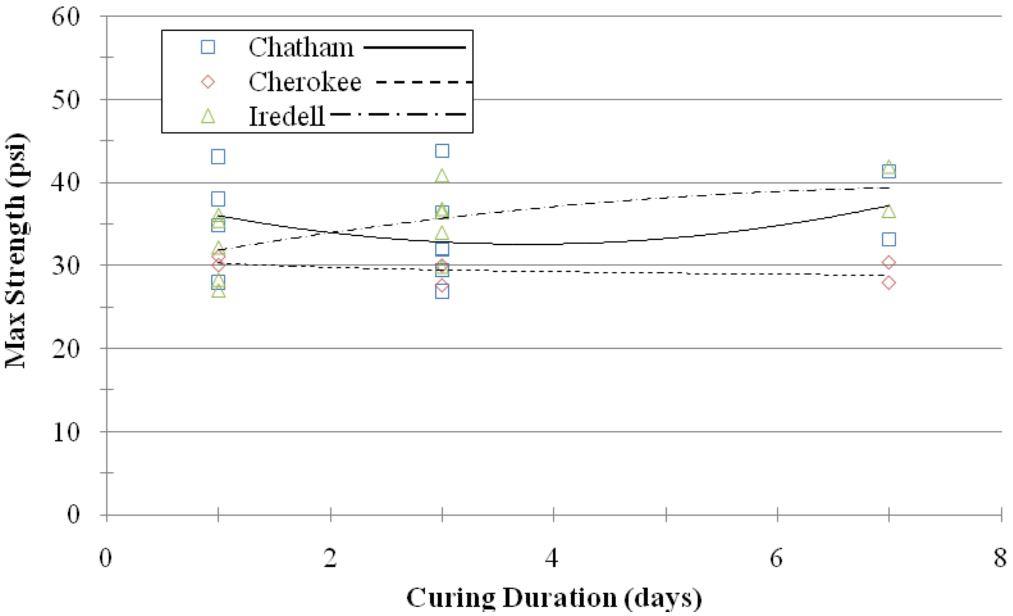

Figure 1-43: UC Strength (Short-Term) vs. Curing Duration (35ºF)

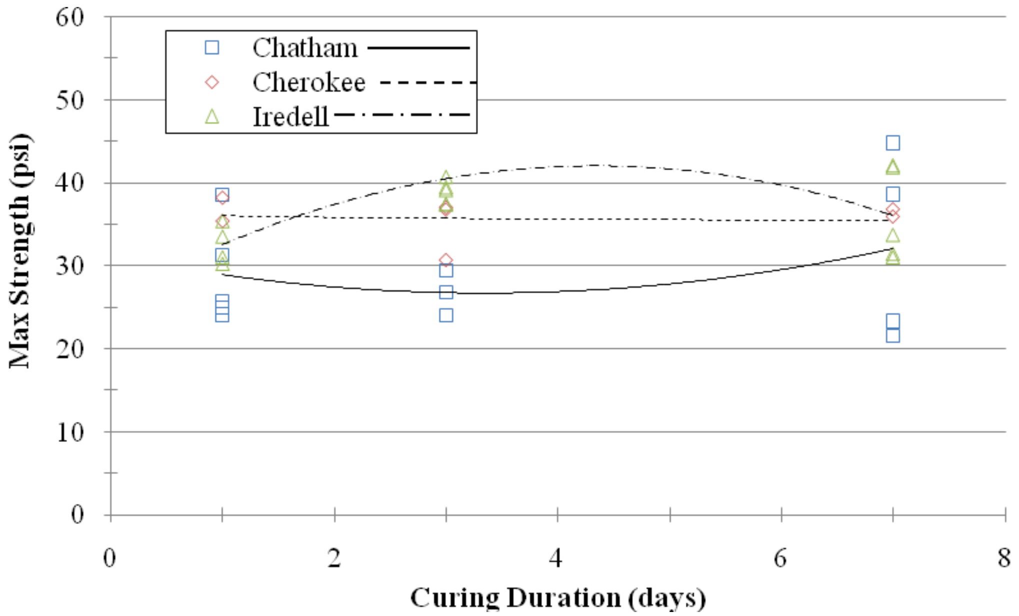

Figure 1-44: UC Strength (Short-Term) vs. Curing Duration (25ºF)

Figures 1-45 and 1-46 illustrate the effect of constant curing temperature and varying long-term curing durations on unconfined compressive strength at 70ºF and 35ºF.

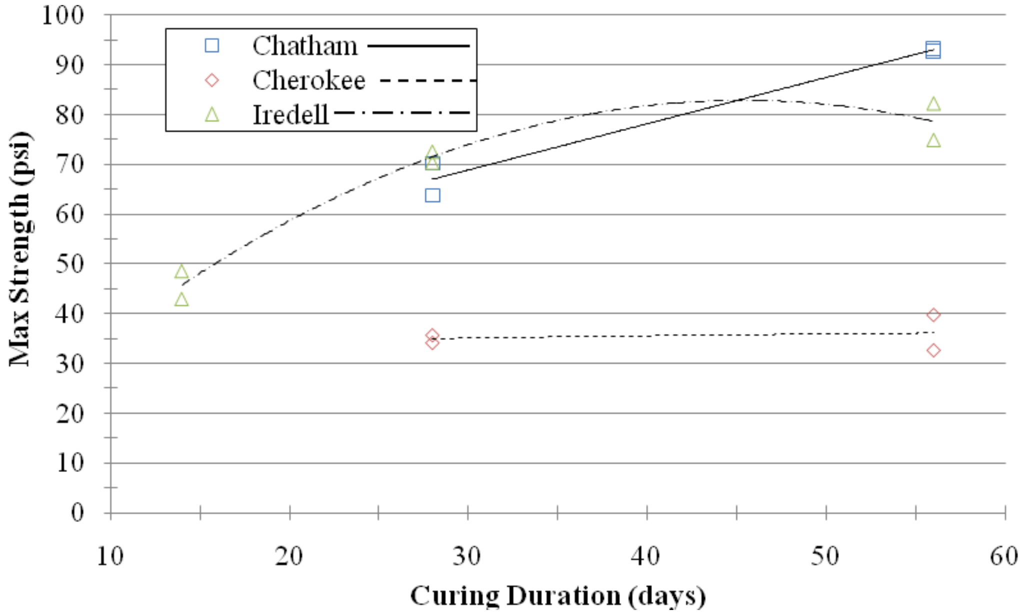

Figure 1-45: UC Strength (Long-Term) vs. Curing Duration (70ºF)

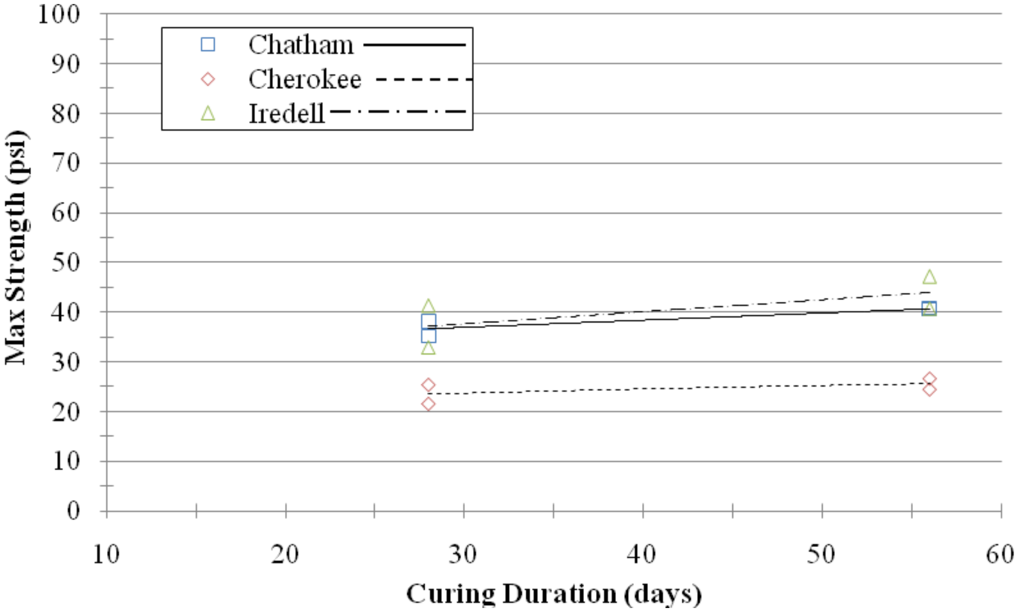

Figure 1-46: UC Strength (Long-Term) vs. Curing Duration (35ºF)

To better demonstrate the "big picture," or the effects of both short-term and long-term curing durations, Figures 1-47 and 1-48 are presented below.

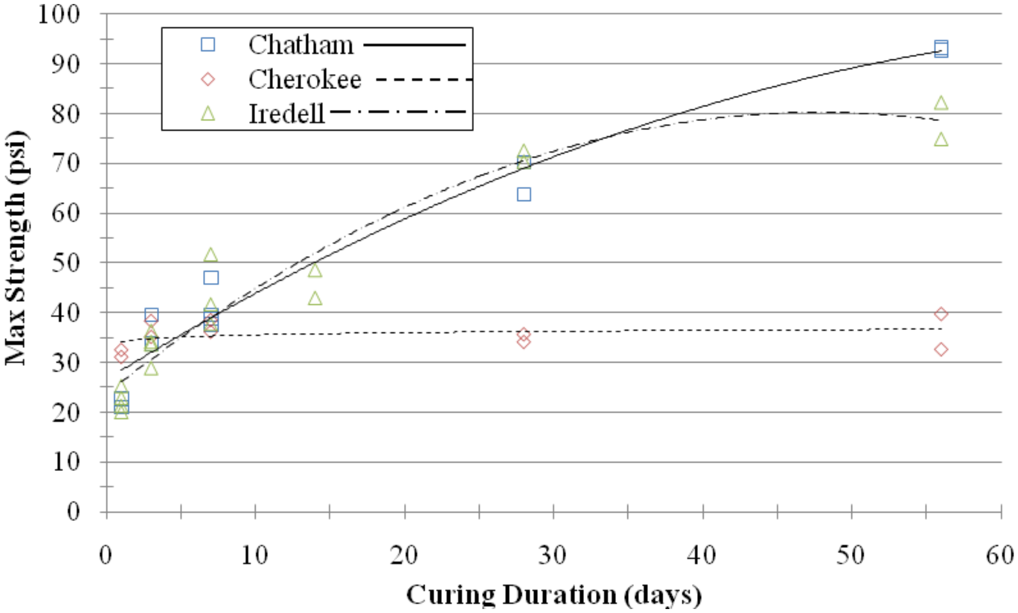

Figure 1-47: UC Strength (Short and Long-Term) vs. Curing Duration (70ºF)

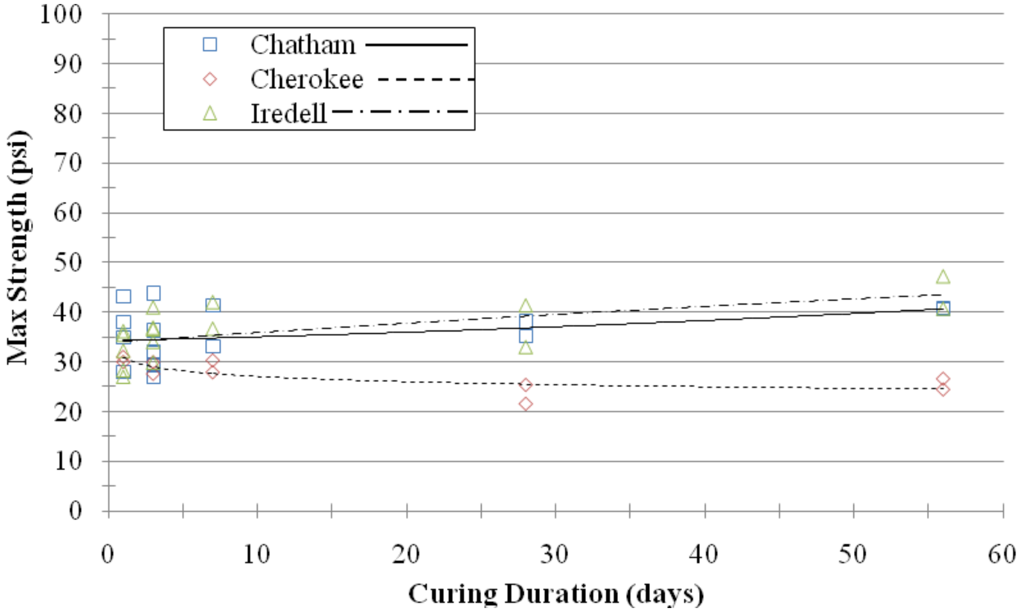

Figure 1-48: UC Strength (Short and Long-Term) vs. Curing Duration (35ºF)

1.4.4.3 Freeze-Thaw

To further investigate unconfined confined compressive strengths of lime-soil mixtures at cooler temperatures, samples were exposed to freeze-thaw cycles as well as a “spring thaw.” All samples were cured as specified in the “Methods and Materials” section. Figures 1-49 and 1-50 illustrate the effects of short-term and long-term, respectively, freeze-thaw cycles on unconfined compressive strength.

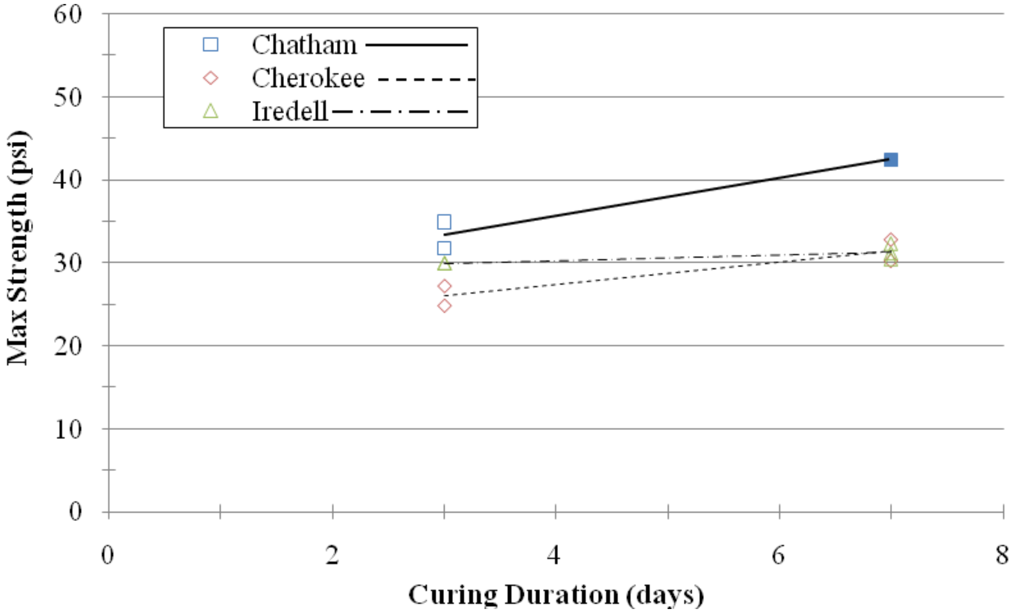

Figure 1-49: UC Strength (Short-Term) vs. Curing Duration (Freeze-Thaw)

All individual samples fell outside of the moisture content range of ±2% of the optimum moisture content for Chatham county soil-lime samples at the curing condition of 7 days with one short-term freeze thaw cycle. Nonetheless, these values were included for comparison purposes. Table 1-9 compares those samples exposed to a short-term freeze thaw cycle and those control samples exposed to no freeze-thaw cycles.

Table 1-9: Freeze-Thaw Effects on UC Strength

Curing Temperature	Curing Duration (days)	Average Unconfined Compressive Strength (psi)		
		Chatham	Cherokee	Iredell
F/T	3	33.3	26.0	30.0
F/T	7	42.4	31.5	31.3
70ºF	3	36.6	36.8	33.3
70ºF	7	40.7	37.4	43.8

Generally, the freeze-thaw cycling led to reduced strength, although the overall difference is relatively modest.

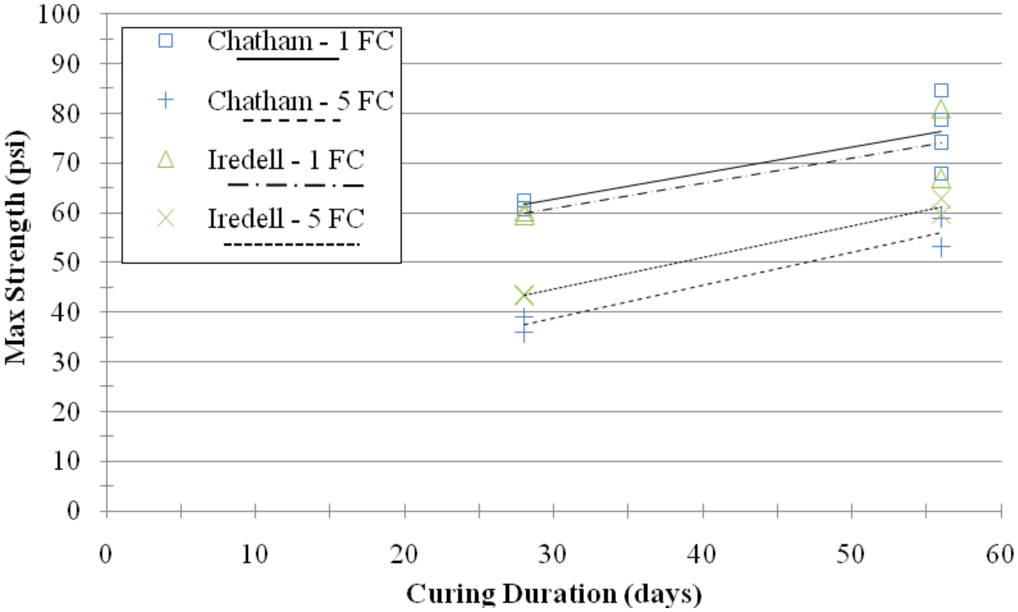

Figure 1-50: UC Strength (Long-Term) vs. Freeze-Thaw Cycles

Figure 1-51 illustrates the effect of a "spring thaw" on lime-soil mixtures. All samples were cured at 35ºF for an initial 28 days followed by an additional 28 or 56 days of curing at 70ºF.

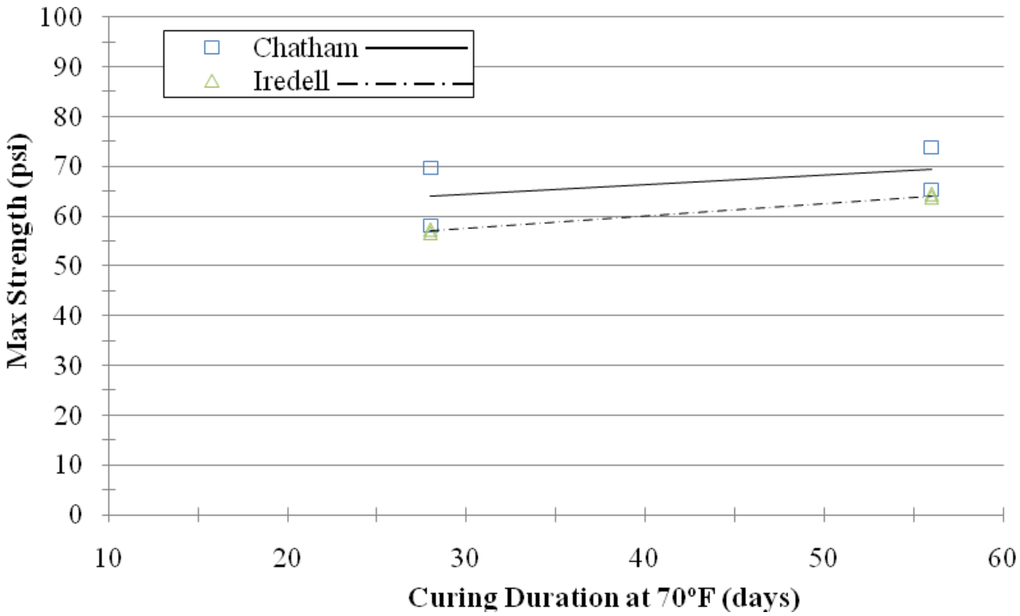

Figure 1-51: UC Strengths (Long-Term) vs. "Spring Thaw"*

*All samples in Figure 1-51 were exposed to an initial 28 days at 35ºF followed by curing at 70ºF. Only the curing duration at 70ºF is shown.

1.4.4.4 Moduli

From unconfined compression testing, secant modulus values were determined. For this chapter, the secant modulus was defined by the slope between the initial point of zero stress and strain to the failure point of max stress and its corresponding strain at failure. The values are summarized in Table 1-10.

Table 1-10: Secant Modulus Results

Curing Temperature	Curing Duration (days)	Secant Modulus (psi)		
		Chatham	Cherokee	Iredell
70ºF	1	244	771	230
70ºF	3	466	846	568
70ºF	7	870	878	815
70ºF	14	-----	-----	-----
70ºF	28	1,781	654	1,969
70ºF (1FC)	28	1,517	-----	1,612
70ºF (5FC)	28	814	-----	774
70ºF	56	2,952	859	2,682
70ºF (1FC)	56	2,126	-----	1,958
70ºF (5FC)	56	1,233	-----	1,704
70ºF*	56	1,736	-----	1,857

Curing Temperature	**Curing Duration (days)**	**Secant Modulus (psi)**		
		Chatham	**Cherokee**	**Iredell**
70ºF**	63	1,386	-----	-----
70ºF*	84	2,063	-----	1,890
70ºF	300	-----	-----	-----
50ºF	1	403	833	584
50ºF	3	372	748	-----
50ºF***	3	853	568	603
50ºF	7	978	843	715
50ºF***	7	1,228	868	672
50ºF	28	-----	843	-----
35ºF	1	569	807	641
35ºF	3	424	752	767
35ºF	7	693	771	955
35ºF	28	725	438	824
35ºF	56	869	510	1,149
25ºF	1	490	1,149	603
25ºF	3	373	1,009	898
25ºF	7	616	1,271	725
25ºF	28	-----	455	-----

*exposed to initial 28 days of 35ºF curing followed by curing at 70ºF until total curing duration

**exposed to initial 35 days of 35ºF curing followed by curing at 70ºF until total curing duration

***exposed to 12 hour freeze period

1.5 Discussion

1.5.1 Soil Characterization

Based on soil characterization testing, all three soils had greater than 50% fines. However, plasticity indices were variable. While Chatham and Iredell soils had plasticity indices of 18% and 16%, Cherokee soils had a plasticity index of only 8%. Chatham soils were the only soil type to classify under USCS as clay, while Iredell and Cherokee soils both classified as silts. From this data, Chatham soil would probably be the ideal candidate for lime stabilization, followed by Iredell soils, and then Cherokee soils. Cherokee soils displayed little reactivity and would not generally be stabilized with lime. This issue will also be discussed in later sections in relation to unconfined compression testing.

1.5.2 Physicochemical Testing

Physicochemical test results were analyzed in regards to curing temperature and curing duration. Initial testing for pH and conductivity was investigated with regards to both lime content and curing temperature. Additional pH and conductivity testing was investigated with regards to lime content and curing temperature as well as curing duration. Curing duration pH and conductivity testing were evaluated using two different sample preparation methods, which were described in the “Methods and Materials” section.

1.5.2.1 pH

Based on pH test results for a specific soil at a specific temperature, the pH of a soil, lime, and water mixture tends to increase drastically until a pH of 12.4 is achieved. Thereafter, the pH may increase somewhat, but generally levels off to a maximum pH. The average corrected pH value of soil alone was 5.71 for Chatham, 5.89 for Cherokee, and 5.51 for Iredell. In some cases, the maximum lime content may not have necessarily resulted in the highest pH.
One of the most significant conclusions from testing for pH against temperature is the effect that temperature has on the minimum lime content for a soil. From the figures displaying pH against lime content based on varying, the minimum lime content at each temperature can be determined. These results are summarized in Table 1-11 below.

Table 1-11: Minimum Lime Content Based on Temperature

Temperature	Chatham	Cherokee	Iredell
115ºF	1.00%	0.70%	1.70%
70ºF	1.80%	1.00%	1.90%
50ºF	1.80%	1.10%	2.00%
35ºF	3.90%	5.40%	3.80%

With pH and temperature correction, the minimum lime content for a soil increases with decreasing temperature. This theory is contrasting from that made by Rogers and Glendinning (2000) which stated that minimum lime content increases with increasing temperature. The basis for their assertion is not clear. The above trend in Table 1-11 seems logical due to the fact that decreased temperatures result in decreased unconfined compressive strengths, which would result in a need for more lime content to achieve stabilization and increase strength. This trend is also supported by the general Arrhenius equation from chemistry, which states that

$$k_2 = k_1\theta^{(T_2-T_1)}$$

Where k is a reaction rate constant dependent upon temperature (Sawyer et al. 1994). Based on this equation, a higher temperature in T_2 would yield a higher k_2 reaction rate constant resulting in a more rapid reaction, and the opposite is true for colder temperatures. Therefore, higher temperatures would have a higher reaction rate constant, which in turn, would need less lime content to achieve a pH of 12.4. From these results, the initial NCDOT lime contents, which were 4% lime (by weight) with Chatham and Iredell soils and 6% lime (by weight) with Cherokee soils, were appropriate lime dosages to provide sufficient strength.

Overall, using the Eades and Grim method of samples preparation and curing resulted in decreases in pH over the 28-day curing period except Cherokee at 35ºF and Iredell at 50ºF. For all soil types cured at 70ºF, the 28-day cure resulted in the lowest pH throughout curing, which would demonstrate the greatest amount of reactivity between soil and lime. Cherokee soil cured at 70ºF had the greatest pH drop between readings of 14 and 28 days. Those soil-lime mixtures cured at 35ºF tended to remain relatively constant throughout curing, while 50ºF samples decreased somewhat, and 70ºF samples decreased the most throughout curing.

Using the alternative method of sample preparation and curing resulted in significant pH drops around 3 days of curing for all three soil types at 70ºF and 35ºF. Similar to the Eades and Grim method, soil-lime mixtures cured at 35ºF tended to remain relatively constant throughout curing, while 50ºF samples decreased somewhat, and 70ºF samples decreased the most throughout curing. Compared to the Eades and Grim method, the alternative method resulted in the lowest pH values in Iredell soils, followed by Chatham soils. While generally displaying the lowest reactivity, Cherokee soils displayed higher pH values in response to the Eades and Grim method as compared to the alternative method. The results from this pH testing using the alternative method were also similar to that of Rao and Shivananda (2005). Although pH readings were taken after approximately one year of curing in their research, the maximum duration for this testing was 28 days. Over this 28 day duration, similar pH drops were observed with respect to curing duration.

1.5.2.2 Conductivity

Overall, using the Eades and Grim method of samples preparation and curing resulted in decreases in conductivity over the 28-day curing period for all soil types at all curing temperatures. Chatham and Cherokee county soils resulted in sharp decreases in conductivity at 6 hours, followed by a sharp increase, and gradual decreasing in conductivity until 28 days of curing. Cherokee county soils exhibited the greatest conductivity decrease between the 14 and 28 day readings. However, Iredell soils increased between these two readings. Overall, Iredell soils still exhibit a decrease in conductivity over the entire 28-day period.

Similar to pH results, using the alternative method of sample preparation and curing resulted in sharp conductivity drops around 3 days of curing for all three soil types. Another similarity with pH results is the relatively constant conductivity for 35ºF samples, followed by some decrease in the 50ºF samples, and the greatest decrease in 70ºF samples. This is especially true for Chatham and Iredell counties, which exhibit the greatest conductivity decrease over the 28-day curing period. Cherokee exhibits the smallest decrease with the trendlines for each temperature paralleling each other. This alternative also compares to that work of Rao and Shivananda (2005), which saw similar decreases of 2mS/cm in conductivity over 28 days curing periods.

1.5.3 Unconfined Compression (UC) Tests

1.5.3.1 Effects of Curing Temperature

In general, unconfined compression testing results maintained fidelity to trends of increasing strength with increasing curing temperature and increasing curing duration. Average unconfined compressive strength values are summarized in Table 1-12 below for each curing condition and each soil type.

Table 1-12: Average UC Strengths for Each Curing Condition

Curing Temperature	Curing Duration (days)	Chatham	Cherokee	Iredell
		Average Unconfined Compressive Strength (psi)		
70ºF	1	21.8	31.9	22.5
70ºF	3	36.6	36.8	33.3
70ºF	7	40.7	37.4	43.8
70ºF	14	-----	-----	45.8
70ºF	28	67.0	35.0	71.5
70ºF (1FC)	28	61.6	-----	59.8
70ºF (5FC)	28	37.4	-----	43.4
70ºF	56	93.0	36.3	78.6
70ºF (1FC)	56	76.3	-----	74.0
70ºF (5FC)	56	56.0	-----	61.2
70ºF*	56	63.9	-----	57.0
70ºF**	63	51.7	-----	-----
70ºF*	84	69.5	-----	64.1
70ºF	300	-----	-----	107.3
50ºF	1	30.5	29.3	30.8
50ºF	3	30.2	30.1	32.8
50ºF***	3	33.3	26.0	30.0
50ºF	7	45.7	31.9	33.2
50ºF***	7	42.4	31.5	31.3
50ºF	28	-----	32.4	-----
35ºF	1	36.0	30.6	31.9
35ºF	3	32.8	28.9	35.7
35ºF	7	37.2	29.2	39.3
35ºF	28	36.6	23.5	37.2
35ºF	56	40.7	25.6	43.9
25ºF	1	28.9	36.8	32.6
25ºF	3	26.8	34.8	40.5
25ºF	7	32.1	36.4	36.0
25ºF	28	-----	24.0	-----

*exposed to initial 28 days of 35ºF curing followed by curing at 70ºF until total curing duration
**exposed to initial 35 days of 35ºF curing followed by curing at 70ºF until total curing duration
***exposed to 12 hour freeze period

To evaluate the effects of temperature on unconfined compressive strength of soil-lime mixtures, all 1, 3, 7, 28, and 56-day samples were compared in terms of strengths based on the curing temperature. Samples were compared based on both short-term curing durations and long-term curing durations. Table 1-13 below illustrates the percent increase from the 25ºF curing strength.

Table 1-13: Temperature Effects on Short-Term UC Strengths

Curing Temperature	Curing Duration (days)	Chatham	Cherokee	Iredell	Chatham	Cherokee	Iredell
		Average Unconfined Compressive Strength (psi)			Percent Change from 25°F Strength		
25ºF	1	28.9	36.8	32.6			
35ºF	1	36.0	30.6	31.9	24.4%	-16.8%	-2.1%
50ºF	1	30.5	29.3	30.8	5.3%	-20.3%	-5.5%
70ºF	1	21.8	31.9	22.5	-24.5%	-13.2%	-31.0%
25ºF	3	26.8	34.8	40.5			
35ºF	3	32.8	28.9	35.7	22.7%	-17.1%	-11.9%
50ºF	3	30.2	30.1	32.8	12.8%	-13.7%	-19.1%
70ºF	3	36.6	36.8	33.3	36.9%	5.8%	-17.7%
25ºF	7	32.1	36.4	36.0			
35ºF	7	37.2	29.2	39.3	16.1%	-19.7%	9.1%
50ºF	7	45.7	31.9	33.2	42.4%	-12.3%	-7.9%
70ºF	7	40.7	37.4	43.8	27.0%	2.9%	21.5%

Based on the 1-day strengths, the only increase in strength from the 25ºF samples occurs in Chatham soils at curing temperatures of 35ºF and 50ºF. Similarly, Chatham soils are the only samples that increase in strength at all temperatures under 3 and 7-day curing periods. The only short-term strength increases in Cherokee occur at a curing temperature of 70ºF, while the only increases in Iredell occur under a 7-day cure. Overall, there is no consistent trend of unconfined compressive strength increasing with increasing temperature based on short-term curing durations. Some of this may have been attributed to the fact that samples exposed to 25ºF were removed from their chambers after the designated curing duration. Due to freezing of moisture within the specimen, these samples were thawed. Some research was done to find the ideal thaw time for a sample. However, if the optimum thaw time was not correct, then the 25ºF samples could have resulted in strength gains from either the remaining frozen effects within the specimen or additional curing times at 70ºF. Generally, from the data presented it can be concluded that no significant pozzolanic reaction occurs within the first 7 days of curing.

After developing no significant trend with those samples cured for short-term duration, the long-term duration specimen were evaluated based on temperature effects. Table 1-14 below displays both average unconfined compressive strengths based on specific curing conditions as well as percent increase between curing temperatures.

Table 1-14: Temperature Effects on Long-Term UC Strengths

Curing Temperature	Curing Duration (days)	Chatham	Cherokee	Iredell	Chatham	Cherokee	Iredell
		Average Unconfined Compressive Strength (psi)			Percent Change from 35°F Strength		
35ºF	7	37.2	29.2	39.3			
70ºF	7	40.7	37.4	43.8	9.4%	28.0%	11.3%
35ºF	28	36.6	23.5	37.2			
70ºF	28	67.0	35.0	71.5	82.8%	48.6%	92.3%
35ºF	56	40.7	25.6	43.9			
70ºF	56	93.0	36.3	78.6	128.7%	41.6%	79.0%

Temperature effects on long-term duration samples are more pronounced. Both Chatham and Iredell soils almost doubled in strength from 35ºF to 70ºF at 28 days of curing. Likewise, 56 days of curing resulted in a greater percent increase for Chatham soils and a significant increase for Iredell soils. It appears that Cherokee soils do have a greater percent increase with longer curing durations, but the long-term strengths themselves are not higher when compared to 7-day curing strengths. To compare short-term and long-term curing durations, 7-day samples cured at 35ºF and 70ºF were included. It can be seen that those samples developed little strength gain under 7 days of curing. This again can be contributed to the fact that little pozzolanic reaction takes place during these initial stages of curing. These 7-day strengths at 70ºF are also comparable to that of Lockett and Moore (1981), which consisted on fine-grained soils mixed with 6% lime resulting in approximately 35 to 45 psi. The 28-day strengths at 70ºF are comparable to that of Lockett and Moore (1981), Tuncer and Basma (1991), Geiman (2005), and Thompson (1967), which were clays, many of which were A-7-5 soils like Iredell, mixed with 3 to 6% lime and resulted in strengths from 70 to 71 psi. The 56-day strengths at 70ºF are comparable to Thompson's work where A-7-6 soils, like Chatham, were mixed with 3 and 5% lime resulting in strength around 100 psi. Overall, long-term curing durations show a more distinct trend of strength increasing with increasing temperature based upon those specimen tested.

Unconfined compressive strengths were also compared based on curing duration. Table 1-15 below demonstrates percent increase in strength from 1-day strengths at a specific curing temperature.

Based on these average strengths, the only temperature that follows the trend of increasing strength with increased curing duration is 70ºF. Almost all samples cured at 50ºF resulted in an increase in strength except for those samples cured for 3 days. However, in comparing the two temperatures, it can be seen that those samples cured at 70ºF gained considerably more strength with respect to curing duration than those samples cured at 50ºF. Cherokee soils had no additional strength gain with respect to curing duration under colder curing temperatures, while Iredell soil developed additional strength under colder curing temperatures. Chatham soils did not develop additional strength with 3 days of curing, but there was a strength gain with 7 days of curing.

Table 1-15: Duration Effects of Short-Term UC Strengths

Curing Temperature	Curing Duration (days)	Chatham	Cherokee	Iredell	Chatham	Cherokee	Iredell
		Average Unconfined Compressive Strength (psi)			Percent Change from 1-Day Strength		
70ºF	1	21.9	31.9	22.5			
70ºF	3	36.6	36.8	33.3	67.7%	15.4%	48. 5%
70ºF	7	40.7	37.4	43.8	86.4%	17.2%	95.0%
50ºF	1	30.5	29.3	30.8			
50ºF	3	30.2	30.1	32.8	-0.9%	2.5%	6.6%
50ºF	7	45.7	31.9	33.2	50.0%	8.7%	7.9%
35ºF	1	36.0	30.6	31.9			
35ºF	3	32.8	28.9	35.7	-8.8%	-5.7%	12.0%
35ºF	7	37.2	29.2	39.3	3.4%	-4.5%	23.4%
25ºF	1	28.9	36.8	32.6			
25ºF	3	26.8	34.8	40.5	-7.5%	-5.4%	24.4%
25ºF	7	32.1	36.4	36.0	10.9%	-1.1%	10.7%

Unconfined compressive strengths were also compared in terms of long-term curing durations. Table 1-16 below exhibits strength development with percent increase from a short-term curing duration of 7 days to more long-term curing durations.

Table 1-16: Duration Effects on Long-Term UC Strengths

Curing Temperature	Curing Duration (days)	Chatham	Cherokee	Iredell	Chatham	Cherokee	Iredell
		Average Unconfined Compressive Strength (psi)			Percent Change from 7-Day Strength		
70ºF	7	40.7	37.4	43.8			
70ºF	28	67.0	35.0	71.5	64.5%	-6.5%	63.3%
70ºF	56	93.0	36.3	78.6	128.3%	-3.1%	79.5%
35ºF	7	37.2	29.2	39.3			
35ºF	28	36.6	23.5	37.2	-1.6%	-19.4%	-5.5%
35ºF	56	40.7	25.6	43.9	9.2%	-12.3%	11.6%

Similar strength gains are developed during long-term curing at 70ºF for both Chatham and Iredell soils. However, Cherokee soils appear to have developed its ultimate strength under 7 days of curing. Similarly, Cherokee soils further reduced in strength at 35ºF. Chatham and Iredell soils also saw strength reductions at 28 days of curing. There was some strength development after 56 days of curing. However, the strength gain at 70ºF when compared to 35ºF was almost 7 times more for Iredell soils and 14 times more for Chatham soils.
Unconfined compressive strengths were compared in terms of both short-term and long-term curing durations. Table 1-17 below demonstrates these total strength gains.

Table 1-17: Duration Effects on Short and Long-Term UC Strengths

Curing Temperature	Curing Duration (days)	Chatham	Cherokee	Iredell	Chatham	Cherokee	Iredell
		Average Unconfined Compressive Strength (psi)			Percent Change from 1-Day Strength		
70ºF	1	21.9	31.9	22.5			
70ºF	3	36.6	36.8	33.3	67.7%	15.4%	48.5%
70ºF	7	40.7	37.4	43.8	86.4%	17.2%	95.0%
70ºF	28	67.0	35.0	71.5	206.6%	9.6%	218.3%
70ºF	56	93.0	36.3	78.6	325.6%	13.6%	249.9%
35ºF	1	36.0	30.6	31.8			
35ºF	3	32.8	28.9	35.7	-8.8%	-5.7%	12.0%
35ºF	7	37.2	29.2	39.3	3.4%	-4.5%	23.4%
35ºF	28	36.6	23.5	37.2	1.8%	-23.1%	16.6%
35ºF	56	40.7	25.6	43.9	12.9%	-16.3%	37.8%

Chatham and Iredell soils see similar increases in strength under curing at 70°F with some increase prior to 7 days. Sometime soon after 7 days, the initial 1-day strength is doubled. At 28 days, initial strengths were tripled. Strength continued to increase up to 56 days with the initial 1-day strength increasing by more than 4 times in Chatham soils and 3.5 times in Iredell soils. However, Cherokee soils never increased by more than 20% from the initial 1-day strength. Chatham and Iredell soils also resulted in similar strengths increased under curing at 35°F. Both saw increases from the initial 1-day strength with the exception of Chatham samples cured for 3 days. However, these increases were nowhere near as much at the strength development that took place under curing at 70°F. Iredell seemed to be the most resistant to colder temperatures, while Cherokee had no strength development under curing at 35°F.

1.5.3.2 Effects of Freeze-Thaw

With the significant hindrance of strength development at cold temperatures, the effects of freeze-thaw cycles were investigated. Table 1-18 below illustrates the percent decrease from samples cured with exposure to no freeze-thaw cycles.

Table 1-18: Freeze-Thaw Effects on UC Strengths

Curing Temperature	Curing Duration (days)	Chatham	Iredell	Chatham	Iredell
		Average Unconfined Compressive Strength (psi)		Percent Change from Zero Freeze-Cycle Strength	
70°F	28	67.0	71.5		
70°F (1FC)	28	61.6	59.8	-8.0%	-16.3%
70°F (5FC)	28	37.5	43.4	-44.1%	-39.3%
70°F	56	93.0	78.6		
70°F (1FC)	56	76.3	74.0	-17.9%	-5.8%
70°F (5FC)	56	56.0	59.7	-39.8%	-24.1%

From the freeze-thaw results, it can be seen that both 28 and 56-day cures have similar results with respect to freeze-thaw cycles. Exposure to one freeze-thaw cycle reduced strength by an average of just over 10%. Exposure to five freeze-thaw cycles reduced strength by approximately 40% under all curing conditions except Iredell soils cured for 56 days.
To better analyze the strength reduction from exposure to freeze-thaw cycles, the amount of strength decrease per freeze-thaw cycle was calculated. These results can be seen in Table 1-19.

Table 1-19: Strength Decrease Per Freeze-Thaw Cycle

Curing Temperature	Curing Duration (days)	Chatham	Iredell	Chatham	Iredell
		Average Unconfined Compressive Strength (psi)		Strength Decrease per Freeze-Thaw Cycle (psi)	
70ºF	28	67.0	71.5		
70ºF (1FC)	28	61.6	59.8	5.4	11.7
70ºF (5FC)	28	37.5	43.4	5.9	5.6
70ºF	56	93.0	78.6		
70ºF (1FC)	56	76.3	74.0	16.6	4.6
70ºF (5FC)	56	56.0	59.7	7.4	3.8

Overall, exposure to five freeze-thaw cycles resulted in an average of about 5 psi reduction in strength per freeze-thaw cycle for both 28 and 56 days of curing. Samples exposed to one freeze-thaw cycle of 12 hours resulted in an average of 8 to 10 psi reduction in strength per freeze-thaw cycle. However, more testing should be done to verify this effect.
Freeze-thaw cycle specimens were also compared in terms of curing duration. Table 1-20 shows the effects of added curing duration on those samples exposed to freeze-thaw cycles.

Table 1-20: Duration Effects on Freeze-Thaw UC Strengths

Curing Temperature	Curing Duration (days)	Chatham	Iredell	Chatham	Iredell
		Average Unconfined Compressive Strength (psi)		Percent Change from 28-Day Strength	
70ºF	28	67.0	71.5		
70ºF	56	93.0	78.6	38.8%	9.9%
70ºF (1FC)	28	61.6	59.8		
70ºF (1FC)	56	76.3	74.0	23.9%	23.7%
70ºF (5FC)	28	37.5	43.4		
70ºF (5FC)	56	56.0	59.7	49.6%	37.6%

Overall, Chatham and Iredell soils had similar strengths after exposure to one freeze-thaw cycle. However, Iredell soils had slightly higher strengths after exposure to five freeze-thaw cycles. From looking at these average unconfined compressive strength graphs with freeze-thaw cycles, it appears as if those samples exposed to both one and five freeze-thaw cycles continued to gain strength at similar rates despite the exposure to harsh conditions. Lime-soil mixtures exposed to one freeze-thaw cycle of 12 hours increased in strength by approximately 24% with an additional 28 days of curing at 70ºF, while those samples exposed to five freeze-thaw cycles of 24 hours increased in strength by an average of 45% with an additional 28 days of curing at 70ºF. Due to the fact that one freeze-thaw cycle had no significant effect on strength, the additional 28 days of curing was not as beneficial to these samples as it was to those samples exposed to five freeze-thaw cycles. When compared to those samples exposed to no freeze-thaw cycles, Iredell soils tend to gain more strength after being exposed to freeze-thaw cycles. Chatham soils had less strength gain in those samples exposed to one freeze-thaw cycle. However, those samples exposed to five freeze-thaw cycles had a greater percentage increase and almost as much strength gain. Looking at average strengths, it appears is if those samples exposed to one freeze-thaw cycle at 28 days of curing have similar strengths to samples exposed to five freeze-thaw cycles at 56 days of curing.

Samples exposed to a "spring thaw" were also evaluated in a similar fashion. Table 1-21 displays average unconfined compressive strengths as well as percent change in strength between curing conditions.

From this data, an additional 28 days of curing at 70°F results in less strength increase for those samples exposed to an initial 28 days of curing at 35ºF, which means that the initial 35°F cure inhibited additional strength gains under 70°F. When looking at those samples cured for 28 and 56 days at 70ºF, an additional 28 days of curing at 35ºF results in approximately 20% decrease in strength. Looking at the initial 28-day cure at 35ºF, an additional 28 and 56 days of curing at 70ºF results in significant strength gains. Lastly, those samples exposed to a total curing duration of 56 days at 70°F had strengths approximately 40% higher than those samples that had a total curing duration of 56 days with 28 days at 35°F and 28 days at 70°F.

Table 1-21: "Spring Thaw" Effects on UC Strengths

Curing Temperature	Curing Duration (days)	Chatham	Iredell	Chatham	Iredell
		Average Unconfined Compressive Strength (psi)		Percent Change	
35ºF/70ºF	28/28	63.9	57.0		
35ºF/70ºF	28/56	69.5	64.1	8.8%	12.4%
70ºF	28	67.0	71.5		
70ºF	56	93.0	78.6	38.8%	9.9%
70ºF	28	67.0	71.5		
35ºF/70ºF	28/28	63.9	57.0	-4.6%	-20.3%
70ºF	56	93.0	78.6		
35ºF/70ºF	28/56	69.5	64.1	-25.2%	-18.5%
35ºF	28	36.6	37.2		
35ºF/70ºF	28/28	63.9	57.0	74.4%	53.4%
35ºF/70ºF	28/56	69.5	64.1	89.8%	72.4%
35ºF/70ºF	28/28	63.9	57.0		
70ºF	56	93.0	78.6	45.5%	37.9%

1.5.3.3 Moduli

Throughout unconfined compression testing, deformation of the soil-lime specimen was monitored. With a known initial height of the specimen, the strain experienced by the sample could be determined. From these stress-strain graphs, the strain on the specimen at failure could be determined. From the replicates at each curing condition, an average maximum stress and an average strain at failure was determined for each curing condition. Figures 1-52 through 1-54 illustrate this relationship between stress and strain at failure.

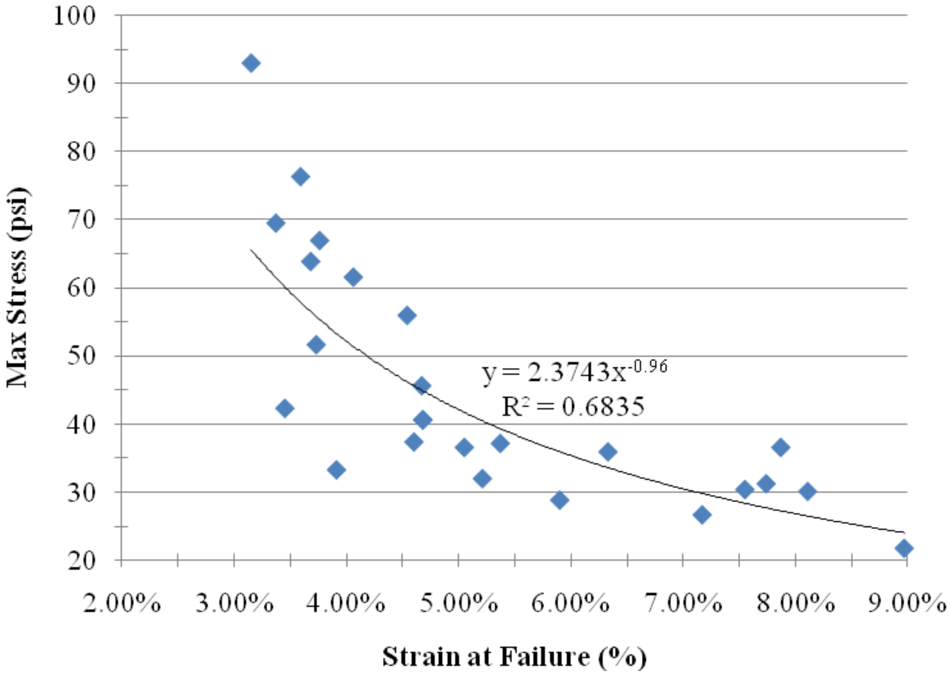

Figure 1-52: Chatham – Stress-Strain at Failure

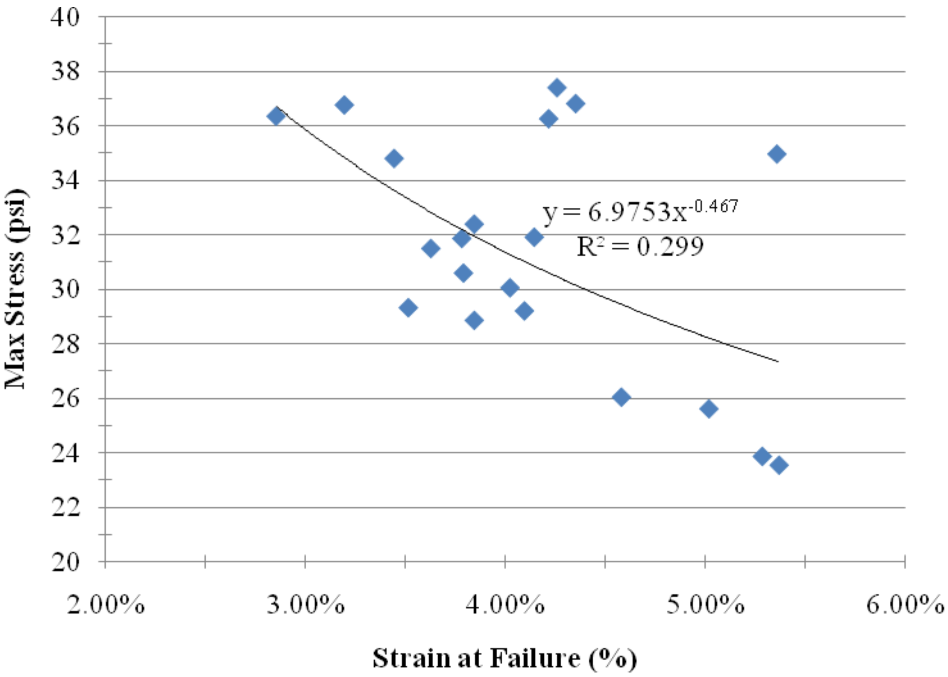

Figure 1-53: Cherokee – Stress-Strain at Failure

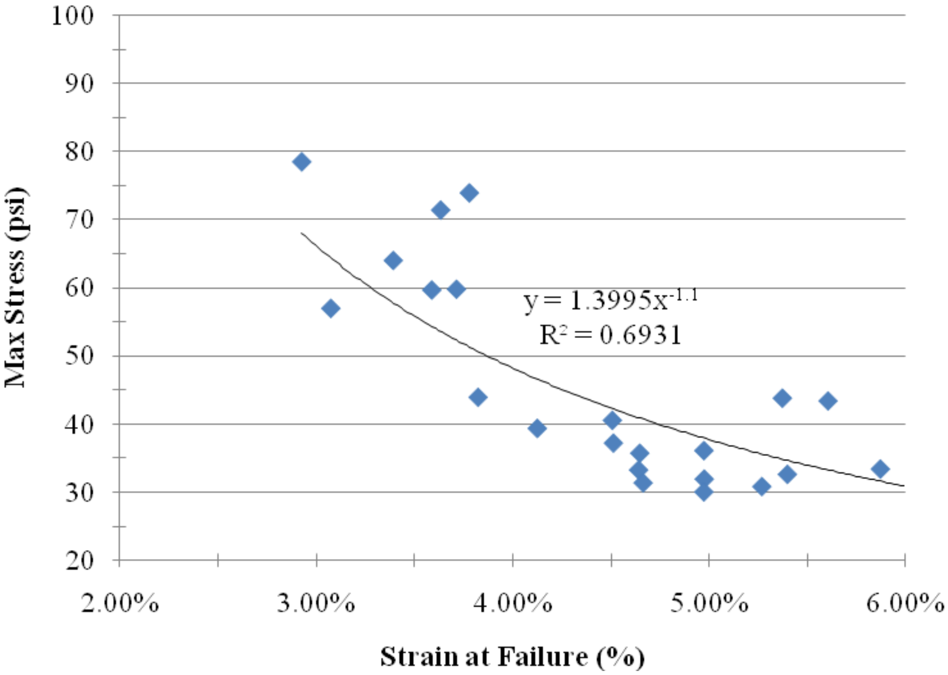

Figure 1-54: Iredell – Stress-Strain at Failure

Chatham and Iredell results show similar trends where those curing conditions resulting in the highest maximum unconfined compressive strengths yielded the smallest strains at failure. The opposite was also valid where lower unconfined compressive strengths yielded larger strains at failure. With both results, it appears as if those samples that failed before 4% strain had unconfined compressive strengths greater than 50 psi, and those samples that failed after 4% strain had unconfined compressive strengths less than 50 psi. This trend seems logical because the amount of strain that a sample can withstand is reflective of its stiffness. Therefore, those samples with higher unconfined compressive strengths that failed at lower strains were stiffer, and had a greater modulus, making them better candidates as subgrade for pavement.

1.6 Conclusions and Recommendations

- Testing for pH and conductivity revealed that there is an increase in both pH and conductivity with increasing temperature and increasing lime content.
- The minimum lime content to achieve a pH of 12.4 decreases with increasing curing temperature.
- Testing for pH and conductivity also revealed that there is a decrease in both pH and conductivity with increasing curing duration for both the Eades and Grim method as well as an alternative method. However, this pH reduction is greatest at 70°F, followed by 50°F, and then 35°F revealing that little to no reactivity between soil and lime occurs over this 28 day curing duration at 35°F.
- Based on unconfined compression testing, there is little to no trend with curing temperature in the first 7 days of curing. However, long-term curing durations of 28 and 56 days illustrate a more distinct trend of increasing strength with increasing curing temperature.
- Under 28 and 56 days of curing, 70°F strengths were approximately twice as much as 35°F strengths for Chatham and Iredell soils.
- Under 70°F curing, 56-day strengths were approximately twice as much as 7-day strengths for Chatham and Iredell soils. However, under 35°F curing, this additional 49 days of curing increased the 7-day strength by approximately 10%. This indicates that prolonged temperatures as low as 35°F significantly hinder strength development over time.
- Increased exposure to freeze-thaw cycles resulted in decreased unconfined compressive strength. Those samples exposed to five freeze-thaw cycles of 24-hour freeze periods resulted in a reduction of about 5 psi per freeze-thaw cycle from those samples not exposed to freeze-thaw cycles.
- All of those samples exposed to freeze-thaw cycles continued to gain strength with curing duration even after freeze-thaw exposure, and this strength gain increased at the same slope regardless of number of freeze-thaw cycles or soil type.
- Those samples exposed to an initial 28 days of curing at 35°F under “spring thaw” conditions had significantly lower strengths than those samples cured only at 70°F. It also appeared that this initial cure hindered strength development over time when compared with those samples cured only at 70°F.
- Several Iredell soil-lime mixtures were allowed to cure at 70°F for approximately 300 days resulting in an average unconfined compressive strength that eclipsed the 100 psi mark, and also resulted in an increase of approximately 36% from the 56-day strengths.
- Chatham soils were representative of that class of soils most suitable for lime stabilization, closely followed by Iredell soils. Cherokee soils had no significant strength development even over long-term curing durations making it an ineffective candidate for lime stabilization. This trend follows the plasticity index results, which were 18% for Chatham, 16% for Iredell, and 8% for Cherokee. When looking at soil candidates for lime stabilization, plasticity index is probably a more telling fact than percent fines.
- Lime-soil specimens with higher unconfined compressive strengths typically failed at lower strains, while lime-soil specimens with lower unconfined compressive strengths typically failed at higher strains. For Chatham and Iredell results, it appeared that failure

before 4% strain resulted in unconfined compressive strengths greater than 50 psi while failure after 4% strain resulted in unconfined compressive strengths less than 50 psi.

- Repeatability is difficult with soil-lime mixtures when testing for unconfined compressive strength. There are many variables, such as soil composition, handling, curing conditions, moisture content, and mixing procedure, which can cause deviations in unconfined compressive strengths.

Overall, these results suggest that current specifications may be modified to allow lime stabilization to proceed in cooler temperatures, provided a corresponding increase in curing time and/or thermal protection is provided prior to loading. That said, these results indicate that current specifications are justified and sufficiently conservative to maximize strength development in lime-stabilized subgrades. The nature of the testing plan and variability of soils and conditions encountered by NCDOT precludes greater specificity in terms of soil types, lime dosage or acceptable temperature ranges. However, the software program developed and described in Chapter 3 of this report may be used to inform case by case decisions as to whether cold weather exceptions can be made. This software relates forecasted air temperatures to likely subgrade temperatures.

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CHAPTER 2: Cement-Stabilized Subgrade Soils

Chapter Summary

Cement is perhaps the most common soil stabilizer, often blended with soils at various proportions to increase strength and durability. However, there are concerns with regard to the impact of low curing temperature on strength and ultimate performance. The purpose of this study is to determine the impact curing temperature has on the unconfined compressive (UC) strength and stress-strain behavior of cement modified soil as well as to consider methods to improve performance in cold-weather conditions. The data indicate that curing soil-cement at lower temperatures will result in lower strengths. For example, the 7 day strength for samples cured at 25^{o}F was less than the strength of samples cured at 50^{o}F or 70^{o}F by a factor ranging from 2-6. Likewise, the 7 day strength for samples cured at 35^{o}F was less than the strength of samples cured at 50^{o}F or 70^{o}F by approximately 20-25%. Additionally, on the basis of 15 repeat tests for 3 and 7 day curing periods, for three different soils, results indicate that the mean strength at 3 days is 84-93% of that for 7 days, in support of a potential change in current subgrade evaluation practice predicated on the longer duration.

2.1 Introduction

Soil stabilization is the process of increasing the quality of in-situ soil so as to make it functional as an engineering material. There are numerous ways to stabilize soils, both mechanically and chemically. One type of chemical stabilization includes adding Portland cement and water to soil, a process known as cement stabilization. At the end of this process, which involves various stages of mixing, compacting, and curing, the resulting material is known as "soil-cement." Soil-cement has been used for decades as a material capable of withstanding large compressive stresses. The Portland Cement Association (PCA) first considered the viability of soil-cement after a scientific study launched by the South Carolina State Highway Department in 1932 revealed promising results (M. D. Catton 1959). Since then, soil-cement has been thoroughly investigated and, in turn, standard methods of determining the various engineering properties of the material have been established by the American Society for Testing Materials (ASTM). The PCA has also produced guidelines for dealing with soil-cement in its publication *Soil-Cement Laboratory Handbook* (PCA 1992). Likewise, the North Carolina Department of Transportation (NCDOT) has extensive experience with soil-cement applications with fully developed guidelines in the Standard Specifications for Roads and Structures. Moreover, the Geopavement Section has developed a Chemical Stabilization Technician Certification Program that provides recommendations on virtually all of the details highlighted in the literature review reported herein.

Soil-cement has become a common building material and is repeatedly examined to enhance its feasibility. To that extent, the North Carolina Department of Transportation (NCDOT), was interested in minimizing the limitations of cement stabilization in roadway construction. Primarily, NCDOT was concerned with the temperature limitations of soil-cement. This limitation has to do with the ambient temperature necessary for *hydration*, the chemical reaction involving cement and water. Current NCDOT regulations stipulate that soil-cement base cannot be constructed: 1) between November 1st and March 15th, 2) if the temperature is below 40°F in the shade, or 3) if weather forecasts indicate the temperature will fall below 40°F within 24 hours (North Carolina Department of Transportation 2002). The genesis of this research project came from NCDOT's desire to extend the construction season for soil-cement, thus making projects involving the material more efficient.

Although there are several states in the country that experience colder climates than that of North Carolina, the seasonal temperatures dip low enough to cause a problem when creating soil-cement material. The climate in the state varies from east to west due to the increasing topographic elevation; the average temperature varies more than 20°F between the extreme eastern regions of the state and the extreme western regions during any season (Boyles, Holder and Raman 2004). More importantly, however, the normal mean temperature range of the state during the colder seasons includes sub-freezing temperatures. Analysis of historical data indicates the following normal mean temperature ranges for the months when colder climates are typically experienced, as shown in Table 2-1 (State Climate Office of North Carolina 2008):

Table 2-1: Historical Temperature Range for North Carolina

October:	45°F -70°F
November:	35°F-55°F
December:	25°F -50°F
January:	25°F -45°F
February:	25°F-50°F
March:	30°F -55°F
April:	40°F -65°F

It is clear from this data that temperature becomes an important design parameter when construction occurs during the colder months of the year.

2.2 Literature Review

The scientific beginning for soil-cement in the United States is attributed to the South Carolina State Highway Department (M. D. Catton 1959). In 1932, the organization began a study on soil-cement in search of low-cost methods of roadway construction. Catton (1959) believes this project was the premier project that indicated soil and cement could be combined to produce a valuable engineering material. Following this soil-cement investigation and others of the time period, PCA embarked on a mission to determine testing standards for the material. The tests employed by these researchers were first adopted as standards by ASTM in 1944 and the American Association of State Highway Officials (AASHTO) followed suit in 1945 (PCA 1992). Almost two decades later, after more than 294,000,000 square yards (2.646 trillion square feet) of soil-cement base and subbase had been built in the United States, PCA researcher Earl J. Felt stated “Soil-cement is an accepted material for a structural base for bituminous wearing surfaces and for a subbase for concrete pavements” (Felt 1961). It has since been generally accepted that soil-cement provides: a) uniform, strong support for pavement, b) resistance to consolidation under traffic loading, c) increased load transfer at pavement joints, d) a stable working surface for heavy machinery placing the concrete roadway, and e) the ability to recycle failed flexible pavements in a new soil-cement base (American Concrete Institute 1990). As of 1990, more than 100,000 miles of equivalent 24 ft. wide pavement (12.672 trillion square feet) with underlying soil-cement base had been constructed (American Concrete Institute 1990). While soil-cement material is primarily used as base underlying pavements, other applications for soil-cement include: controlling erosion and water seepage in ditches, lake shores, reservoirs, and earth dams, and the stabilization of dikes and foundations (American Concrete Institute 1990).

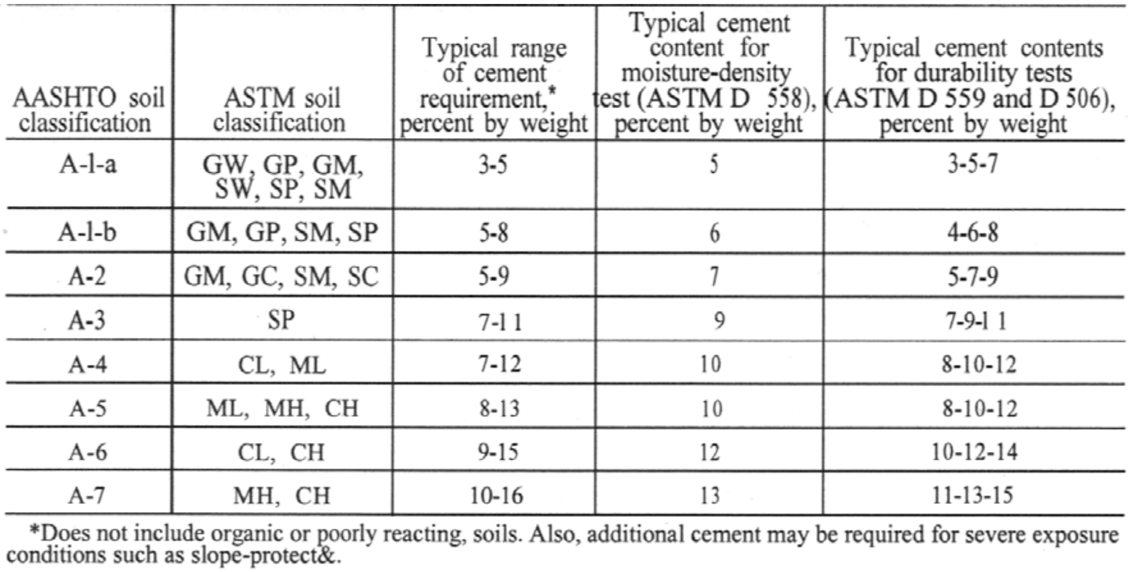
2.2.1 Definition of Soil-Cement

Catton (1959) defines soil-cement as “a tightly compacted mixture of pulverized soil, Portland cement, and water which, as the cement hydrates, forms a hard, durable structural material.” The process of making soil-cement material begins with the pulverization of soil. Catton (1959) explains that soil is considered an aggregate and thus its pulverization is important to ensure a thorough mixture of water and cement can occur. The amount of moisture added to the soil and cement is based on moisture-density tests done on the specific soil being used. The material is finally compacted with the desired effort and allowed to cure into soil-cement.

Similarities can be drawn between soil-cement and concrete in several ways. Both are a mixture of aggregate, water, and cement. The cement reaction in both materials is the same. One of the big differences however, lies in how the soil-cement matrix interacts to attain its finished strength. The cement particles in soil-cement become surrounded by soil particles. After hydration, this creates the soil-cement agglomerates that give the material increased strength. Alternatively, the cement particles in concrete, paired with a sufficient amount of water, coat the fine and coarse aggregate (M. D. Catton 1962).

While most soil types can be stabilized with cement, there are a few exceptions that exist. The American Concrete Institute (ACI) recommends that soil-cement should not be made with organic soils, highly plastic clays, or poorly reacting sandy soils (American Concrete Institute 1990). While the ideal soil is a granular one, by varying the percentage of cement added to the soil, an adequate soil-cement material can be created with varying soil types. Table 2-2 outlines the typical amount of cement to add based on the classification of the soil.

Table 2-2: The typical cement content necessary for making soil-cement based on soil classification (American Concrete Institute, 1990).

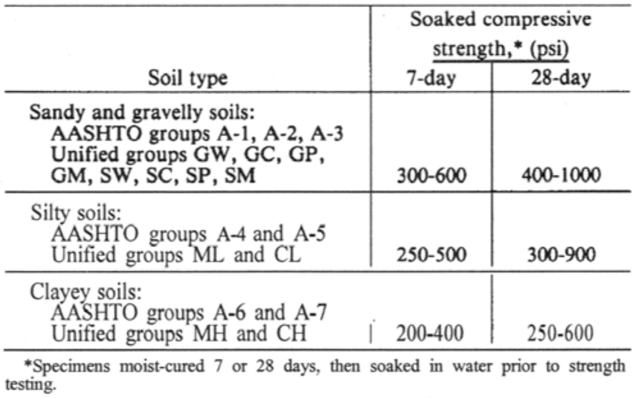


AASHTO soil classification	ASTM soil classification	Typical range of cement requirement,* percent by weight	Typical cement content for moisture-density test (ASTM D 558), percent by weight	Typical cement contents for durability tests (ASTM D 559 and D 506), percent by weight
A-l-a	GW, GP, GM, SW, SP, SM	3-5	5	3-5-7
A-l-b	GM, GP, SM, SP	5-8	6	4-6-8
A-2	GM, GC, SM, SC	5-9	7	5-7-9
A-3	SP	7-1 1	9	7-9-1 1
A-4	CL, ML	7-12	10	8-10-12
A-5	ML, MH, CH	8-13	10	8-10-12
A-6	CL, CH	9-15	12	10-12-14
A-7	MH, CH	10-16	13	11-13-15

*Does not include organic or poorly reacting, soils. Also, additional cement may be required for severe exposure conditions such as slope-protect&.

ACI has also summarized the typical strengths the researcher should encounter when testing hardened soil-cement material. Table 2-3 summarizes the typical UC strengths obtained by soil-cement material for different types of soil and curing periods (American Concrete Institute 1990).

Table 2-3: UC strength for soil-cement with different soil types (American Concrete Institute 1990)



Soil type	Soaked compressive strength,* (psi)	
	7-day	28-day
Sandy and gravelly soils: **AASHTO groups A-1, A-2, A-3** **Unified groups GW, GC, GP, GM, SW, SC, SP, SM**	300-600	400-1000
Silty soils: AASHTO groups A-4 and A-5 Unified groups ML and CL	250-500	300-900
Clayey soils: AASHTO groups A-6 and A-7 Unified groups MH and CH	200-400	250-600

*Specimens moist-cured 7 or 28 days, then soaked in water prior to strength testing.

Felt and Abrams (1957) identified the five factors that affect the structural properties of cement stabilized soil:

1. the physical and chemical composition of the soil
2. the amount of cement added to the mixture
3. the amount of water added to the mixture just prior to compaction
4. the density of the mixture after compaction
5. the age of the material and the conditions under which the material was cured

As described in the following paragraphs, most of these factors have been investigated exhaustively. Furthermore, there is no conclusive evidence that any one factor or group of these factors is more important than the others (M. D. Catton 1962). However, it is apparent that there exists a lack of information on the effects that the curing conditions of soil-cement have on the properties of the hardened material (number 5 in the previous list). Particularly, it is unclear what effect the *ambient temperature during the curing stage* has on the quality of hardened soil-cement.

2.2.2 Factors that Affect Soil-Cement

2.2.2.1 Physical and chemical composition of soil

The gradation of the soil to be stabilized has a great impact on the amount of cement used as well as the other properties of the soil-cement mixture. A smaller grain size will increase the optimum moisture content and decrease the maximum density along with requiring additional cement for compacted soil-cement (M. D. Catton 1962).

The types of soil involved in creating soil-cement material have a significant impact on the properties of the finished material (Felt 1955). Early on in the development of soil-cement, engineers suspected that surface chemistry of the soil particles might influence the properties of soil-cement. Catton (1959) stated that while H. F. Winterkorn's pioneering study (Winterkorn, Gibbs and Fohrman 1942) failed to produce a quantitative evaluation, the effort confirmed that surface chemistry did indeed affect the properties of hardened soil-cement. The Michigan State Highway Department took a pedological (soil science) approach by examining the influence of soil formation on the physical properties of soil and how these properties relate to soil-cement (Housel 1937). Catton (1959) cited a report by Hicks (1939) that summarizes the North Carolina State Highway Department's use of a more pedological method of classifying soils to conclude that soils not of the same physical classification but belonging to the same soil series and horizon do react to cement similarly. Additionally, Felt (1961), while stating that gradation of the soil is the most important factor in designing soil-cement, he also highlights the importance of using a pedological classification of the soil because surface chemical properties of the soil have an impact on the reaction with cement. To support his statement, he refers to the example that A-horizon topsoil of many podzolic sandy soils has a relatively ineffective interaction with cement while the C-horizon parent materials from the same soil profile work extremely well with cement. He adds that lateritic clays (red and yellow soils formed in warm, humid climates) work well with cement while northern podzolic and chernozem clayey soils do not. While there are reliable testing and evaluation procedures for soil-cement material, questions remain with respect to understanding the physical chemistry of soil-cement (M. D. Catton 1959).

Lastly, after evaluating several of the initial soil-cement projects in the United States that were in service at that time, Catton (1962) believed that the chemical and physico-chemical reactions of the material were permanent, meaning the material was reliable because these reactions were irreversible.

2.2.2.2 Amount of Cement Added to Mixture

An important distinction exists between soils that are stabilized with different amounts of cement. "Soil-cement" refers to a material that is stabilized with a high percentage of cement. "Cement-modified" material is material that is stabilized with a relatively low percentage of cement that does not harden the material sufficiently to achieve "soil-cement" properties. Of course, the percentage of cement used to create soil-cement is dependent on the physical and chemical composition of the soil (i.e. using AASHTO classification, an A-2 soil may be considered "soil-cement" when stabilized with 5% cement, while an A-6 soil stabilized with 5% would typically be considered "cement-modified" instead). These terms are used with the understanding that this distinction is relative to the specific soil type in question and the resulting physical properties of the finished material (Felt 1955). One area of concern during the early stages of the development of soil-cement was whether cement harmed the moisture-density relationship of the soil once it had been integrated in the mixture. However, it has been suggested, by studying several soil types, that the moisture-density relationship of the original soil is not damaged by the chemical or physico-chemical interaction with unhydrated cement (M. D. Catton 1959).

The effects of the amount of cement added on the overall quality of soil-cement have been investigated thoroughly. Felt (1955) was interested in the effect cement content had on different types of soils (sandy, silty, and clayey). For each soil, he used cement contents (by volume) of 6, 10, 14, 18, 22, 26, and 30 percent. He then tested the compressive strength of the samples after 2, 7, 28, 120, and 365 days of curing. He found that, with a few inconsistencies, as the cement content increased the compressive strength and durability did also.

Felt and Abrams (1957) used four different types of soils (sand, sandy loam, clayey sand, and silt loam) to determine a correlation between cement content and strength of the resulting soil-cement material. With a litany of tests, they determined that an increase in cement content resulted in an increase in modulus of rupture, modulus of elasticity, and compressive strength. In 1959, Melvin S. Abrams published a report detailing his investigation into the strength and durability of two substandard granular soils stabilized with varying amounts of cement. The data in the report show a clear positive relationship between the amount of cement added and compressive strength (Abrams 1959).

Furthermore, Catton (1961) refers to several studies that show the rate and efficiency of cement hydration in soil-cement and the strength of the finished material are positively correlated to the amount of cement added to the material.

To finalize the discussion on the effect cement content has on the compressive strength of hardened soil-cement, Figure 2-1 is a chart that depicts the positive correlation between cement content and 28-day compressive strength. Also, it can be seen in Figure 2-1 that the strength increase is greater for coarse-grained soils than it is for fine-grained soils.

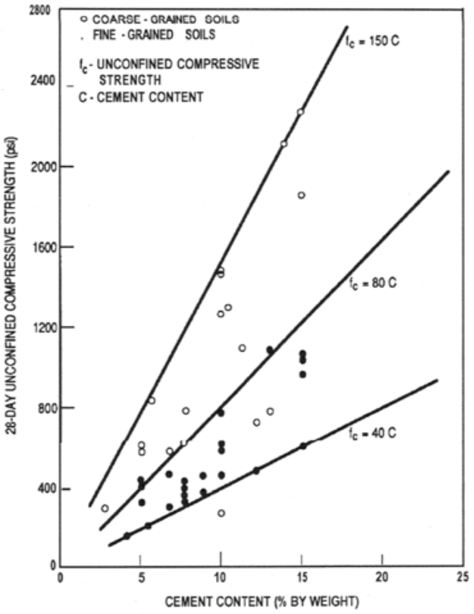


Figure 2-1: Relationship between cement content and UC strength for soil-cement (American Concrete Institute 1990)

By way of reviewing previous NCDOT experience with cement, a sampling of strength data was obtained manually and tabulated on July 12, 2006. These data are given presented as Table 2-4:

Table 2-4: Representative Experience with Regard to Cement Stabilization in North Carolina

Soil Classification	Location	Strength	% Cement Added
A-2-4(0)	Wake	258.5 psi	6
A-2-4(0)	Wake	324.0 psi	8
A-2-4(0)	Wake	340.1 psi	6
A-2-4(0)	Wake	404.2 psi	8
A-2-4(0)	Wake	353.5 psi	6
A-2-4(0)	Wake	437.2 psi	8
A-2-4(0)	Wake	407.5 psi	6
A-2-4(0)	Wake	429.5 psi	8
A-2-4(0)	Mecklenburg	2319.9 kPa	6
A-2-4(0)	Forsyth	281.7 psi	4
A-2-4(0)	Davidson	446.0 psi	6
A-2-4(0)	Davidson	543.4 psi	8
A-2-4(0)	Chatham	318.4 psi	6

Soil Classification	**Location**	**Strength**	**% Cement Added**
A-2-4(0)	Chatham	439.8 psi	8
A-2-4(0)	Wayne	623.1 psi	6
A-2-4(0)	Wayne	721.0 psi	8
A-2-4(0)	Wilson	559.6 psi	6
A-2-4(0)	Wilson	716.1 psi	8
A-2-4(0)	Cabarrus	415.0 psi	6
A-2-4(0)	Cabarrus	441.4 psi	8
A-2-6(0)	Wilson	3098.1 kPa	6
A-2-6(0)	Wilson	3476.8 kPa	8
A-2-6(0)	Wilson	3943.6 kPa	6
A-2-6(0)	Wilson	4730.2 kPa	8
A-2-6(1)	Wilson	3787 kPa	6
A-2-6(1)	Wilson	4186.4 kPa	8
A-4-2(2)	Wake	2059.9 kPa	6
A-4(0)	Wake	2115.3 kPa	6
A-4(0)	Wake	2164.0 kPa	8
A-4(0)	Wake	2591.8 kPa	6
A-4(0)	Wake	2810.3 kPa	8
A-4(0)	Wake	2513.0 kPa	6
A-4(0)	Wake	3165.9 kPa	8
A-4(0)	Wake	2104.4 kPa	6
A-4(0)	Wake	2173.7 kPa	8
A-4(0)	Wake	250.3 psi	6
A-4(0)	Wake	274.2 psi	8
A-4(0)	Chatham	2489.6 kPa	6
A-4(0)	Chatham	2930.5 kPa	8
A-4(0)	Rowan	240.1 psi	6
A-4(0)	Rowan	253.6 psi	8
A-4(0)	Gaston	1959.0 kPa	9
A-4(0)	Durham	275.1 psi	6
A-4(0)	Durham	357.2 psi	8
A-4(0)	Mecklenburg	249.0 psi	6
A-4(0)	Mecklenburg	272.4 psi	8
A-4(1)	Rowan	402 psi	6
A-4(1)	Rowan	436.3 psi	8
A-4(1)	Rowan	256.9 psi	6
A-4(1)	Rowan	314.6 psi	8
A-4(1)	Wake	1557.9 kPa	6
A-4(1)	Wake	2079.0 kPa	8
A-4(1)	Wake	289.6 psi	6
A-4(1)	Wake	342.2 psi	8
A-4(1)	Rowan	264.7 psi	6
A-4(1)	Rowan	321.0 psi	8

Soil Classification	**Location**	**Strength**	**% Cement Added**
A-4(1)	Rowan	276.6 psi	6
A-4(1)	Rowan	289.3 psi	8
A-4(1)	Gaston	3007.8 kPa	9
A-4(1)	Wake	2359.6 kPa	6
A-4(1)	Wake	2520.5 kPa	8
A-4(1)	Mecklenburg	355.4 psi	6
A-4(1)	Mecklenburg	402.4 psi	8
A-4(2)	Rowan	316.1 psi	6
A-4(2)	Rowan	361.3 psi	8
A-4(2)	Rowan	350.5 psi	6
A-4(2)	Rowan	398.4 psi	8
A-4(2)	Wake	1951.8 kPa	6
A-4(2)	Wake	2504.0 kPa	8
A-4(2)	Wake	281.7 psi	6
A-4(2)	Wake	320.0 psi	8
A-4(2)	Rowan	284.6 psi	6
A-4(2)	Rowan	324.6 psi	8
A-4(2)	Cherokee	2097.9 kPa	6
A-4(2)	Cherokee	2355.7 kPa	8
A-4(3)	Stanly	1624.7 kPa	6
A-4(3)	Stanly	1881.2 kPa	8
A-4(3)	Cherokee	1117.2 kPa	6
A-4(3)	Cherokee	1397.5 kPa	8
A-4(3)	Wake	1727.4 kPa	6
A-4(3)	Wake	226.5 psi	6
A-4(3)	Wake	248.7 psi	8
A-4(4)	Cherokee	924.6 kPa	6
A-4(4)	Cherokee	1157.5 kPa	8
A-4(5)	Cherokee	1071.5 kPa	6
A-4(5)	Cherokee	1388.3 kPa	8
A-4(5)	Cherokee	155.4 psi	6
A-4(5)	Cherokee	201.4 psi	8
A-4(6)	Wake	209.8 psi	6
A-4(6)	Wake	230.2 psi	8
A-4(7)	Rowan	224.4 psi	6
A-4(7)	Rowan	239 psi	8
A-5(0)	Gaston	2173.7 kPa	9
A-5(0)	Lincoln	1255.6 kPa	6
A-5(0)	Lincoln	1606.3 kPa	8
A-5(1)	Wake	282.6 psi	6
A-5(1)	Wake	292.1 psi	8
A-5(2)	Lincoln	2180.2 kPa	6
A-5(2)	Lincoln	2365.6 kPa	8

Soil Classification	Location	Strength	% Cement Added
A-5(3)	Wake	254.3 psi	6
A-5(3)	Wake	265.7 psi	8
A-5(4)	Lincoln	1228.8 kPa	6
A-5(4)	Lincoln	1678.6 kPa	8
A-5(5)	Guilford	1807.1 kPa	6
A-5(5)	Guilford	2224.4 kPa	8
A-5(5)	Wake	1356.8 kPa	6
A-5(5)	Wake	2070.8 kPa	8
A-5(5)	Wake	1346.1 kPa	6
A-5(5)	Wake	1558.6 kPa	8
A-5(7)	Lincoln	1264.3 kPa	6
A-5(7)	Lincoln	1504.7 kPa	8
A-5(7)	Wake	443.4 kPa	6
A-5(7)	Wake	1061.5 kPa	8
A-6(1)	Chatham	3004.4 kPa	6
A-6(1)	Chatham	3428.9 kPa	8
A-6(1)	Mecklenburg	2009.7 kPa	6
A-6(1)	Mecklenburg	2159.0 kPa	8
A-6(1)	Richmond	305.3 psi	6
A-6(1)	Richmond	348.4 psi	8
A-6(3)	Wake	3033.4 kPa	6
A-6(3)	Wake	3416.9 kPa	8
A-6(3)	Mecklenburg	459.1 psi	6
A-6(3)	Mecklenburg	505.2 psi	8
A-6(4)	Cabarrus	1598.9 kPa	6
A-6(4)	Cabarrus	2162.8 kPa	8
A-6(4)	Rowan	254.8 psi	6
A-6(4)	Rowan	1573.0 kPa	6
A-6(4)	Rowan	2054.2 kPa	8
A-6(4)	Stanly	1386.1 kPa	6
A-6(4)	Stanly	1483.3 kPa	8
A-6(4)	Wake	2352.7 kPa	6
A-6(4)	Wake	2435.0 kPa	8
A-6(4)	Chatham	389.8 psi	6
A-6(4)	Chatham	449.5 psi	8
A-6(5)	Wake	1825.5 kPa	6
A-6(5)	Wake	2151.6 kPa	8
A-6(6)	Rowan	1941.4 kPa	8
A-6(6)	Rowan	313.9 psi	6
A-6(6)	Rowan	372.3 psi	8
A-6(6)	Rowan	1493.2 kPa	6
A-6(6)	Chatham	2549.3 kPa	6
A-6(6)	Chatham	2866.4 kPa	8

Soil Classification	Location	Strength	% Cement Added
A-6(7)	Rowan	1586.2 kPa	6
A-6(7)	Rowan	2102.9 kPa	8
A-6(9)	Stanly	1636.9 kPa	6
A-6(9)	Stanly	1706.5 kPa	8
A-6(9)	Wake	1878.0 kPa	8
A-6(10)	Stanly	1097.8 kPa	6
A-6(10)	Stanly	1171.9 kPa	8
A-6(12)	Stanly	693.7 kPa	6
A-6(12)	Stanly	744.4 kPa	8
A-6(14)	Lee	169.6 psi	6
A-6(14)	Lee	217.9 psi	8
A-7-6(8)	Rowan	249.1 psi	6
A-7-6(8)	Rowan	291.5 psi	8

As Table 2 demonstrates, NCDOT typically specifies 6% or 8% cement by weight, for soil stabilization.

2.2.2.3 Amount of Water Added Prior to Compaction

The amount of water included in making soil-cement has a large impact on the quality of the hardened material. There needs to be enough water in the mixture to ensure that cement hydration can occur. The question lies in whether the water content specified by the moisture density tests (ASTM D 558) that define the soil-cement mixture's optimum moisture content (OMC) is enough for the reaction. It has been suggested that the amount of water specified by the OMC is indeed enough for the hydration reaction to occur. Felt (1955) stipulates that for high quality mixtures of soil-cement using silty or clayey soils the material must contain *at least* the amount of water indicated by the OMC. However, he also concluded from his tests on sandy soils, that moisture contents at or slightly below the OMC were most desirable (similar to concrete properties). Several others since have confirmed that the OMC of soil-cement material is and adequate amount of moisture for cement hydration (M. D. Catton 1959).

Davidson et al. (1962) found similar results in testing sandy and clayey soils. They determined that most of the sandy soils reached maximum strength with a moisture content at or just below the OMC for the soil-cement mixture. In addition, they found that for the clay soils, maximum strength was reached by including 1 to 2 percent more moisture than the OMC. Lightsey et al. (1970) concluded that the OMC of the soil-cement mixture does not guarantee the maximum strength or durability of the finished material.

It is important that the water used in the mixture is free from significant amounts of alkalies, acids, or organic matter; but it does not necessarily need to be potable. As a matter of fact, seawater has not only been successfully used in the past, but the American Concrete Institute (1990) has reported that the presence of chlorides might lead to increased early strength.

2.2.2.4 Density of mixture after compaction

Felt (1955) determined that when all other factors (soil type, cement content, water content) are held constant, an increase in the density to which the material is compacted will result in increased durability and compressive strength. For the soils he tested, which included sandy, silty, and clayey types, Felt determined that an increase in 1pcf of density led to about 15 to 25 psi additional compressive strength. Shen and Mitchell (1966) also showed that for cohesionless soils with the same cement content, compaction to a higher density lead to a higher compressive strength upon testing. Interestingly, however, is the effect that low temperatures have on the density compacted soil. It has been suggested that the density of the soil is dependent on the temperature of the soil matrix. More specifically, the viscosity of the water increases as its temperature falls and thus the soil particles have a more difficult time moving into a tighter configuration (Lovell and Osborne 1968).

2.2.2.5 Age of Material and Curing Conditions

It is important to understand the conditions under which the soil-cement material cures because the conditions of curing (time, temperature, and moisture content) have significant impact on the strength gain of the material (American Concrete Institute 1990). It has been documented that the time spent mixing the soil, cement, and water prior to compaction has an impact on the strength of the finished soil-cement material. Felt (1955) studied the effect of waiting 2, 4, and 6 hours after mixing the material to compact it. He found that in almost all cases the extended period prior to compaction was detrimental to the quality of soil-cement. He did find that intermittent mixing during the waiting time period was helpful in retaining some durability and strength of the finished material, but these samples still failed to perform as well as immediately compacted soil-cement.

Arman and Saifan (1967) found that after a delay in compaction of at least 2 hours the compressive strength of the hardened soil-cement suffered greatly. The purpose of their study was to determine proper steps for contractors in the field to make when preparing a road subbase with soil-cement. In this study, they found that the durability, compressive strength, and density of the soil-cement mixes were substantially and uniformly lower for at least a 2 hour delay in compaction.

ACI stipulates that soil-cement not be allowed to freeze and has recommendations for contractors on how to insulate the material while it cures. However, ACI gives no guidance as to what temperatures above freezing are safe for the soil-cement.

2.3 Materials and Methods

The procedures followed for this research were developed in consultation with NCDOT (see Appendix 1-1 – presented in Chapter 1) as well as recommendations from AASHTO, ASTM and the open literature.

2.3.1 Materials

The primary material used in this study was soil gathered from the various regions of the state. Three soil samples were collected for testing, with one sample each from the Mountain, Western Piedmont and Eastern Piedmont regions of North Carolina. More specifically, soil collected from Buncombe County represented the Mountain region; soil collected from Guilford County represented the Western Piedmont region; and soil collected from Johnston County represented the Eastern Piedmont region. Each sample consisted of eight 55-gallon drums of soil. Once obtained, a portion of each sample was returned to the Division of Highway Materials and Tests Unit of the NCDOT for classification. Each drum was labeled, securely tightened and closed until needed. The locations of these three soils can be seen in Figure 2-2, while more details are provided in Table 2-5.

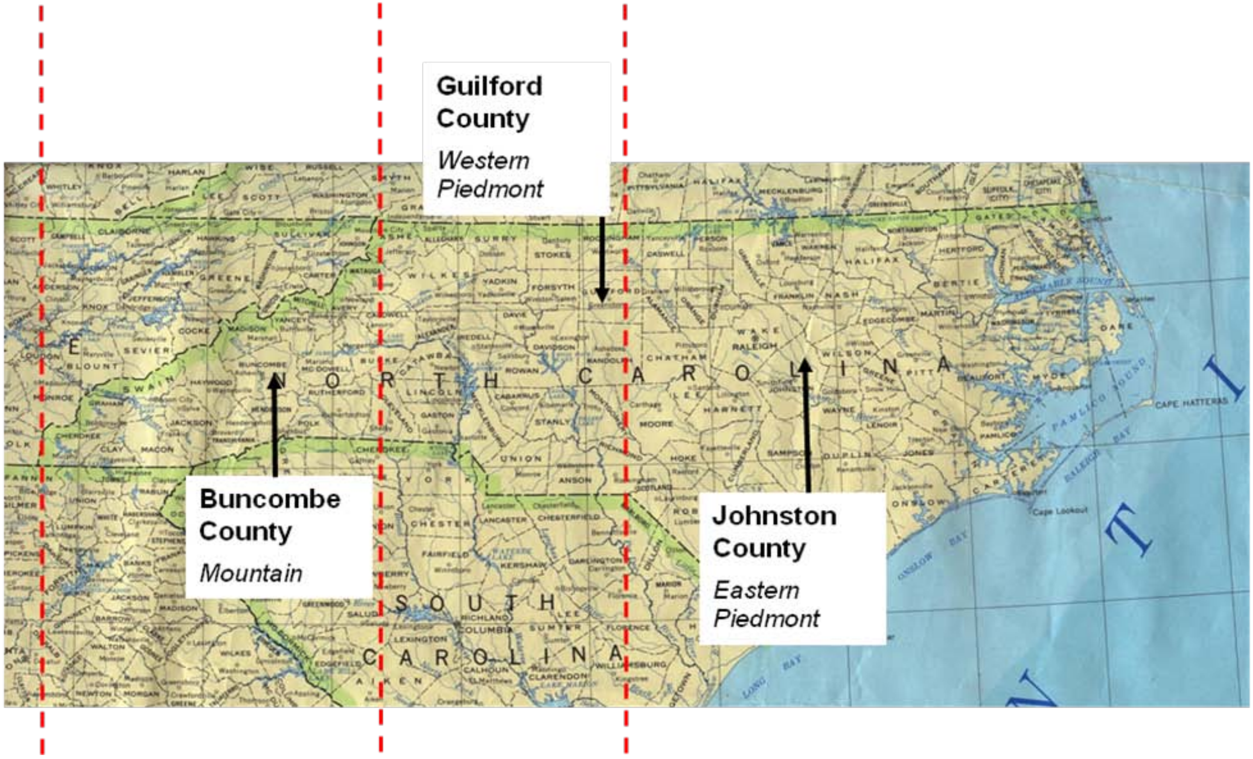


Figure 2-2: Soil Locations

Table 2-5: Soil collected for lime stabilization research

No.	Dates Collected	Region	County	Location	AASHTO Soil Classification	Sampling
1	7/12/06, 8/25/06	Eastern Piedmont	Johnston	U.S. 70, near Clayton	A-4, A-6	
2	7/26/06	Mountain	Buncombe	I-40, near Asheville	A-2-4	
3	10/24/06	Western Piedmont	Guilford	I-40, near Greensboro	A-4	

After the soils were classified, soil from each county was mixed with Type I Portland cement and tap water and subjected to moisture-density tests. Historically, NCDOT has recommended adding 60 g – 80 g of cement per 1 kg of dry soil mass (6%-8%), e.g., as suggested by Table 2-4. Based on discussions with NCDOT it was agreed that testing for the project would focus, a priori, on 6% cement content per the dry soil mass. This decision was made with full appreciation of the guidelines given by the PCA (PCA 1992). For purposes of this research, the amount of cement addition was based largely on NCDOT past practices.

2.3.2 Methods

2.3.2.1 Moisture-Density Relationships and Unconfined Compression (UC) Testing

The procedure followed for determining the maximum dry unit weight, and therefore the optimum moisture content (OMC), is described in ASTM D 558 “Standard Test Methods for Moisture-Density (Unit Weight) Relations of Soil-Cement Mixtures.” ASTM D 558 is virtually the same as ASTM D 698 “Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400ft-lbf/ft^3 (600kN-m/m^3))” with a few exceptions. First, ASTM D 698 stipulates the maximum particle size allowed for the 4 in. sample mold be 3/8 in. Meanwhile, ASTM D 558 allows a maximum particle size of ¾ in. for the same size mold. Second, ASTM D 698 does not allow for the reuse of material left over from a previous moisture content determination in the next trial; ASTM D 558 does allow for the re-use of the extra material. Lastly, ASTM D 558 allows for the “scalp and replacement” technique, while ASTM D 698 does not. This technique involves discarding material that does not pass the ¾ in. sieve

and replacing it with an equal amount of material that will pass the ¾ in. sieve and then passing that material through the No. 4 sieve in order to use (ASTM International 2004).
After the OMC was determined for each soil-cement mixture for each region, the process of making samples began with the oven-drying of soil. Once allowed to dry for 24 hours at 140°F, the soil was removed and pulverized. The soil was passed through a No. 4 sieve and the retained material discarded. At this point, a mass of approximately 4 kilograms (kg) was collected in order to make duplicate specimens.

The amount of Portland cement to add to the mixture was determined by calculating 6% of the dry soil mass. Then, that amount of cement was measured and set aside. Next, the amount of water to add was calculated. Based on the OMC for the particular soil sample, the mass of water to add was determined by taking the percent based on mass of dry soil plus the mass of cement. It is important to note that even oven dried soil contains a certain amount of water. Therefore, the soil, after the typical drying period, was tested to determine its water content prior to mixing. To immediately determine the moisture content of the soil, ASTM D 4643 "Standard Test Method for Determination of Water (Moisture) Content of Soil by the Microwave Oven Heating" was followed. Any moisture existing in the oven dried specimen was subtracted from the mass of water to add in order to measure out the appropriate amount of water for the mixture.

Once all the components of the mixture were measured out, the dry soil and cement were thoroughly mixed together, as shown in Figure 2-3.

Figure 2-3: Mixing of the dry soil and cement prior to adding water

The two were mixed until the conglomerate mixture was a uniform color and no soil balls or clumps of cement were visible. Next, the water was gradually added to the mixture and thoroughly blended together (Figures 2-4 and 2-5).

Figure 2-4: Adding water the dry soil-cement matrix

Figure 2-5: Thoroughly mixing the water into the material

Again, the materials were mixed until the color was uniform and there were no visible signs of soil or cement balls. Upon completion of the mixing, the soil-cement material was ready for molding into the test specimens (Figure 2-6).

Figure 2-6: Soil-cement material ready for compaction into the molds

The procedure followed for making the soil-cement specimens was the same procedure that was followed for the moisture-density tests, ASTM D 558. Each sample was approximately 4.0” in diameter and 4.584” in height. These dimensions correlate to an approximate cross-sectional area of 12.7 in^2 and an approximate volume of 57.6 in^3 for each sample. The only deviation from the test procedure was that once the soil specimen had been appropriately compacted in the mold and then ejected, the specimen was not disintegrated for moisture content determination. The ejected soil specimen was placed into a re-sealable plastic bag along with a damp sponge, closed, and moved to the curing chamber (Figures 2-7 through 2-9). The curing chambers were Cincinnati Sub-Zero Temperature Chambers (Model Number ZHS-16-2-H/AC).

Figure 2-7: Soil-cement specimen ejected from the mold using a hydraulic ejector

Figure 2-8: Soil-cement specimen and damp sponge in the curing bag

Figure 2-9: Curing chambers used for curing the samples

The plastic bag was labeled with the specimen identification (soil region, cement content, curing temperature, and during duration), the specimen creation date, and the specimen test date. The purpose of the damp sponge was to provide a humid environment for the duration of the specimen's curing period. This "sponge in a bag approach" was tested and shown to have the capacity to maintain a relative humidity in excess of 95%.

When a specimen had reached its designated curing duration it was removed from the curing chamber and the preparation for unconfined compression (UC) testing began using a rate of 1.270 mm/min. Following the directives of ASTM D 1633 "Standard Test Methods for

Compressive Strength of Molded Soil-Cement Cylinders," the specimen was immediately placed in a water bath at room temperature. After soaking for 4 hours, the samples were removed and subjected to a UC test in an electronic load frame. Figure 2-10 is a photo of a typical sample at failure.

Figure 2-10: Specimen tested to failure; the LVDT (back left) and the load cell (top, center) measured the deflection and load respectively

A computer and data acquisition system was used to measure deflection and load via an LVDT and load cell respectively. The computer captured the data in the form of a spreadsheet and the raw data was analyzed to determine the maximum stress experienced by the sample (its UC strength). Once the strength of the specimen was recorded, the broken specimen was discarded. After making several specimens and analyzing their performance versus the control specimens, it was determined that a change to the procedure needed to be made. In order to model the conditions in the field, it was decided that the materials should be at the design temperature *prior* to mixture. When the temperature in the field is cold, the soil to be stabilized will have been conditioned by the ambient temperature. Ideally, researchers would be able to conduct all of the mixing at the design temperature (i.e. material storage and sample making at 35°F instead of room temperature). This was not feasible; therefore the materials (soil, cement, and water) were cooled in the curing chambers to the design temperature (for at least 24 hours) prior to mixing. During the mixing stage the materials typically spent 30 minutes at room temperature while the specimens were molded before being returned to the curing chambers. The motivation for this change was also discussed in Chapter 1, and in particular Figure 1-9 serves as a relevant indication of the significance of "pre-cooling."

2.3.2.2 Tube Suction Test

The tube suction test (TST) provides an indication of the potential for capillary rise in the stabilized material as well as the state of moisture within the matrix. TST data allow comment on the extent to which water adsorption is expected to lead to breakdown of the matrix. Specifically, measurements of dielectric constant (which in turn is a measure of the moisture content), can be correlated to the arrangement of water molecules in and around mix components (Scullion and Saarenketo 1996).

The TST procedure followed is based primarily on that proposed by Barbu and Scullion 2006:

1. Arrange molds, hammer, porous stone, membrane, filter paper, plastic sheet, sieved soil, cement, deionized water and percometer.
2. Mix the soil with deionized water (at optimal moisture content), making 2 specimens every time with standard compaction effort. Because of different size of mold typically used, the following equation (refer to Tex-113-E, 08/99) is used to determine the compactive effort. For example, the mold with 7 inch height and 6 inch diameter should be compacted at four layers, 25 drops per layer with the 10lb hammer with a 18 inch drop height.

$$C.E. = \frac{Ht.\ of\ Drop\,(m\ or\ ft) \bullet Wt.\ of\ Hammer\,(kN\ or\ lb) \bullet \#Drops \bullet \#Layers}{Volume\ of\ Mold\,(m^3\ or\ in^3)}$$

3. After extruding the specimen, measure the weight of each specimen and the dielectric value.
4. Put the two specimens and the porous stones together into the 60°C chamber to dry for 48 hours.
5. After 48 hours, remove specimens from chamber and allow them to equilibrate to room temperature for 2 hours.
6. Measure the initial dielectric value and the dry weight of each specimen.
7. Wrap the specimen and porous stone together with a membrane, as shown in Figure 2-11:

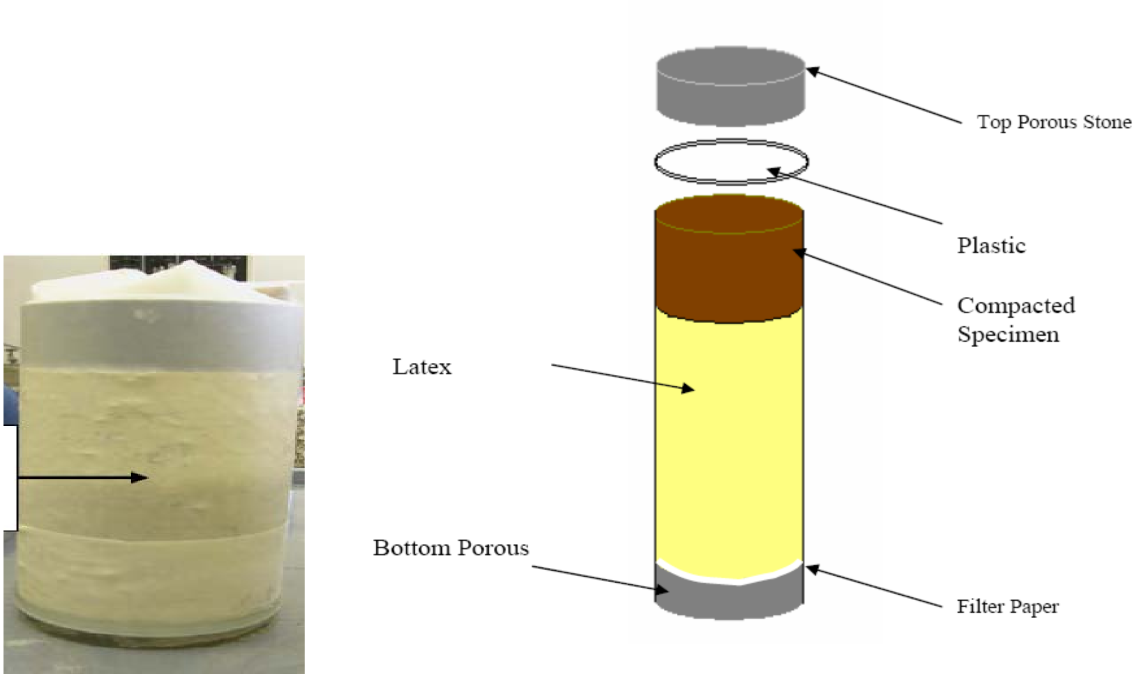


Figure 2-11: TST Specimen Preparation (Source: Barbu and Scullion 2006)

8. Place specimens in a flat-bottomed stainless steel pan, filled with de-ionized water. The water depth should be approximately 0.25 inch above the top of the bottom stone.
9. Take five dielectric values reading on the top surface of the specimen daily for 10-20 days. Take the reading at the same time during each day, if at all possible.
10. Draw the curve of the dielectric value to determine the moisture susceptibility of the mixture.

The picture in Figure 2-12 was taken from samples during the hydration/capillary rise phase of the test (i.e., after Step 8 above.)

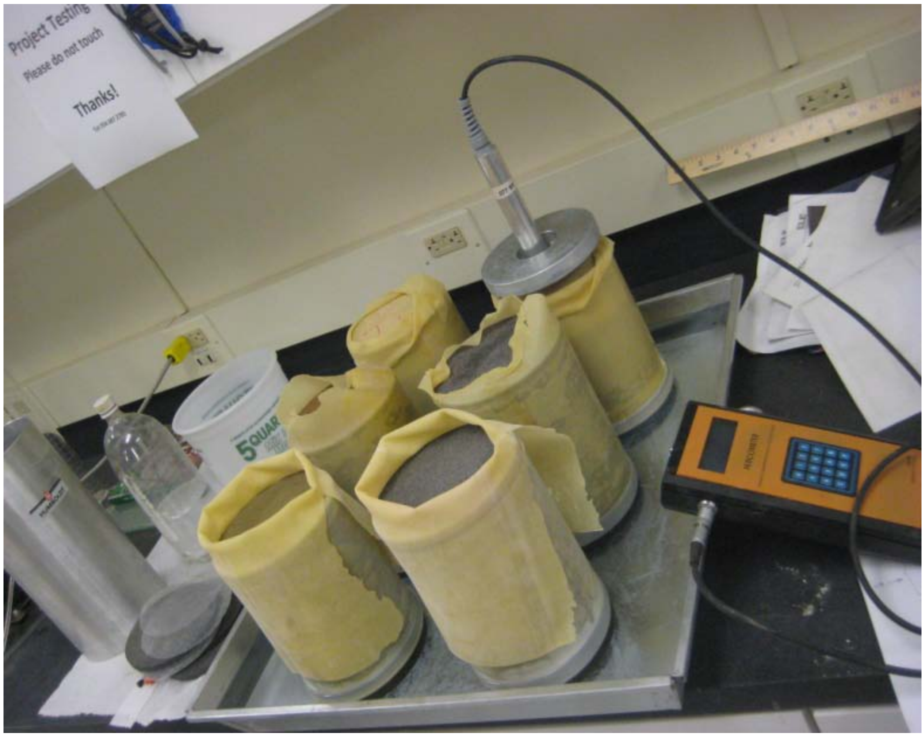


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Figure 2-12: TST Specimen Subjected to Capillary Rise

2.4 Results

2.4.1 Characterization

The NCDOT Materials and Tests Unit classified the soil samples obtained for this project. A summary of their findings is provided in Table 3 below.

Table 2-6: Soil sample classifications

		M	WP	EP*	
Soil Sample		Buncombe	Guilford	Johnston (1)	Johnston (2)
Retained #4 Sieve	%	10	-	-	-
Passing #10 Sieve	%	86	98	98	100
Passing #40 Sieve	%	74	82	75	79
Passing #200 Sieve	%	32	46	40	45
P. L.		29	28	27	22
L. L.		33	37	35	39
P. I.		4	9	8	17
AASHTO Classification		**A-2-4(0)**	**A-4(2)**	**A-4(0)**	**A-6(4)**

*The Johnston County soil was obtained on two different occasions from the same site,
every attempt was made to mix the two soils prior to making specimens

2.4.2 Moisture Density Relationships

Prior to creating the specimens for UC testing, it was necessary to determine the moisture density relationship for each soil-cement material. Figures 2-13 through 2-15 show the moisture density relationship for Buncombe County, Guilford and Johnston County soils (each with 6% cement) with the design moisture content identified by a dashed vertical line.

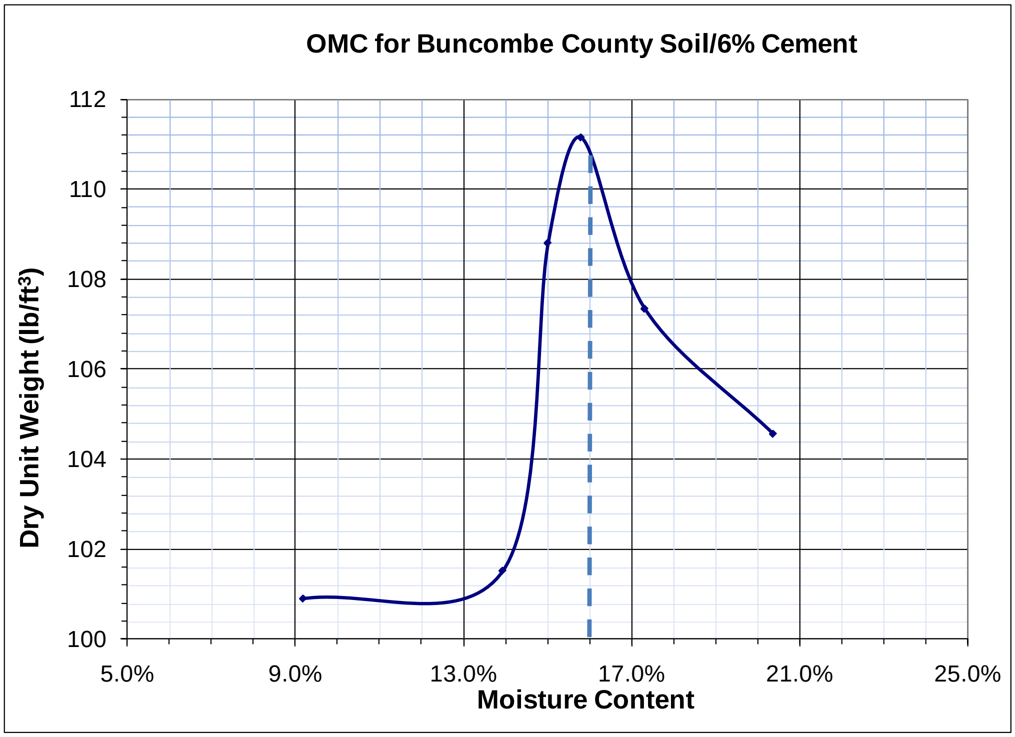

Figure 2-13: Moisture Density Relationship for Buncombe County soil-cement

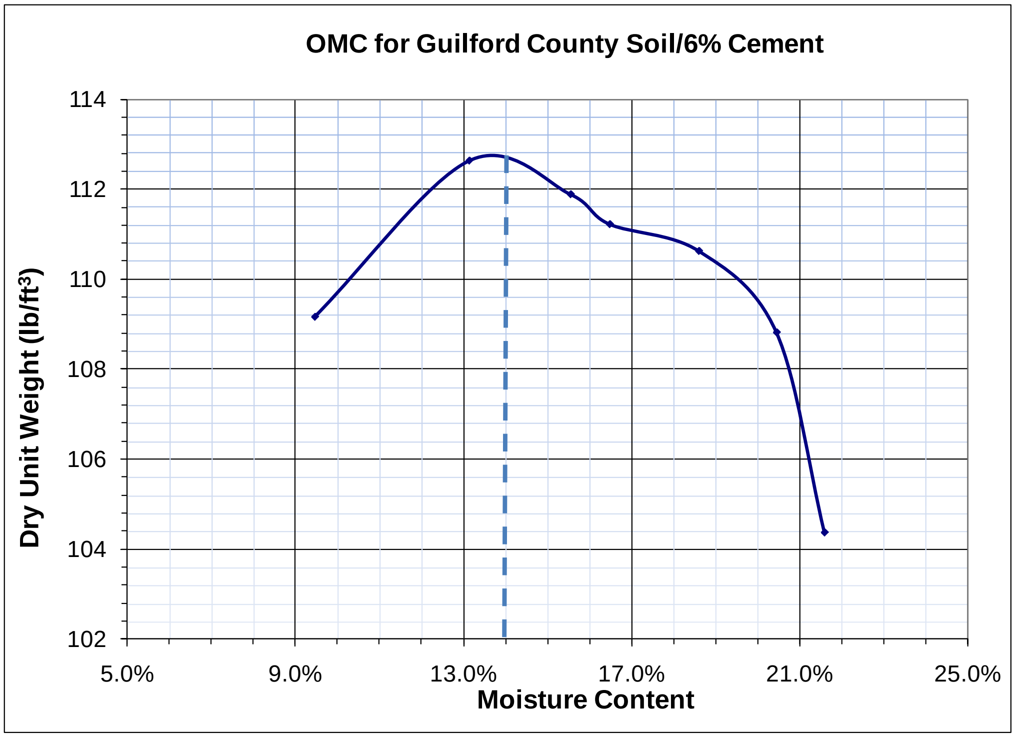

Figure 2-14: Moisture Density Relationship for Guilford County soil-cement

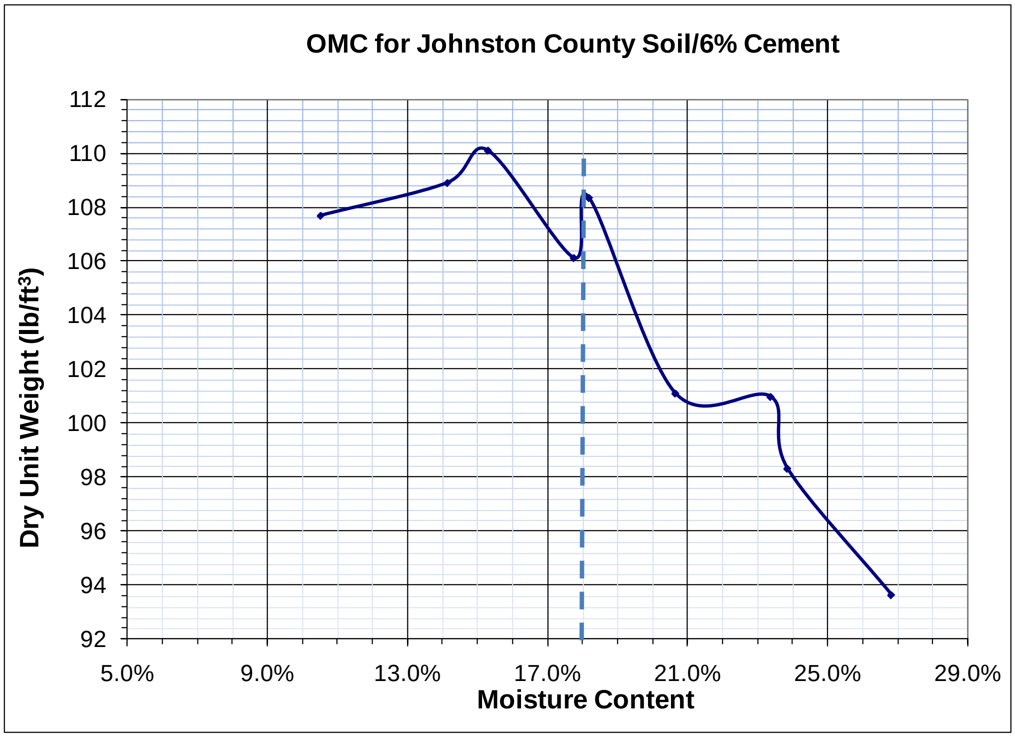

Figure 2-15: Moisture Density Relationship for Johnston County soil-cement

2.4.3 Unconfined Compression Strength

The primary test conducted as part of this investigation was the unconfined compression test. The results are displayed for each county in terms of tables (Tables 2-7 through 2-9) as follows. Note that average values are presented, with the number of replicates ranging from a minimum of 2 samples to as many as 15 samples.

Table 2-7: Buncombe County Soil-Cement - UC Strength versus curing temperature and duration (average values shown)

Curing Temperature (F)	Curing Duration (days)		
	1	3	7
	UC Strength (psi)		
25	90.9	71.7	94.7
35	99.7	124.0	200.2
50	115.7	147.5	262.3
70	120.7	201.4	217.1

Table 2-8: Guilford County Soil-Cement - UC Strength versus curing temperature and duration (average values shown)

Curing Temperature (F)	Curing Duration (days)		
	1	3	7
	UC Strength (psi)		
25	76.4	100.7	46.5
35	98.9	155.5	154.0
50	158.1	162.3	146.1
70	111.8	247.1	293.0

Table 2-9: Johnston County Soil-Cement - UC Strength versus curing temperature and duration (average values shown)

Curing Temperature (F)	Curing Duration (days)		
	1	3	7
	UC Strength (psi)		
25	93.1	108.2	109.5
35	111.8	163.1	177.0
50	128.6	198.1	261.0
70	139.8	212.3	245.2

In addition, 15 specimens were made under ideal curing conditions (70°F and 100% humidity) to determine the correlation between 3 day compressive strength and 7 day compressive strength of soil-cement samples for all three counties. The data from these tests appear in the Tables 2-10 through 2-12.

Table 2-10: Buncombe County Soil-Cement UC Strength – 70^{o}F / 3 Day / 7 Day Strength Statistical Analysis

Buncombe County - 70F Curing Temperature		
	Unconfined Compressive Strength (psi)	
Sample #	3 day cure	7 day cure
1	194.0	222.0
2	204.4	239.2
3	211.4	217.5
4	210.5	231.5
5	191.7	247.0
6	201.6	229.9
7	217.9	215.3
8	208.0	198.4
9	205.9	203.9
10	196.0	200.6
11	216.7	220.7
12	197.0	205.5
13	183.2	204.1
14	189.6	195.1
15	193.3	225.0
Average	201.4	217.1
Standard Deviation	10.3	15.7
Coefficient of Variation	0.05	0.07
3d / 7d	**92.8%**	

Table 2-11: Guilford County Soil-Cement UC Strength – 70°F / 3 Day / 7 Day Strength Statistical Analysis

Guilford County - 70F Curing Temperature		
	Unconfined Compressive Strength (psi)	
Sample #	3 day cure	7 day cure
1	245.5	304.9
2	253.9	269.7
3	228.9	318.8
4	256.7	275.8
5	251.1	276.7
6	250.5	242.2
7	232.1	300.4
8	231.8	299.5
9	256.9	314.8
10	267.6	273.0
11	251.3	296.4
12	249.3	304.3
13	233.6	311.4
14	247.6	293.9
15	249.1	313.4
Average	247.1	293.0
Standard Deviation	11.0	21.3
Coefficient of Variation	0.04	0.07
3d / 7d	**84.3%**	

Table 2-12: Johnston County Soil-Cement UC Strength – 70^{o}F / 3 Day / 7 Day Strength Statistical Analysis

Johnston County - 70F Curing Temperature		
	Unconfined Compressive Strength (psi)	
Sample #	3 day cure	7 day cure
1	223.2	244.7
2	218.9	285.1
3	216.3	282.5
4	224.7	211.8
5	228.8	236.8
6	216.4	245.4
7	219.8	202.9
8	195.1	186.9
9	226.3	205.2
10	214.4	185.9
11	201.9	272.8
12	198.0	288.6
13	204.7	278.5
14	193.3	274.7
15	203.0	276.5
Average	212.3	245.2
Standard Deviation	11.9	37.9
Coefficient of Variation	0.06	0.15
3d / 7d	**86.6%**	

Given that maximum strength values provide only one data point to describe a soil sample's behavior, moduli (initial tangent and secant) were tabulated to provide more insight into the relative stiffness/ductility of a given sample over a broader range of stress and strain. These moduli are given in Tables 2-13 through 2-15. By definition, the initial tangent modulus is the initial slope of the stress-strain curve. The secant modulus was defined in this chapter as the slope of the line which passes from the origin through 50% of the maximum stress.

Table 2-13: Summary of different moduli computed on sample for Buncombe County Soil-Cement

Curing Temp (°F)	**Curing Duration (Days)**	**Average Initial Tangent Modulus (psi)**	**Average Secant Modulus (psi)**
70	1	4911	9822
70	3	11332	19212
70	7	4737	17549
50	1	6825	7993
50	3	1571	5691
50	7	7315	15649
35	1	4690	5751
35	3	7382	8760
35	7	2924	10184
25	1	2204	3570
25	3	1055	2571
25	7	1345	4108
Freeze Cycle	3	4558	9918
Freeze Cycle	7	7481	12351

Table 2-14: Summary of different moduli computed on sample for Guilford County Soil-Cement

Curing Temp (°F)	**Curing Duration (Days)**	**Average Initial Tangent Modulus (psi)**	**Average Secant Modulus (psi)**
70	1	4811	4968
70	3	12165	6062
70	7	13934	5340
50	1	1061	5561
50	3	2665	6151
50	7	5759	7128
35	1	1668	2607
35	3	875	3793
35	7	4987	6968
25	1	1555	2353
25	3	662	1846
25	7	1603	2551
Freeze Cycle	3	9985	14986
Freeze Cycle	7	7192	13603

Table 2-15: Summary of different moduli computed on sample for Johnston County Soil Cement

Curing Temp (°F)	Curing Duration (Days)	Average Initial Tangent Modulus (psi)	Average Secant Modulus (psi)
70	1	6416	6260
70	3	7060	9171
70	7	1271	17862
50	1	6953	7975
50	3	5021	9391
50	7	16319	15604
35	1	6431	5791
35	3	2791	4915
35	7	11087	10954
25	1	4428	4733
25	3	354	1943
25	7	3785	4851
Freeze Cycle	3	8366	12549
Freeze Cycle	7	8258	13838

2.4.4 Tube Suction Test

The results of the TST are presented in Figures 2-16 through 2-18 as well as associated data in Table 2-16.

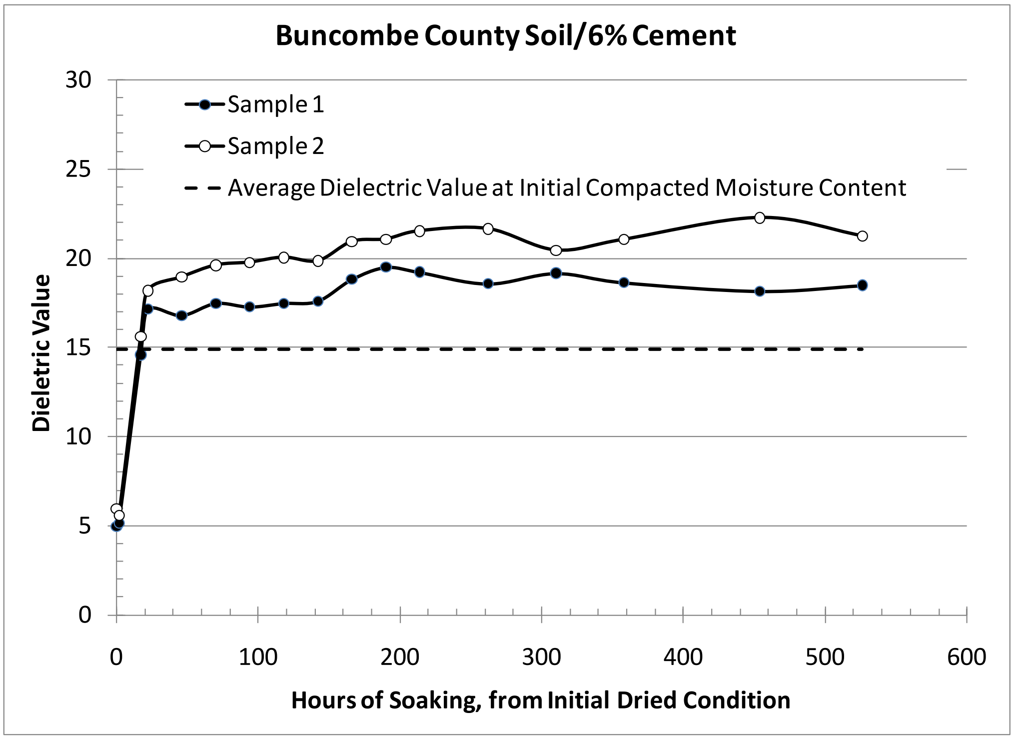

Figure 2-16: TST Results for two specimens (Buncombe County)

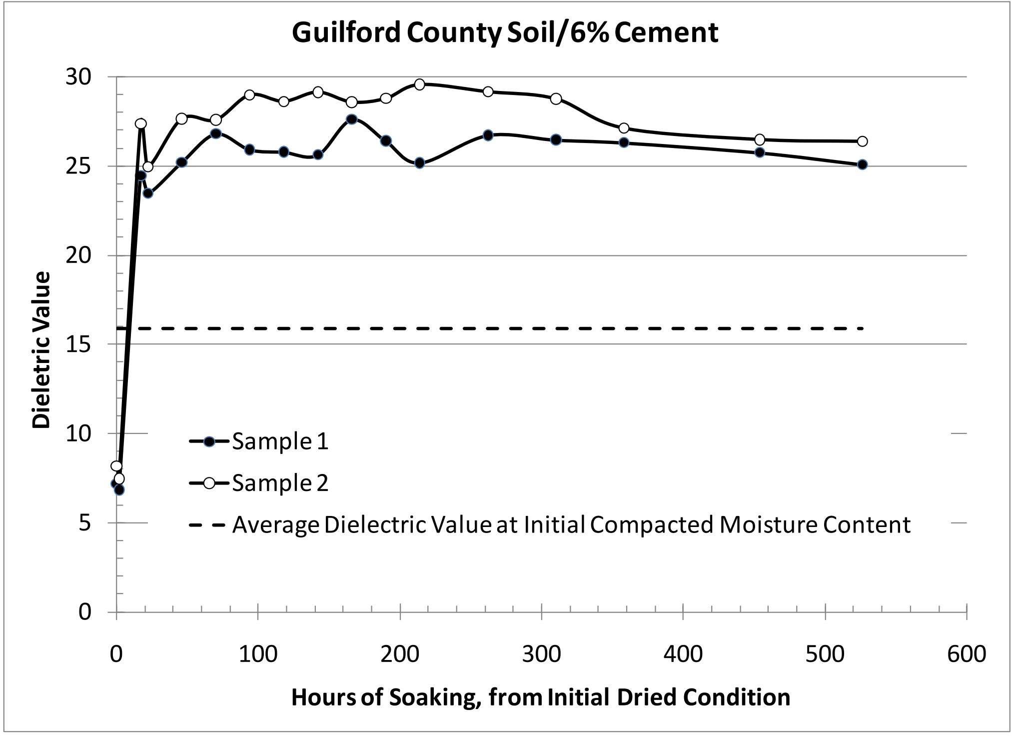

Figure 2-17: TST Results for two specimens (Guilford County)

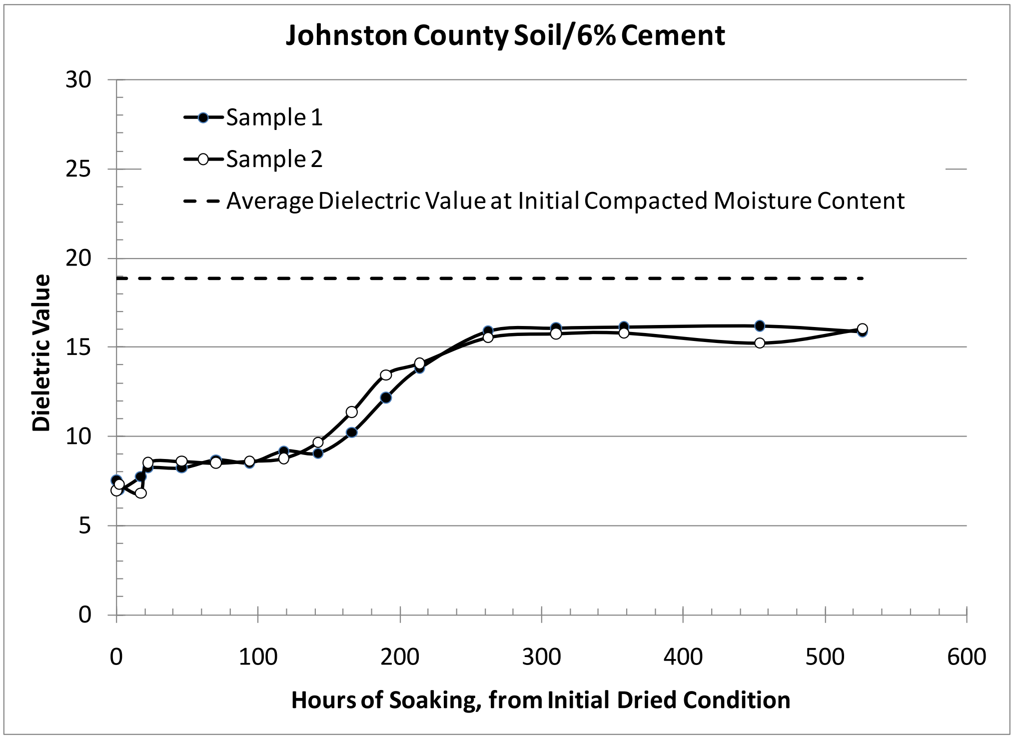

Figure 2-18: TST Results for two specimens (Johnston County)

Table 2-16: Comparison of DV and EC at different counties and status

	J1		J2		B1		B2		G1		G2	
	DV	**EC**	**DV**	**EC**	**DV**	**EC**	**DV**	**EC**	**DV**	**EC**	**DV**	**EC**
Original	19.18	12.98	18.56	25	14.02	37.8	15.8	73.2	15.76	67.2	16.04	105.2
Dry status	7.536	0	6.962	0	4.976	0	5.956	0	7.186	0	8.188	0
Final	13.84	6	14.1	8	19.22	64.8	21.52	133.2	25.18	247.8	29.56	331.2

DV is dielectric value, EC is electric conductivity, "Original" means the status of the specimen just after compaction. "Dry status" means the status that after 24 hours dry and cool to room temperature. "Final" means after 10 days testing in water.

DV values may be taken as an indicator of moisture content. The foregoing data indicate that Johnston County soil-cement mix is the least susceptible to moisture-induced damage, while Guilford and Buncombe County soil-cement mixes are more susceptible. In the case of Johnston County, the rate of DV increase is slow and does not reach the original DV value corresponding to the moisture content at compaction. By contrast, the rate of DV value increase is considerably higher for Guilford and Buncombe County soil-cement mixes. In addition, these soils attain steady state DV values well in excess that of the initial compaction condition. In essence, the Buncombe and Guilford County soil-cement mixes attract, through capillary action, more water than present when compacted near the optimum water content.

2.5 Discussion, Conclusions and Recommendations

For purposes of discussion, Tables 2-17, 2-18 and 2-19 contain the UC strength data for Buncombe, Guilford, and Johnston county samples (respectively) expressed as a percentage of the ideal curing temperature (70°F) for each design curing duration. With a few exceptions, there is a clear trend of reduced strength with lower curing temperature. Where differences arise, they are primarily attributed to issues with sample repeatability and statistical variation.

Table 2-17: Average UC strength of Buncombe County samples expressed as a percentage of samples cured at 70°F for the respective duration.

Curing Temperature (°F)	Curing Duration (days)		
	1	3	7
25	75.3%	35.6%	43.6%
35	82.6%	61.6%	92.2%
50	95.9%	73.2%	120.8%

Table 2-18: Average UC strength of Guilford County samples expressed as a percentage of samples cured at 70°F for the respective duration.

Curing Temperature (°F)	Curing Duration (days)		
	1	3	7
25	68.3%	40.8%	15.9%
35	88.5%	62.9%	52.6%
50	141.4%	65.7%	49.9%

Table 2-19: Average UC strength of Johnston County samples expressed as a percentage of samples cured at 70°F for the respective duration.

Curing Temperature (°F)	Curing Duration (days)		
	1	3	7
25	66.6%	51.0%	44.7%
35	80.0%	76.8%	72.2%
50	92.0%	93.3%	106.4%

Part of the original vision for this research was to provide a basis to estimate strength of stabilized subgrade as a function of temperature. The foregoing data provide such a basis and can be incorporated into a model that relates air temperature to subgrade temperature. In particular, Figures 2-19 through 2-21 show the UC strength data plotted as a function of temperature with the corresponding trendline equations. These equations are used in the modeling presented in Chapter 3. In general, greater reliability is ascribed to the 3-day and 7-day data. More testing would be required to improve the reported coefficients of correlation.

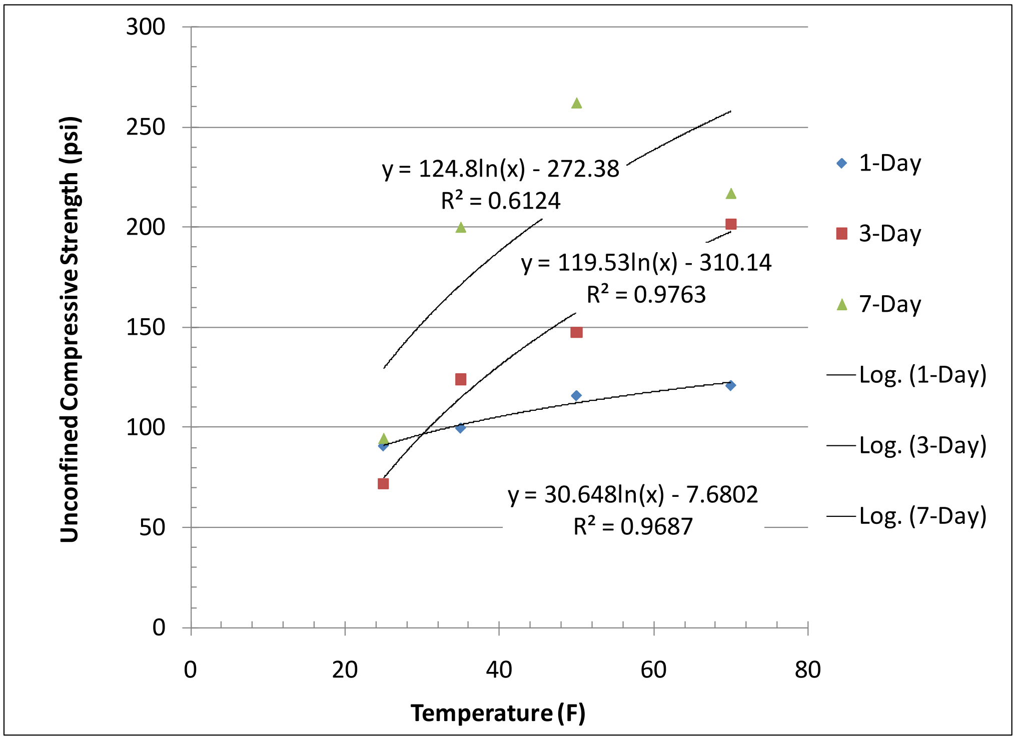

Figure 2-19: UC Strength as a function of temperature (Buncombe County)

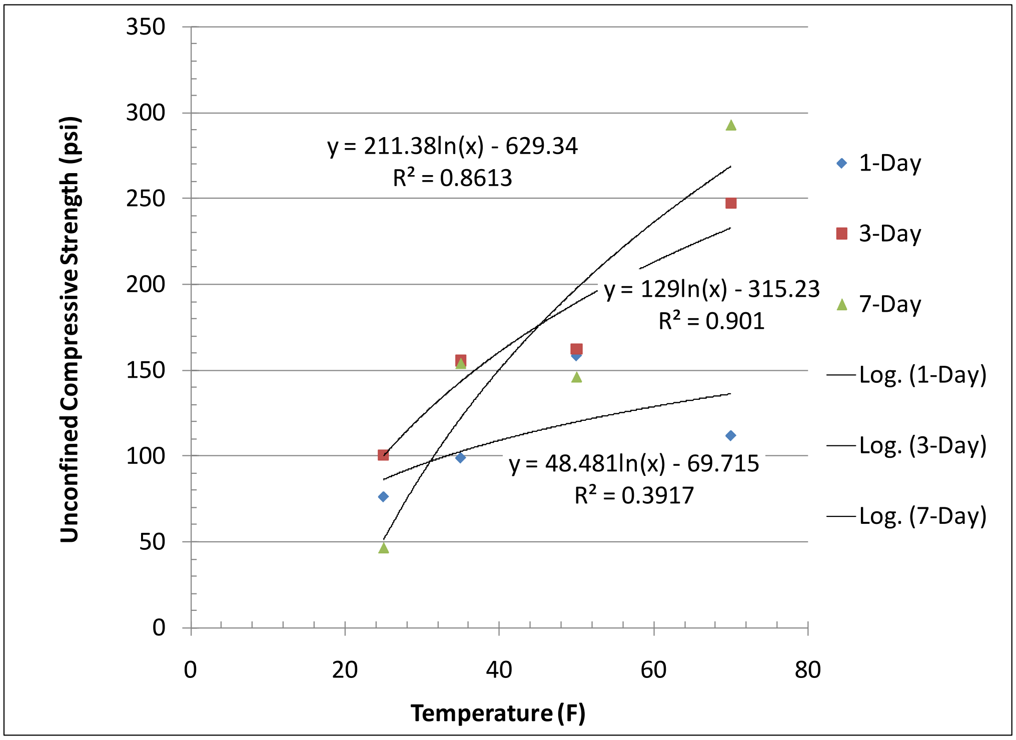

Figure 2-20: UC Strength as a function of temperature (Guilford County)

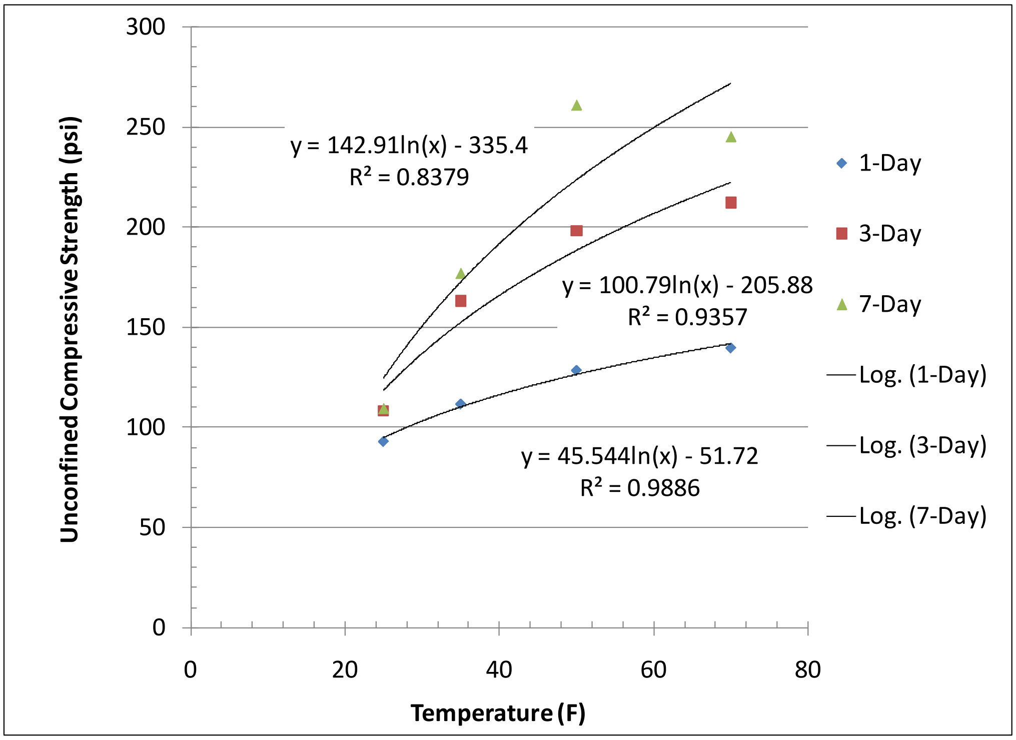

Figure 2-21: UC Strength as a function of temperature (Johnston County)

In general, the cement-stabilized soil exhibited behavior that was analogous to concrete. The lower the curing temperature the less strength and stiffness gained at the end of curing. In addition, the longer the curing period the more strength and stiffness gained by the end of curing. By way of comparison, the effect of temperature and curing duration on concrete can be seen in Figure 2-22 (Klieger 1958).

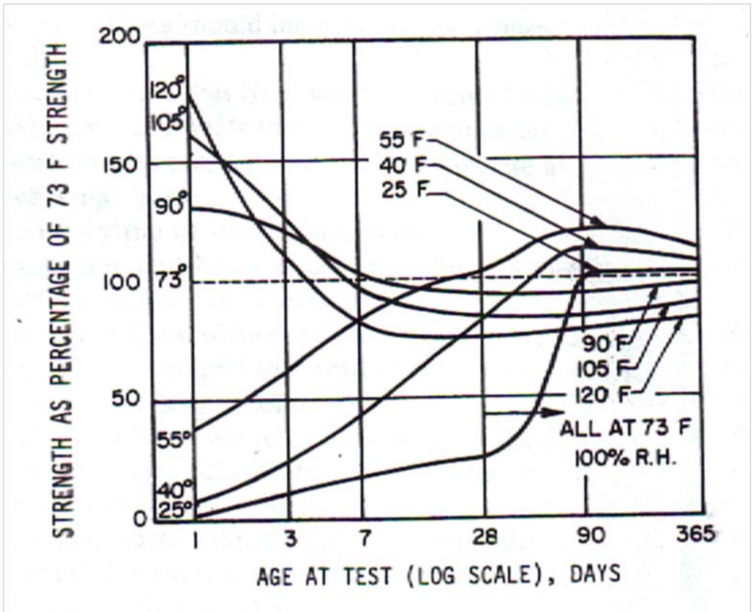

Figure 2-22: The effect of curing temperature on strength of concrete (Klieger 1958)

Figure 2-22 expresses the strength of concrete cured at 25°F, 40°F, and 55°F as a percentage of the strength of concrete cured under ideal conditions. At every curing duration, as the temperature increases, the strength of the concrete increases (i.e. for 7 days of curing, the strengths at 25°F, 40°F, and 55°F are 19%, 46%, and 89% respectively). However, for the soil-cement data presented herein, occasionally specimens cured at 50°F outperformed the samples cured at 70°F for the same curing duration. For example, Buncombe County specimens cured at 50°F for 7-day exhibited 125% of the 70°F 7-day cure strength. Clearly, the data from the cement-stabilized soil is less consistent than that associated with concrete. This can be partly attributable to the inherent variability of soil, meaning that two specimens taken from the same sample vary in composition. This can also be attributed to the cement content of the material. Cement-stabilized soil often contains 1/3 to 2/3 less cement than concrete (Rollings et al. 2002). Thus, there is less thermal gain from hydration allowing the temperature of the stabilized soil matrix to equilibrate with the environment sooner which, in turn, hinders further hydration. Regardless of these anomalies, the majority of the tests indicate that a decrease in curing temperature has an adverse effect on the strength of the finished material.

1. It appears that an especially critical curing temperature range is 25°F to 35°F, an intuitive concern given the point at which water freezes (32°F). In particular, water that is frozen is unable to participate in the reactions that contribute to strength development. In addition, formation of ice within pore space causes fraction and severe damage within the stabilized soil matrix. Studies on concrete have determined that once the material reaches a compressive strength of 500 psi it has adequate strength to resist the damage of the first freeze (Powers 1962). Thus, if concrete is susceptible to damage prior to reaching 500 psi, cement stabilized material is as well. None of the stabilized soil samples reached a compressive strength of 500 psi; in fact, the maximum average strength exhibited in the test was 318 psi. It is also important to note that after curing at 25°F the specimen itself experienced sloughing and crumbling once removed from the curing chambers. It is evident that the formation of ice within the material’s matrix compromised the integrity of the specimen.
2. These data suggest that investigating the impact of additives (e.g., Chapter 4) on cement stabilization of soil during the cold weather months is valuable because there is a significant difference between the strength of a sample cured below 40°F (NCDOT’s limiting temperature for cement stabilization) and that of material cured above 40°F.
3. On the basis of 15 repeat tests for 3 and 7 day curing periods, for three different soils, results indicate that the mean strength at 3 days is 84-93% of that for 7 days, in support of a potential change in current subgrade evaluation practice predicated on the longer duration.
4. The soils tested have varying levels of moisture susceptibility, with Guilford County soils being the most vulnerable to moisture-induced material weakening.

The data presented herein suffered from several inconsistencies that can be explained by a few points. Each specimen was made by following the directives of ASTM D 558 “Standard Test Methods for Moisture-Density (Unit Weight) Relations of Soil-Cement Mixtures.” Therefore, hypothetically each specimen experiences the same amount of compaction force. Any deviation from the directions of ASTM D 558 may cause a reduction or increase in the amount of effort used to create the specimen. For example, if the hammer is not dropped and rather forced onto

the sample, the amount of effort imparted on the material is increased. Also, the thorough mixture for all the constituents was critical to the quality of the final product. The complete integration of the cement in the dry soil and of the water in the dry soil-cement mixture was always the intent. However, the procedure for exactly measuring this simply was not possible for the scope of this project. It was left to the experimenter to decide when the mixing stage was complete. As in any study, the data obtained from these tests were subject to human error.
The physical and chemical properties of the soil are also critical factors to consider when stabilizing with cement. In order for a particular soil to meet the design specifications, the appropriate amount of cement needs to be prescribed. Chapter 2 of the Portland Cement Association's *Soil-Cement Laboratory Handbook* is devoted to describing how to select cement contents. In general, the amount of cement needed to properly stabilize a soil increases with the increase in silt and clay contents; while a gravelly or sandy soil typically requires less cement than that of silts and clays. In particular, the *Soil-Cement Laboratory Handbook* suggests 7% cement by weight for an A-2 (Buncombe), 10% cement for an A-4 (Guilford and Johnston) (PCA 1992) – as opposed to the 6% cement used for all soils on the basis of typical NCDOT practice.

Similar to the comments made in Chapter 1, these results indicate that current specifications are justified and sufficiently conservative to maximize strength development in cement-stabilized subgrades. The nature of the testing plan and variability of soils and conditions encountered by NCDOT precludes greater specificity in terms of soil types, cement dosage or acceptable temperature ranges. However, the software program developed and described in Chapter 3 of this report may be used to inform case by case decisions as to whether cold weather exceptions can be made. This software relates forecasted air temperatures to likely subgrade temperatures.

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CHAPTER 3: Temperature Modeling of Stabilized Subgrade Soil

Chapter Summary

Specifications for stabilization work have often been based on air temperature measurements, however the performance of lime or cement treated soil is expected to be more closely related to the in situ temperature. As such, the purpose of this work was to document the relationship between air and subgrade (soil) temperatures, under conditions relevant to NCDOT stabilization projects and then develop a model that relies on easily obtained parameters. This work was designed to complement the research which evaluated the relationship between strength and temperature of chemically stabilized subgrade soils. This research has found that the thermal diffusivity of both lime and cement-stabilized subgrades varies from 3.8 x 10^{-7} m^2/s (2.14 in^2/hr) to 9.8 x 10^{-7} m^2/s (5.46 in^2/hr). These data were incorporated into a model that relates air and soil temperatures. A computer application was developed to use the model to make predictions of subgrade temperatures and cured strength.

3.1 Introduction

The purpose of this phase of the work was to document the relationship between air and subgrade (soil) temperatures, under conditions relevant to NCDOT stabilization projects and then develop a model that could be implemented with readily accessible parameters. Such a model would then complement the data developed in Chapters 1 and 2 such that given little more than a weather forecast, one could estimate the anticipated subgrade strength.   This chapter is divided into the following main sections: Field Data Collection Activities, Results and Analysis, Model Development and Computer Application

3.2 Field Data Collection

For this project, six different sites were instrumented at varying depths. The location of these sites is given by Figure 3-1. Four of the locations were instrumented in the fall of 2006 (Table 3-1), while Union County data were obtained in the fall of 2008 (Table 3-2) and Lincoln County data were obtained in the summer of 2009 (Table 3-3).

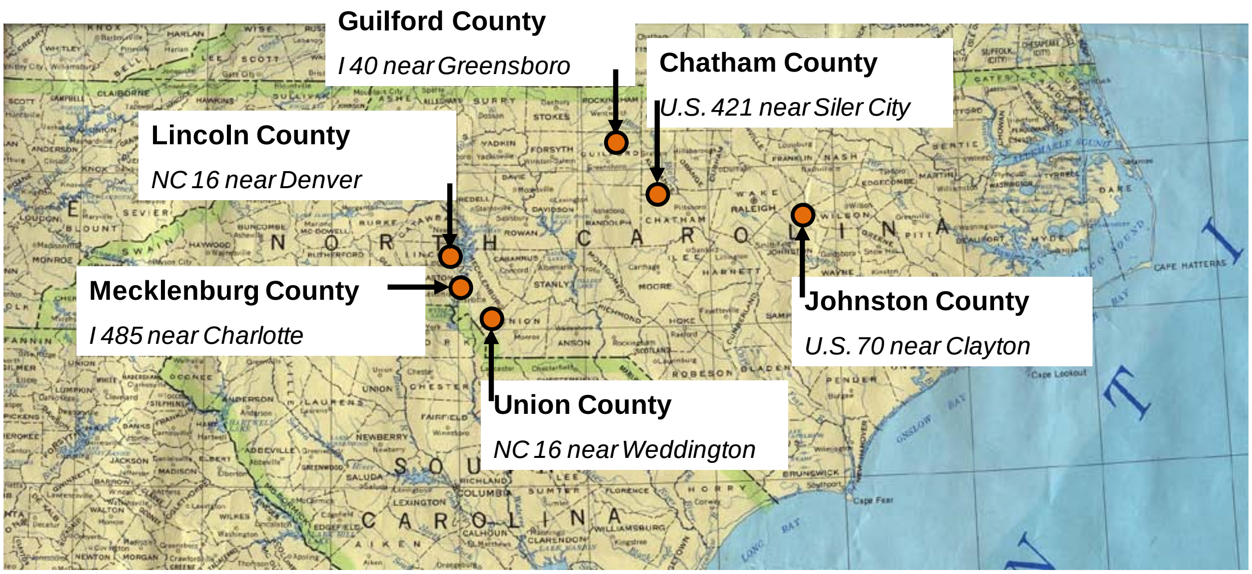


Figure 3-1: Monitoring Location Data

Table 3-1: Monitoring Location Data-Fall 2006

Parameter	**Location**			
	U.S. 70, near Clayton, NC (Johnston County)	**U.S. 421, near Siler City, NC (Chatham County)**	**I-40, near Greensboro, NC (Guilford County)**	**I-485, near Charlotte, NC (Mecklenburg County)**
Station	132+00	94+20	45+25	303+90
Soil Type	A-7-5	A-7-5	A-7-6	A-4
Type of Stabilization	Lime Slurry	Lime Slurry	Cement	Cement
Rate of Application	13 kg/m^2 (hydrated lime) 6.5 kg/m^2 (quicklime "sweetener")	14 kg/m^2 (hydrated lime)	8%, by dry weight	8%, by dry weight
Depth of Stabilization	8"	8"	12"	7"
Optimum Moisture Content (%)	26.1	28	18	15.5
Maximum Dry Density (lb/ft^3)	95.6	~95	113.9	114.8
Date Stabilizer Added	10/4/06 (lime slurry) 10/12/06 (quicklime)	10/10/06	10/24/06	11/14/06
Date Tack Coat Applied	10/13/06	10/16/06	10/24/06	11/14/06
Date Instrumentation Placed / Start of Data Collection	10/16/06	10/11/06	10/25/06	11/15/06
Date Next Layer Placed, type	10/24/06, Stone	10/23/06, Stone	11/30/06, Asphalt Pavement	Date not recorded, Asphalt Pavement
Subgrade Temperature Measurements	1.75", 3.75", 7.25"	1.75", 2.75", 4.25"	1.75", 4.75", 8.25"	1.75", 3.75", 6.25"

Table 3-2: Monitoring Location Data-Fall 2008

Parameter	**NC 16, near Weddington, NC (Union County)**
Station	23+50 - 25+00 (LT)
Soil Type	A-7-5
Type of Stabilization	QuickLime
Rate of Application	5%, by dry weight
Depth of Stabilization	10”
Date Stabilizer Added	11/10/08
Date Instrumentation Placed / Start of Data Collection	11/13/08
Date Next Layer Placed, type	Asphalt, ~ 12/10/08
Subgrade Temperature Measurements	~1”, 3”, 5”, 7”, 9”

For Union County, instrumentation was actually installed twice. The first installation of temperature sensors on October 31, 2008 was removed on November 4, 2008 because the instrumented section did not meet the density requirements and was therefore re-mixed and re-compacted. Temperature sensors were reinstalled on November 13, 2008 as noted in the above Table. Data were recorded in both cases.

Table 3-3: Monitoring Location Data-Summer 2009

Parameter	**Location**			
	NC 16, near Denver, NC (Lincoln County) Control Section 1	**NC 16, near Denver, NC (Lincoln County) Low Dosage $CaCl_2$**	**NC 16, near Denver, NC (Lincoln County) High Dosage $CaCl_2$**	**NC 16, near Denver, NC (Lincoln County) Control Section 2**
Station (approx)	170+29 (LT)	171+21 (LT)	172+15 (LT)	173+06 (LT)
Soil Type	A-4*	A-4*	A-7-6**	A-7-6**
Type of Stabilization	Cement	Cement	Cement	Cement
Rate of Application	28 kg/m^2	28 kg/m^2	28 kg/m^2	28 kg/m^2
Depth of Stabilization	180 mm (~7”)	180 mm (~7”)	180 mm (~7”)	180 mm (~7”)
Optimum Moisture Content (%)	17.4^t	17.4^t	18.6^{tt}	18.6^{tt}
Maximum Dry Density (lb/ft^3) (approx)	108.3	108.3	108.3	108.3
Date Stabilizer Added	6/10/09	6/10/09	6/10/09	6/10/09
Date Tack Coat Applied	On or before 6/15/09	On or before 6/15/09	On or before 6/15/09	On or before 6/15/09
Date Instrumentation Placed / Start of Data Collection	6/10/09	6/10/09	6/10/09	6/10/09
Date Next Layer	Asphalt – after	Asphalt – after	Asphalt – after	Asphalt – after

Parameter	**Location**			
	NC 16, near Denver, NC (Lincoln County) Control Section 1	**NC 16, near Denver, NC (Lincoln County) Low Dosage $CaCl_2$**	**NC 16, near Denver, NC (Lincoln County) High Dosage $CaCl_2$**	**NC 16, near Denver, NC (Lincoln County) Control Section 2**
Placed, type	7/10/09	7/10/09	7/10/09	7/10/09
Subgrade Temperature Measurements	1.0”, 3.0”, 5.0”, 8.0”, 10”	1.0”, 3.0”, 5.0”, 8.0”, 10”	1.0”, 3.0”, 5.0”, 8.0”, 10”	1.0”, 3.0”, 5.0”, 8.0”, 10”

*Based on sample 45Q Collected at Station 169+00 LT

**Based on sample 45X Collected at Station 172+00 LT

^{t} Based on sample 21X Collected at Station 169+00 RT

^{tt} Based on sample 46X Collected at Station 172+00 LT

In general, each site was instrumented for air temperature, subgrade temperature and on occasion, moisture content. Air temperatures were generally recorded with two sensors. The first was an EchoTemp sensor from Decagon Devices, Inc. This sensor has a range from -40^{0}C to +60^{0}C and is accurate to ± 0.25^{0}C at temperatures above 0^{0}C, and ± 1^{0}C at -40^{0}C. The temperature sensor was outfitted with a radiation shield to mitigate the effects of localized radiated and reflected heat. The EchoTemp sensor was connected to EM5 datalogger, also from Decagon Devices and data were recorded every 30 minutes. A second temperature sensor for measuring air temperatures was an intelliRock temperature sensor from Engius, LLC. This sensor has a range from -5^{0}C to +85^{0}C and is accurate to ± 1^{0}C. Data were recorded every 60 minutes by the datalogger which is part of the sensor itself. Many of the intelliRock sensors were provided by NCDOT, Materials and Test Unit. The intelliRock sensors were also used to measure subgrade temperatures, and three sensors were placed at different depths (ranging from ~1” below the surface ~10”) for each site. The moisture sensors used were Echo Model EC-10 from Decagon Devices. When used, only one moisture sensor was placed at a given site, at approximately 2” below the surface. These devices use capacitance measurements to determine changes in the dielectric constant of the surrounding medium, which in turn is related to the amount of moisture in the measurement zone. For example, water has a dielectric constant of about 80 while most soils have a value of about 4. Air has a dielectric constant of 1. At the I-485 site, a rain gauge was installed on site to assist in tracking climatic conditions. The rain gauge used was EchoRain from Decagon Devices. This device is self-emptying and frostproof with a resolution of 1 mm of rain. Instrumentation was installed primarily with a drill and serrated knife. The intelliRock sensors appear as cylindrical rubber stoppers with wires emanating from the top. They have a diameter of about 1 inch and a length of 1.75 inches. For subgrade placement, a hole was drilled to the desired depth, and the sensors were tapped into place with a hammer and then soil was backfilled and compacted over top of the sensor. The lead wires were then extended, labeled and placed out from roadway to a median or shoulder. The overall process is shown in Appendix 3-1 and 3-2. For purposes of broader perspective, Appendices 3-3 through 3-8 show pictures from the site locations.

3.3 Field Data Results and Analysis

The primary output from the field data collection effort is a series of temperature versus time plots which document the relationship between air and subgrade (soil) temperatures, under conditions relevant to NCDOT stabilization projects. These data are then used to back-calculate values of thermal diffusivity for use in predicting subgrade temperatures on the basis of weather forecasts. General results are shown in Tables 3-4 through 3-6, while temperature versus time profiles are presented in Figures 3-2 through 3-11. Notwithstanding an interest in keeping data point markers and scales identical for ease in comparison, different sites are plotted with variation according to that which was appropriate to the number of sensors and observed temperature range. For example, the temperature range was considerably warmer for measurements made in Lincoln County (summer 2009) as compared to the other sites which were generally monitored in the fall and winter.

Table 3-4: Observed average, maximum and minimum temperatures (Fall 2006)

Observed Temperatures (°F)	**Location**			
	U.S. 70, near Clayton, NC ***10/16/06-10/25/06***	**U.S. 421, near Siler City, NC** ***10/11/06-10/23/06***	**I-40, near Greensboro, NC** ***10/25/06-11/5/06***	**I-485, near Charlotte, NC** ***11/30/06-12/10/06***
Average Air Temperature	59.8	58.3	49.9	41.7
Average Soil Temperature at 1.75” Depth	66.7	61.5	56.4	49.8
Maximum Air Temperature	90.6*	81.7	75.4	75.8
Maximum Soil Temperature at 1.75”	89.6	86.0	82.4	73.4
Minimum Air Temperature	31.7	31.6	27.3	16.7
Minimum Soil Temperature at 1.75”	46.4	42.8	37.4	30.2

*A radiation shield was not employed at this site during this time frame, sensors were enclosed in a 5-gallon bucket.

Table 3-5: Observed average, maximum and minimum temperatures (Fall 2008)

Observed Temperatures (^{o}F)	**NC 16, near Weddington, NC**	
	10/31/08-11/4/08	***11/15/08-11/21/08***
Average Air Temperature	55.1	43.4
Average Soil Temperature at 1.0” Depth	55.4	49.4
Maximum Air Temperature	80.6	73.4
Maximum Soil Temperature at 1.0”	69.8	71.6
Minimum Air Temperature	37.4	21.2
Minimum Soil Temperature at 1.0”	44.6	33.8

Table 3-6: Observed average, maximum and minimum temperatures (Summer 2009)

Observed Temperatures (^{o}F)	**NC 16, near Denver, NC (6/10/09-6/18/09)**			
	Control Section 1	**Low Dosage $CaCl_2$**	**High Dosage $CaCl_2$**	**Control Section 2**
Average Air Temperature	76.8	76.8	76.8	76.8
Average Soil Temperature at 1.0” Depth	87.6	89.6	90.3	90.0
Maximum Air Temperature	98.6	98.6	98.6	98.6
Maximum Soil Temperature at 1.0”	125.6	123.8	122.0	127.4
Minimum Air Temperature	64.4	64.4	64.4	64.4
Minimum Soil Temperature at 1.0”	71.6	73.4	73.4	73.4

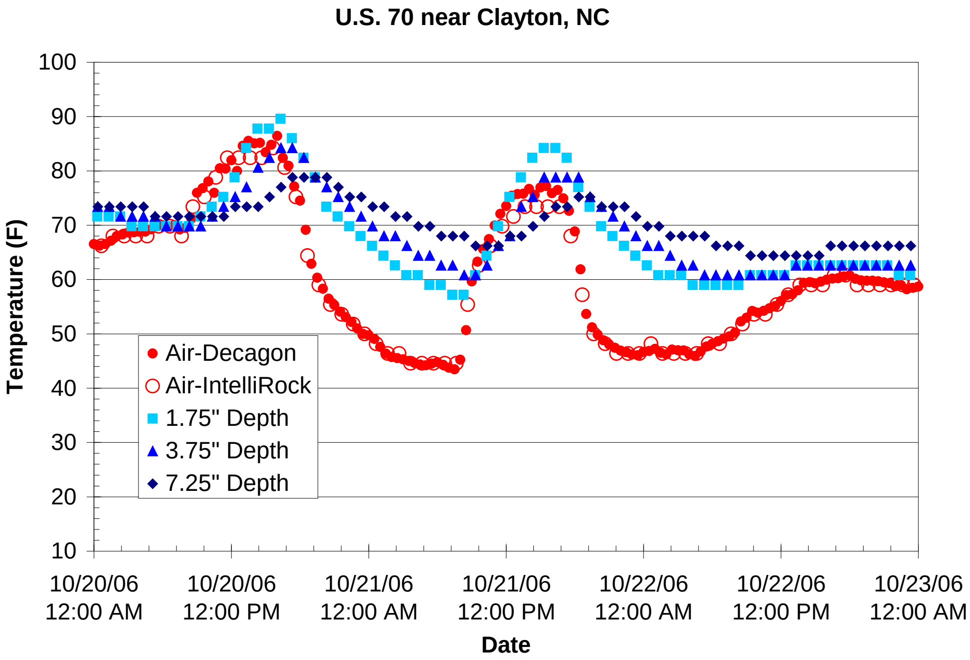


Figure 3-2: Example temperature versus time data for U.S. 70 near Clayton, NC from 10/20/06 – 10/23/06

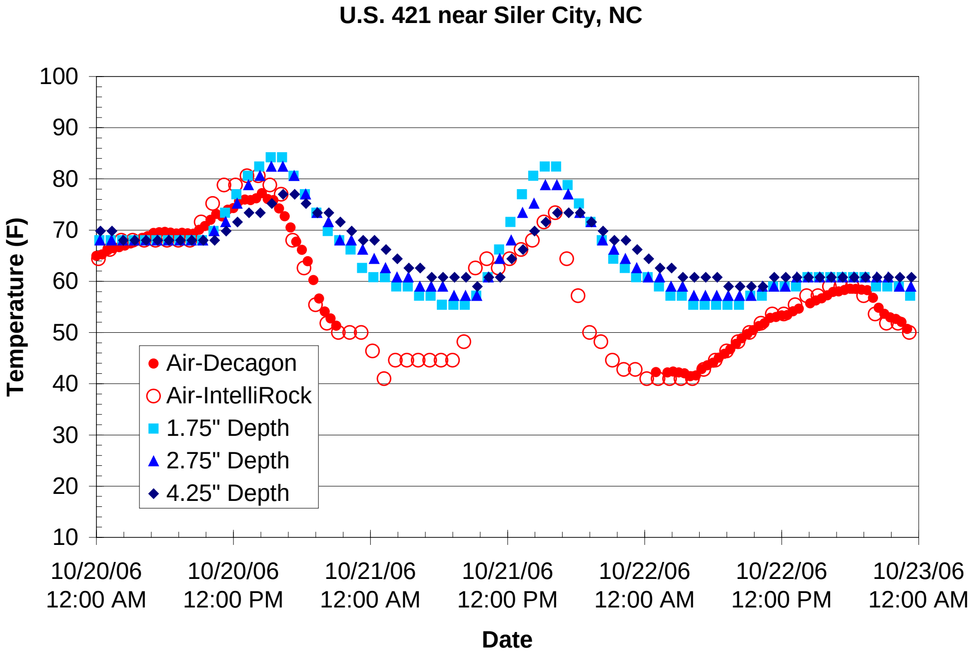


Figure 3-3: Example temperature versus time data for U.S. 421 near Siler City, NC from 10/20/06 – 10/23/06

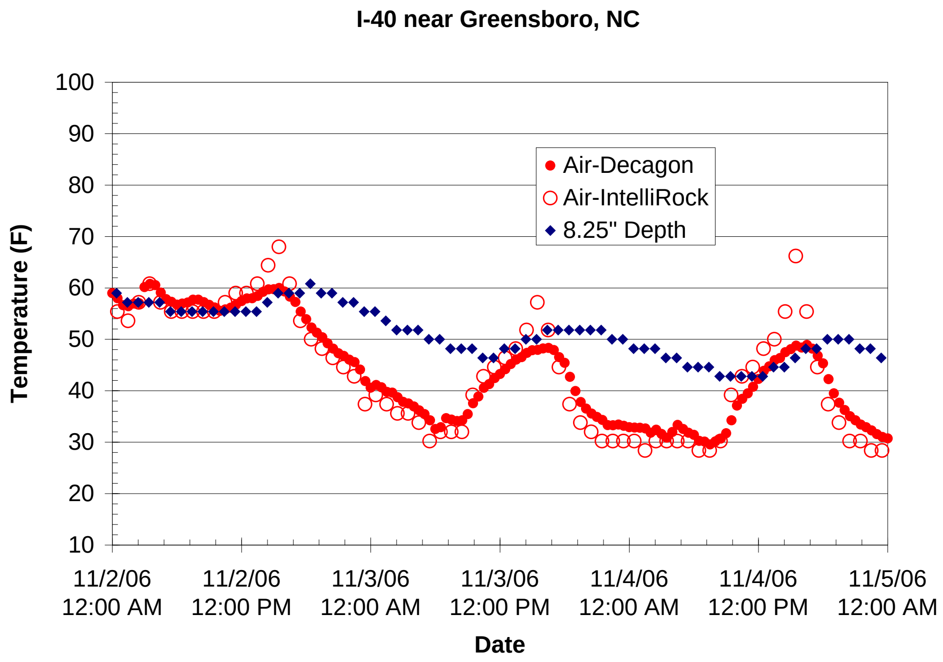


Figure 3-4: Example temperature versus time data for I-40 near Greensboro, NC from 11/2/06 – 11/5/06

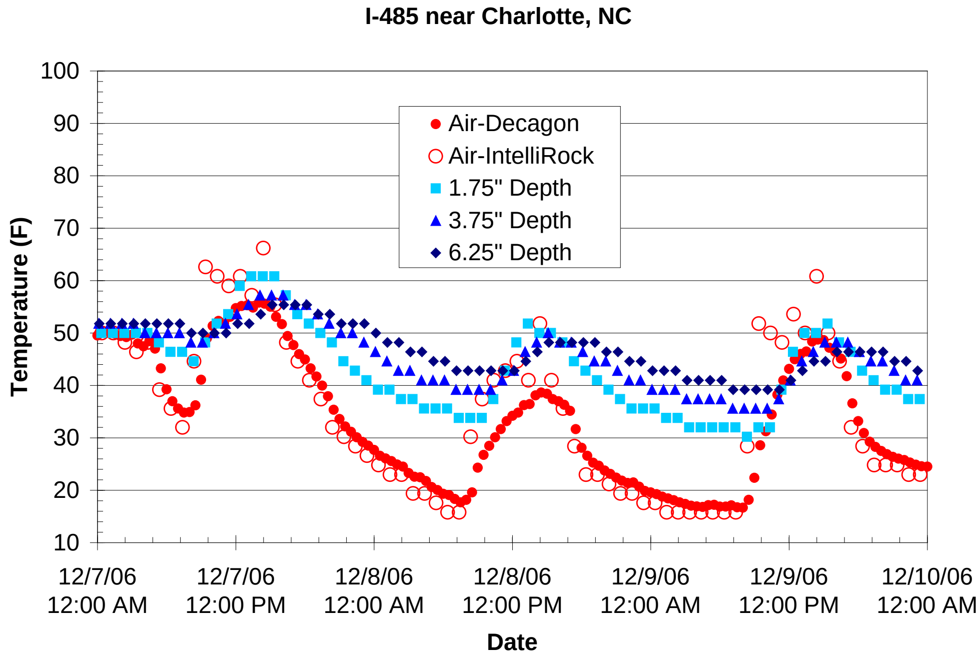


Figure 3-5: Example temperature versus time data for I-485 near Charlotte, NC from 12/7/06 – 12/10/06

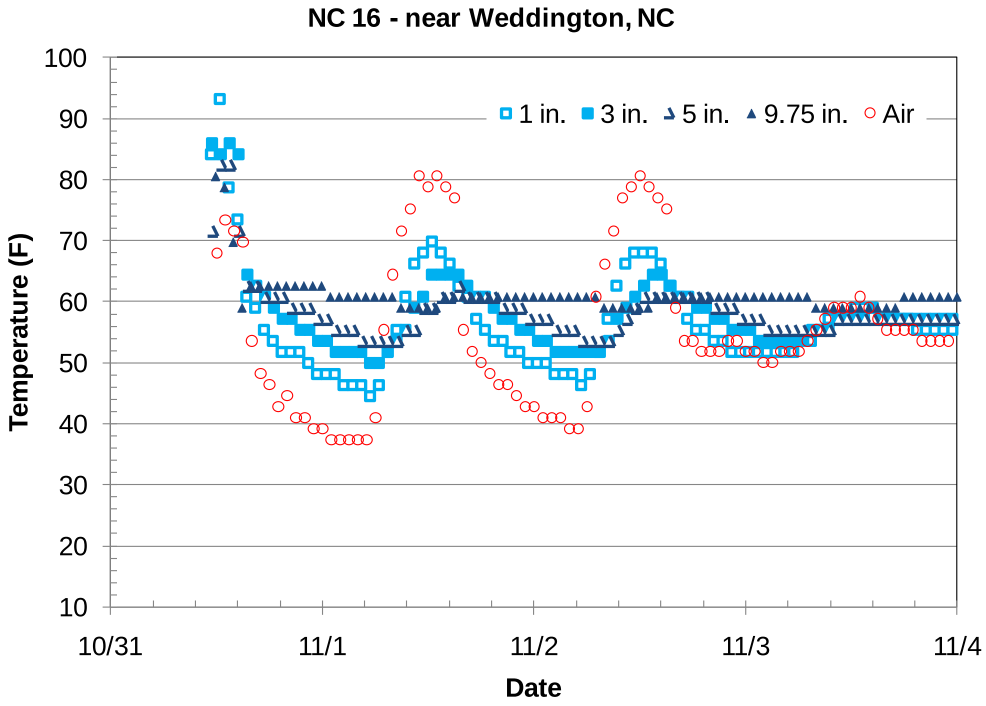


Figure 3-6: Example temperature versus time data for NC 16 near Weddington, NC from 10/31/08 – 11/4/08 (first installation)

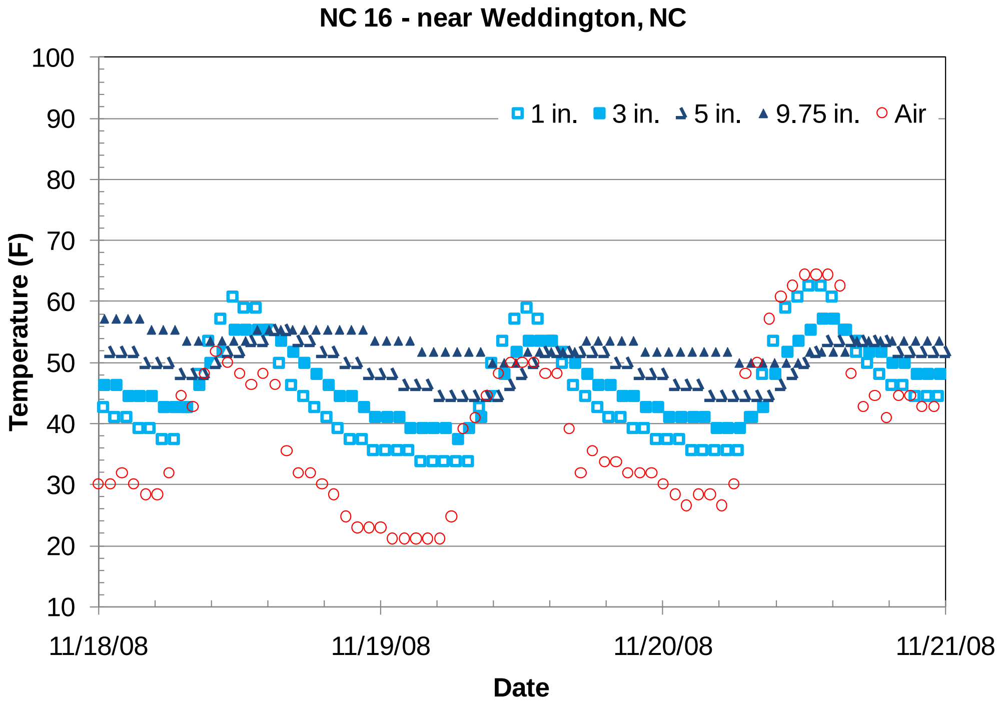


Figure 3-7: Example temperature versus time data for NC 16 near Weddington, NC from 11/18/08 – 11/21/08 (second installation)

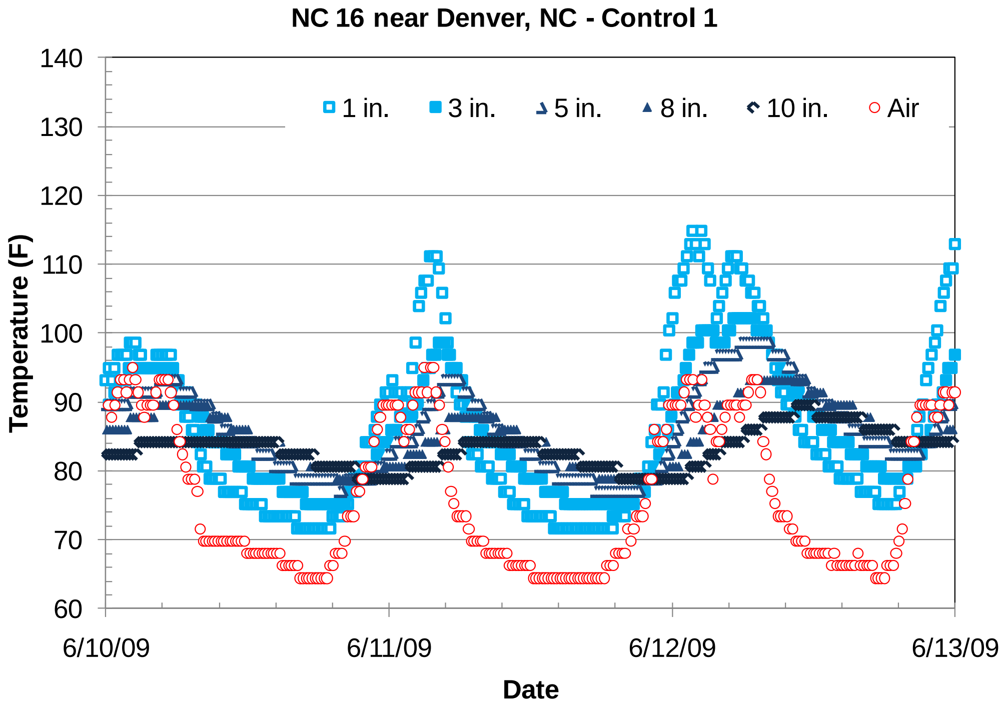


Figure 3-8: Example temperature versus time data for NC 16 near Denver, NC from 6/10/09 – 6/13/09, Control Section 1

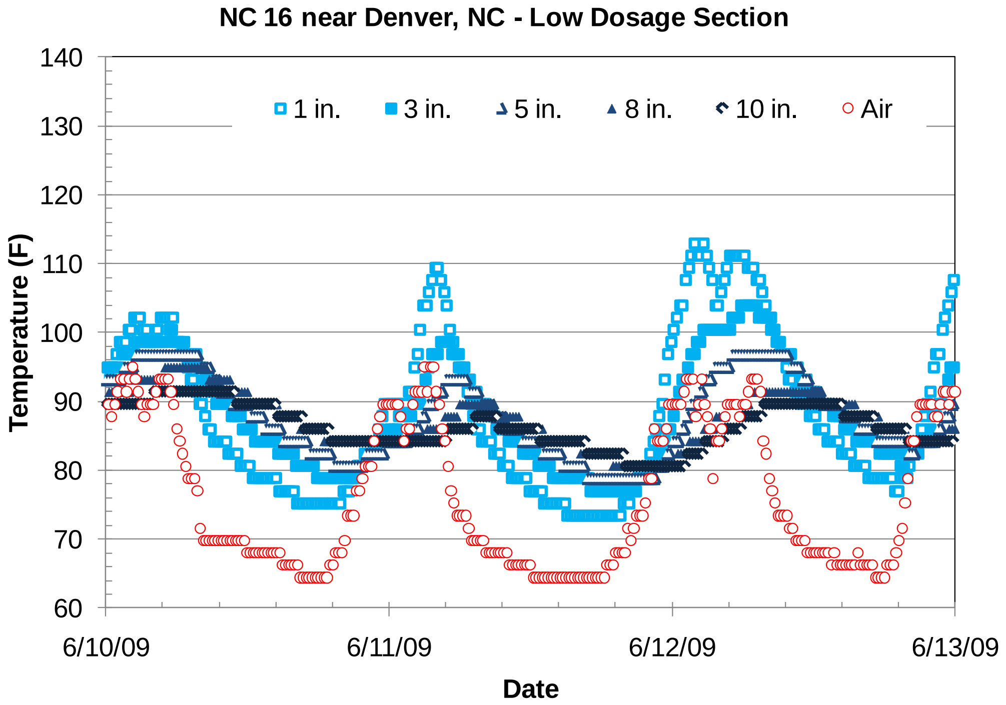


Figure 3-9: Example temperature versus time data for NC 16 near Denver, NC from 6/10/09 – 6/13/09, Low Dosage Section

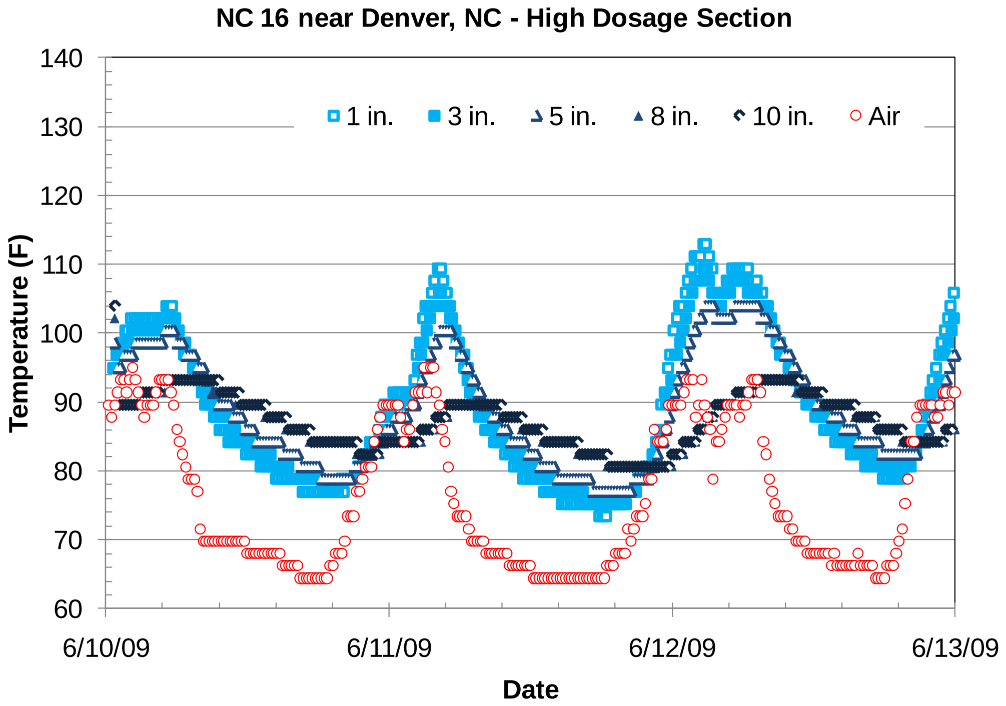


Figure 3-10: Example temperature versus time data for NC 16 near Denver, NC from 6/10/09 – 6/13/09, High Dosage Section

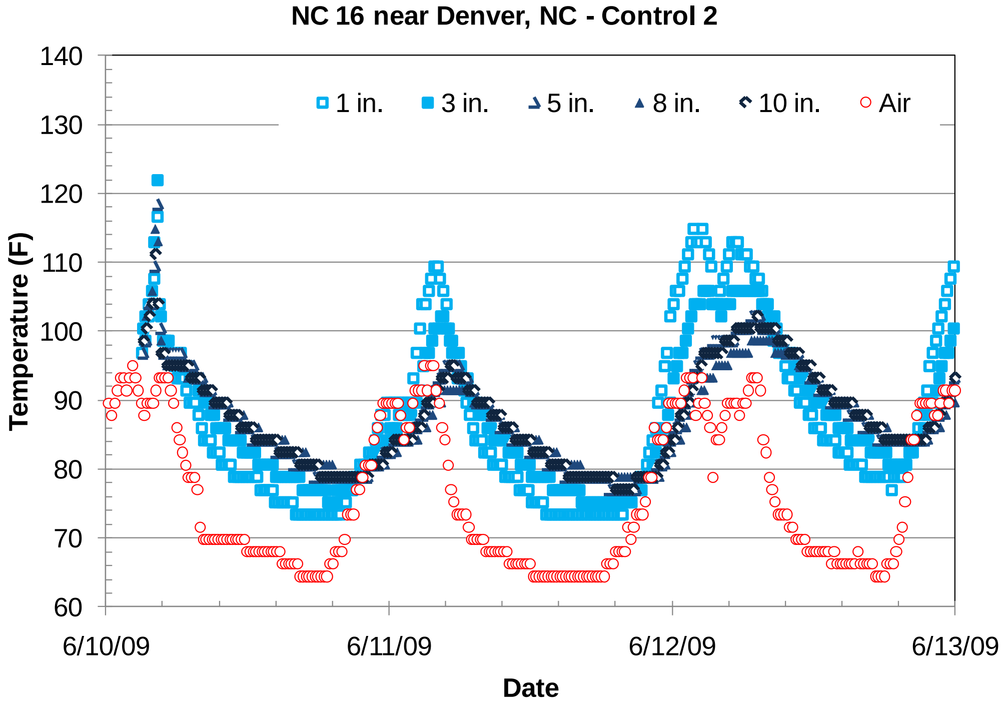


Figure 3-11: Example temperature versus time data for NC 16 near Denver, NC from 6/10/09 – 6/13/09, Control Section 2

As previously noted, two measurements of air temperature were generally taken. Typically, use of the intelliRock sensors without a radiation shield resulted in consistently higher observed maximum temperatures, as compared to the EchoTemp sensors with a radiation shield or as compared to temperatures recorded at other climate stations. For example in the case of I-485, we have the following data for the period of 12/1/06 through 12/10/06:

Table 3-7: Variation between air temperature measurements

Value	**Air Temperatures, ^{o}F, 12/1/06 – 12/10/06**		
	I-485, near Charlotte, NC EchoTemp Sensor with Radiation Shield On-site	**I-485, near Charlotte, NC intelliRock Sensor without Radiation Shield On-site**	**Charlotte Douglas International Airport**
Average	39.2	41.2	42.2
Maximum	75.8	84.2	76
Minimum	16.7	15.8	14

As may be observed, the average and minimum temperatures are all within approximately 2^{o}F, while the observed maximum temperature is considerably higher (+8^{o}F) for the intelliRock sensors without a radiation shield. These values are likely higher due to localized heating of the sensor itself. As such, lack of a radiation shield affects the maximum temperature more so than the average or minimum temperatures. While it is an artifact of the sensor, the higher temperatures observed by the unshielded intelliRock sensors appear to mirror the higher temperatures observed in the subgrade at shallow depths (e.g., <3”). In particular, the subgrade temperatures increase because of direct solar radiation in addition to the ambient air temperatures, and this is frequently observed where the maximum temperature at shallow depths is higher than the temperature noted by the shielded EchoTemp sensors (or as measured by a nearby airport or other climate monitoring station). This surface heating effect was also documented by comparing the subsurface temperature in stabilized subgrade versus that of the surrounding native, grass-covered soil at NC 16 near Weddington in Union County, as shown in Figure 3-12.

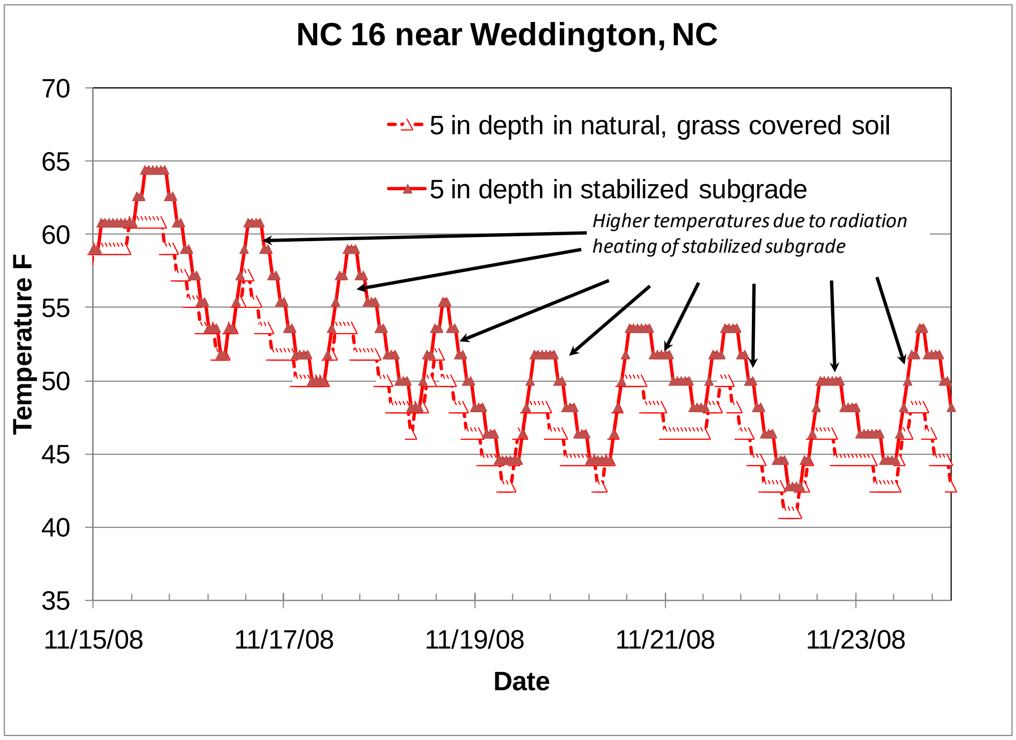


Figure 3-12: Influence of surface heating of exposed stabilized subgrade versus natural grass covered soil.

Notwithstanding the above noted surface heating, the effect of increasing depth is a reduction in overall temperature amplitude, i.e., the subgrade temperature fluctuates less than the air temperature.

Not all of the sensors performed as expected. For example, there were times when the battery in the dataloggers failed prematurely, impacting the Decagon sensors (moisture content and shielded air temperature). Likewise the intelliRock sensors failed to report all of their data for the I-40/Greensboro site, and so only the depth at 8.25 inches is reported herein. And in the case of Lincoln County, it is not clear that the sensors were installed at the precise depth reported for the 8 and 10 inch depths in Control Section 2. In any case, the collected data provide insight into the relationship between air and subgrade temperature. This relationship is explored further by considering the amplitude attenuation with depth. For example, consider Figure 3-13, which shows the temperature profile for three different dates.

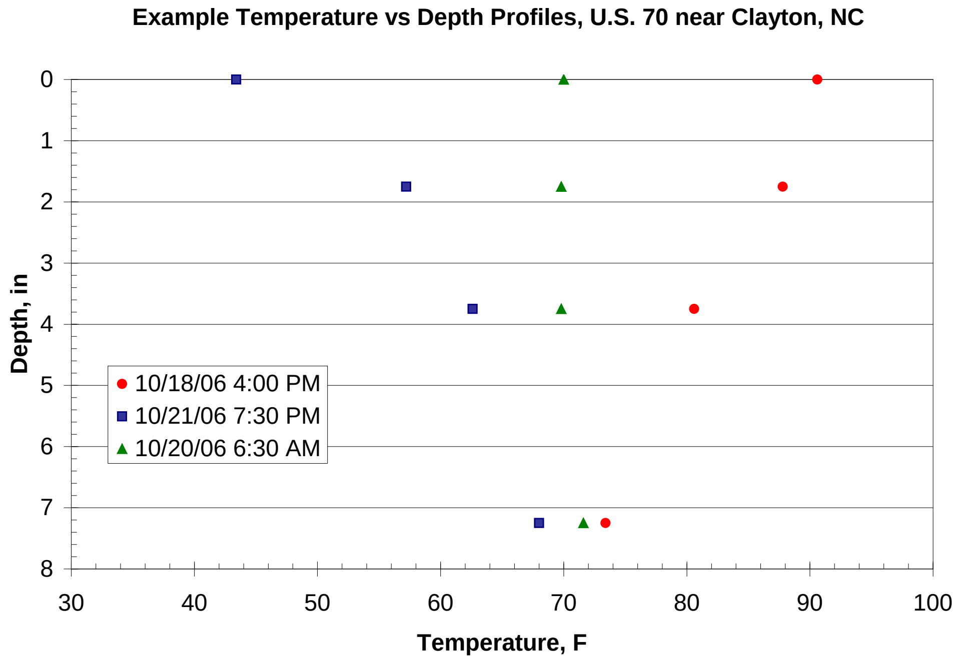


Figure 3-13: Temperature profiles at under warming (10/18), cooling (10/21) and neutral conditions (10/20) at U.S. 70 near Clayton, NC

As noted above, these three profiles reflect the behavior under warming, cooling and temperature-neutral conditions. In essence, the data tend to follow a diurnal change in temperature, which may be modeled as a sinusoidal function. This analysis can be extended to include more data, by averaging the amplitudes at various depths, as shown in Tables 3-8, 3-9 and 3-10.

Table 3-8: Temperature amplitude with depth and location (Fall 2006)

Depth	**Value**	**Temperatures (^{o}F)**			
		U.S. 70, near Clayton, NC ***10/17/06-10/24/06***	**U.S. 421, near Siler City, NC** ***10/12/06-10/22/06***	**I-40, near Greensboro, NC** ***10/26/06-11/4/06***	**I-485, near Charlotte, NC** ***12/1/06-12/9/06***
Air*	**Average Amplitude**	**13.5**	**11.7**	**12.0**	**13.4**
	Standard Deviation	4.4	2.8	2.9	2.3
	Minimum Amplitude	6.4	7.5	7.9	10.5
	Maximum Amplitude	18.2	17.2	16.0	16.1

Depth	**Value**	**Temperatures (^{o}F)**			
		U.S. 70, near Clayton, NC ***10/17/06-10/24/06***	**U.S. 421, near Siler City, NC** ***10/12/06-10/22/06***	**I-40, near Greensboro, NC** ***10/26/06-11/4/06***	**I-485, near Charlotte, NC** ***12/1/06-12/9/06***
Depth 1 (1.75 in)	**Average Amplitude**	**8.9**	**8.7**	**12.9*****	**9.9**
	Standard Deviation	4.3	3.3	2.8***	1.3
	Minimum Amplitude	1.8	2.7	8.1***	8.1
	Maximum Amplitude	13.5	13.5	16.2***	11.7
Depth 2 (2.75 – 4.75 in)**	**Average Amplitude**	**6.0**	**7.0**	********	**5.5**
	Standard Deviation	2.5	2.6	****	0.9
	Minimum Amplitude	1.8	1.8	****	4.5
	Maximum Amplitude	9.0	10.8	****	7.2
Depth 3 (4.25-8.25 in)**	**Average Amplitude**	**3.7**	**4.9**	**3.4**	**3.8**
	Standard Deviation	1.0	1.6	1.1	0.8
	Minimum Amplitude	1.8	2.7	0.9	2.7
	Maximum Amplitude	4.5	7.2	4.5	4.5

Notes:

*Air as measured from EchoTemp sensor with radiation shielding. Depths 1, 2 and 3 are as listed in Table 5-1.

Data range is compressed slightly (as indicated) from Table 5-1 to include only full 24 hour periods.

** Actual depths 1, 2 and 3 for each site are as listed in Table 3-1.

***Sensor recorded data that appears out of phase with time, but perhaps accurate with actual temperature.

****No data, sensor failed to report any data

Table 3-9: Temperature amplitude with depth and location (Fall 2008)

Depth	Value	Temperatures (^{o}F) NC 16, near Weddington, NC	
		10/31/08-11/4/08	***11/15/08-11/21/08***
Air	**Average Amplitude**	**15.9**	**15.4**
	Standard Deviation	9.1	1.7
	Minimum Amplitude	5.4	14.4
	Maximum Amplitude	21.6	18.9
Depth 1 (1 in)	**Average Amplitude**	**9.0**	**12.6**
	Standard Deviation	4.8	0.7
	Minimum Amplitude	3.6	11.7
	Maximum Amplitude	12.6	13.5
Depth 2 (3 in)	**Average Amplitude**	**5.4**	**7.8**
	Standard Deviation	2.4	0.7
	Minimum Amplitude	2.7	7.2
	Maximum Amplitude	7.2	9.0
Depth 3 (5 in)	**Average Amplitude**	**3.3**	**4.0**
	Standard Deviation	1.4	0.5
	Minimum Amplitude	1.8	3.6
	Maximum Amplitude	4.5	4.5
Depth 4 (9.75 in)	**Average Amplitude**	**1.2**	**1.5**
	Standard Deviation	0.5	0.4
	Minimum Amplitude	0.9	0.9
	Maximum Amplitude	1.8	1.8

Table 3-10: Temperature amplitude with depth and location (Summer 2009)

Depth	**Value**	**NC 16, near Denver, NC (6/10/09-6/18/09)**			
		Control Section 1	**Low Dosage $CaCl_2$**	**High Dosage $CaCl_2$**	**Control Section 2**
Air	**Average Amplitude**	**13.9**	**13.9**	**13.9**	**13.9**
	Standard Deviation	2.3	2.3	2.3	2.3
	Minimum Amplitude	9.9	9.9	9.9	9.9
	Maximum Amplitude	16.2	16.2	16.2	16.2
Depth 1 (1.0 in)	**Average Amplitude**	**16.5**	**15.8**	**15.1**	**18.4**
	Standard Deviation	5.2	4.7	4.8	5.9
	Minimum Amplitude	8.1	6.3	5.4	4.5
	Maximum Amplitude	24.3	21.6	20.7	25.2
Depth 2 (3.0 in)	**Average Amplitude**	**10.2**	**9.9**	**13.2**	**14.6**
	Standard Deviation	4.4	3.5	4.2	5.2
	Minimum Amplitude	2.7	3.6	4.5	4.5
	Maximum Amplitude	17.1	15.3	18.0	23.4
Depth 3 (5.0 in)	**Average Amplitude**	**7.2**	**7.0**	**10.8**	**10.5**
	Standard Deviation	3.4	2.5	3.2	4.5
	Minimum Amplitude	1.8	2.7	4.5	3.6
	Maximum Amplitude	11.7	9.9	15.3	19.8
Depth 4 (8.0 in)	**Average Amplitude**	**4.8**	**3.9**	**4.3**	**9.8**
	Standard Deviation	2.2	1.7	1.7	3.6
	Minimum Amplitude	1.8	0.9	0.9	3.6
	Maximum Amplitude	8.1	5.4	6.3	16.2

Depth	**Value**	**NC 16, near Denver, NC (6/10/09-6/18/09)**			
		Control Section 1	**Low Dosage $CaCl_2$**	**High Dosage $CaCl_2$**	**Control Section 2**
Depth 5 (10.0 in)	**Average Amplitude**	**3.2**	**3.3**	**4.3**	**8.7**
	Standard Deviation	1.4	1.2	1.3	4.2
	Minimum Amplitude	1.8	1.8	1.8	2.7
	Maximum Amplitude	5.4	5.4	5.4	18

Temperature profiles can be constructed to better reflect the typical temperature profile throughout the study period. Using data from Tables 3-8 through 3-10, along with additional average temperatures at depth, Figures 3-14 through 3-23 were developed for each of the sites.

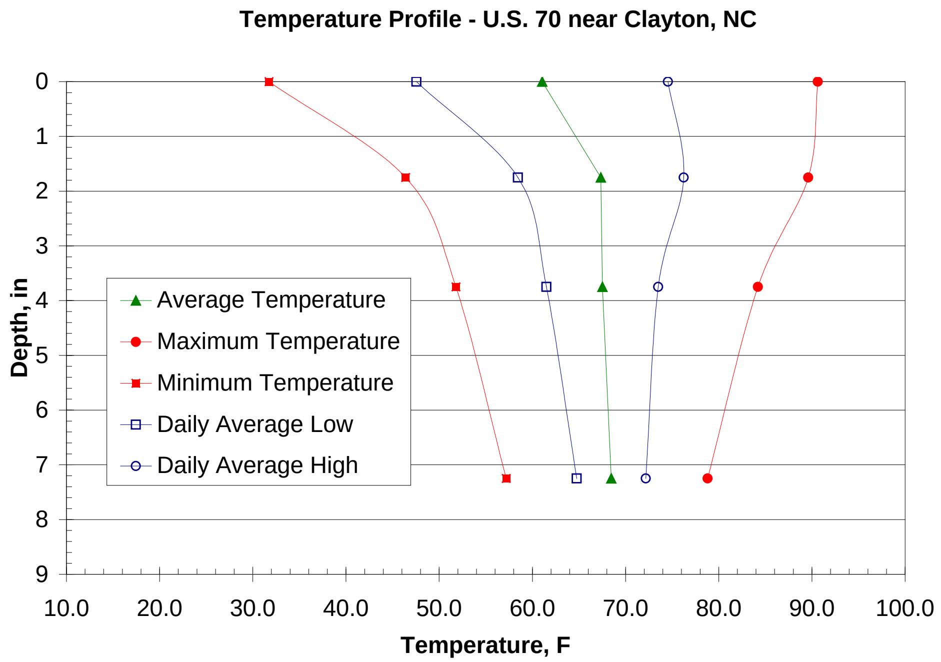


Figure 3-14: Temperature profile at U.S. 70 near Clayton, NC for 10/17/06 – 10/24/06

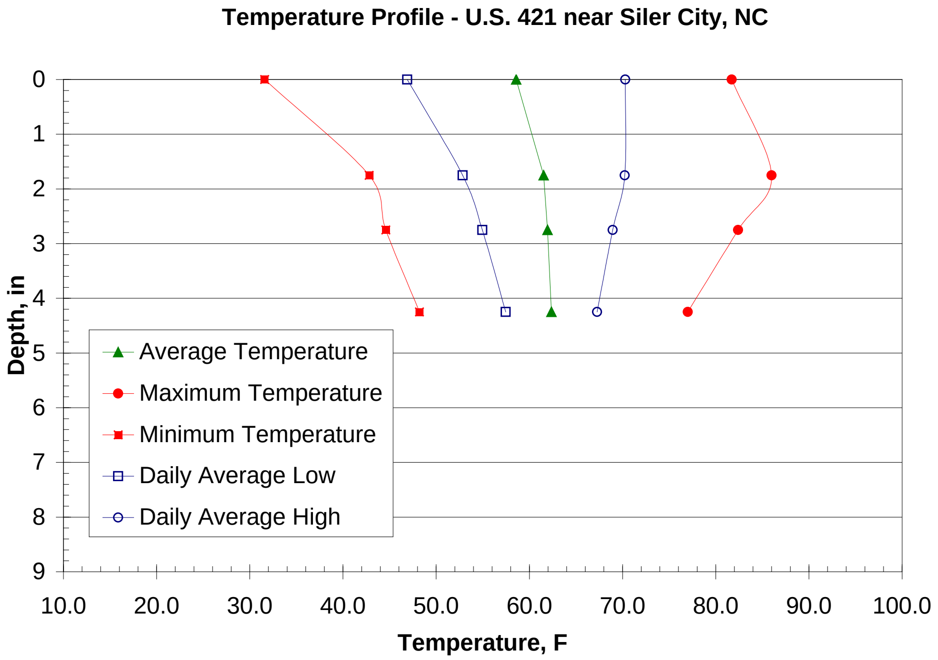


Figure 3-15: Temperature profile at U.S. 421 near Siler City, NC for 10/12/06 – 10/22/06.

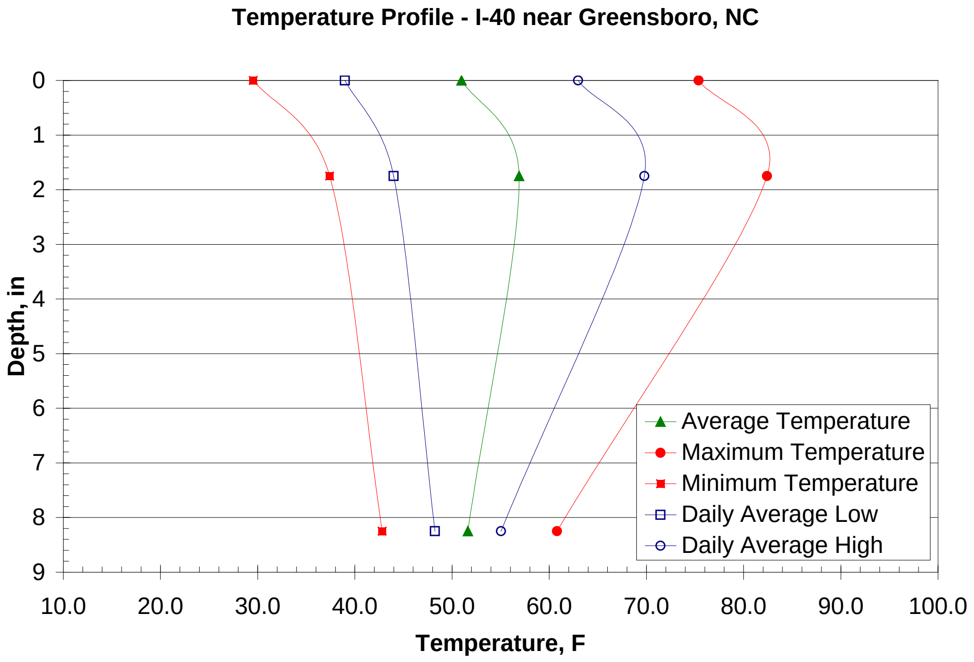


Figure 3-16: Temperature profile at I 40 near Greensboro, NC for 10/26/06 – 11/4/06

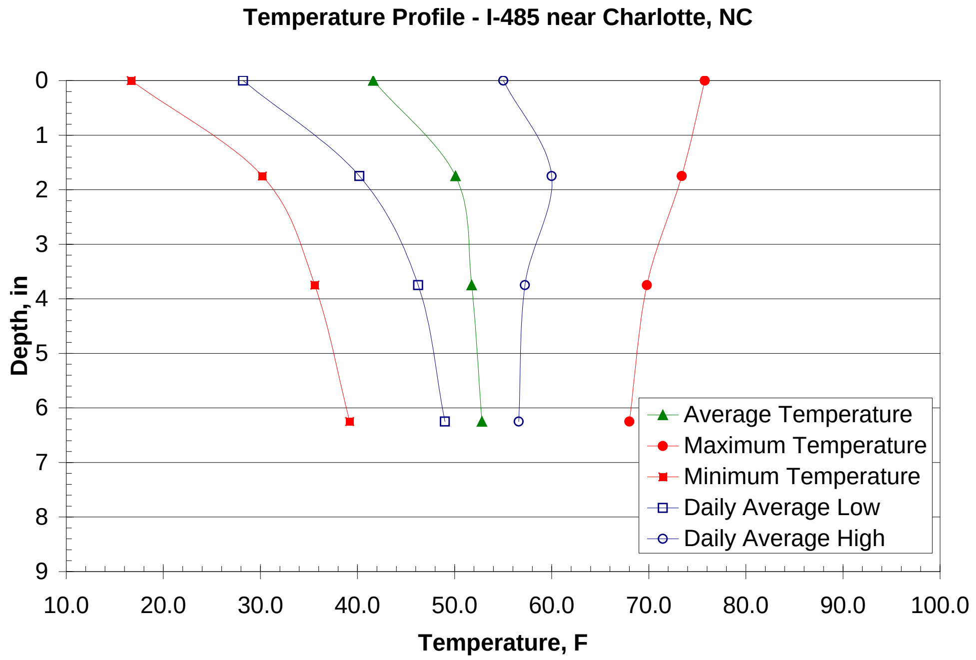


Figure 3-17: Temperature profile at I 485 near Charlotte, NC for 12/1/06 – 12/9/06

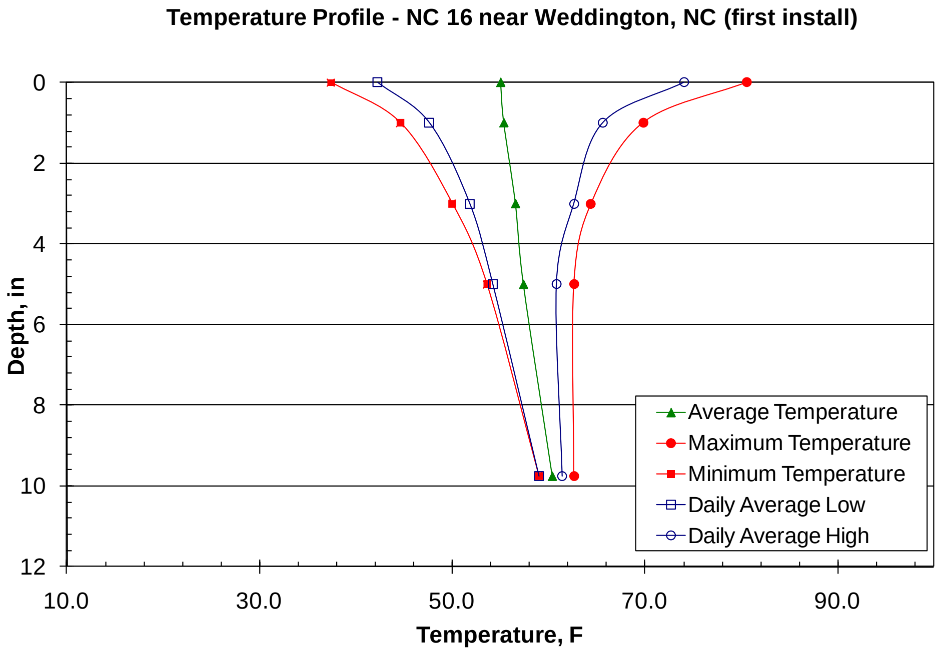


Figure 3-18: Temperature profile at NC 16 near Weddington, NC for 10/31/08 – 11/4/08

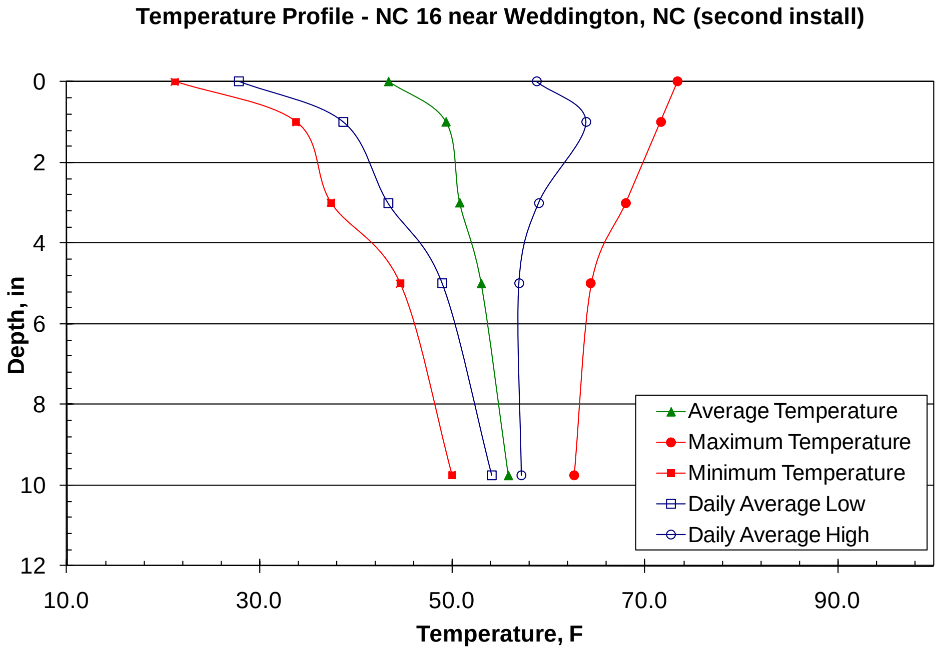


Figure 3-19: Temperature profile at NC 16 near Weddington, NC for 11/15/08 – 11/21/08

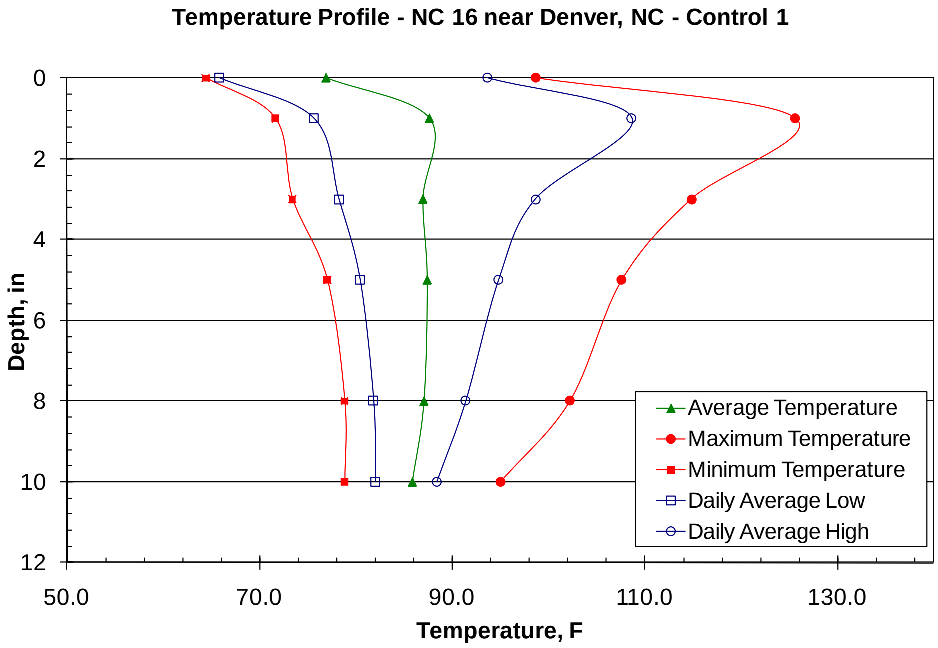


Figure 3-20: Temperature profile at NC 16 near Denver, NC for 6/10/09 – 6/18/09, Control Section 1

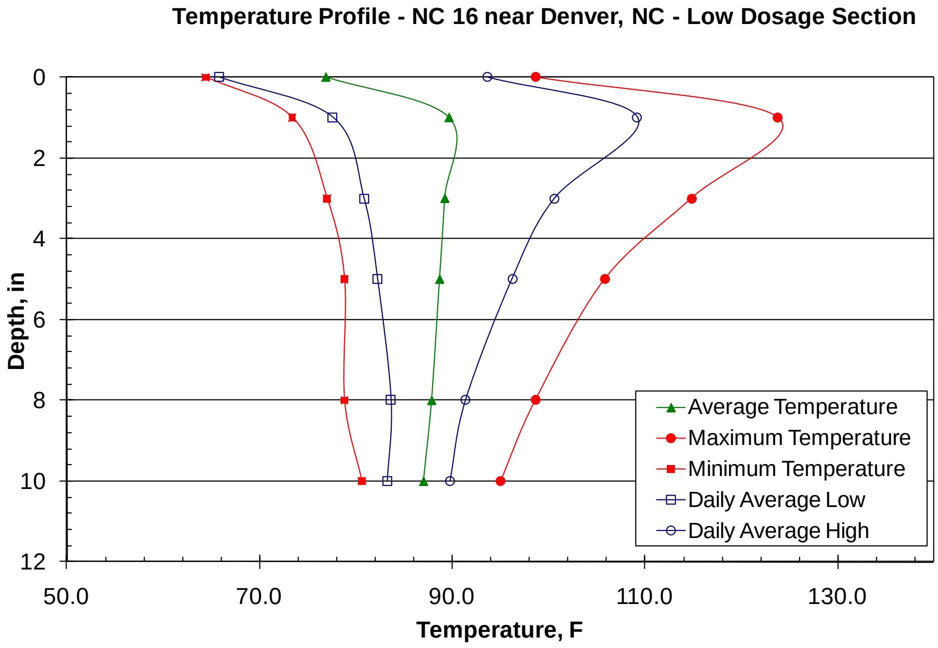


Figure 3-21: Temperature profile at NC 16 near Denver, NC for 6/10/09 – 6/18/09, Low Dosage Section

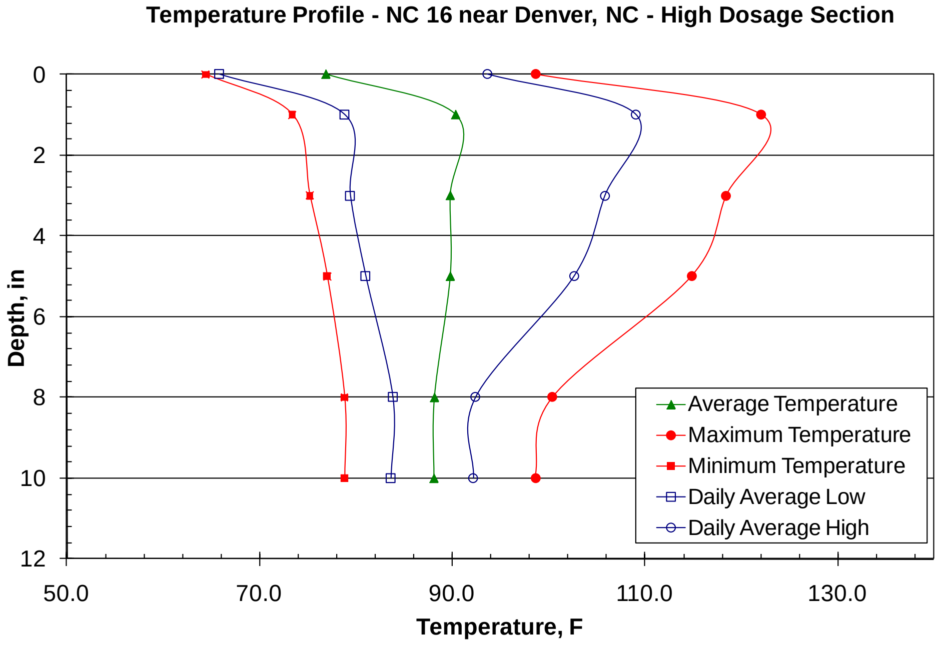


Figure 3-22: Temperature profile at NC 16 near Denver, NC for 6/10/09 – 6/18/09, High Dosage Section

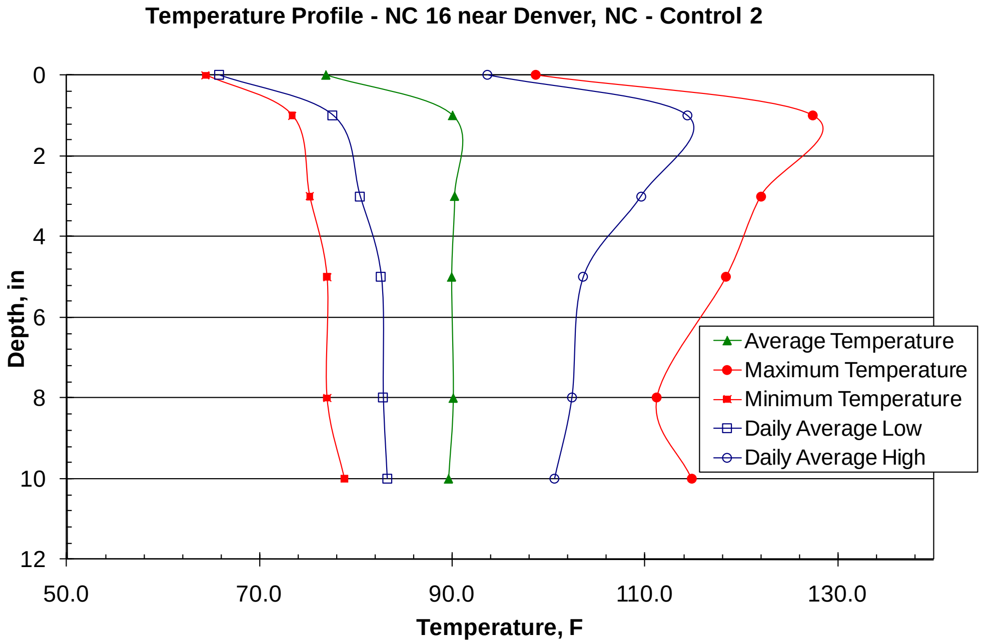


Figure 3-23: Temperature profile at NC 16 near Denver, NC for 6/10/09 – 6/18/09, Control Section 2

3.4 Model Development

Although each of the sites experienced different thermal regimes, the above data can be used to determine material specific values, including thermal diffusivity and thermal conductivity. This information is in turn useful when developing a predictive relationship. In particular, the following relationships may be used to determine thermal diffusivity (Andersland and Ladanyi 2004):

$$\alpha = \frac{p}{4\pi}\left(\frac{z_2 - z_1}{t_2 - t_1}\right)^2 \qquad \text{Eq. 3-1}$$

Or:

$$\alpha = \frac{p}{\pi}\left(\frac{z_2 - z_1}{\ln\left(\frac{A_{z1}}{A_{z2}}\right)}\right)^2 \qquad \text{Eq. 3-2}$$

Where:

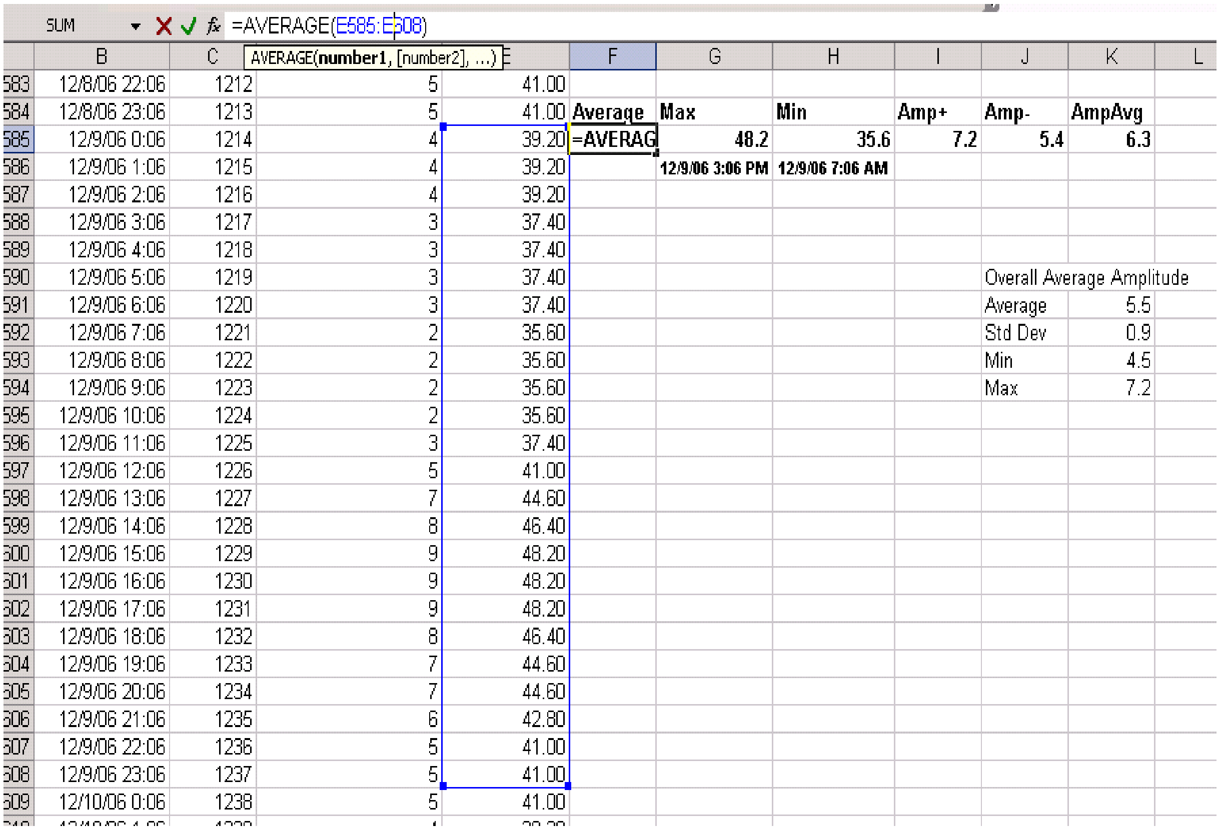
α = thermal diffusivity = k / c ρ
k = thermal conductivity
c = heat capacity
ρ = bulk density
p = period (e.g., 24 hours)
z_1,z_2 = depth at location 1, 2
t_1,t_2 = time lag at location 1, 2
A_{z1},A_{z2} = temperature amplitude at location 1, 2

The data collected allow for either of the above computations to be made. Thermal diffusivity values were obtained for each of the four sites by the use of Eq. 3-2, because temperature amplitudes (A_{z1}, A_{z2}) are more readily and accurately available from the collected data. The amplitude at a given depth (z_1, z_2) is calculated by averaging hourly data over a 24 hour period to compute mean, maximum and minimum values. The positive amplitude is taken as the maximum temperature minus the mean, while the negative amplitude is taken as the mean minus the minimum temperature. An overall daily amplitude is then computed as the average of these two values. An example of this calculation is given by the screen capture of the excel spreadsheet, presented as Figure 3-24.

This calculation is repeated for as much of the data as available for a given site, and then all of these daily amplitude values are averaged to compute an overall value for the site. Note that determination of thermal diffusivity via Eq. 3-2 requires two amplitude values, while the collected data, in general, provide four amplitude values, i.e., temperature measurements were made at the surface and then at three different levels of depth. Therefore, the thermal diffusivity can be calculated on the basis of the various combinations:

1. Air – Depth 1
2. Air – Depth 2
3. Air – Depth 3
4. Depth 1 – Depth 2
5. Depth 1 – Depth 3
6. Depth 2 – Depth 3
7. And so forth for the various depths available at different sites

However in some cases the air temperatures were provided only by intelliRock sensors which are subject to artifact radiation heating. And in other cases the actual depth of sensor placement was questioned. In these instances and in any other where the calculated value of thermal diffusivity appeared erroneous, the value was disregarded. Table 3-11 presents a summary of the thermal diffusivities while Figure 3-25 presents them in terms of bar chart.



SUM ▼ ✕ ✓ fx =AVERAGE(E585:E608)

	B	C	AVERAGE(**number1**, [number2], ...)	E	F	G	H	I	J	K	L
583	12/8/06 22:06	1212	5	41.00							
584	12/8/06 23:06	1213	5	41.00	**Average**	**Max**	**Min**	**Amp+**	**Amp-**	**AmpAvg**	
585	12/9/06 0:06	1214	4	39.20	**=AVERAG**	**48.2**	**35.6**	**7.2**	**5.4**	**6.3**	
586	12/9/06 1:06	1215	4	39.20		**12/9/06 3:06 PM**	**12/9/06 7:06 AM**				
587	12/9/06 2:06	1216	4	39.20							
588	12/9/06 3:06	1217	3	37.40							
589	12/9/06 4:06	1218	3	37.40							
590	12/9/06 5:06	1219	3	37.40					Overall Average Amplitude		
591	12/9/06 6:06	1220	3	37.40					Average	5.5	
592	12/9/06 7:06	1221	2	35.60					Std Dev	0.9	
593	12/9/06 8:06	1222	2	35.60					Min	4.5	
594	12/9/06 9:06	1223	2	35.60					Max	7.2	
595	12/9/06 10:06	1224	2	35.60							
596	12/9/06 11:06	1225	3	37.40							
597	12/9/06 12:06	1226	5	41.00							
598	12/9/06 13:06	1227	7	44.60							
599	12/9/06 14:06	1228	8	46.40							
600	12/9/06 15:06	1229	9	48.20							
601	12/9/06 16:06	1230	9	48.20							
602	12/9/06 17:06	1231	9	48.20							
603	12/9/06 18:06	1232	8	46.40							
604	12/9/06 19:06	1233	7	44.60							
605	12/9/06 20:06	1234	7	44.60							
606	12/9/06 21:06	1235	6	42.80							
607	12/9/06 22:06	1236	5	41.00							
608	12/9/06 23:06	1237	5	41.00							
609	12/10/06 0:06	1238	5	41.00							

Figure 3-24: Example calculation of temperature amplitude for a given 24 hour period.

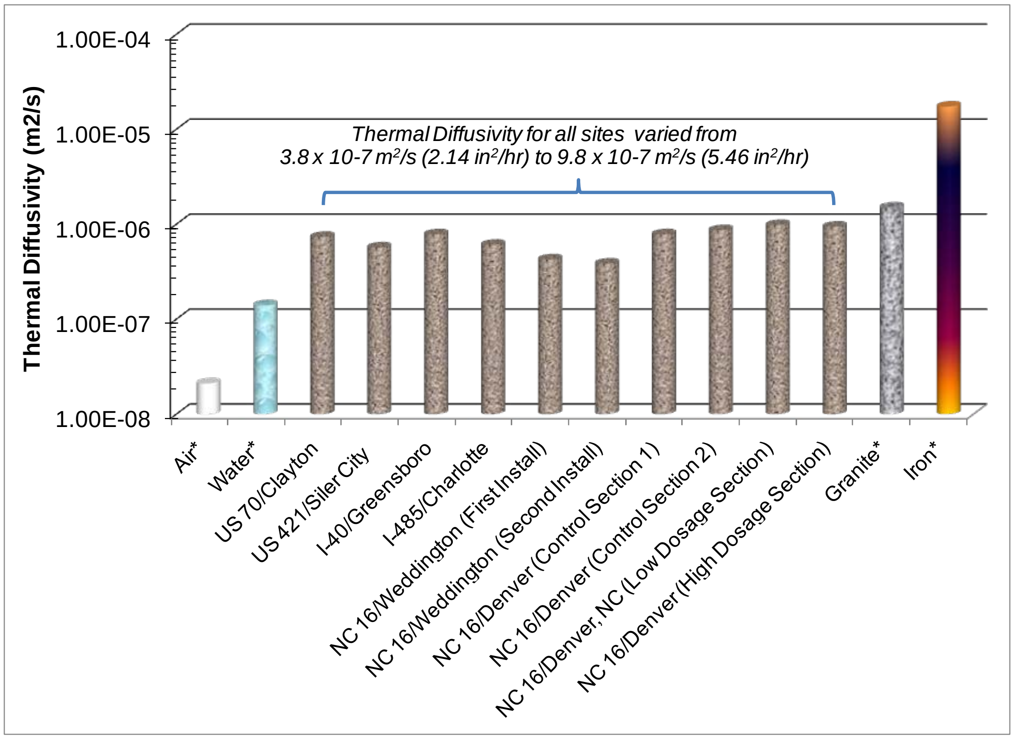


Figure 3-25: Comparison of thermal diffusivities calculated from field monitoring of stabilized subgrade with referenced in Table 3-11.

Table 3-11: Comparison of calculated thermal diffusivity values for each site, along with values for common materials from literature

Material or Location of Stabilized Subgrade	**Thermal Diffusivity**	
	m^2/s	**in^2/hr**
Air*	2.10E-08	0.12
Water*	1.40E-07	0.78
US 70/Clayton (Soil-Lime)	7.34E-07	4.10
US 421/Siler City (Soil-Lime)	5.68E-07	3.17
I-40/Greensboro (Soil-Cement)	7.81E-07	4.36
I-485/Charlotte (Soil-Cement)	6.07E-07	3.38
NC 16/Weddington (First Install) (Soil-Lime)	4.25E-07	2.37
NC 16/Weddington (Second Install) (Soil-Lime)	3.84E-07	2.14
NC 16/Denver (Control Section 1) (Soil-Cement)	7.82E-07	4.36
NC 16/Denver (Control Section 2) (Soil-Cement)	8.64E-07	4.82
NC 16/Denver, NC (Low Dosage Section) (Soil-Cement)	9.78E-07	5.46
NC 16/Denver (High Dosage Section) (Soil-Cement)	9.4E-07	5.25
Granite*	1.50E-06	8.37
Iron*	1.73E-05	96.50

*For comparison, from Table 2-13 in *Frozen Ground Engineering*, Andersland and Ladanyi, 2004, actual data from Terzaghi (1952) and De Vries (1966)

Having obtained site-specific values of thermal diffusivity, the following relationship to predict the temperature as a function of time or depth can be employed:

$$T_{z,t} = T_m + A_s \exp\left(-z\sqrt{\frac{\pi}{\alpha p}}\right)\sin\left(\frac{2\pi t}{p} - z\sqrt{\frac{\pi}{\alpha p}}\right) \qquad \text{Eq. 3-3}$$

Where:

$T_{z,t}$ = temperature as a function of specified time (t) and depth (z)
T_m = average air temperature
A_s = surface temperature amplitude

The above equation presumes a sinusoidal change in air/surface temperature whereby the minimum occurs at time t=0, or for our period of 24 hours, at 12:00 AM. However, the lowest air temperatures typically occur just before dawn. Eq. 6-3 can be modified to incorporate this time lag by introducing another cosine term, given as:

$$T_{z,t} = T_m + A_s \exp\left(-z\sqrt{\frac{\pi}{\alpha p}}\right)\cdot\left(-\cos\left(\frac{2\pi t}{p} - \frac{2\pi\phi}{p} - z\sqrt{\frac{\pi}{\alpha p}}\right)\right) \qquad \text{Eq. 3-4}$$

In Eq. 3-4, the soil temperature varying with different depth and time can be predicted with the aid of the surface air temperature and thermal diffusivity and is an approach that has been widely used (Elimoel 2004, Orlando 2004, Tessy 2002, Coskun et al. 2004, Gao 2007).

Notwithstanding its ease of application, Eq. 3-4, retains several attributes that limit its accuracy. Firstly, it may be observed that Eq. 3-4 implies diurnal variations in temperature are symmetrical with time, e.g., for 24 hour periods. However, the rate of heating is generally higher than the rate of cooling for a given diurnal change in temperature. Secondly, the use of this model across multiple days gives rise to a step function disconnect from one day to the next according to the average temperature for a given period. As such, the predicted soil temperature is not continuous through time. Previous studies applied this model for a relatively short period of 1-2 days (Tessy 2002, Cheng et al. 2007), in which this error is not obvious. Thirdly, Eq. 3-4 assumes the temperature amplitude is attenuated with depth according to a fixed negative exponential function over multiple days. This requires the maximum soil temperature to be less than the maximum air temperature. While this is often the case, there are times when the soil temperature is greater than the air temperature. Finally, there is no mechanism to readily adjust the amplitude in response to moisture content variations.

While the foregoing deficiencies are inherent with default application of Eq. 3-4, several improvements can be made to increase the accuracy dramatically while keeping input requirements at a minimum. These are discussed as follows.

3.4.1 Model improvement

3.4.1.1 Symmetric to Asymmetric

The variation in soil temperature is not symmetric across any given period (e.g., diurnal, seasonal). Usually, the lowest observed temperature occurs just before dawn, while the highest temperature occurs in the early afternoon, as dictated by the minimum and maximum levels of solar radiation, respectively. So an hourly temperature model whose period is 24 hours is not a simple sinusoidal variation as described in Eq. 3-4. An improved method is to divide the temperature-time relationship according to successive maximum and minimum values. This results in a more uniform fit between modeled and actual temperatures. The first part is from 0:00 (12:00 AM) to t_{min} (time of the lowest temperature between the two adjacent days), the second is from t_{min} to t_{max} (time of the highest temperature between the two adjacent days), the third is from t_{max} to 24:00 (12:00 PM). The sinusoidal sub function in the raw model can be designed as sub function K(t), as shown in Eq. 3-5 and 3-6.

$$K(t)=\left|\begin{array}{ll} f_1(t) & \text{if } \left(t_{\min}-\frac{z}{2}\sqrt{\frac{p}{\pi\alpha}} \le \operatorname{mod}\left[\left(t-\frac{z}{2}\sqrt{\frac{p}{\pi\alpha}}\right),24\right] < t_{\min}\right) \\ f_2(t) & \text{if } \left(t_{\min} \le \operatorname{mod}\left[\left(t-\frac{z}{2}\sqrt{\frac{p}{\pi\alpha}}\right),24\right] < t_{\max}\right) \\ f_3(t) & \text{if } \left(t_{\max} \le \operatorname{mod}\left[\left(t-\frac{z}{2}\sqrt{\frac{p}{\pi\alpha}}\right),24\right] < 24\right) \end{array}\right. \qquad \text{Eq.3-5}$$

Where:

$$f_1(t)=-\cos\left(2\cdot\pi\cdot\frac{\operatorname{mod}\left[\left(t-\frac{z}{2}\sqrt{\frac{p}{\pi\alpha}}\right),24\right]-t_{\min}}{p_0}\right)$$

$$f_2(t)=-\cos\left(2\cdot\pi\cdot\frac{\operatorname{mod}\left[\left(t-\frac{z}{2}\sqrt{\frac{p}{\pi\alpha}}\right),24\right]-t_{\min}}{p_1}\right)$$

$$f_3(t)=\cos\left(2\cdot\pi\cdot\frac{\operatorname{mod}\left[\left(t-\frac{z}{2}\sqrt{\frac{p}{\pi\alpha}}\right),24\right]-t_{\max}}{p_2}\right)$$

Eq.3-6

$\frac{z}{2}\sqrt{\frac{p}{\pi\alpha}}$ is the time lag, $p=24$, $p_0=2\times(24+t_{min}-t_{max-1})$, $p_1=2\times(t_{max}-t_{min})$, $p_2=2\times(24+t_{min+1}-t_{max})$, Mod() is a function that returns the remainder after a number is divided by divisor.

3.4.1.2 Discontinuous to Continuous

While there is no consideration of antecedent conditions in Eq. 3-4, it is known that the variation of soil temperature should generally be continuous with time, and the current temperature is influenced by the previous temperature. Use of Eq. 3-6 addresses this concern. In this approach, the mean air temperature is calculated from the daily maximum and minimum air temperature values, which may be conveniently obtained from a typical weather forecast. While calculating the mean temperature or amplitude, the parameters (observed air temperature) of the current day and adjacent day are used, not only a given day's data. For example (Figure 3-38), the mean temperature (T_{mean1}) of the first part equals ½(T_{max}+T_{min}) and the mean temperature (T_{mean2}) of the second part (from t_{max} to 24:00) which is mainly affected by the current T_{max} (maximum temperature) and the next T_{min1} (minimum temperature), is determined as ½(T_{max}+T_{min1}).

3.4.1.3 Daily Variation between Soil temperature and Air temperature

Air temperature is quite sensitive to albedo, cloud cover, rain, wind, etc., and the relationship between the soil temperature and air temperature is complex, i.e., soil temperature is not always some fraction of air temperature. Zheng et al. (1993) and Feng et al. (2004) used a linear relationship derived from a statistical analysis of collected data. However, the potential inaccuracy in a given site-specific application renders this approach less suitable without further investigation. Alternatively, based on the relationship between air temperature amplitude (A_A) and soil temperature amplitude (A_S) may be given as (Andersland, et al.2004):

$$A_S = A_A \cdot \exp(-z \cdot \sqrt{\frac{p}{\pi\alpha}}) \qquad \text{Eq.3-7}$$

Where $A_A = \frac{1}{2}(T_{A\max} - T_{A\min}))$ and $A_{A1} = \frac{1}{2}(T_{A\max} - T_{A\min 1}))$

One may write

$$T_{smean1} = T_{smean0} + \frac{1}{2} \cdot (T_{A\max} - T_{A\max 0}) \cdot \exp(-z \cdot \sqrt{\frac{p}{\pi\alpha}})$$

$$T_{smean2} = T_{smean1} + \frac{1}{2} \cdot (T_{A\min 1} - T_{A\min}) \cdot \exp(-z \cdot \sqrt{\frac{p}{\pi\alpha}}) \qquad \text{Eq.3-8}$$

Where T_{smean} is the mean soil temperature, T_{Amax} and T_{Amin} are the maximum and minimum air temperature between two adjacent days. Eq.3-8 indicates that the relationship between air temperature and soil temperature changes daily, and this can be used to capture such variations with better resolution.

3.4.1.4 Amplitude variation

The soil temperature amplitude may be influenced by the soil moisture content and prevailing solar radiation. The thermal diffusivity of the soil is the ratio of the thermal conductivity of the soil to the volumetric heat capacity of the soil. The thermal conductivity and volumetric heat capacity increase with increasing water content so the diffusivity is also dependent upon soil moisture content. Reichard et al. (1972) developed the relationship between the soil thermal diffusivity (α) and soil moisture content (Θ),

$$\alpha = \alpha_0 \cdot \exp\left(\beta \cdot \frac{\theta - \theta_0}{\theta_s - \theta_0}\right) \qquad \text{Eq.3-9}$$

Where Θ_s is the maximum moisture content (i.e. under saturated conditions), β is a constant depending on the soil, α_0, Θ_0 are the thermal diffusivity and moisture content at dry condition, respectively. The dry soil thermal diffusivity, α_0, can be obtained by Eq.3-2 shown previously.

In addition, based on the regression analysis of sample data, a variation coefficient (C) with regard to moisture content was developed:

$$C = \left(\frac{\theta_n}{\theta}\right)^2$$

Eq. 3-10

Where Θ_n is a "normal" average soil moisture content, taken as being the average of the ½(Θ_s-Θ_0) and Θ represents the existing moisture content. This amplitude damping coefficient can be incorporated as:

$$A_S = A_A \cdot \exp(-z \cdot \sqrt{\frac{p}{\pi\alpha}}) \cdot C = A_A \cdot \exp\left(-z \cdot \sqrt{\frac{p}{\pi \cdot \alpha_0 \cdot \exp\left(\beta \cdot \frac{\theta - \theta_0}{\theta_s - \theta_0}\right)}}\right) \cdot \left(\frac{\theta_n}{\theta}\right)^2$$

Eq.3-11

Based on the above improvements for the specific problems, the improved model developed herein may be written as:

$$T_{Z,t} = T_{Smean} + \frac{1}{2}\left(T_{Am1} - T_{Am}\right) \cdot \exp(-z \cdot \sqrt{\frac{p}{\pi\alpha}}) + A_A \cdot \exp(-z \cdot \sqrt{\frac{p}{\pi\alpha}}) \cdot C \cdot K(t)$$

Eq.3-12

Where T_{Am1} or T_{Am} (mean values) may be substituted for T_{Amax} or T_{Amin} (maximum and minimum values).

3.4.2 Model application and discussion

The foregoing model has been applied to virtually all sites reported herein as well as sites for which data exist from general weather monitoring, e.g., North Carolina weather stations which include at least one soil temperature measurement. As an example, consider the data collected October to November in 2008 at NC 16, near Weddington, North Carolina. A sample of ten days of observed data is presented to illustrate application of the improved model. The input parameters to the model include soil depth, thermal diffusivity and moisture content as well as the time and value of the maximum and minimum air temperature of each day.

Figure 3-26 illustrates the model results at the 1.0 inch depth (2.5 cm) by the default equation (Eq. 3-4) and the improved model (Eq. 3-12). The actual data and predicted results are difficult to distinguish as they are virtually the same.

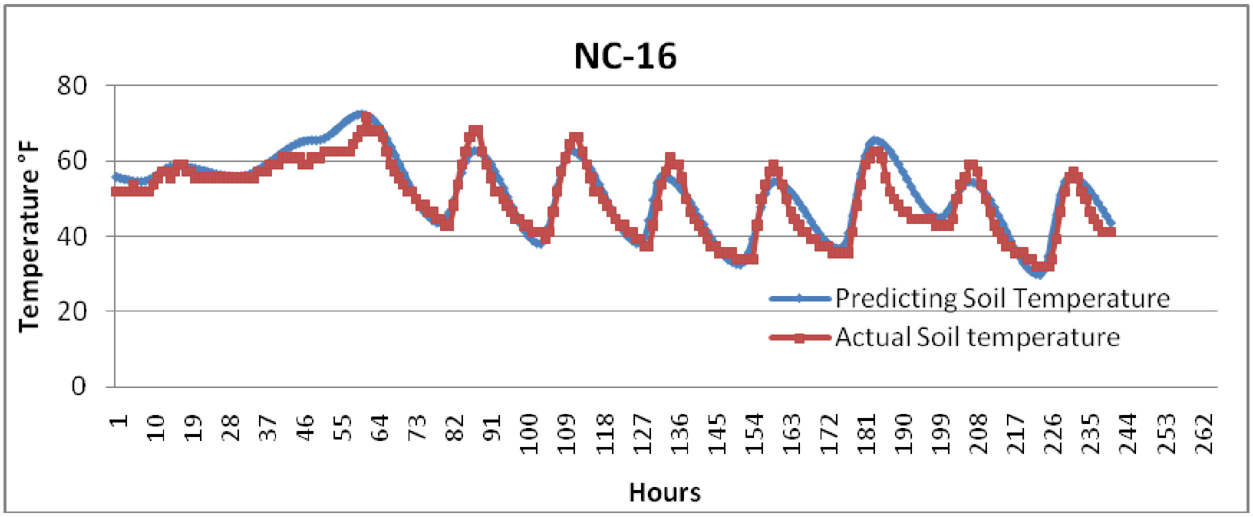


Figure 3-26: Example of predicted and actual temperatures at a 1 inch depth in lime-stabilized subgrade in NC16 near Weddington, NC, Fall 2008.

3.5 Computer Application and Implementation

A computer application was developed to apply the foregoing model and enable NCDOT personnel to use a weather forecast and several accessible parameters to estimate temperature as a function of depth. This application also incorporates the results from Chapter 2 such that estimates can also be made on the in situ strength as a function of anticipated curing temperature. The model has been provided separately as an .exe file (STSTP.exe) to be installed as desired on NCDOT computers. There are help files and explanatory notes in the application that can be consulted. Moreover, the PI will be pleased to provide advice on how to use the application, as well as its strengths and limitations. By way of introduction to this program, screen captures and commentary are provided as follows.

Once the program is loaded, the first task is to enter forecasted air temperature data (e.g., from weather.com or some other appropriate meteorological data source) as well as information about the depth and stabilized subgrade. The first screen presented is shown in Figure 3-27:

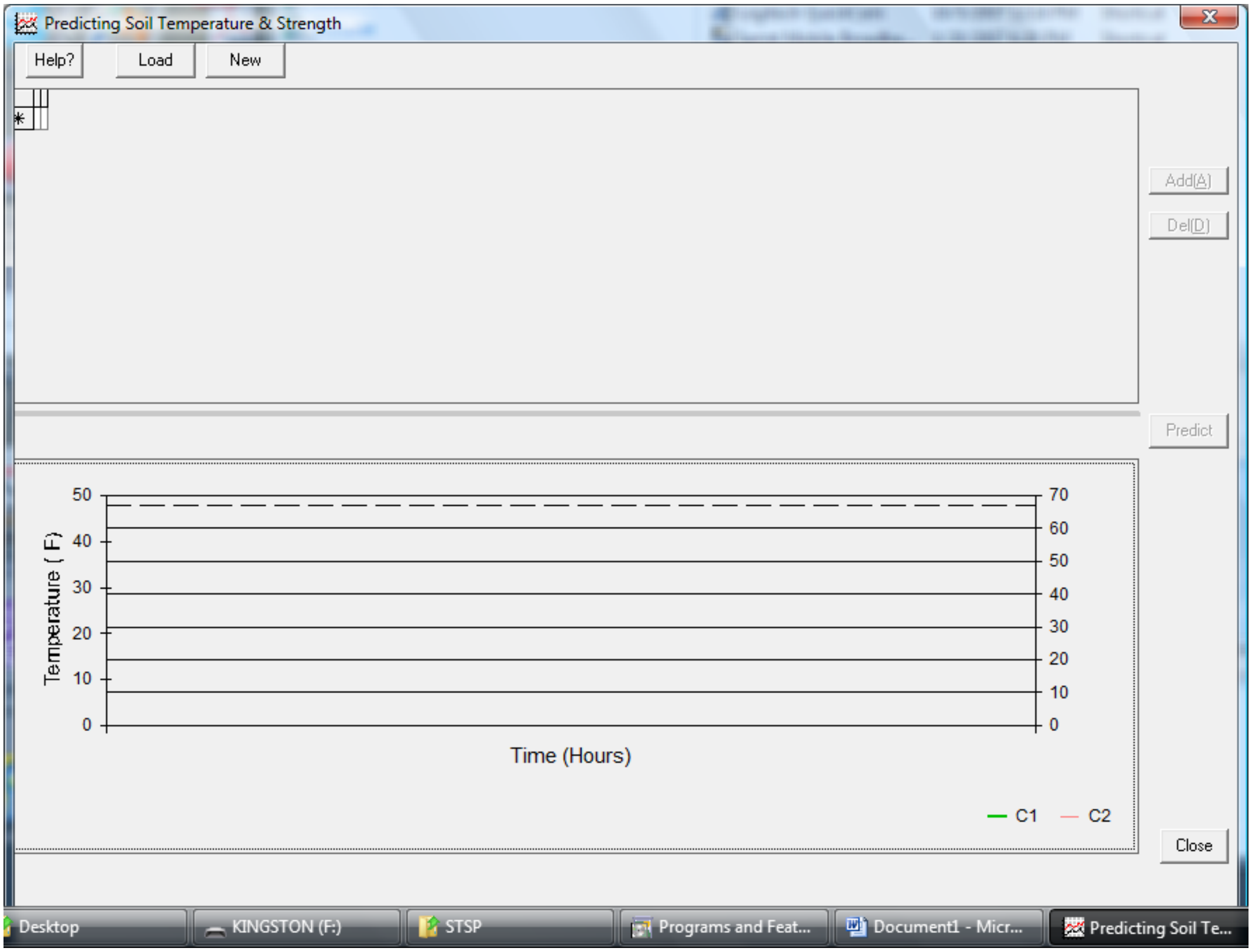
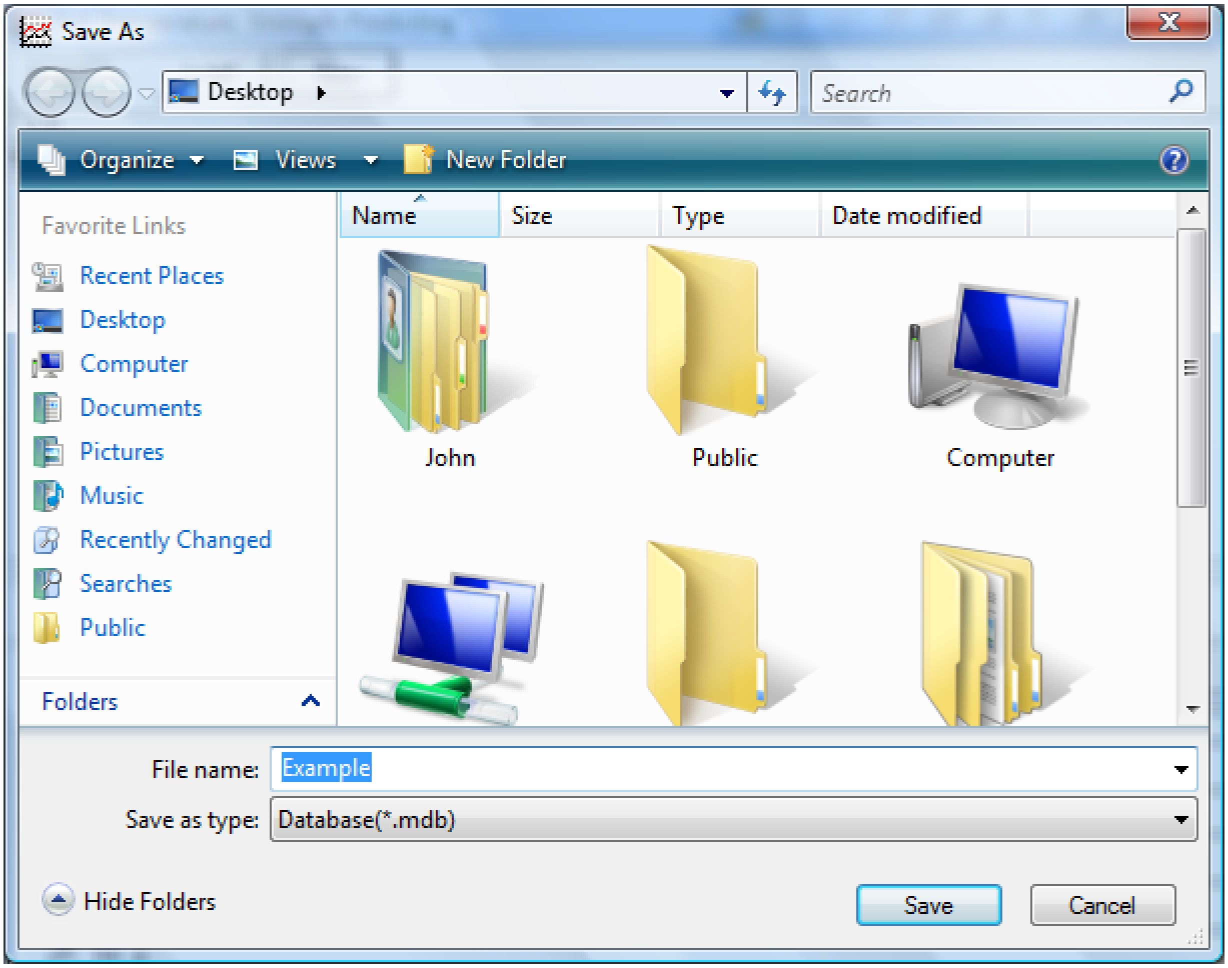


Figure 3-27: Opening Screen in STSTP Computer Application

The user has the option to load an existing data set or to create one. This example illustrates creating a new data set. The next step is to select “New,” then as prompted in Figure 3-28, the user is asked for a file name (in .mdb format) with which to save the data that is about to be entered. The name “Example” was typed for this illustration.



File name: Example

Save as type: Database(*.mdb)

Figure 3-28: Dialog box prompt for file name for data to be entered
After naming the file, one row will become visible, awaiting data input, as shown in Figure 3-29.

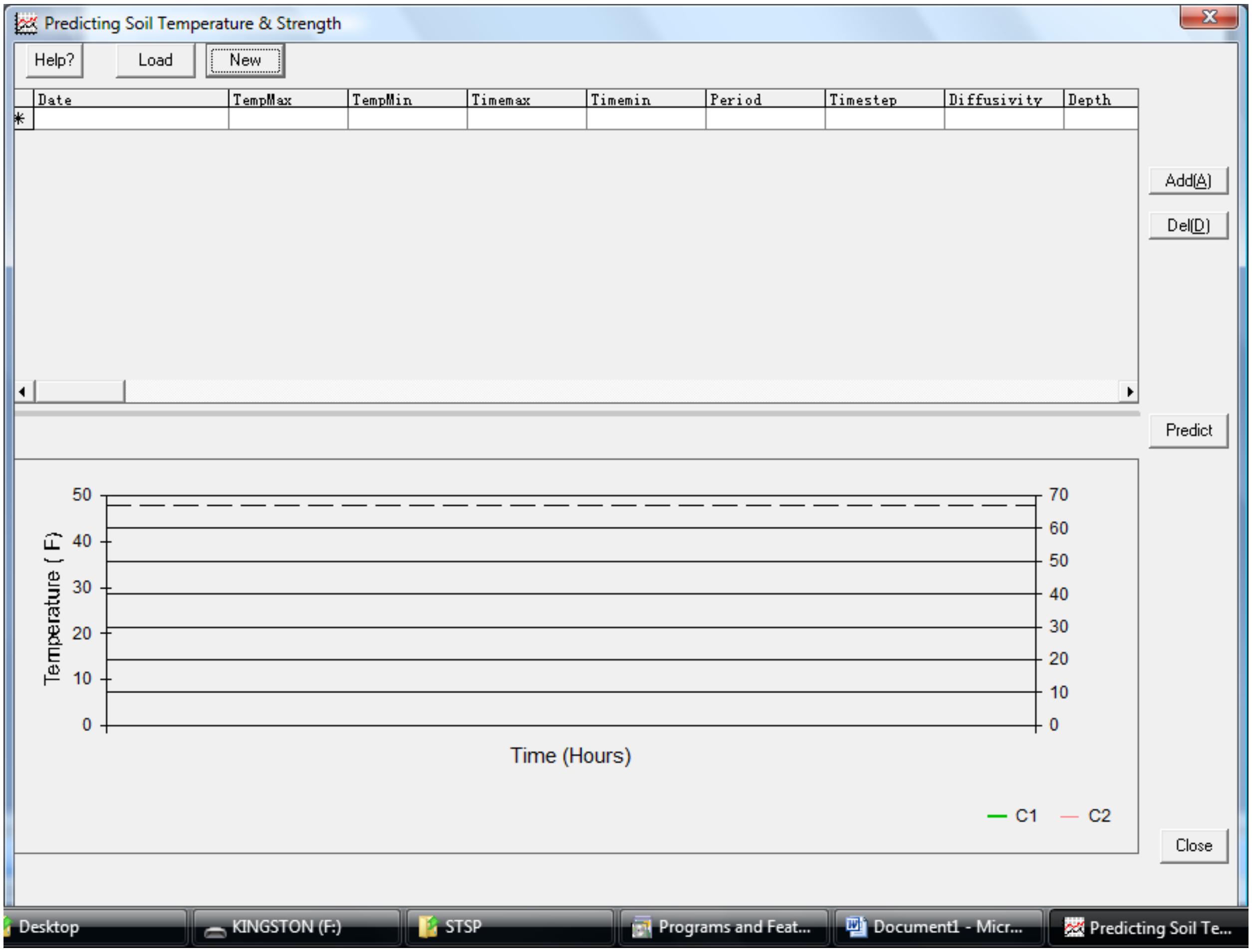
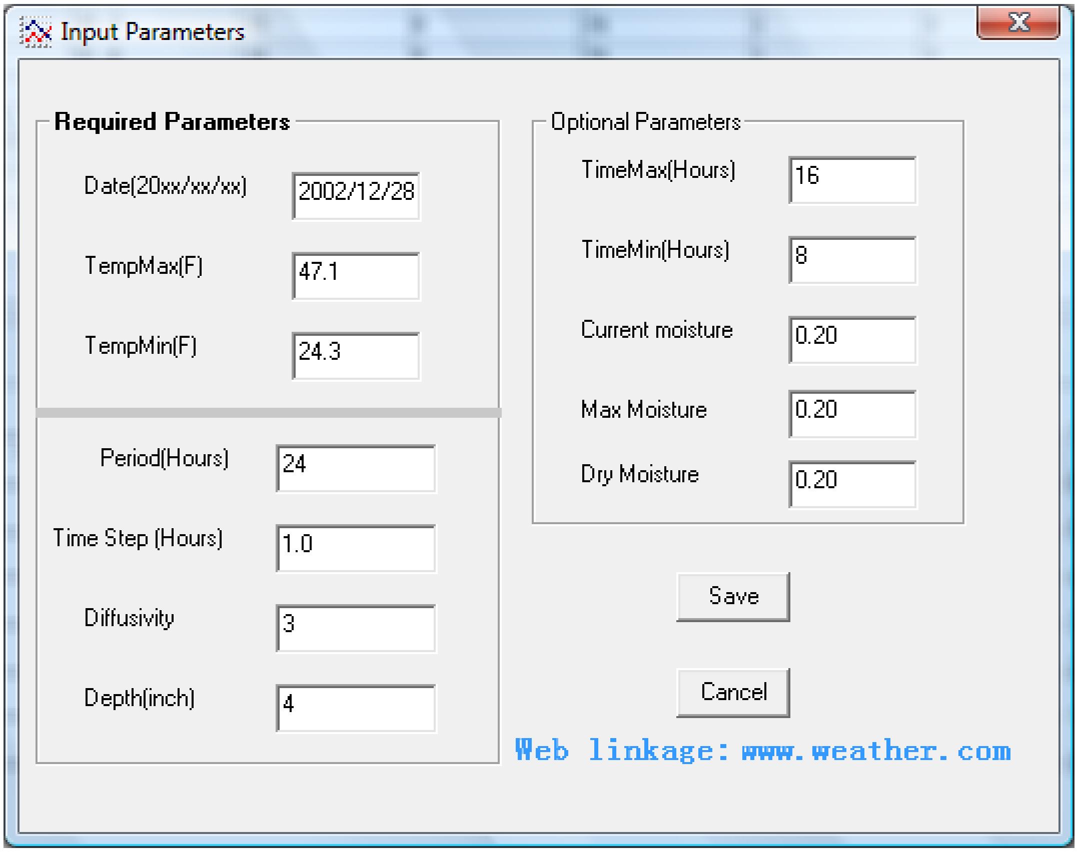


Figure 3-29: One record (first row) showing, awaiting data input.

Next, click “Add” to input data. A dialog box appears as shown in Figure 3-30.



Required Parameters

Date(20xx/xx/xx) 2002/12/28
TempMax(F) 47.1
TempMin(F) 24.3
Period(Hours) 24
Time Step (Hours) 1.0
Diffusivity 3
Depth(inch) 4

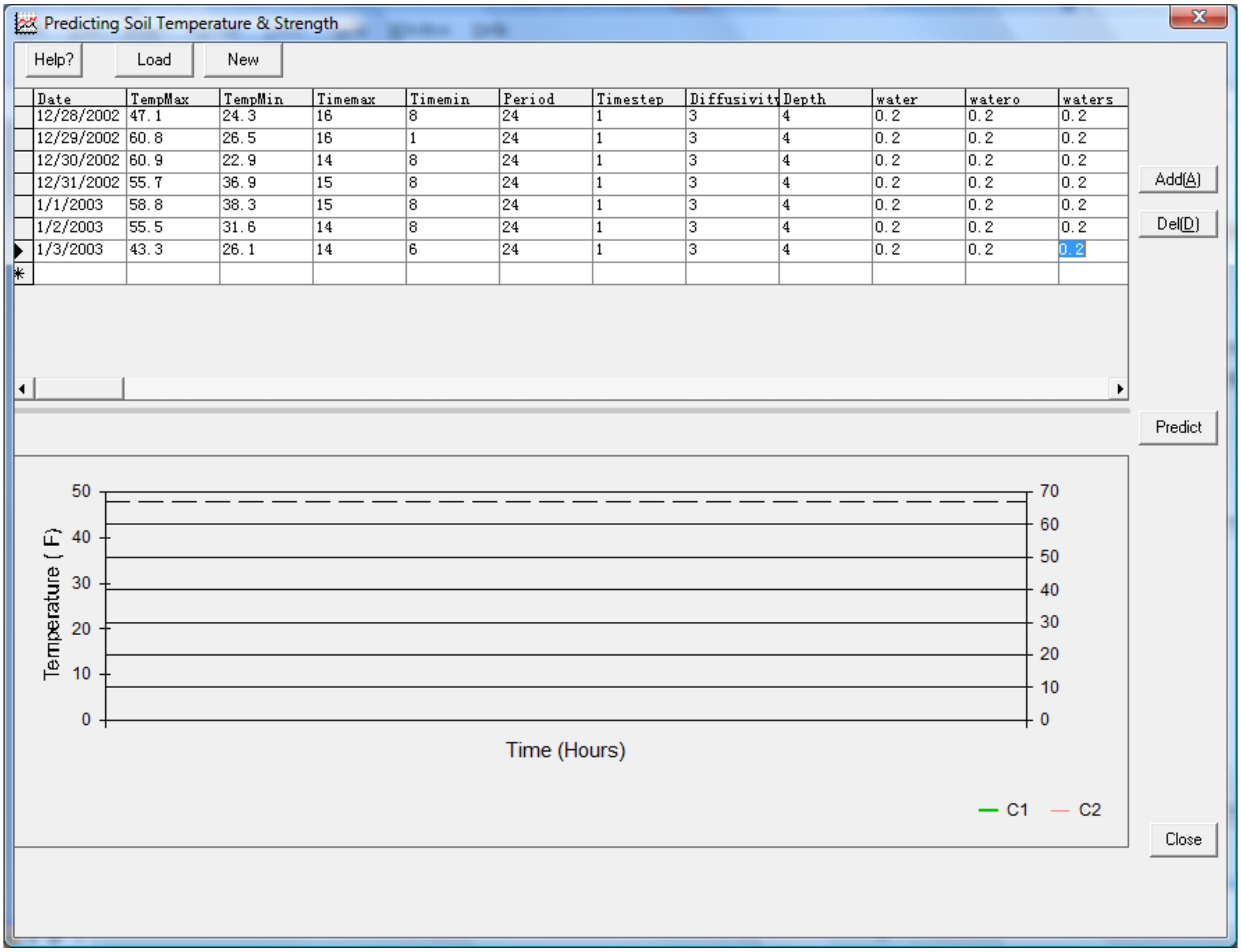
Optional Parameters

TimeMax(Hours) 16
TimeMin(Hours) 8
Current moisture 0.20
Max Moisture 0.20
Dry Moisture 0.20

Web linkage: www.weather.com

Figure 3-30: Input parameter dialog box.

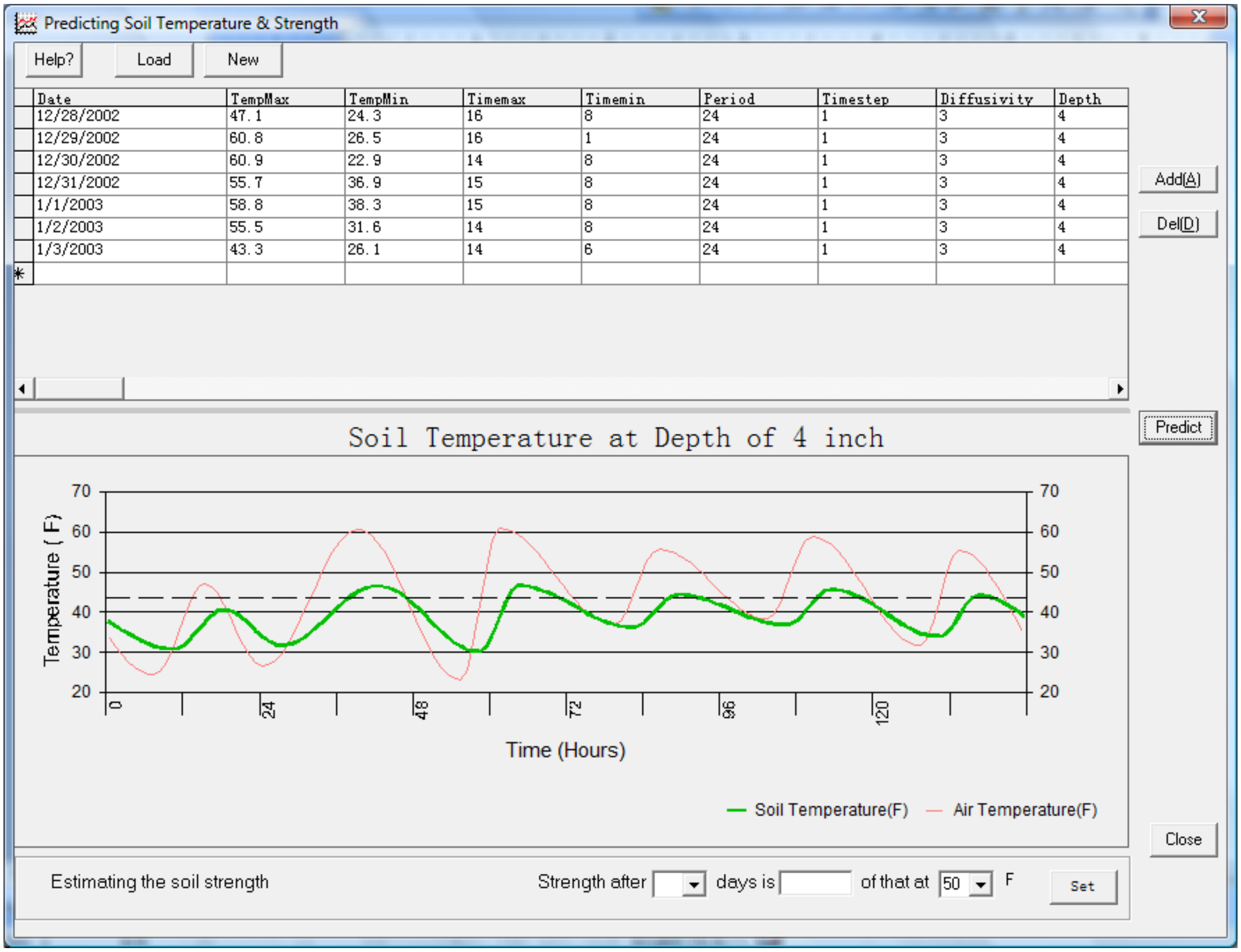
The data to be entered include the date in (Year/Month/Day) format, along with the minimum and maximum temperatures, the period (e.g. 24 hour days), the time step (e.g., calculate temperature every 1 hour), the thermal diffusivity (in^2/hour), the desired depth of soil temperature prediction. Default parameters can be used for moisture content (in this case the the moisture content is estimated as one value, with no information regarding the dry or maximum values. The TimeMax and TimeMin refer to the hours past 12 midnight when the maximum and minimum temperatures occur, respectively. For example, the above indicates the maximum temperature occurs at 4:00 PM (late afternoon) and the minimum temperature occurs at 8:00 AM (morning).   When such detailed data are unknown, they are simply estimated, with the estimates informed by the data generated during this research or other available data. Figure 3-31 shows the table that is completed after data for an entire week.



Date	TempMax	TempMin	Timemax	Timemin	Period	Timestep	Diffusivity	Depth	water	watero	waters
12/28/2002	47.1	24.3	16	8	24	1	3	4	0.2	0.2	0.2
12/29/2002	60.8	26.5	16	1	24	1	3	4	0.2	0.2	0.2
12/30/2002	60.9	22.9	14	8	24	1	3	4	0.2	0.2	0.2
12/31/2002	55.7	36.9	15	8	24	1	3	4	0.2	0.2	0.2
1/1/2003	58.8	38.3	15	8	24	1	3	4	0.2	0.2	0.2
1/2/2003	55.5	31.6	14	8	24	1	3	4	0.2	0.2	0.2
1/3/2003	43.3	26.1	14	6	24	1	3	4	0.2	0.2	0.2

Figure 3-31: Data entered and ready for prediction.

After the data have been entered, the user selects "Predict" and the results are displayed as shown in Figure 3-32:



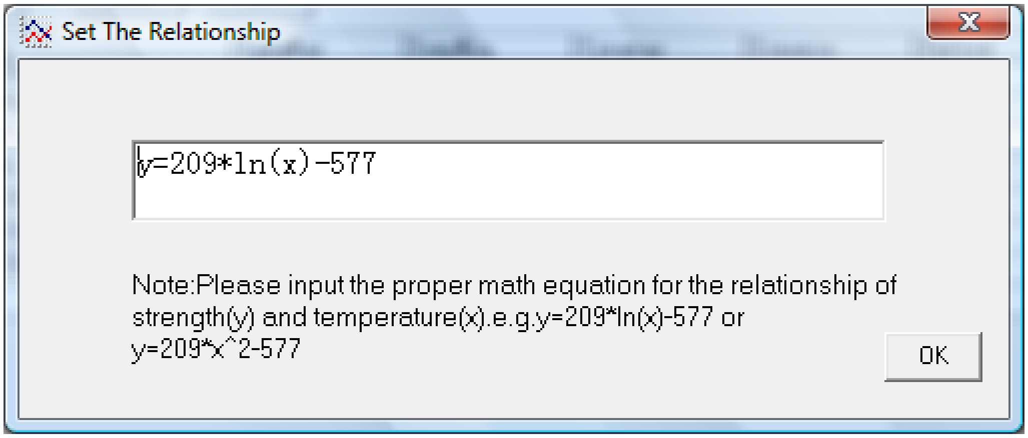
Date	TempMax	TempMin	Timemax	Timemin	Period	Timestep	Diffusivity	Depth
12/28/2002	47.1	24.3	16	8	24	1	3	4
12/29/2002	60.8	26.5	16	1	24	1	3	4
12/30/2002	60.9	22.9	14	8	24	1	3	4
12/31/2002	55.7	36.9	15	8	24	1	3	4
1/1/2003	58.8	38.3	15	8	24	1	3	4
1/2/2003	55.5	31.6	14	8	24	1	3	4
1/3/2003	43.3	26.1	14	6	24	1	3	4

Estimating the soil strength

Strength after ☐ days is ☐ of that at 50 F

Figure 3-32: Prediction of subgrade soil temperature (green) plotted with air temperature (red).

Given the predicted subgrade soil temperature with depth, the user can incorporate the results from this research that relate strength to temperature with regression equations. For example the user can click “Set” and see the default equation as presented in Figure 3-33.

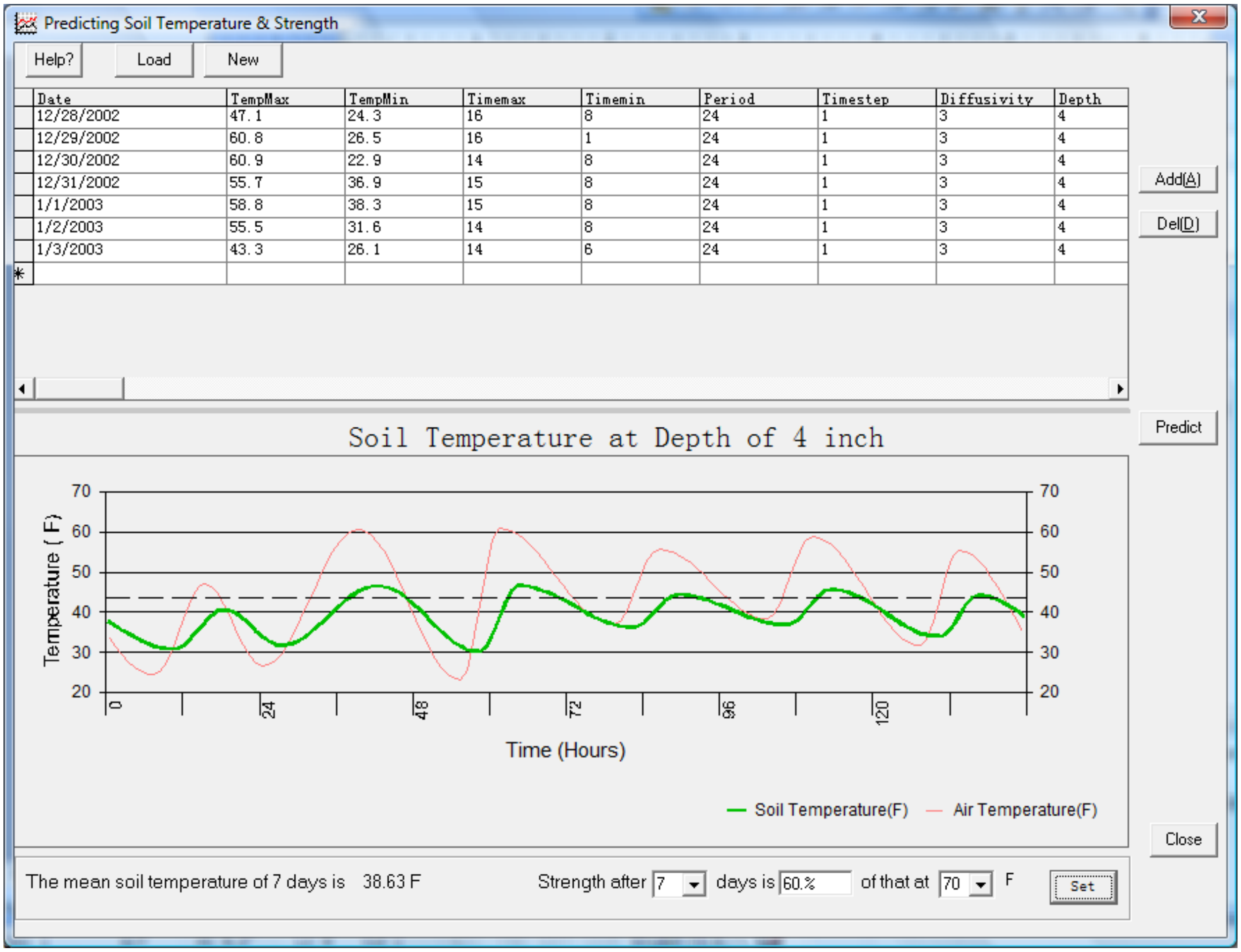


y=209*ln(x)-577

Note:Please input the proper math equation for the relationship of strength(y) and temperature(x).e.g.y=209*ln(x)-577 or y=209*x^2-577

Figure 3-33: Default equation used to relate cured strength to temperature.

Once confirming the equation or entering a new relationship, the user can specify the desired estimate in terms of days and temperature with the resulting estimate provided as a percent of another condition (e.g., ideal 70°F) , as shown in Figure 3-34.



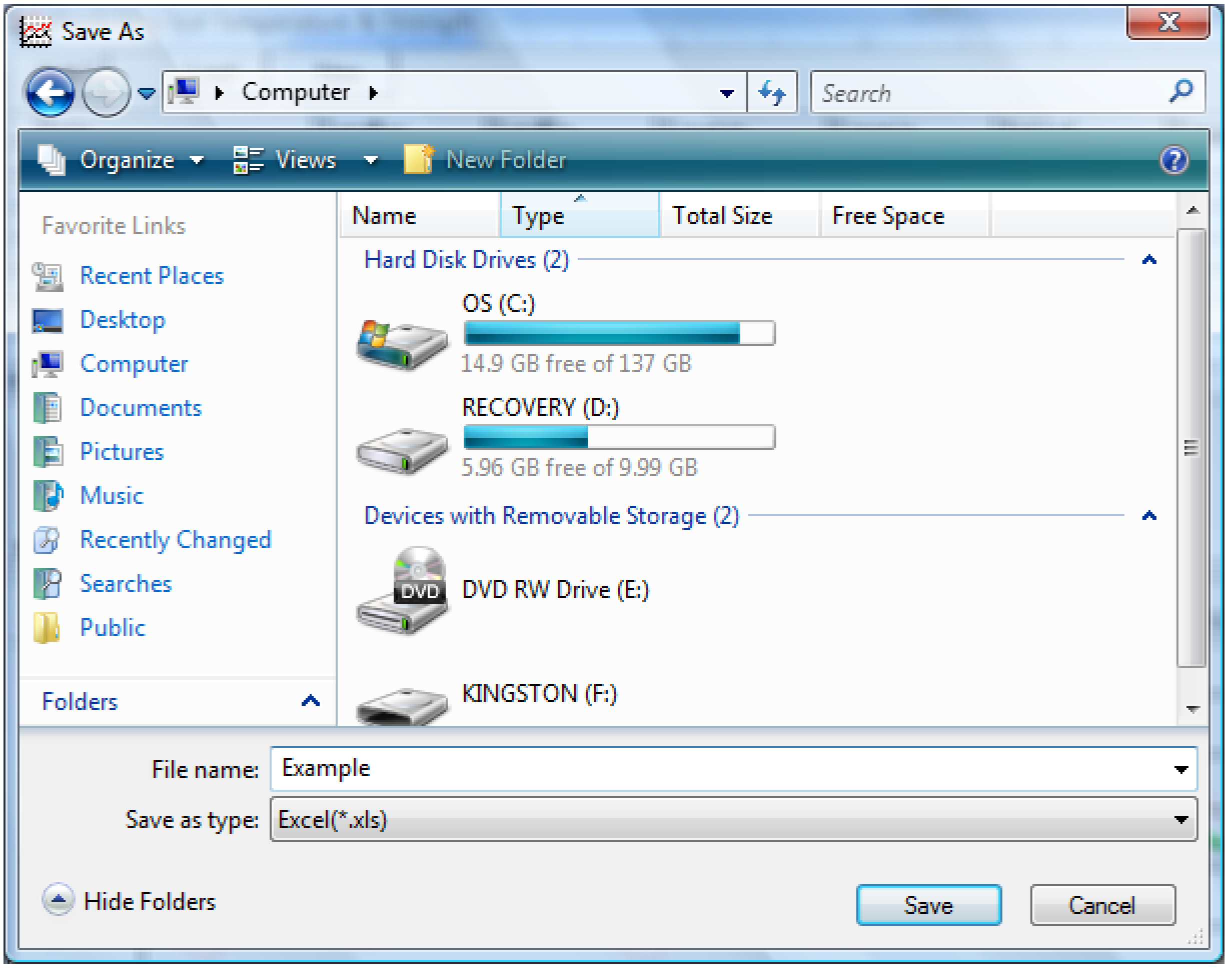
Date	TempMax	TempMin	Timemax	Timemin	Period	Timestep	Diffusivity	Depth
12/28/2002	47.1	24.3	16	8	24	1	3	4
12/29/2002	60.8	26.5	16	1	24	1	3	4
12/30/2002	60.9	22.9	14	8	24	1	3	4
12/31/2002	55.7	36.9	15	8	24	1	3	4
1/1/2003	58.8	38.3	15	8	24	1	3	4
1/2/2003	55.5	31.6	14	8	24	1	3	4
1/3/2003	43.3	26.1	14	6	24	1	3	4

The mean soil temperature of 7 days is 38.63 F

Strength after 7 days is 60.% of that at 70 F

Figure 3-34: Estimate of Strength (displayed in bottom of dialog box).

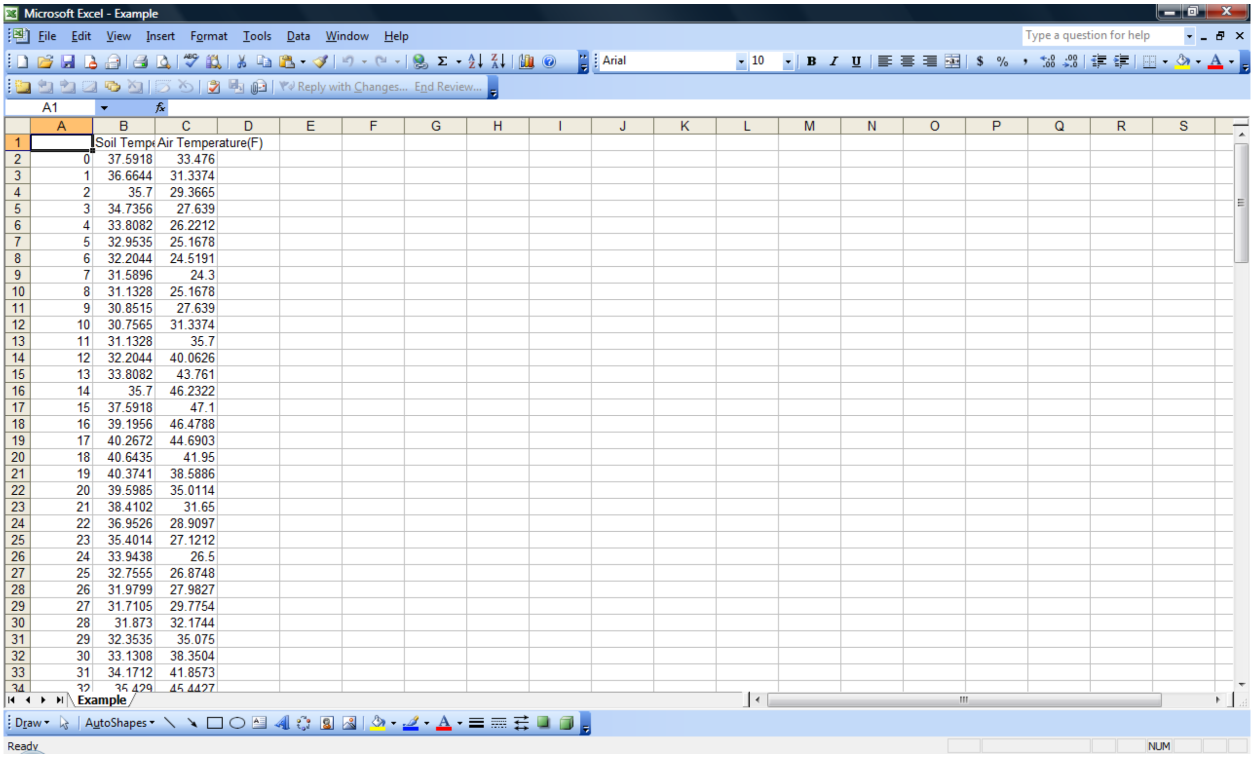
As noted in Figure 3-34, the modeled data suggest the strength of this stabilized subgrade, to be 60% of that which would occur if the material were allowed to cure at an average temperature of 70°F. If the user wishes to inspect the modeled data for further analysis, then right clicking on the graph provides an option to save the data as an Excel spreadsheet, as shown in Figure 3-35 and 3-36.



File name: Example

Save as type: Excel(*.xls)

Figure 3-35: Option to save modeled data as Excel spreadsheet.



	A	B	C
1		Soil Tempe	Air Temperature(F)
2	0	37.5918	33.476
3	1	36.6644	31.3374
4	2	35.7	29.3665
5	3	34.7356	27.639
6	4	33.8082	26.2212
7	5	32.9535	25.1678
8	6	32.2044	24.5191
9	7	31.5896	24.3
10	8	31.1328	25.1678
11	9	30.8515	27.639
12	10	30.7565	31.3374
13	11	31.1328	35.7
14	12	32.2044	40.0626
15	13	33.8082	43.761
16	14	35.7	46.2322
17	15	37.5918	47.1
18	16	39.1956	46.4788
19	17	40.2672	44.6903
20	18	40.6435	41.95
21	19	40.3741	38.5886
22	20	39.5985	35.0114
23	21	38.4102	31.65
24	22	36.9526	28.9097
25	23	35.4014	27.1212
26	24	33.9438	26.5
27	25	32.7555	26.8748
28	26	31.9799	27.9827
29	27	31.7105	29.7754
30	28	31.873	32.1744
31	29	32.3535	35.075
32	30	33.1308	38.3504
33	31	34.1712	41.8573
34	32	35.429	45.4427

Figure 3-36: Modeled data in Excel Spreadsheet.

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CHAPTER 4: Calcium Chloride for Modification of Cement-Stabilized Subgrade Soils

Chapter Summary

This chapter presents background information and describes laboratory and field efforts to increase the strength of cement-stabilized subgrade with chemical admixtures, primarily calcium chloride ($CaCl_2$). Previous literature had suggested a wide range of possible effective doses and a number were explored for soil-cement mixes with soils from Buncombe, Guilford, Johnston and Lincoln counties. A window of efficacy was observed for Buncombe, Guilford and Johnston county soil cement mixes, with ideal ranges at 50^0F curing conditions of 0.25% – 1.0%, 0.25%-0.75% and 1.25%-1.75%, respectively (percentages reflect mass of $CaCl_2$ per mass of cement). However these optimum ranges vanished or changed to levels untested at 35^0F curing conditions. A field trial was conducted with $CaCl_2$ doses of 2.3% and 8.3%, and both of these dosage levels resulted in weakening of the material, as evaluated by in situ dynamic cone penetration tests and unconfined compression testing of field-mixed samples. Laboratory mixed samples of the same material resulted in strength increases. $CaCl_2$ modification increases the electrical conductivity and dielectric value of soil-cement mixtures which might imply increased susceptibility to longer term moisture-induced weakening at high dosage levels. The cost of $CaCl_2$ modification at effective doses is likely to be less than 10% of cement costs. The overall body of research presented in this report suggests that $CaCl_2$ modification of soil-cement is not a mature enough approach to serve as a method for mitigating the effects of low temperatures on strength gain. Additional data are required to probe the sensitivity of temperature, mixing method and soil type.

4.1 Introduction

As observed in Chapter 2, lower curing temperatures result in decreased unconfined compressive strength for cement-stabilized soil. This chapter explores the use of calcium chloride to mitigate the effects of temperature, following the recommendations of a technical assistance agreement report (Daniels and Janardhanam 2005) and the original proposal (included in appendix for easy access) for this research. The general rationale for this approach, with preliminary data, has been discussed in those documents as well as others by the PI (Daniels et al. 2009; Daniels and Janardhanam 2007). The basic idea is that calcium chloride is diluted in water and mixed with soil cement prior to compaction. Calcium chloride accelerates the rate of cement hydration which is responsible for strength development. This accelerated curing can compensate for otherwise low-strength development at low temperatures.

4.2 Background

4.2.1 Calcium Chloride in Concrete

The decision to use calcium chloride was based largely on consideration of the cold weather concreting literature and work led by Charles J. Korhonen at the Cold Regions Research and Engineering Laboratory of the U.S. Army Corps of Engineers Engineer Research and Development Center. When a set accelerating admixture is added to a cement mixture such as concrete, the effects are threefold: there is an increase the rate of hydration of cement, the time of setting of concrete is shortened, and the rate of early strength development is increased (Dodson

1990). These characteristics are also attractive for cold weather soil-cement stabilization. Set accelerators are considered Type C admixtures according to ASTM C 494. Substances that act as set accelerators are alkali hydroxides, silicates, fluorosilicates, organic compounds, calcium formate, calcium nitrate, calcium thiosulfate, aluminum chloride, potassium carbonate, sodium chloride, and calcium chloride. Due to its availability, low cost, predictable performance characteristics and successful application over several decades, calcium chloride is the most widely used accelerator. The accelerating effect is related mainly to the tricalcium silicate (C3S) phase of the cement-water mixture. $CaCl_2$ is unique because the cation-anion combination of $Ca^{+2} – Cl^{-1}$ ranks as one of the best accelerators for the hydration of C3S. The set accelerating potential of the following cations and anions, respectively, are diagrammed in as follows (Ramachandran 1984):

$$Ca^{+2} > Sr^{+2} > Ba^{+2} > Li^{+} > Na^{+} > K^{+}$$

$$SO4^{-2} > OH^{-} > Cl^{-} > Br^{-} > I^{-} > NO_3^{-} > CH_3COO^{-}$$

Set accelerators are mainly used in cold weather concreting and are sometimes referred to as antifreeze admixtures. It is true that the addition of $CaCl_2$ results in the depression of the freezing point of water. For example, the addition of 0.25, 0.5, 1.0, 2.0, 3.0, 5.0, or 7.0% anhydrous calcium chloride to water, should lower the freezing point of water by 0.1, 0.2, 0.4, 0.9, 1.4, 2.3, or 3.4°C respectively. However $CaCl_2$ dosages for cold weather concreting are typically much less than 3% solution and so the depression of the freezing point is essentially negligible (Ramachandran 1984). Various suppliers of $CaCl_2$ provide estimates of the benefit of dosage versus strength as well as recommendations as a function of temperature. For example, Dow Chemical (2006) provides the following guidelines (note that references to dosage percents have units of mass of $CaCl_2$ per mass of cement):

- *The amount of calcium chloride used should not exceed 2% and preferably, should be added in solution form to the aggregates in the mixer drum or as a portion of the mixing water, reducing the total mixing water required by the amount of solution being used.*

- *Concentrated calcium chloride should not come into direct contact with dry cement as it may cause flash set.*

- *For best results, calcium chloride application rates should vary with temperature as follows:*
  >90°F – no more than 1%
  70°F to 90°F – 1.0 to 1.5%
  <70°F – up to 2%
  Never exceed 4% due to flash set probability

- *The specific effects produced by chemical admixtures may vary with the properties of the other ingredients in the concrete.*

The foregoing illustrates that $CaCl_2$ can increase the rate of strength gain and moreover, the benefits are related to temperature.

4.2.2 Calcium Chloride in Soil

The application of cold-weather concreting concepts to soil cement stabilization, as presented in this report, is relatively new (Daniels and Janardhanam 2007). However, the general process of adding chemicals to soil-cement to enhance its hardened properties, is not a new idea, as noted by the following discussion.

Laguros (1962) describes an investigation of several chemicals on soil-cement made with various soils from around the country. Sodium hydroxide, calcium sulfate, magnesium sulfate, sodium sulfate, calcium chloride, magnesium oxide, sodium orthosilicate, and sodium carbonate were added to soil-cement mixtures using soils from Iowa, Wisconsin, Illinois, Texas, Michigan, North Carolina, and Washington. The researcher concluded that just about every soil tested was able to attain additional strength with the addition of at most 2% of a selected chemical. Also, the results were highly dependent on the type of soil and the chemistry of the soil. The specimens were cured under ideal conditions; the study offered no insight on a chemical additive's ability in assisting cement stabilization under conditions where temperature limits the effectiveness of hydration.

Calcium chloride has been successfully used to neutralize organics in soil to facilitate the creation of soil-cement without adding excessive amounts of cement (Ness 1966). Furthermore, it has been suggested that the addition of $CaCl_2$ is very simple (it can be added at any time when mixing the soil, cement and water) and does not harm normally reacting soils (Catton and Felt 1943). $CaCl_2$ has also been used in full-depth reclamation projects where the existing deteriorated asphalt roadway is crushed with a specific amount of its underlying base and recycled to form the new road surface. $CaCl_2$ offers several benefits in this capacity including uniform moisture control, increased density of compacted material, and extension of the season for full-depth reclamation projects (Pickett 1991).

For soil-cement applications, Rollings et al. (2002) recommends a general dosage of 2% (mass of $CaCl_2$ per mass of cement). In particular, Rollings (undated) investigated the use of $CaCl_2$ on the unconfined compression strength of a silty sand stabilized with 12% cement as part of research conducted for Joint Rapid Airfield Construction (JRAC) program within the U.S. Army. The control specimen was cured at 70^oF. Rollings observes that the basic effect of $CaCl_2$ modification at 70^oF is to gain the same strength as the unmodified control, but in approximately one-third of the time. At 35^oF and 50^oF, $CaCl_2$ modification results in approximately the same or greater strength as unmodified specimens cured at 70^oF. Strength gain also occurred at $25\,^oF$ for $CaCl_2$ modified samples, achieving sufficient strength (in this case 750 psi, a high value intended to support aircraft) in 24 days. Rollings further recommends that in the absence of data, the effect of $CaCl_2$ modification can be estimated to reduce the time required to achieve the target strength by 50% at normal temperatures or it will allow the same strength achievable under normal temperatures to occur at temperatures that approach freezing.

Uppot et al.(2008) investigated the use of $CaCl_2$ modification on soil cement for four different soils that ranged from highly plastic to non-plastic silty sand. Laboratory (unconfined compression) and field tests (DCP) were conducted. Cement ratios of 6% and 8% by weight of soil were used and the $CaCl_2$ ratios varied from 9.5% to 38.0% (mass of $CaCl_2$ per mass of cement), or as defined in the paper 2.0% to 6.0% solution in dry weight of soil. The authors explain that for their experiments, "A 2% calcium chloride means 2% of the {source 38%} calcium chloride solution in the dry weight of soil treated. This mixture of soil and calcium chloride solution, on dry weight basis, will actually contain 0.76% calcium chloride. So in other words, 2% by their definition means 0.76% (mass of $CaCl_2$ per mass of soil) or at 6% cement, this means 12.7% (mass of $CaCl_2$ per mass of cement). These are relatively high dosage values as compared to recommendations by the concrete literature or by Rollings at the US Army (<2% mass of $CaCl_2$ per mass of soil). Surendra K. Mishra, one of the co-authors of Uppot et al. (2008) suggests that more $CaCl_2$ is needed than would be specified on the basis of concreting because of the reactivity and greater specific surface area of the soil particles (personal communication). The resulting unconfined compression strength was relatively high for all dosage levels. Table 4-1 presents data for "McNeese Farm Road Soil" which was a Non-plastic, Sandy Silt – ML.

Table 4-1: Effect of $CaCl_2$ on soil cement unconfined compression strength (Source: Adapted* from Uppot et al., 2008).

No. of curing days	**UC Strength (psi) for soil cement (6% Cement)**			
	0% $CaCl_2$	**12.7% $CaCl_2$**	**25.3% $CaCl_2$**	**38% $CaCl_2$**
1	34.6	43.5	68.2	87.4
2	42.5	54.5	86.2	94.4
3	49.5	57.2	99.9	113.4
7	77.8	85.0	215.7	235.8
14	164.0	182.0	481.5	500.7

*Dosages converted to mass of $CaCl_2$ per mass of cement for consistency and comparison with other data presented in report.

Another source of data relevant to evaluating the effect of $CaCl_2$ is the original work that motivated the proposal for the project discussed herein, Daniels and Janardhanam (2005, 2007). To summarize, that work involved unconfined compression testing of soil-cement (6% cement) with three chemical additives procured from the W.R. Grace Company, namely Polarset, Gilco and Daraccel, the latter of which includes $CaCl_2$. The Material Safety Data Sheet describes Daracel as "Aqueous Solution of Calcium Chloride with Triethanolamine." Daraccel contains 0.291 grams of chloride per mL of solution, or 0.454 grams of anhydrous $CaCl_2$ per mL of solution. Considering only the $CaCl_2$ fraction of the solution and the levels tested during the original research, three levels of dosage were investigated, 0.3%, 0.7% and 1.1% (as mass of $CaCl_2$ per mass of cement). The results of these tests are summarized in Figures 4-1 and 4-2 for 28°F and 36 °F curing conditions, respectively.

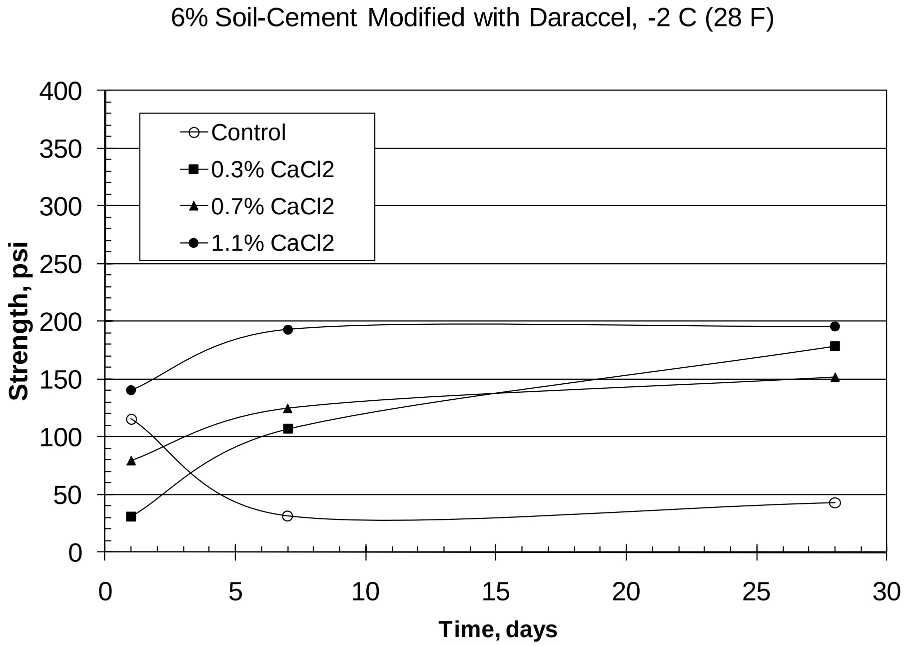

Figure 4-1: UC Strength at 28F curing conditions as a function of time and cement and $CaCl_2$ (data from Daniels and Janardhanam 2005)

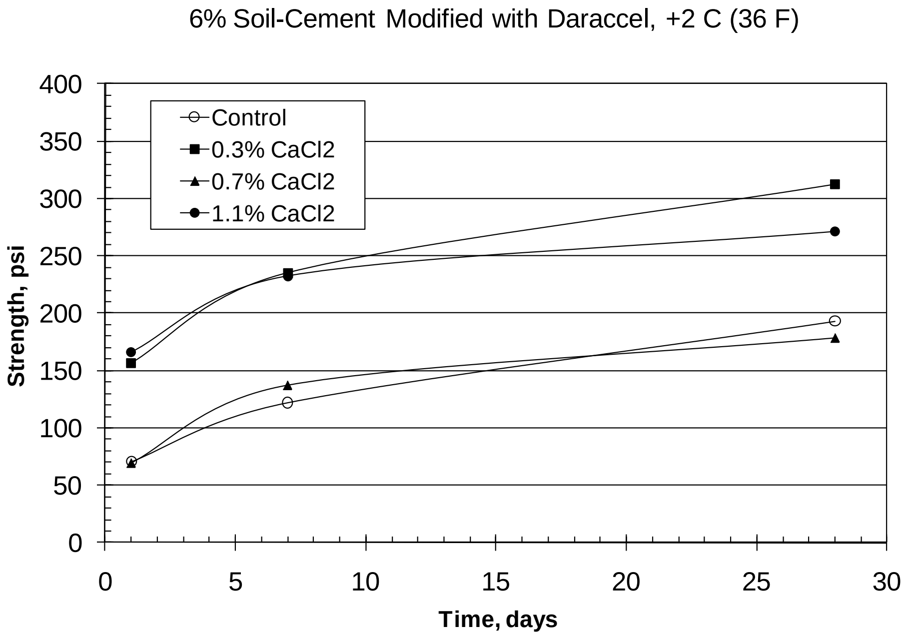

Figure 4-2: UC Strength at 36F curing conditions as a function of time and cement and $CaCl_2$ (data from Daniels and Janardhanam 2005)

The above results suggest that the $CaCl_2$ range tested (0.3% - 1.1%) generally resulted in significant increases in strength, with the exception of the intermediate dosage cured at 36F.

4.3 Materials and Methods

4.3.1 Laboratory-Based Analysis (Buncombe County, Guilford County, Johnston County)

The methods used were essentially identical to that described in Chapter 2 for soil cement, e.g., for moisture-density relationships, unconfined compression, tube suction testing. The distinction in this chapter is the addition of calcium chloride. The $CaCl_2$ used in this research was generally a 38% solution obtained from Tetra Technologies, Houston, TX, although throughout the research solid flake calcium chloride was used (and dissolved) as well as different source concentrations according to availability and ordering schedules. In all cases solutions were prepared and the concentration was adjusted to reach the desired dosage level in the soil. As with the methods, the soils tested herein for the effect $CaCl_2$ modification of were the same as reported in Chapter 2 (e.g., Buncombe County, Guilford County, Johnston County).

As noted in previous quarterly reports, several chemical additives were considered, including sodium nitrate/nitrate, sodium chloride and calcium chloride. As noted by the title of this chapter, only calcium chloride was systematically investigated, although limited unconfined compression data were generated for sodium chloride, which was of laboratory grade quality, obtained from Fisher Scientific. No data were collected for sodium nitrate/nitrate.

4.3.2 Field and Laboratory-Based Analysis (Lincoln County)

The viability of incorporating $CaCl_2$ modification into typical road construction and stabilization activities was investigated through an evaluation of representative field equipment in the fall of 2006. Likewise, a field test was conducted in Lincoln County, as part of the construction of NC16 near Denver, NC in the summer of 2009.

4.3.2.1 Field Evaluation of Mixing Equipment

Field implementation of $CaCl_2$ modification involves incorporating dilute aqueous concentrations of $CaCl_2$ into the subgrade during the mixing phase of construction. This approach was suggested in part because it is straightforward to blend the additives with the subgrade soils at the same time as moisture conditioning, i.e., the chemicals would be blended with the supplied water. Moisture conditioning is often required to reach density requirements and so the addition of chemicals into the water can occur without significant delay in the construction process. Conversely, the use of dry additives would require an extra unit process during subgrade preparation, in that another vehicle would have to spread the additive, much like spreading of the lime or cement.

Still, it has been noted that there are many scenarios whereby the existing moisture content is already close to or beyond the optimum moisture content. In such situations, it is not desirable to add more fluids. Moreover, the ability to mix chemical additives in the field is clearly different than that in the laboratory. In the laboratory for example, chemical additives may be dosed in concentrated form and this adds approximately 0.1% to molding moisture content. With that in mind, the objective of this section of the report is to verify the minimum volume that can be reasonably metered from rotary mixing equipment, and to discuss this impact on field implementation.

Equipment used to blend lime or cement with subgrade soils can vary considerably from one job to the next. However, for purposes of developing a practical sense of what is possible, it is reasonable to evaluate a conventional mixer and water truck. In particular, on July 21, 2006, UNC Charlotte researchers met with Jason Cuneo, Superintendent, Soil Stabilization Division, Propst Construction Company to discuss this approach and to calibrate moisture control under typical field conditions. An RS-425 mixer and water truck were used, as shown in Figure 4-3. To control water distribution with the mixer, the RS-425 pushes the water truck, which is left in neutral. The two vehicles are connected with a steel bar and also with a 2-3” water line. The water truck has a power take-off (PTO) driveshaft, which in turn pumps the water to the mixer. A master valve controls how much water is actually delivered to the nozzles on the sprayer bar located above the rotating tyned-drum While details can vary according to equipment type, moisture delivery is most easily controlled through the (1) the master valve that connects the water line from the water truck to the mixer (located on the mixer) and (2) the rate of travel, i.e., how quickly the mixer-water truck is moved over a given area.

Four experimental runs were conducted to measure how much water could be metered from this arrangement, as follows:

- Valve half open, 45 feet/minute rate

- Valve half open, 20 feet/minute rate
- Valve fully open, 45 feet/minute rate
- Valve fully open, 20 feet/minute rate

In each run, two galvanized steel pans (depth = 4” (10.2 cm), area = 4.04 ft^2 (3753 cm^2)) were carefully placed in the path of the mixer/water truck to collect water distributed via the sprayer bar. This is shown in Figures 4-4 through 4-6. The volume in each of the steel pans was recorded as shown in Table 1.

Table 4-2: Volume collected from RS-425 Mixer as a function of valve position and rate of travel

Setup	**Average Volume* (gal/yd^2)**	**Average Volume* (L/m^2)**	**Average Depth/Unit Area (inches)**	**Average Depth/Unit Area (cm)**
Valve half open, 45 feet/minute rate	1.01	4.56	0.18	0.46
Valve half open, 20 feet/minute rate	1.57	7.10	0.28	0.71
Valve fully open, 45 feet/minute rate	2.46	11.16	0.44	1.12
Valve fully open, 20 feet/minute rate	6.12	27.70	1.09	2.77

Note: Average Volume collected in metal pan that was approximately 4.04 ft^2 (3753 cm^2) in area

The above scenarios are expected to bracket many field-relevant situations. In terms of the impact on moisture content, consider the following example. Considering a 1 ft^2 area, a 7 in. mixing depth and a dry unit weight of 100 lb/ft^3 and the condition where the valve is half open at a speed of 45 ft/minute, we have:

$$M_s = \gamma_d \cdot V_T$$

$$V_T = area \cdot depth = 1ft^2 \cdot 7in. \cdot \frac{1ft}{12in.} = 0.583\,ft^3$$

$$M_s = 100\frac{lb}{ft^3} \cdot 0.583\,ft^3 = 58.33lb$$

$$V_w = area \cdot depth = 1ft^2 \cdot 0.18in \cdot \frac{1ft}{12in.} = 0.015\,ft^3$$

$$M_w = \gamma_w V_w = 62.4\frac{lb}{ft^3} \cdot 0.015\,ft^3 = 0.936lb$$

$$\Delta w = \frac{M_w}{M_s} = \frac{0.936}{58.33} = 1.6\%$$

As noted, the above scenario would result in a moisture content increase of 1.6%. Of course lower levels of moisture increase are feasible should the valve be closed further or if the mixer travels with greater speed. If the soil is too far beyond the optimum moisture content, then this is a problem that will have to be addressed regardless of whether the chemical additives are specified. The simplest approaches to that problem include the addition of more dry material (e.g., lime or dry soil if available) as well as aeration through a harrow disc type operation. In any case the above data are useful when making preliminary assessments as to what may be accomplished with field equipment.

Figure 4-3: Typical connection between RS-425 Mixer and Water Truck

Figure 4-4: Placement of metal pans under mixer, prior to passing

Figure 4-5: Collection of water from sprayer bar on mixer, after passing

Figure 4-6: Measurement of collected water in metal pans

4.3.2.2 Field Trial - construction of NC16 near Denver, Lincoln County, NC

The purpose of the field trial was to (1) develop NC-field relevant data for use in comparison with previous findings from the laboratory and open literature; (2) evaluate the extent to which $CaCl_2$ modification influences the rate of curing and strength gain of field applied and mixed soil-cement; (3) develop data to bracket the dose-response relationship and (4) evaluate the ease with which this approach integrates with field operations for road construction. The field plan developed before the trial is given as Appendix 4-1. This plan details much of the testing and schedule as well as roles and responsibilities.

Briefly, four test sections (each 92 meters long, 9.2 meters wide) were identified for extensive sample collection and instrumentation. Cement was mixed with the soil at a rate of 28 kg cement per square meter to a depth of 180 mm. For an estimated dry density of 17 kN/m^3 this corresponds to approximately 9% cement by dry weight of soil. These four test sections were constructed chronologically, identified as Control Section 1 (Location W for NCDOT UC Strength Data), Low Dosage Section (Location X for NCDOT UC Strength Data), High Dosage Section (Location Y for NCDOT UC Strength Data) and Control Section 2 (Location Z for NCDOT UC Strength Data). Notes and images taken during construction are given as Appendix 4-2.

On the basis of field-recorded volumes (i.e., estimating the volume of water and calcium chloride used in water trucks), the low and high dosage concentrations were estimated to be 2.7% and 7.3%, (mass of cement per mass of soil) respectively. Samples collected from the field and measured by Duke Energy’s analytical laboratory suggest a wider concentration range of 1.9% and 9.3%. The dosage percentages are expressed as the mass of dry $CaCl_2$ per mass of cement. Both of these measurements are subject to errors. Estimates of the volume discharged by the water truck were made visually. The analytical measurements are more precise but they are based on relatively small samples and are predicated on measurements of calcium alone. All dissolved calcium (Ca) was presumed to be associated with $CaCl_2$, however naturally occurring sources could have been present as well. Considering an average of 2.3% for the low dosage section indicates that 545 kg (1201 lb) of $CaCl_2$ was applied to the low dosage section by mixing 1,124 liters (297 gallons) of 38% $CaCl_2$ solution with water for a total of 13,247 liters (3500 gallons) of water/solution that was mixed with the subgrade. Similarly, using an average 8.3% for the high dosage section indicates that 1967 kg (4333 lb) of $CaCl_2$ was applied to the low dosage section by mixing 4,055 liters (1071 gallons) of 38% $CaCl_2$ solution with water for a total of 11,166 liters (2950 gallons) of water/solution that was mixed with the subgrade.

4.4 Results

4.4.1 Soil Classification

The NCDOT Materials and Tests Unit classified the Lincoln County soils obtained for this project. A summary of their findings is provided in Table 4-3 below. The moisture density curve for Lincoln County soil, is presented in Figure 4-7. Classifications and moisture density curves for Buncombe, Guilford, and Johnston County were provided in Chapter 2.

Table 4-3: Soil sample classifications – Lincoln County

		High Dosage Section		Low Dosage Section		Control Section 1		Control Section 2	
Proj. Sample No.		**Sample 1**	**Sample 2**	**Sample 1**	**Sample 2**	**Sample 1**	**Sample 2**	**Sample 1**	**Sample 2**
Lab. Sample No.		**760080**	**760084**	**760081**	**760085**	**760082**	**760086**	**760083**	**760087**
Retained #4 Sieve	%	**1**	**3**	**4**	**7**	**1**	**-**	**2**	**-**
Passing #10 Sieve	%	**96**	**93**	**93**	**89**	**97**	**98**	**95**	**97**
Passing #40 Sieve	%	**84**	**73**	**84**	**77**	**70**	**89**	**85**	**86**
Passing #200 Sieve	%	**50**	**30**	**50**	**46**	**22**	**58**	**49**	**52**

MINUS NO. 10 FRACTION

SOIL MORTAR - 100%									
Coarse Sand Ret - #60	%	**22.8**	**35.4**	**22.2**	**24.0**	**45.9**	**18.7**	**21.7**	**21.5**
Fine Sand Ret - #270	%	**25.2**	**32.7**	**25.2**	**24.4**	**31.9**	**23.0**	**27.0**	**25.0**
Silt 0.05 - 0.005 mm	%	**27.6**	**25.8**	**24.2**	**27.2**	**18.1**	**25.8**	**32.9**	**29.1**
Clay < 0.005 mm	%	**24.4**	**6.1**	**28.5**	**24.4**	**4.1**	**32.5**	**18.3**	**24.4**
Passing #40 Sieve	%	-	-	-	-	-	-	-	-
Passing #200 Sieve	%	-	-	-	-	-	-	-	-

L. L.	**39**	**44**	**38**	**41**	**50**	**39**	**39**	**40**
P. I.	**9**	**NP**	**8**	**11**	**NP**	**11**	**6**	**13**
AASHTO Classification	**A-4(3)**	**A-2-5(0)**	**A-4(2)**	**A-7-5(3)**	**A-2-5(0)**	**A-6(5)**	**A-4(1)**	**A-6(5)**

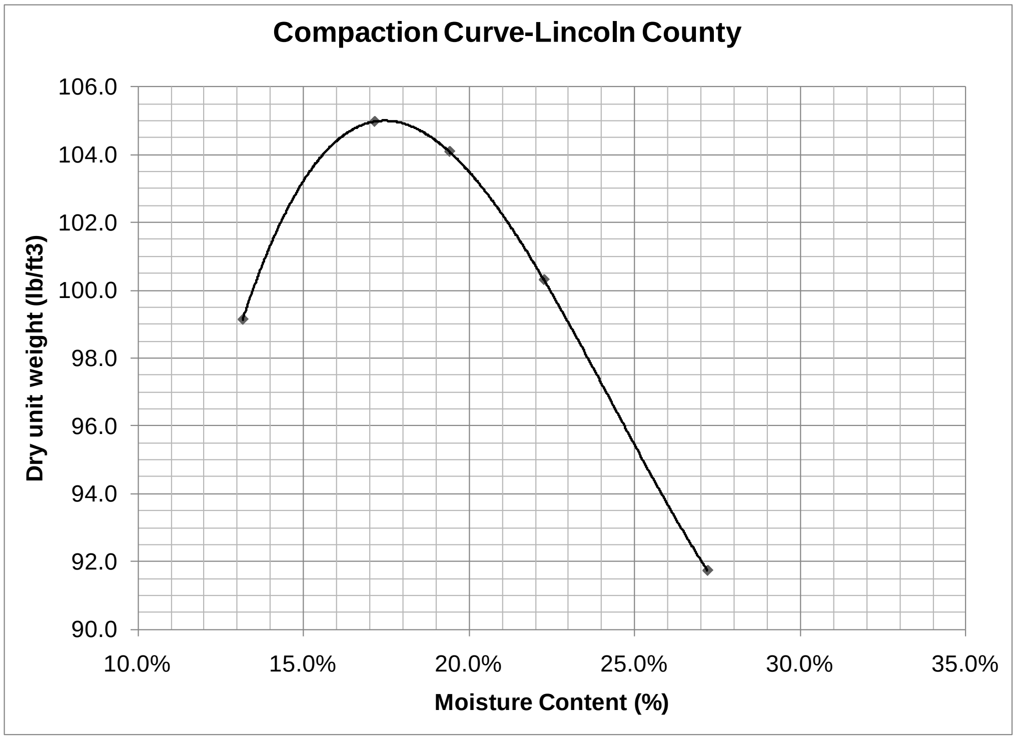

Figure 4-7: Compaction curve for Lincoln County soil (9% cement)

4.4.2 Influence of Additive on Moisture Density Relationships

In general, the concentration of $CaCl_2$, while high enough to influence cement hydration, was too low to meaningfully impact moisture density relationships. An exception to this are data that were generated early in the research which explored high dosage levels (0.5% - 3.0% of $CaCl_2$ by weight of *soil*). The data from the moisture density tests on the Buncombe, Guilford, and Johnston county soils appears in Figures 4-8 through 4-10 respectively.

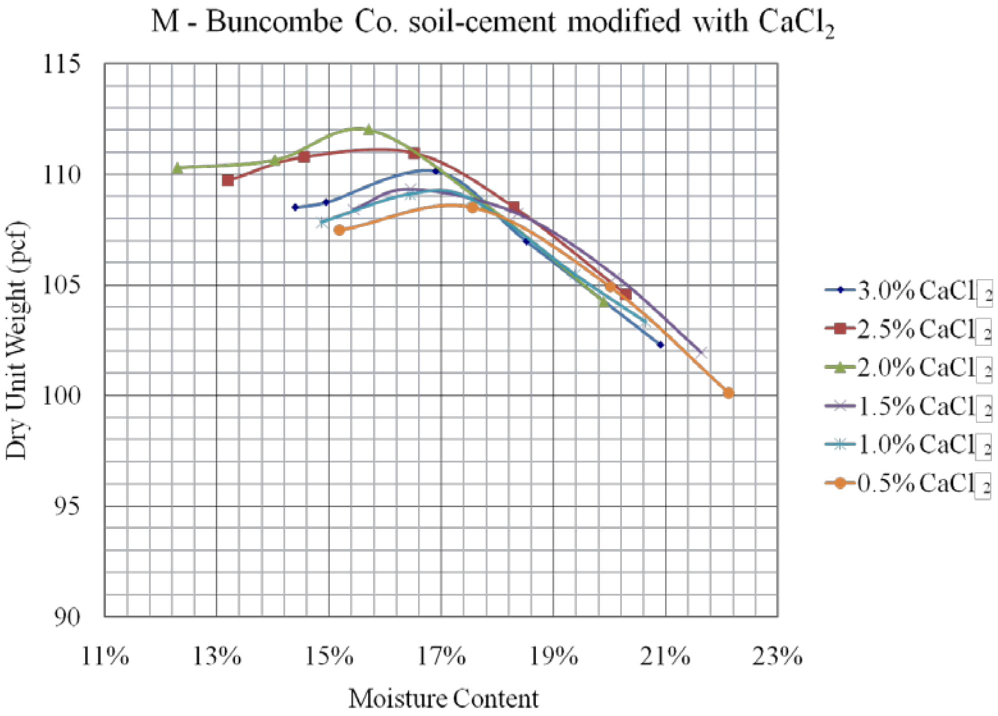

Figure 4-8: Moisture density curves for Buncombe County soil-cement plus additive per mass of dry soil

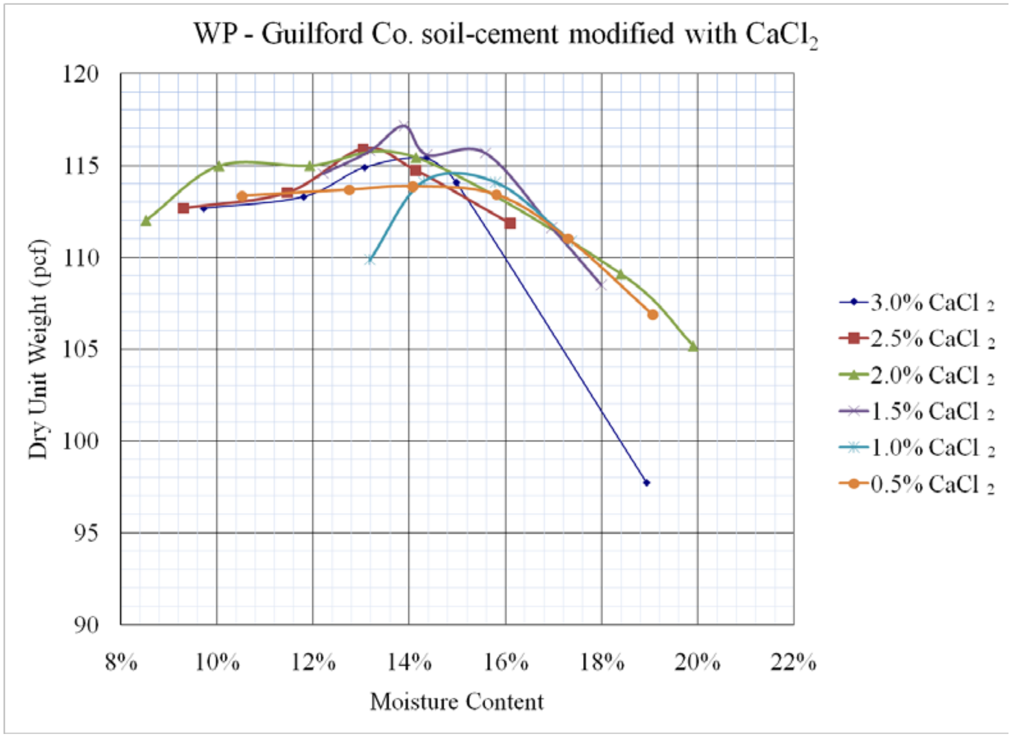

Figure 4-9: Moisture density curves for Guilford County soil-cement plus additive per mass of dry soil

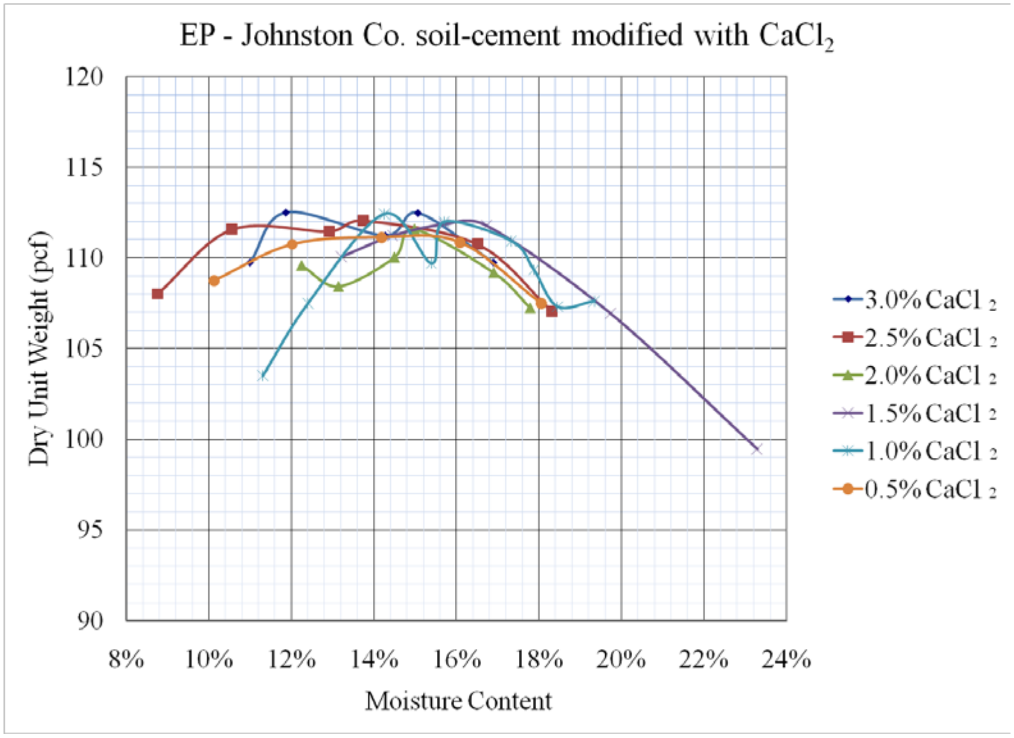

Figure 4-10: Moisture density curves for Johnston County soil-cement plus additive per mass of dry soil

Figure 4-11 provides a summary of Figures 4-8 through 4-10 by plotting the maximum dry density for each moisture density test for each county versus the $CaCl_2$ percentage added.

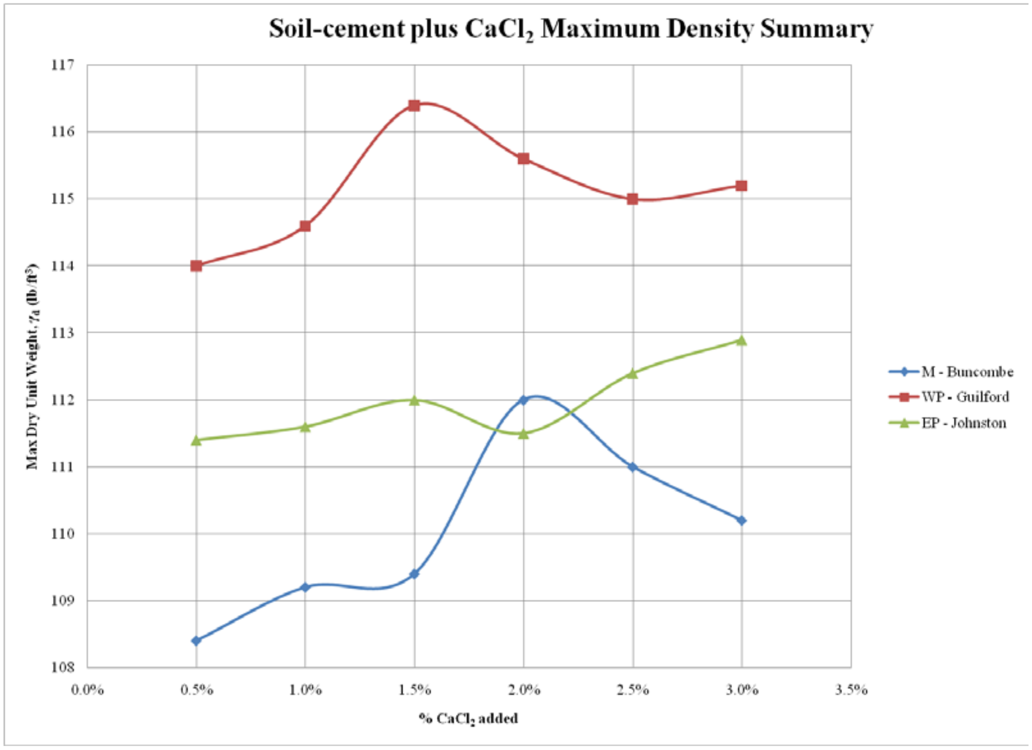

Figure 4-11: Summary graph displaying the maximum density for each $CaCl_2$ percentage added

Early in the research, the foregoing data were used to select the dosage range of $CaCl_2$ modification, i.e., selecting the concentration which maximizes density. While appropriate for optimizing moisture content and physical methods of soil stabilization, a more effective method of optimizing $CaCl_2$ is based on unconfined compressive strength. The principal effect of $CaCl_2$ on soil cement is accelerated cement hydration, not increased density.

4.4.3 Laboratory-Based Analysis (Buncombe County, Guilford County, Johnston County)

The results for the laboratory-based $CaCl_2$ modification are reported in two categories, namely, (1) samples which include "non-homogenized" soil that was taken straight from the original sampling barrels (prior to the fall of 2008) and include a wide range of $CaCl_2$ dosages (0-50%, mass of $CaCl_2$ per mass of cement) and (2) samples which were "homogenized" and tested within a narrow range (0-2.0%, mass of $CaCl_2$ per mass of cement).

In addition, a limited amount of data was generated on the influence of sodium chloride (NaCl) on the unconfined compression strength of soil-cement, and this is presented in tabular form in Appendix 4-3. In particular, for the range tested (0.5, 1.0 and 2.0 % by weight of *water*) NaCl amendment resulted in significant increases in strength for Guilford County soil-cement and to a lesser extent for Johnston County soil-cement for two different curing durations (3 days, 7 days) and temperatures (35^0F, 50^0F). NaCl amendment resulted in mostly decreases for Buncombe County soil-cement. By way of example, Figure 4-12 provides a comparison of the effects of various concentrations of NaCl and $CaCl_2$ on the unconfined compression strength of Guilford County soil-cement cured at 35^0F for 7 days. As can be observed, while NaCl improves the strength relative to the control, $CaCl_2$ achieves even greater strengths for Guilford County soil-cement. As might be expected, different additives may have different effects on different soils, and the reverse behavior was observed for Johnston County soil, i.e., greater strength was obtained with NaCl as compared to $CaCl_2$, as shown in Figure 4-13 for specimens cured at 50^0F for 7 days (there were more data available to illustrate this point at 50^0F). There was no significant difference between NaCl or $CaCl_2$ for Buncombe County soil-cement as shown in Figure 4-14 for specimens cured at 50^0F (again, there were more data available to illustrate this point at 50^0F). These data indicate NaCl increases strength beyond the control in two of the three soils (Guilford and Johnston), while $CaCl_2$ accomplished this for only one of the soils (Guilford). Neither additive improved the control strength for Buncombe County soil. At this stage in the research, the concentration range was not necessarily optimized, and based on the literature it was presumed that such an optimum range could be found for either additive through expanded testing. $CaCl_2$ was chosen for this expanded testing. The rationale for pursuing $CaCl_2$ over NaCl involved considerations of availability, cost and supporting literature as well as experimental simplicity and tractability.

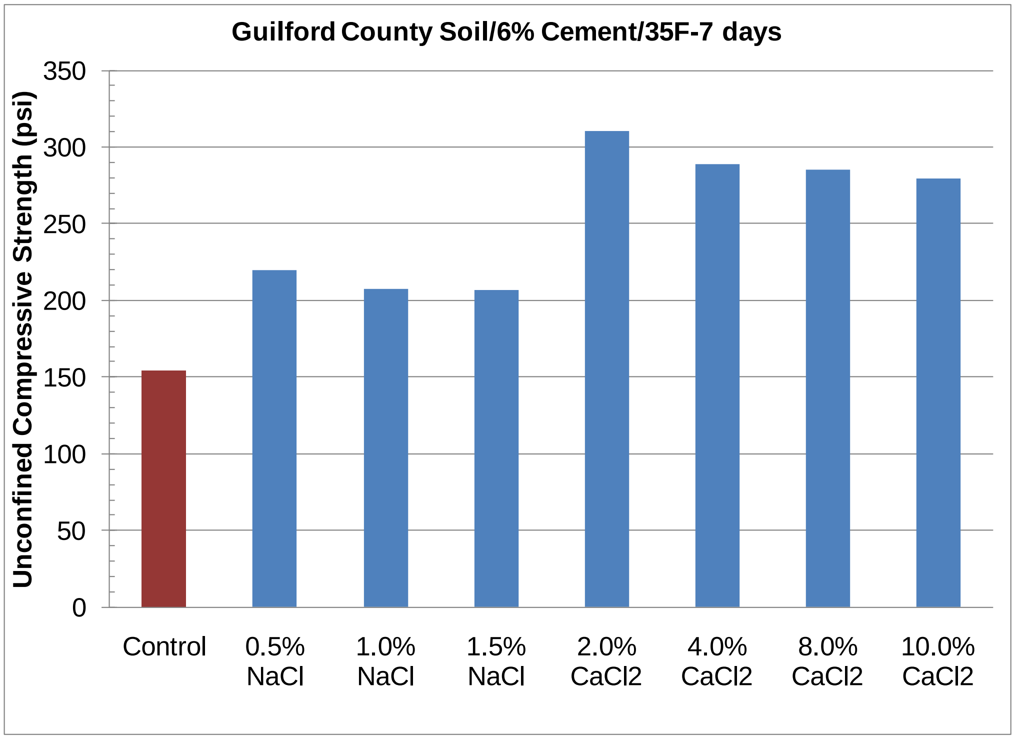

Figure 4-12: UC Strength for Guilford County Soil-Cement as a function of NaCl or $CaCl_2$ (Example Comparison: cured at 35^0F for 7 days)

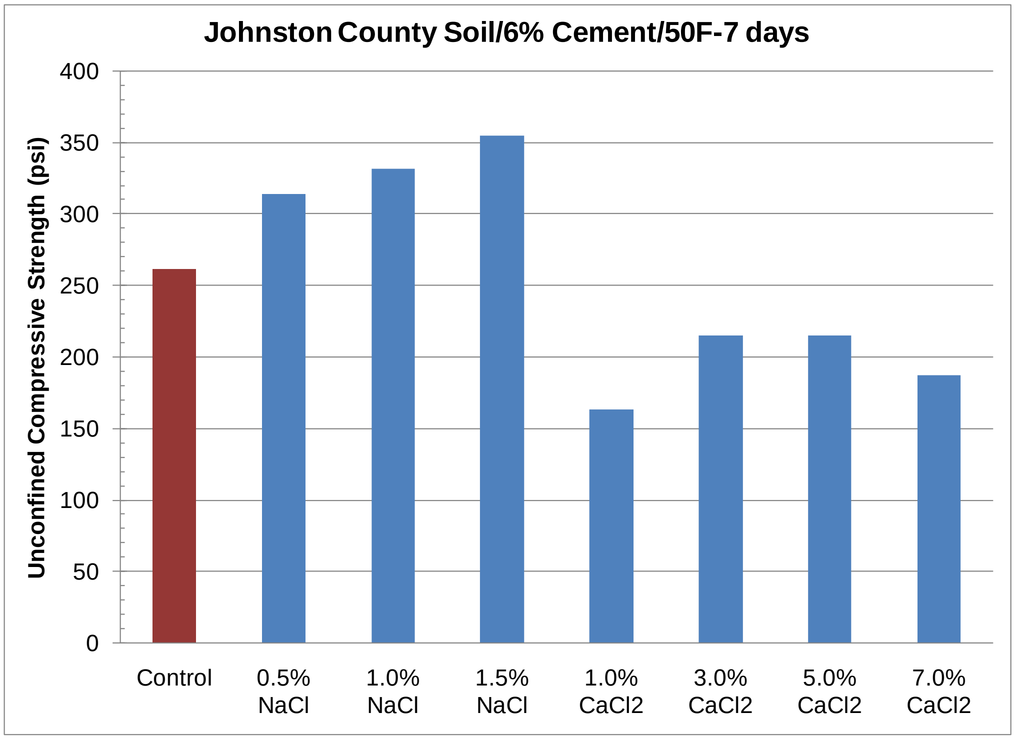

Figure 4-13: UC Strength for Johnston County Soil-Cement as a function of NaCl or $CaCl_2$ (Example Comparison: cured at 50^0F for 7 days)

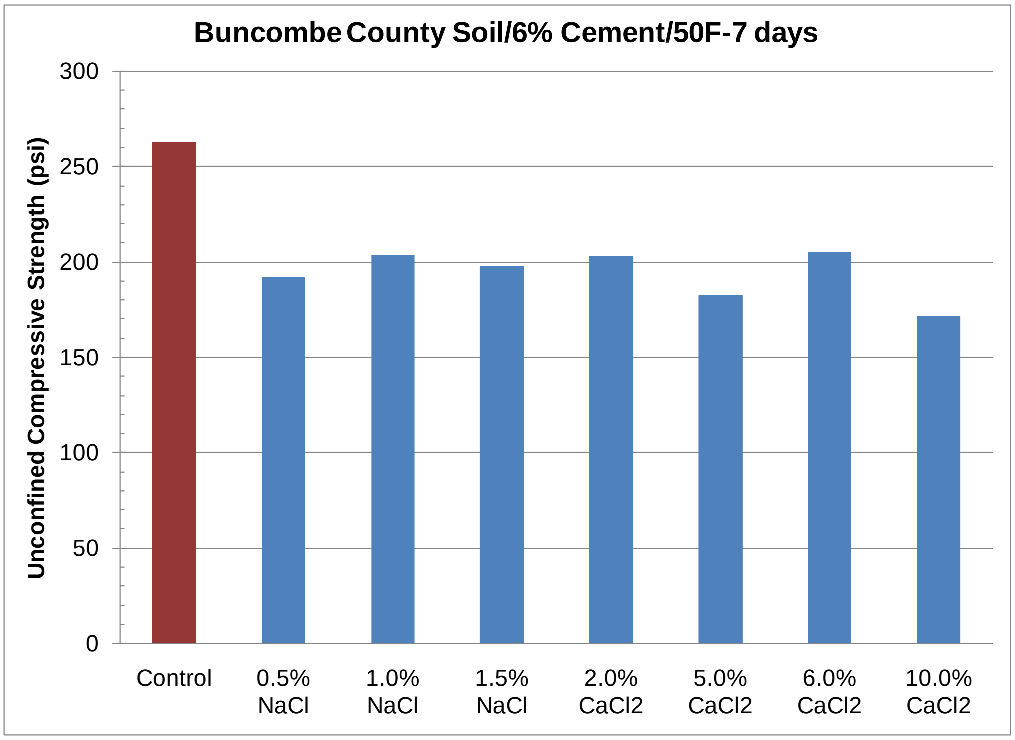

Figure 4-14: UC Strength for Buncombe County Soil-Cement as a function of NaCl or $CaCl_2$ (Example Comparison: cured at 50^0F for 7 days)

4.4.3.1 UC Strength, wide range (0-50%), including non-homogenized samples

As the critical reader will observe, $CaCl_2$ dosage amendment has been described in three different units, namely (1) mass of $CaCl_2$ per mass of cement, e.g., as in Section 4.2, (2) mass of $CaCl_2$ per mass of soil e.g., as in Section 4.4.2, (3) mass of $CaCl_2$ per mass of water, e.g., as in Section 4.4.3. Such variability reflects the literature and different approaches to soil-cement modification. Following the logic that $CaCl_2$ acts primarily to accelerate cement hydration, and that that is the fraction which is most significant, most of the references to $CaCl_2$ modification in this report are expressed as mass of $CaCl_2$ per mass of cement. In the case of data that were previously reported as mass of $CaCl_2$ per mass of soil or water, they have been converted to units of mass of $CaCl_2$ per mass of cement. For example, initially some specimens were tested as high as 3% (mass of $CaCl_2$ per mass of soil) and this corresponds to 50% (mass of $CaCl_2$ per mass of cement) for 6% cement.

Testing was intended to document trends with the effect of $CaCl_2$ on soil cement strength as a function of temperature and curing duration. The experimental design was adjusted as results were obtained. Depending on the results and the emerging pattern (or lack thereof), some soils were tested under a wider range of concentrations and conditions than others. Results for Buncombe, Guilford and Johnston County soils are presented in tabular form in Appendix 4-4, with representative figures for the broadest dosage range available (which varies according to soil type) presented in Figures 4-15, 4-16 and 4-17, respectively.

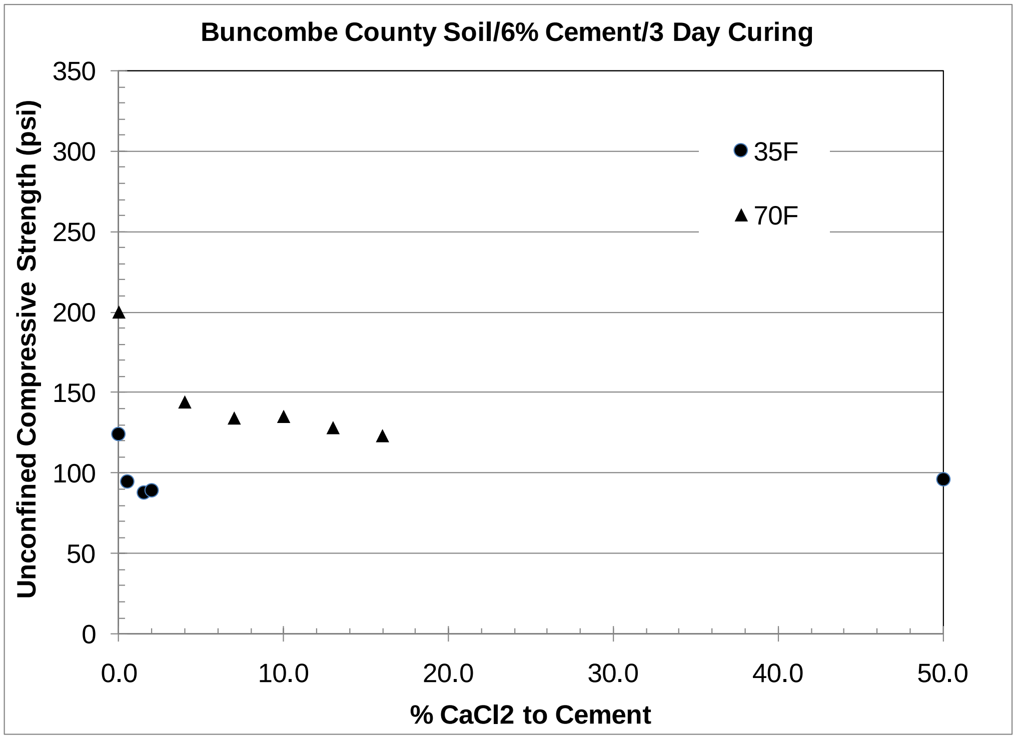

Figure 4-15: UC Strength for Buncombe County Soil-Cement as a function of $CaCl_2$ (Example Comparison: cured for 3 days at either 35^0F or 70^0F)

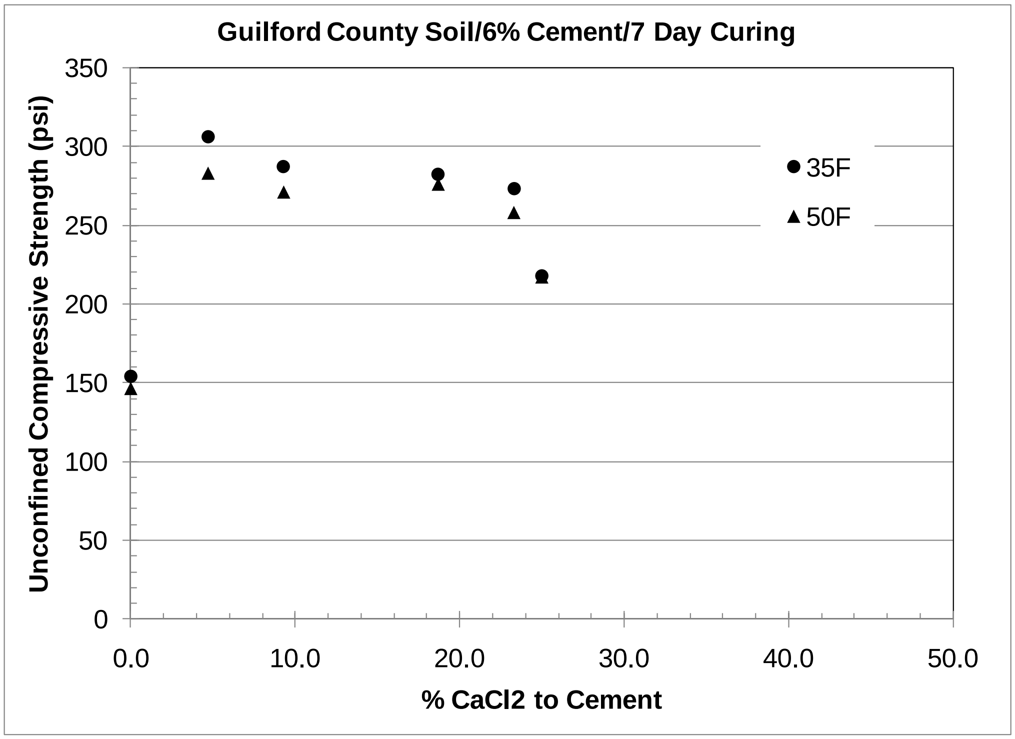

Figure 4-16: UC Strength for Guilford County Soil-Cement as a function of $CaCl_2$ (Example Comparison: cured for 7 days at either 35^0F or 50^0F)

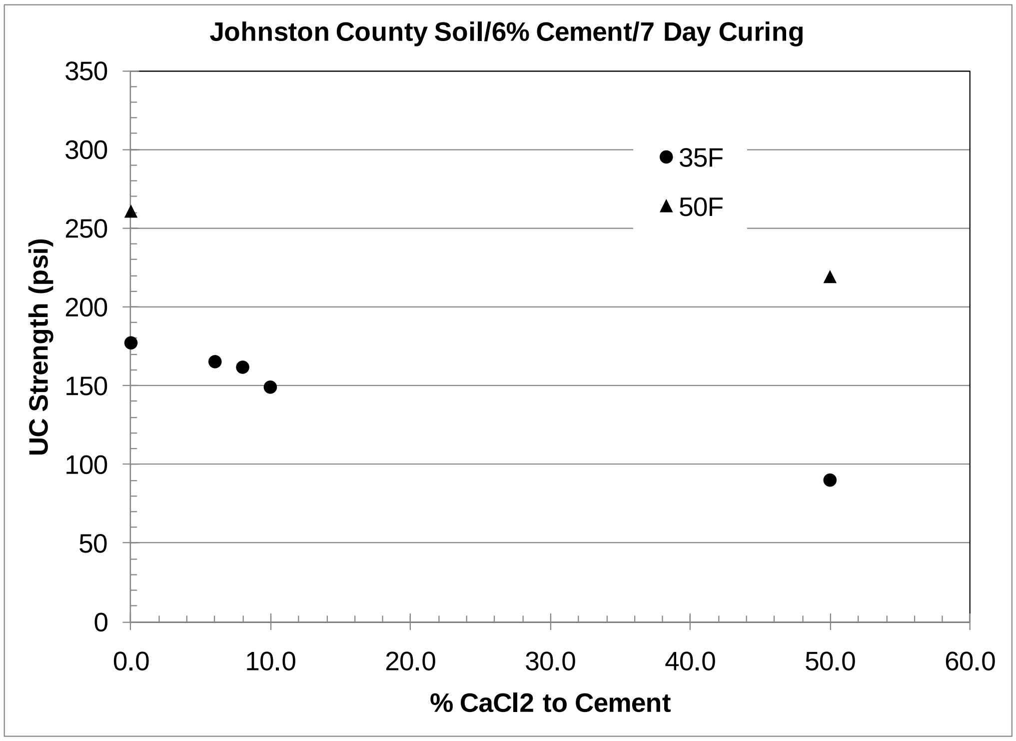

Figure 4-17: UC Strength for Johnston County Soil-Cement as a function of $CaCl_2$ (Example Comparison: cured for 7 days at either 35^0F or 50^0F)

As the foregoing data indicate, the use of $CaCl_2$ resulted in higher unconfined compression strengths for Guilford County soils (at least for the 35^0F or 50^0F curing temperatures shown above) but generally mixed results or decreased strength for Buncombe and Johnston County soils. Several factors were considered during this phase of the research, namely, (1) the effect of soaking the specimens (i.e., might the soaking phase after curing but before testing serve to leach $CaCl_2$ and decrease its ability to accelerate strength gain), (2) the effect of moisture content being optimized for density as opposed to water to cement ratio, the latter being relevant to concrete, (3) significance of compaction energy by the manual method versus an automatic compactor, (4) heterogeneity across the eight barrels (~4500 lb) of soil collected for a given county, (5) significance of 3 day data versus 7 day data in making assessments of $CaCl_2$ efficacy, (6) likely concentration range based upon previous work and available literature. After considering each of the above items in turn, some with abbreviated experimental campaigns, the conclusion was to focus on items (4) through (6) through an additional suite of testing of a narrow $CaCl_2$ concentration range. This is discussed in the next section.

4.4.3.2 UC Strength, narrow range (0-2%), homogenized samples

For this suite of experiments, all of the remaining soil that was previously collected in several drums was completely re-mixed so that the entire batch could be presumed to be more uniform. Prior to this, it is conceivable that the actual soil classification varied from one drum to another or from the top to the bottom of a given drum. Likewise, the basis of comparison for this suite of comparisons was 7 days. Curing durations shorter than this may be too short for the effects of $CaCl_2$ to manifest, as reported by Uppot (2007, 2008). Likewise curing durations longer than 7 days would require more time than available to the project. The concentration range to be investigated was selected as 0 – 2.0%, as this appeared the most promising based on the preliminary data and/or the literature. The idea was to fill out this range for all three soils, as shown in Table 4-3.

Table 4-3: Experimental Design for Narrow Range of $CaCl_2$ Modification

All Samples Cured for 7 Day (3 samples prepared per County)						
Dosages (mass $CaCl_2$ / mass cement, %)	**Johnston County 35^0F**	**Johnston County 50^0F**	**Guilford County 35^0F**	**Guilford County 50^0F**	**Buncombe County 35^0F**	**Buncombe County 50^0F**
0.0	3	3	3	3	3	3
0.2	3	3	3	3	3	3
0.5	3	3	3	3	3	3
1	3	3	3	3	3	3
1.5	3	3	3	3	3	3
2	3	3	3	3	3	3
Total	**18**	**18**	**18**	**18**	**18**	**18**

The results of this testing is reported in Appendix 4-5 and as average strength plotted as a function of dosage in Figures 4-18 and 4-19. Given the tighter range of concentrations tested, the individual data points are connected with a smooth line although the true relationship between concentration and strength is not necessarily known.

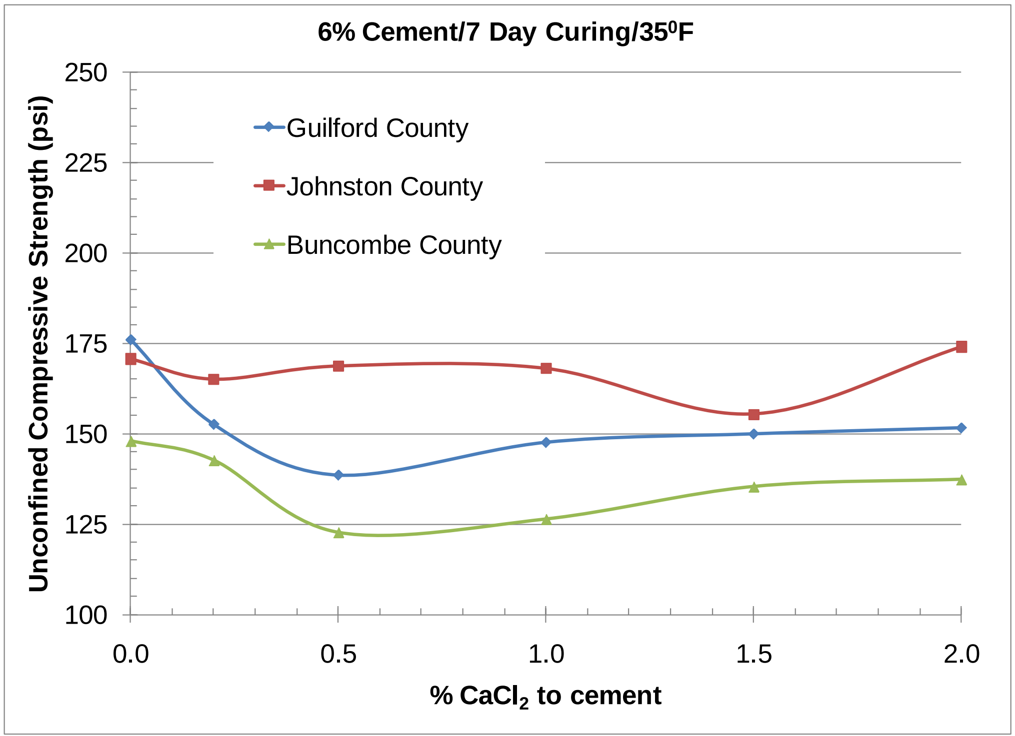

Figure 4-18: UC Strength as a function of $CaCl_2$ (cured for 7 days at 35^0F)

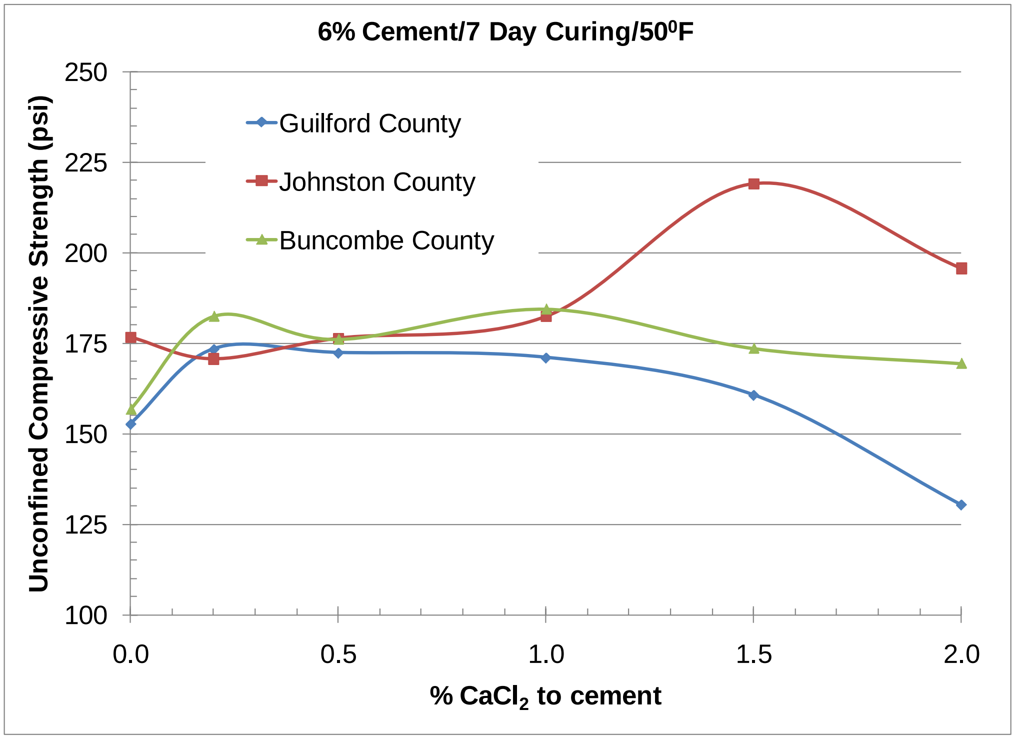

Figure 4-19: UC Strength as a function of $CaCl_2$ (cured for 7 days at 50^0F)

The above data suggest decreasing strength with $CaCl_2$ modification for curing at 35^0F while there appears to be a modest window of efficacy for curing at 50^0F, unique for each soil. Temperature is known to affect cement hydration and its susceptibility to $CaCl_2$ modification, so the above may suggest that higher concentrations (beyond the 2.0% level tested) are needed at the lower temperature. There remain gaps in plausibly effective concentrations tested in this research.

4.4.4 Tube Suction Test Results

TST testing was conducted as a possible indicator of whether $CaCl_2$ modification influences the moisture susceptibility and durability of soil-cement mixes. The dosage levels were selected to bracket the maximum range of $CaCl_2$ modification for field use. At the time the tests were conducted, this range was deemed to have 10% as its upper bound and so the effect of $CaCl_2$ modification was evaluated by testing 5 and 10% dosage levels. The results of the TST are presented in Figures 4-20 through 4-25 as well as associated data in Tables 4-4 through 4-6.

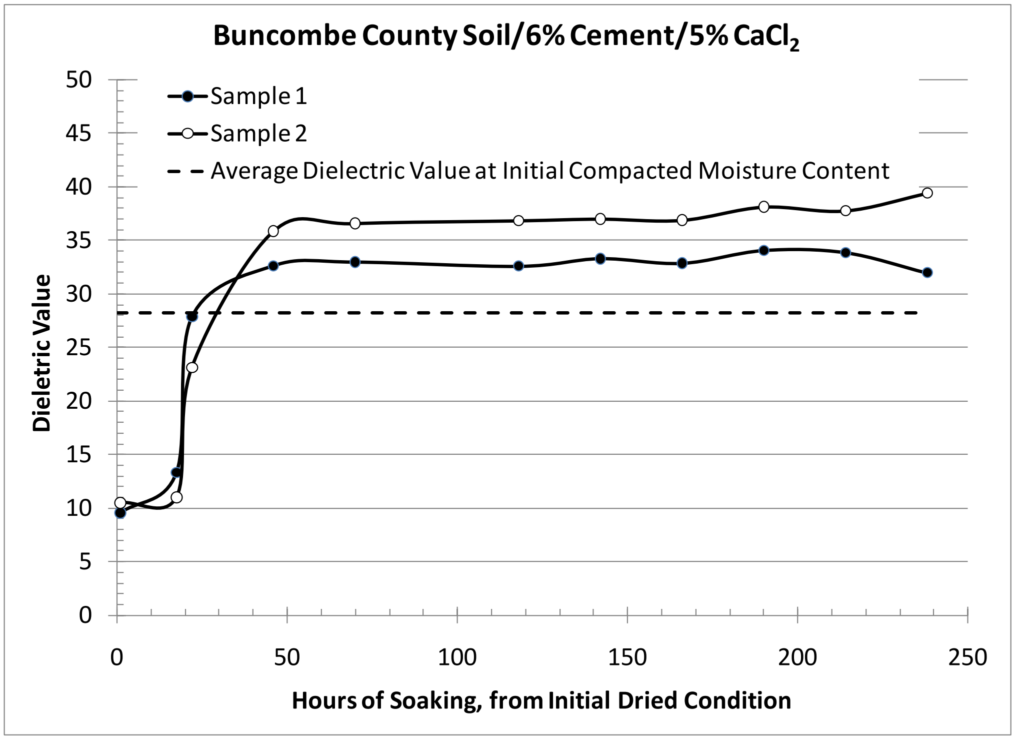

Figure 4-20: TST Results for two specimens (Buncombe County – 5% $CaCl_2$)

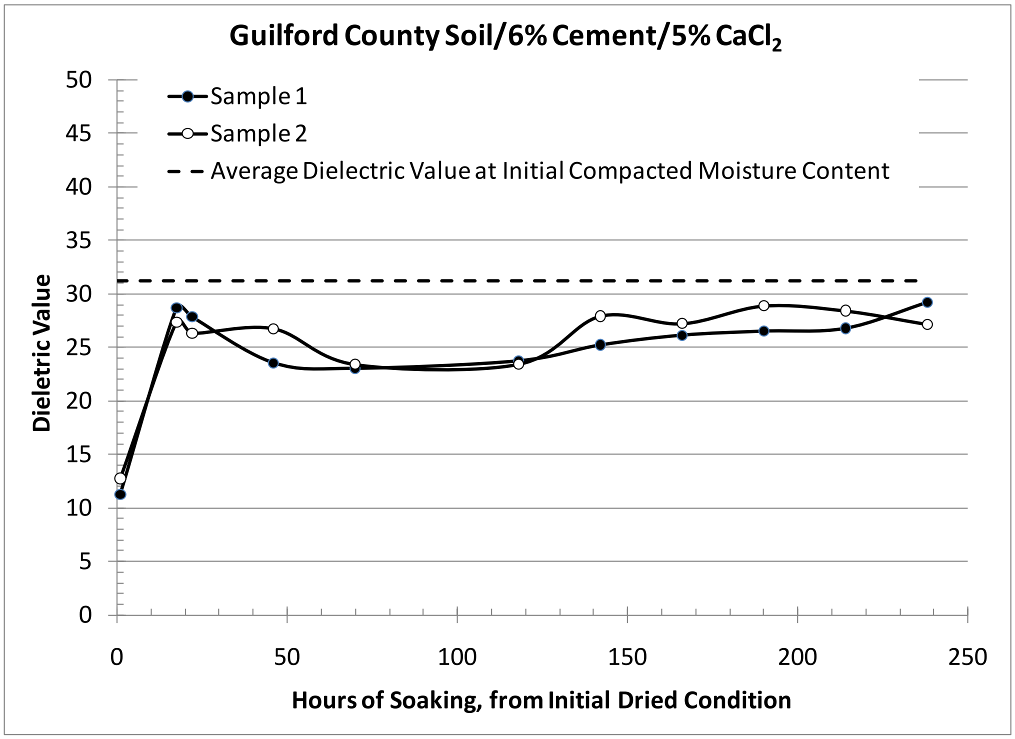

Figure 4-21: TST Results for two specimens (Guilford County – 5% $CaCl_2$)

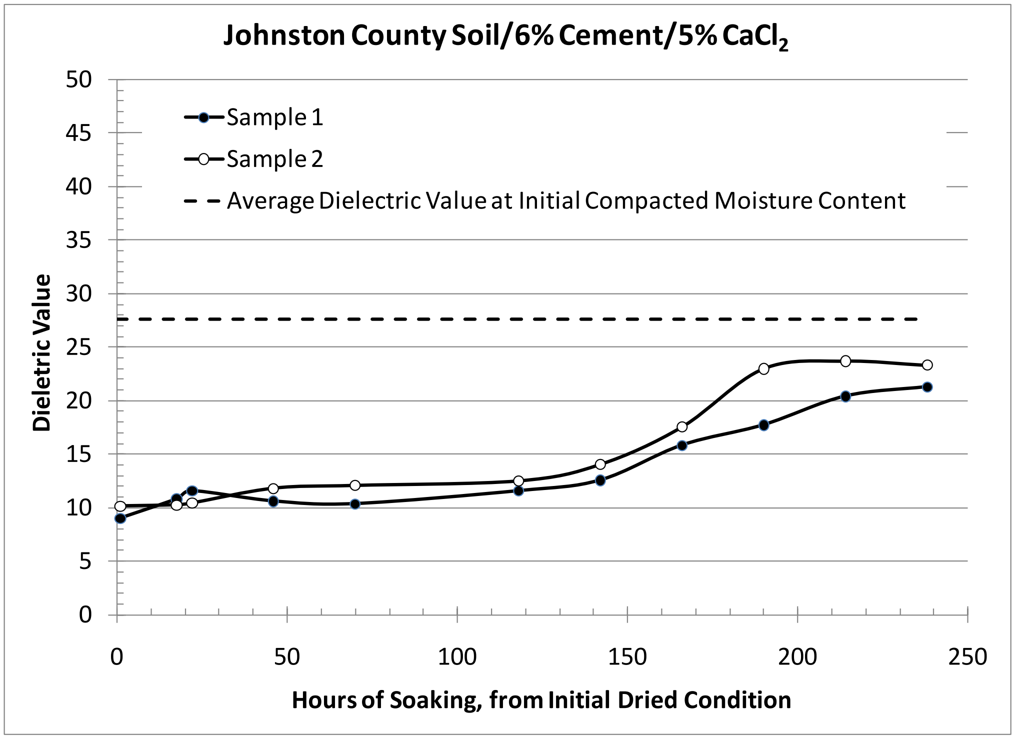

Figure 4-22: TST Results for two specimens (Johnston County– 5% $CaCl_2$)

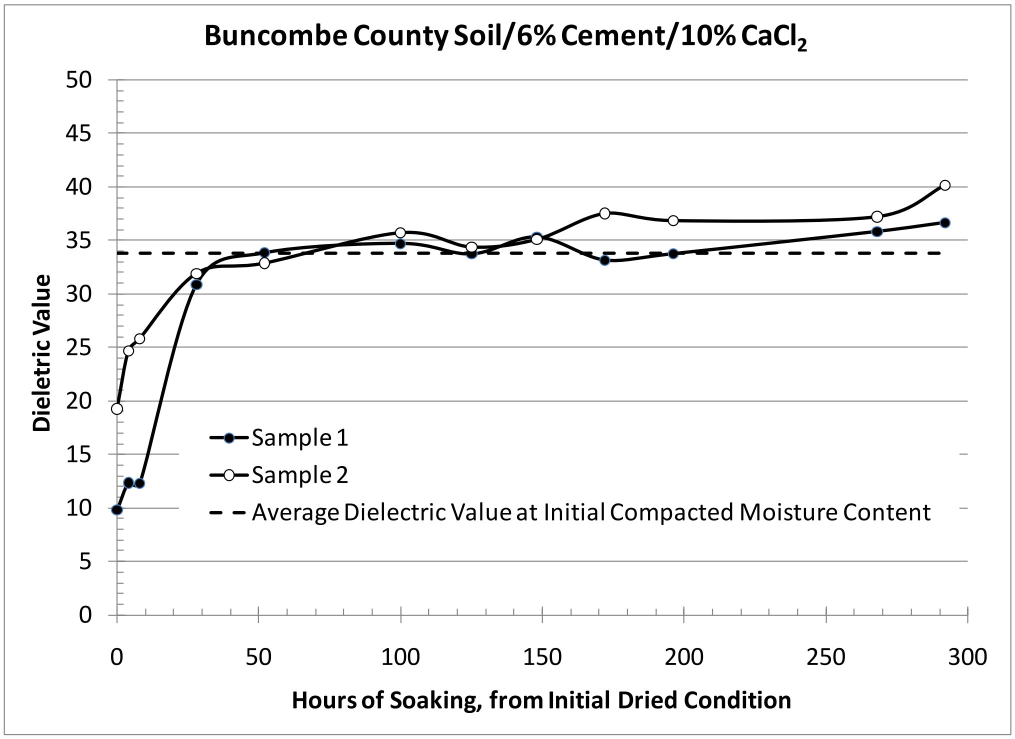

Figure 4-23: TST Results for two specimens (Buncombe County – 10% $CaCl_2$)

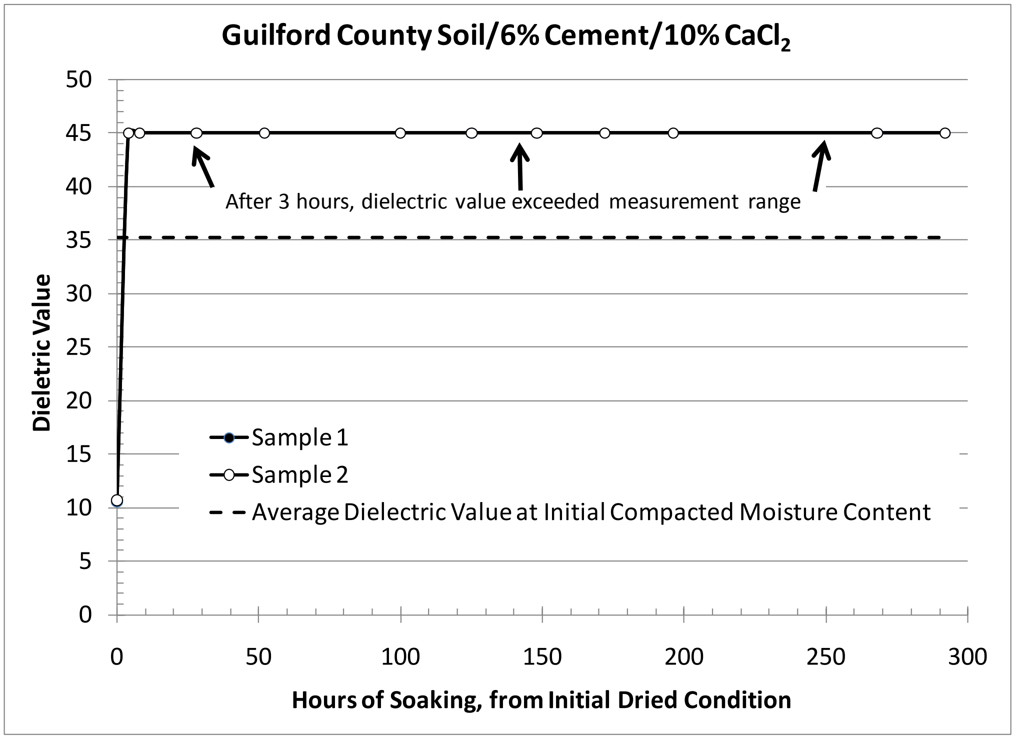

Figure 4-24: TST Results for two specimens (Guilford County – 10% $CaCl_2$)

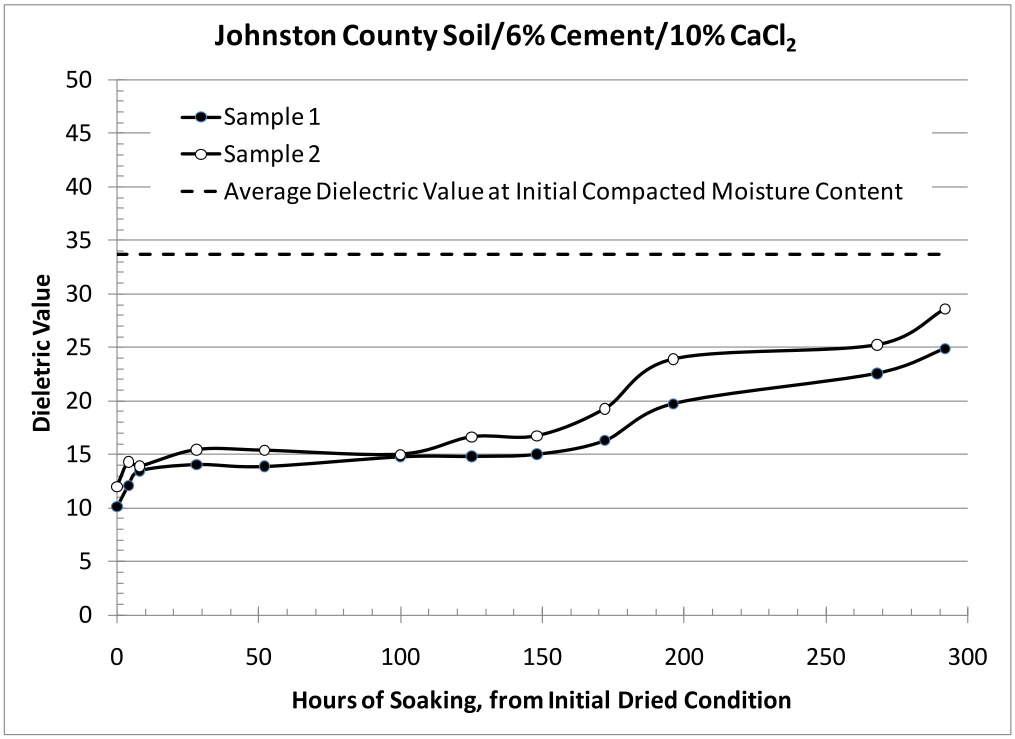

Figure 4-25: TST Results for two specimens (Johnston County– 10% $CaCl_2$)

Table 4-4: Comparison of DV and EC as a function of $CaCl_2$ – Buncombe County

	Buncombe County											
	Control				5% $CaCl_2$				10% $CaCl_2$			
	B1		B2		B1		B2		B1		B2	
	DV	EC	DV	EC	DV	EC	DV	EC	DV	EC	DV	EC
Original	14.02	37.8	15.8	73.2	25.58	625.8	30.88	818	32.64	1062	34.98	951.4
Dry status	4.976	0	5.956	0	9.57	1.2	10.49	1.4	9.81	0	19.24	11.8
Final	19.22	64.8	21.52	133.2	31.96	824	39.4	1344.8	36.66	1525.6	40.17	1871.5

DV is dielectric value, EC is electric conductivity (µS/cm), "Original" means the status of the specimen just after compaction. "Dry status" means the status that after 24 hours dry and cool to room temperature. "Final" means after at least 10 days testing in water.

Table 4-5: Comparison of DV and EC as a function of $CaCl_2$ – Guilford County

	Guilford County											
	Control				5% $CaCl_2$				10% $CaCl_2$			
	G1		G2		G1		G2		G1		G2	
	DV	EC	DV	EC	DV	EC	DV	EC	DV	EC	DV	EC
Original	15.76	67.2	16.04	105.2	29.74	669.8	32.72	746.8	35.94	949	34.56	999.4
Dry status	7.186	0	8.188	0	11.28	1.8	12.74	1.8	10.56	1.4	10.68	1.0
Final	25.18	247.8	29.56	331.2	29.2	2837.2	27.16	3333.4	>45	4892	>45	4892

DV is dielectric value, EC is electric conductivity (µS/cm), "Original" means the status of the specimen just after compaction. "Dry status" means the status that after 24 hours dry and cool to room temperature. "Final" means after at least 10 days testing in water.

Table 4-6: Comparison of DV and EC as a function of $CaCl_2$ – Johnston County

	Johnston County											
	Control				**5% $CaCl_2$**				**10% $CaCl_2$**			
	J1		**J2**		**J1**		**J2**		**J1**		**J2**	
	DV	**EC**	**DV**	**EC**	**DV**	**EC**	**DV**	**EC**	**DV**	**EC**	**DV**	**EC**
Original	19.18	12.98	18.56	25	27.98	596.7	27.32	594.5	30.86	329.2	36.62	523.8
Dry status	7.536	0	6.962	0	9.0	0.6	10.15	0.4	10.10	0	12.0	0.2
Final	13.84	6	14.1	8	21.28	90.4	23.3	133	24.88	176.8	28.6	232.2

DV is dielectric value, EC is electric conductivity (µS/cm), "Original" means the status of the specimen just after compaction. "Dry status" means the status that after 24 hours dry and cool to room temperature. "Final" means after at least 10 days testing in water.

DV values may be taken as an indicator of moisture content, although they may well be sensitive to other components in the soil-water-cement-additive matrix. As with the data presented in Chapter 2 (and partially repeated in the above tables for the control data), the foregoing data indicate that Johnston County soil-cement mix is the least susceptible to moisture-induced damage, while Guilford and Buncombe County soil-cement mixes are more susceptible. Guilford and Buncombe County soils are also more sensitive to $CaCl_2$ modification, as noted by the increases in dielectric value and electrical conductivity as well as the final dielectric value divided by the initial dielectric value at initial compaction. From the data in Tables 4-4 through 4-6, it is apparent that $CaCl_2$ modification increases the final, steady-state dielectric value, i.e., it ostensibly attracts more moisture. However the effect of $CaCl_2$ modification is normalized according to the dielectric value at initial compaction. For example, the unmodified Guilford County soil attracts more moisture through capillary action than exists at compaction. When modified by 5% $CaCl_2$, it theoretically attracts less moisture through capillary action than exists at compaction – the difference being that the dielectric value at compaction was higher for the $CaCl_2$ modified specimens. However, given that the initial moisture contents (gravimetrically determined) were the same, this increase in dielectric value likely reflects the presence of $CaCl_2$ and/or its influence on the association of water with soil particles and cement hydration. It is quite clear that these dosage levels of $CaCl_2$ increase electrical conductivity.

4.4.5 Field Trial - construction of NC16 near Denver, Lincoln County, NC

The principal basis for assessing the influence of $CaCl_2$ modification was (1) UC strength of samples collected from the field (i.e. field mixed, compacted with Proctor hammer in field), (2) UC strength tests, whereby soil was collected from the field site, but soil-cement was mixed and compacted in the laboratory using two additional $CaCl_2$ dosage levels (1% and 5%) beyond that actually tested in the low dosage (2.3%) or high dosage (8.3%) sections and (3) dynamic cone penetration (DCP) tests conducted in the field, the latter was led by Tom Hearne with NCDOT.

4.4.5.1 UC strength of samples collected from the field

UC strength was measured by NCDOT and UNC Charlotte researchers. Results from NCDOT 7-day UC strength are presented in Table 4-7. UNC Charlotte data are presented as Appendix 4-6 and as Figures 4-26 and 4-27.

Table 4-7: NCDOT unconfined compression strength data

(7 day curing, strength denoted by "PSI" column)

Location		Spec#	**PSI**	Station Number	Made By	Remarks
Control Section 1	W	32A	**299**	170+08 LT	RMR	OPT+1
Control Section 1	W	33E	**349**	170+32 LT	JHM	OPT+1
Control Section 1	W	34 I	**170**	170+56 LT	DMP	OPT+4
Low Dosage	X	35A	**322**	171+00 LT	RMR	OPT
Low Dosage	X	36E	**139**	171+24 LT	JHM	OPT+1
Low Dosage	X	37 I	**143**	171+48 LT	DMP	OPT+2
High Dosage	Y	38A	**478**	171+94 LT	RMR	OPT-1
High Dosage	Y	39E	**100**	172+18 LT	JHM	OPT+1
High Dosage	Y	40 I	**84**	172+42 LT	DMP	OPT
Control Section 2	Z	41A	**162**	172+85 LT	RMR	OPT+1
Control Section 2	Z	42E	**225**	173+09 LT	JHM	OPT+1
Control Section 2	Z	43 I	**200**	173+33 LT	DMP	OPT

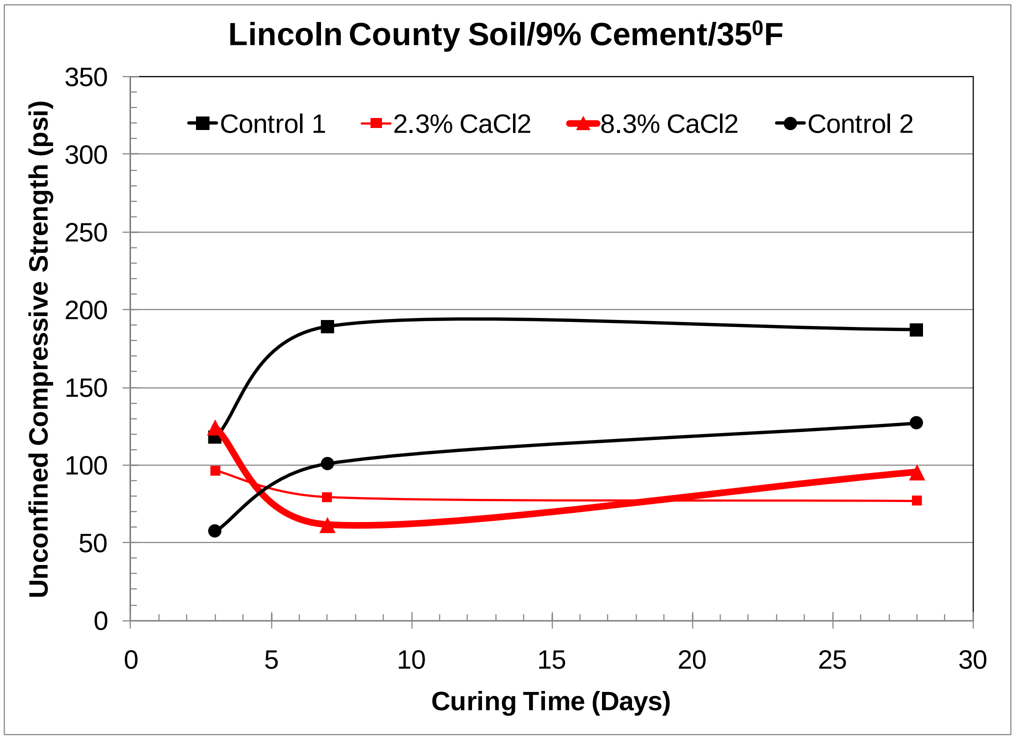

Figure 4-26: UC Strength Development Comparison for field samples (cured at 35^{0}F)

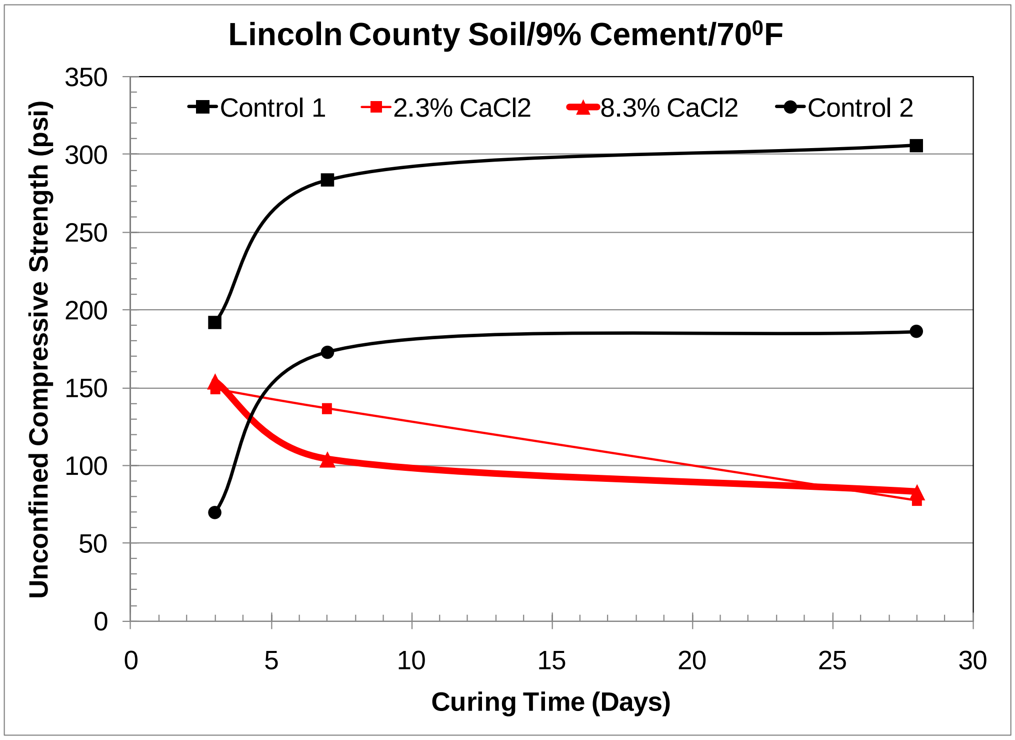

Figure 4-27: UC Strength Development Comparison for field samples (cured at 70^{0}F)

The above data indicate that at the dosage levels tested, $CaCl_2$ modification resulted in clear decreases in UC strength. These data indicate that if modification has merit, it would be at much lower dosages than suggested by Uppot et al. (2008), perhaps more like proposed by Rollings et al. (2002) and tested as part of the narrow range (0-2%) in this research.

4.4.5.2 UC strength of samples collected from the field

The results of samples made of soil collected from the construction site at Lincoln County, but mixed with cement and compacted in the lab are presented as Appendix 4-7 and as Figures 4-28 and 4-29. In all cases, strength increases with curing duration. $CaCl_2$ modification results in increases in UC strength at 28 days for the 1% dosage at 35^0F while both 1% and 5% increase the UC strength for all curing durations at 70^0F. These results, especially the 5% dosage level are in contrast to the values observed with samples that were field-mixed and compacted. By definition, one potential explanation is the variation between field mixing and laboratory mixing, i.e., the latter could have led to better soil-chemical-cement interaction and thus better strength. Also, while the curing conditions were the same for laboratory-mixed and field-mixed samples (35^0F and 70^0F), the field mixed samples were much warmer at initial mixing and collection, as ambient daytime temperatures were in excess of 90^0F, as noted in Chapter 3. Warmer temperatures may be analogous to increasing the $CaCl_2$ concentration to a deleterious level, as discussed in Dow (2006). In addition, the soil conditions varied across the four test sections and so what defines the “control” sample is not quite the same when comparing the field and laboratory mixed samples.

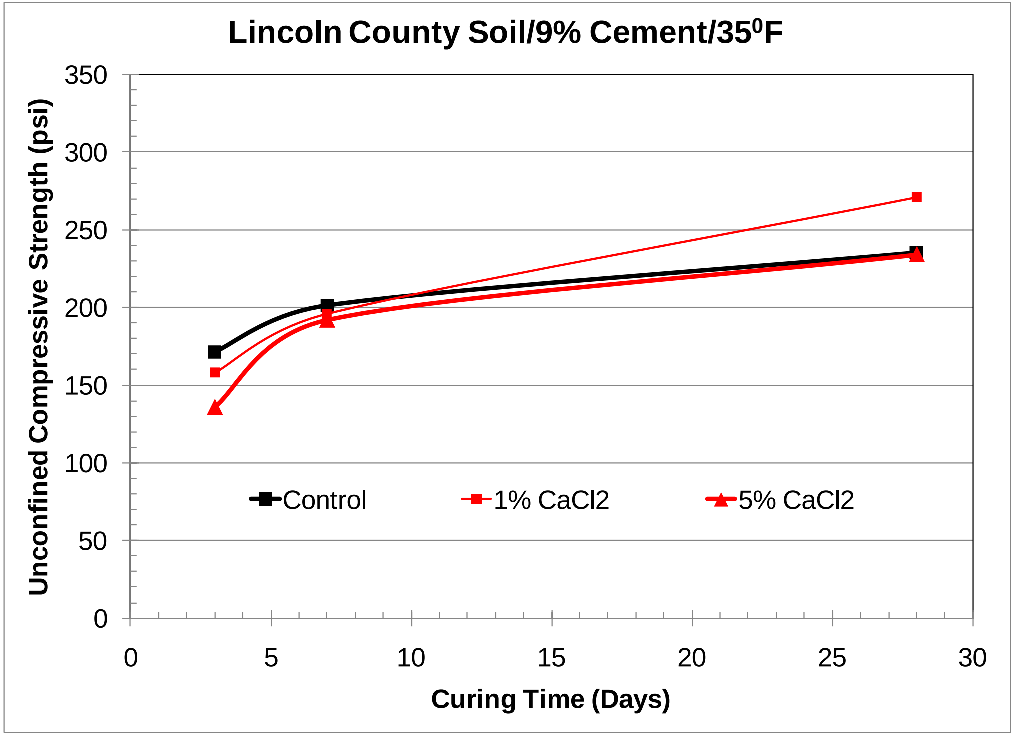

Figure 4-28: UC Strength Development Comparison for laboratory samples (cured at 35^0F)

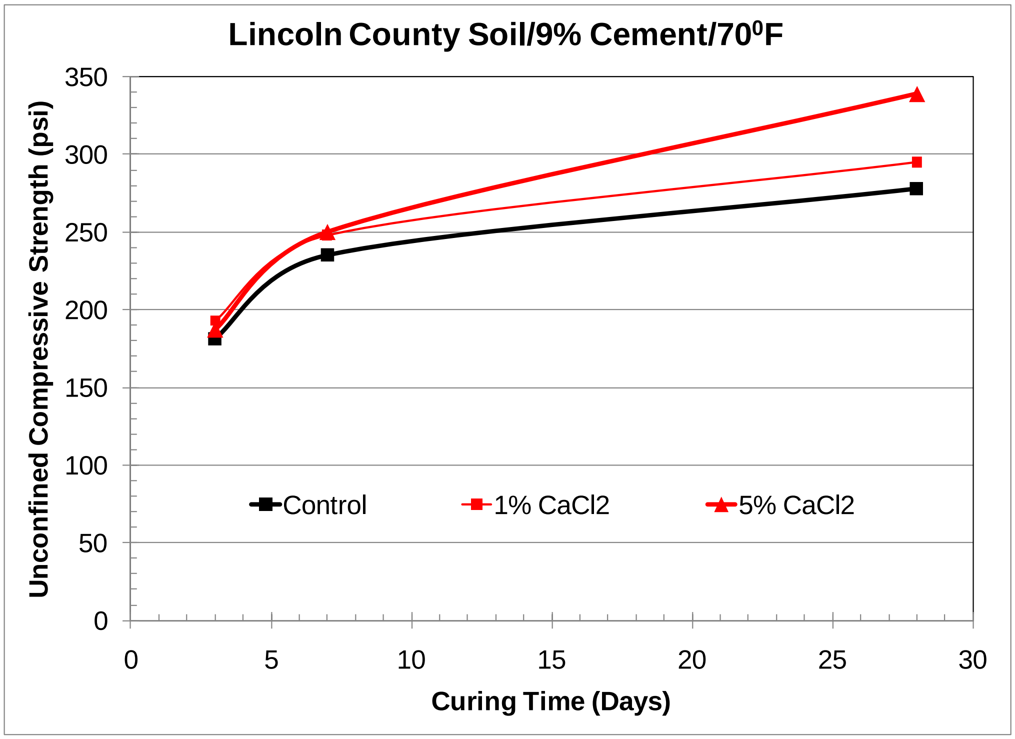

Figure 4-29: UC Strength Development Comparison for laboratory samples (cured at 70^{0}F)

4.4.5.3 Dynamic Cone Penetration Results

Results for DCP testing, expressed as CBR values as prepared by Tom Hearne (NCDOT), are presented in the Appendix 4-8 and as Figures 4-30, 4-31, 4-32 and 4-33 for Control Section (Section W), Low Dosage Section (Section X), High Dosage Section (Section Y) and Control Section 2 (Section Z), respectively.

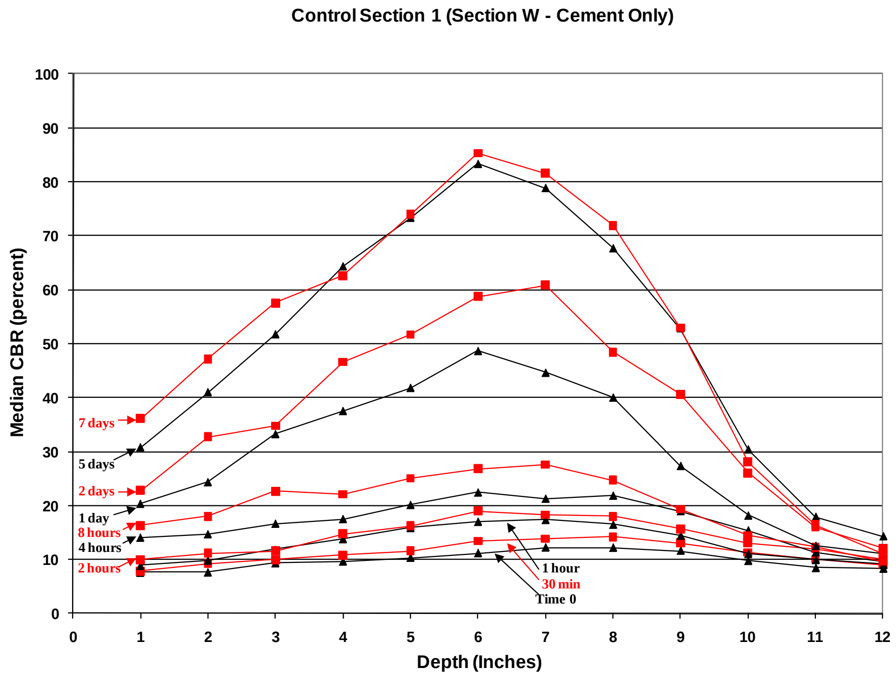
Control Section 1 (Section W - Cement Only)

Figure 4-30: DCP Data (expressed as CBR values) as a function of depth and time – Control Section 1.

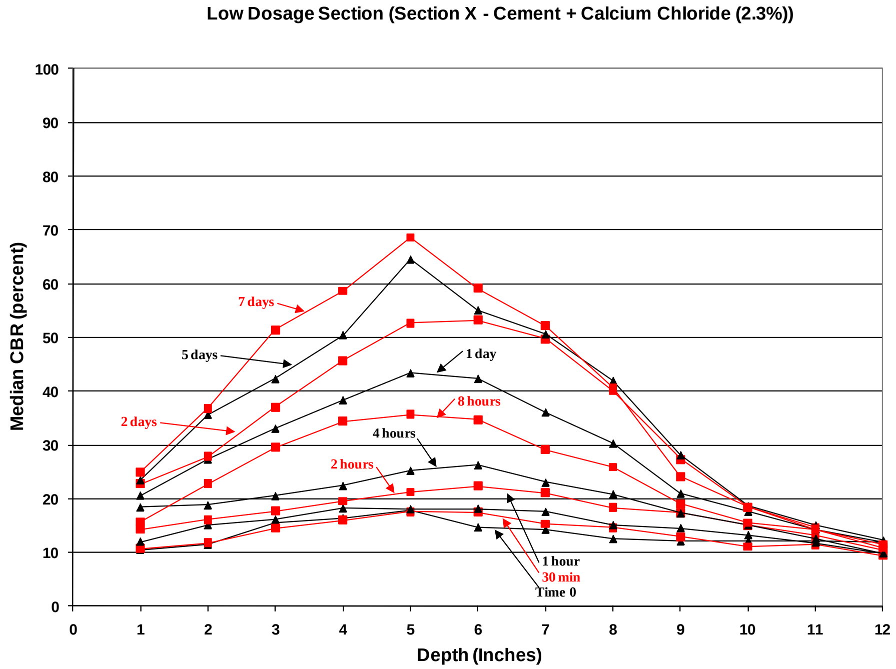
Low Dosage Section (Section X - Cement + Calcium Chloride (2.3%))

Figure 4-31: DCP Data (expressed as CBR values) as a function of depth and time – Low Dosage Section

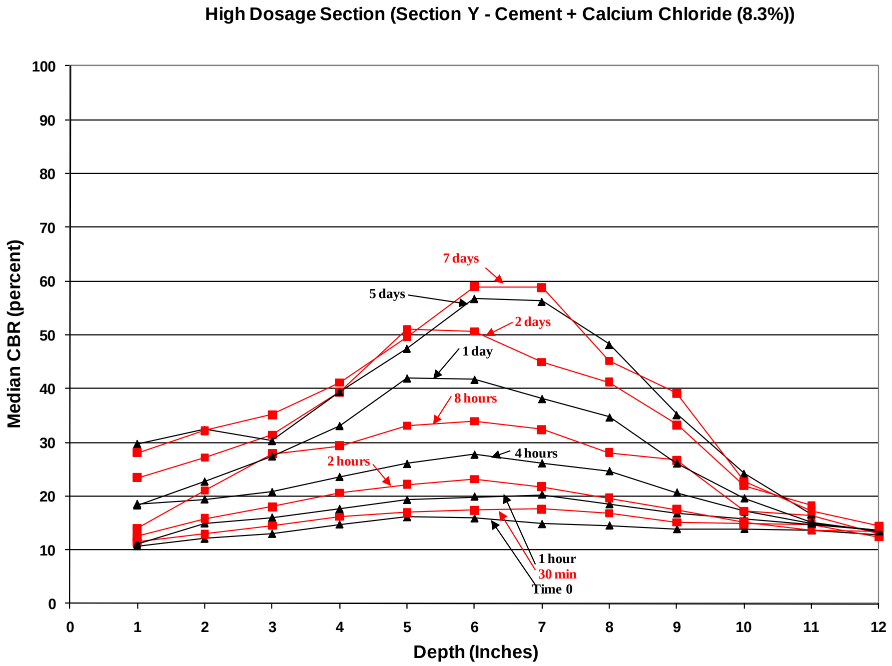

Figure 4-32: DCP Data (expressed as CBR values) as a function of depth and time – High Dosage Section

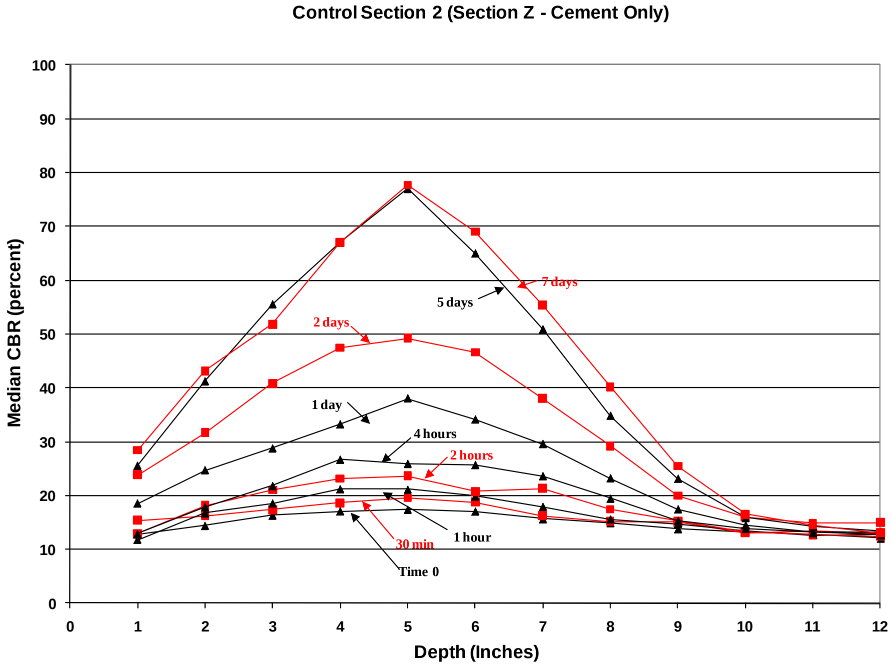

Figure 4-33: DCP Data (expressed as CBR values) as a function of depth and time – Control Section 2.

The foregoing data comprehensively track the development of strength for the various test sections and suggest the net effect of $CaCl_2$ modification, as tested, to be negative, i.e., it tends to reduce strength. This impact was observed to be time dependent, however. For example, Figures 4-34, 4-35 and 4-36 show the strength developed by all four sections at 2 hours, 1 day and 7 days. At 2 hours, $CaCl_2$ modification (either low or high) results in CBR values that are often 50% higher than Control Section 1 (Section W) and within 15% of the strength of Control Section 2 (Section Z). This corroborates the observations in the field by equipment operators who observed that the Low Dosage and High Dosage sections “set up” and became noticeably stiffer much sooner than Control Section 1. At one day, there is less disparity in CBR values among all four test sections and CBR values for Control Section 1 (Section W) are nearly as high or higher than those for the Low Dosage and High Dosage sections. At 7 days, both Control Sections have higher CBR values as compared to the Low Dosage and High Dosage sections, with the higher concentration correlating with the lowest strength. These data suggest that $CaCl_2$ modification may have accelerated strength gain initially (over a period of hours) and retarded strength gain over the longer term (over a period of days), consistent with observations in the concrete industry (Hales et al., undated). Germane to these data are the in situ subgrade temperatures, which as noted in Chapter 3 and by data collected by Tom Hearne at NCDOT, exceeded 125^0F. Such high temperatures would probably have been better served by lower dosages. This possibility was understood from the outset of field testing, however the opposing hypothesis was that because soils have a much higher surface area than the aggregate material in concrete, greater dosage levels of $CaCl_2$ would be required, e.g., as in Uppot et al. (2008). According to this logic, a certain amount of calcium chloride is needed to react with the soil beyond what may be needed to accelerate the cement hydration.

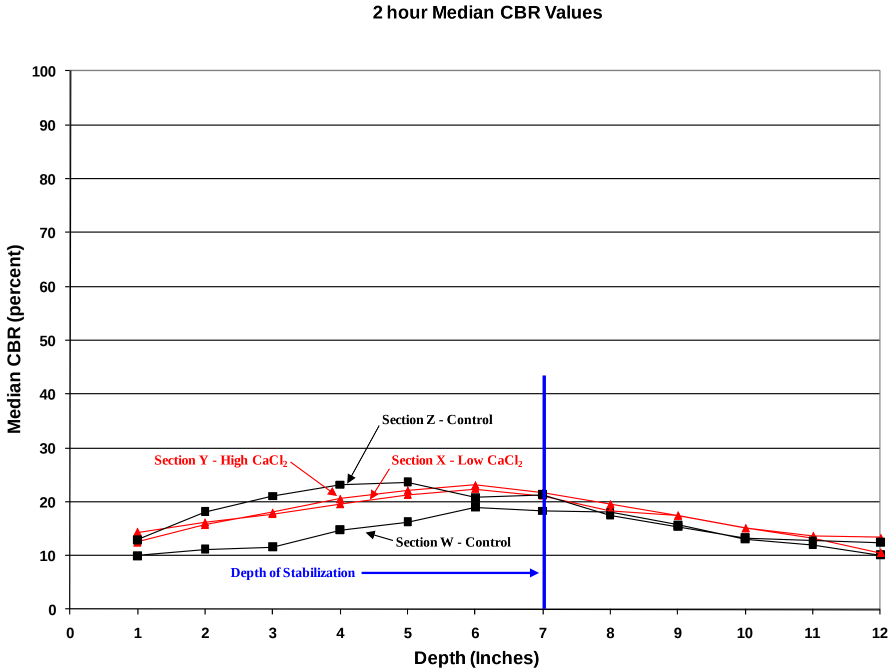

Figure 4-34: DCP Data (expressed as CBR values) as a function of depth for all test sections at two hours post construction.

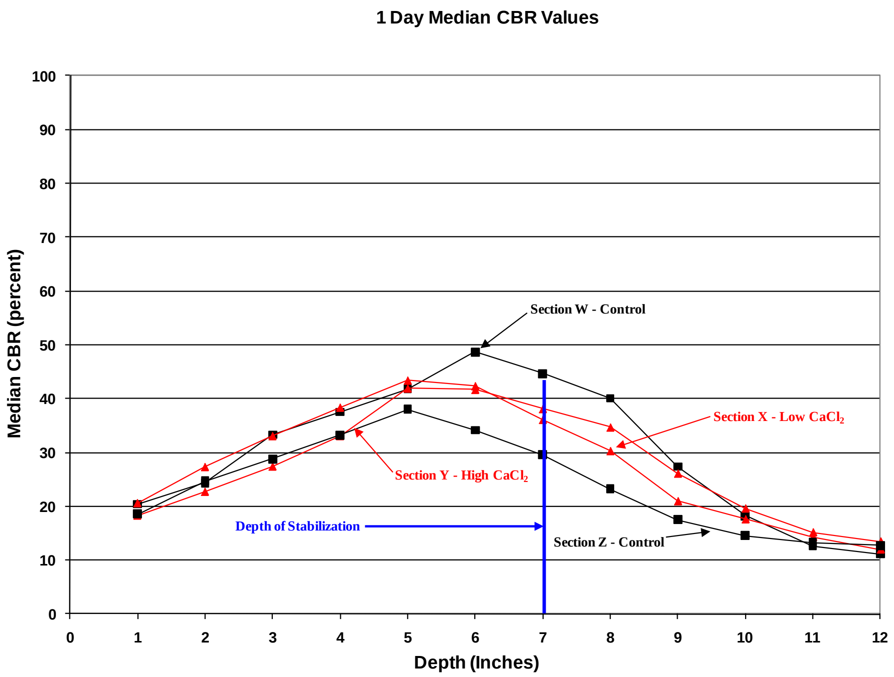

Figure 4-35: DCP Data (expressed as CBR values) as a function of depth for all test sections at one day post construction.

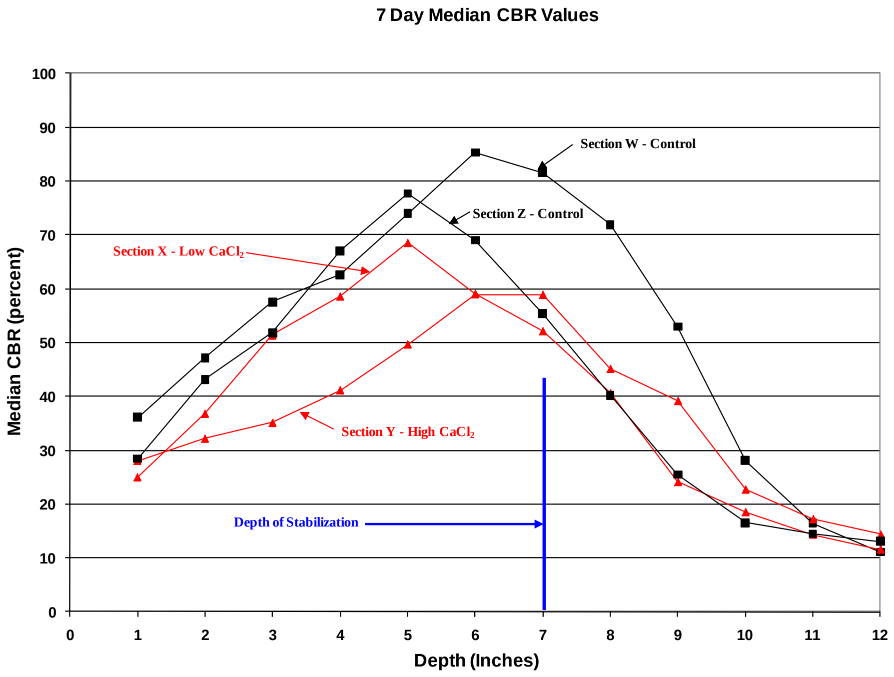

Figure 4-36: DCP Data (expressed as CBR values) as a function of depth for all test sections at seven days post construction.

4.5 Discussion

The overall body of research presented in this report suggests that $CaCl_2$ modification of soil-cement is not a mature enough approach to serve as a method for mitigating the effects of low temperatures on strength gain. However, experimental and mechanistic evidence exists to suggest $CaCl_2$ modification retains potential for use in soil-cement applications. Experimentally, data generated by Rollings et al. (2002), Daniels and Janardhanam (2005, 2007), Uppot et al. (2008), and this report indicate that strength gains are possible with $CaCl_2$ modification of soil cement. Results from the field trial provide a clear indication of a maximum dosage level, beyond which deleterious performance may occur, i.e., the concentrations used were beyond the optimum for the prevailing soil and weather conditions. Mechanistically, the notion that an optimum concentration of $CaCl_2$ exists is supported by the interaction between $CaCl_2$, water, cement and soil. $CaCl_2$ is surrounded by water of hydration (i.e. H_2O) and is electrostatically attracted to cement particles such that it serves to "deliver" water to cement faster than would occur in the absence of $CaCl_2$. In addition, $CaCl_2$ reduces the repulsive forces between particles, which in turn attracts them into closer relative proximity, reducing the distance across which cementitious reactions need to bridge void space. These attributes explain why $CaCl_2$ modification, at the correct dosage, can be effective. If the dosage is too high, however, the strength can be reduced. The presence of a threshold concentration, beyond which strength weakening rather than strengthening occurs, may be partially explained according to the extent to which excessive $CaCl_2$ serves to attract and retain H_2O, thereby interfering with cement hydration. Simply stated, at high levels of $CaCl_2$ modification, there is competition between $CaCl_2$ and cement for H_2O molecules. To the extent that $CaCl_2$ is successful in this competition, there is weakening of the soil-cement.

This research was predicated on the notion that a window of efficacy existed for $CaCl_2$ modification such that one could iteratively test and find the appropriate range, much like moisture density testing. And indeed such a window was observed in Figure 4-19. Regrettably, this window varies or was not yet identified across different temperatures (e.g., Figure 4-18) such that it deprives one of confidence for suggesting blanket specifications at this point.

This research has demonstrated that in addition to the sensitivity of mixing condition and temperature, a potentially key attribute that governs $CaCl_2$ efficacy is soil type. It makes sense that the optimal concentration varies with mineralogy, chemical composition and particle size. Kroyer et al. (2003) observes that clay minerals may accelerate cement hydration and therefore, possibly, strength gain. This could mean that a certain fraction of clay is analogous to adding calcium chloride. It may be that one needs to evaluate a given soil's cation exchange capacity (CEC) relative to its reactivity to cement and $CaCl_2$ (e.g., as a function of UC strength).

Another factor that brackets the ideal concentration is cost. At the beginning of this research, the general idea was to provide an alternative that was within approximately 10-20% of the costs of the general chemical stabilization. Given that the field trial involved concentrations that define an upper bound for technical viability – i.e., it is not likely that more than those levels would be required – it is useful to compare those costs, as below:

Cement dosage: 28 kg cement / m^2
Area for one section: 92 m x 9.2 m = 846.4 m^2
Depth of stabilization: 180 mm
Mass of cement used for one section: 23,700 kg (52,200 lb)
Cost of cement: \$120/metric ton (\$108.86/ short ton)
(1 short ton = 2000 lb = 907.18 kg)
Total cost of cement used for one section: \$2,844
Cost of cement/volume treated: \$18.66/m^3
$CaCl_2$ dosage: 0.023 kg $CaCl_2$ / kg cement (2.3% - Low Dosage)
Mass of $CaCl_2$ required: 545 kg
Volume of $CaCl_2$ solution required: 1,124 liters (297 gal)
Cost of $CaCl_2$: \$0.40/gal (38% solution)
Total cost of $CaCl_2$ for one section: \$118.76
Cost of $CaCl_2$/volume treated: \$0.78/m^3
Cost as a percent of cement costs: ~4.1%

Repeating the above calculation for the high dosage section (8.3%) yields a cost as a percent of cement of ~15.0%. Given that it is unlikely that higher dosages would be effective, these estimates suggest $CaCl_2$ modification will remain financially feasible.

4.6 References

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Appendix – Chapter 1 – Lime-Stabilized Subgrade Soils

List of Appendices – Chapter 1:

Appendix 1-1: Notes – 7/12/06 – NCDOT Materials and Test Unit, Raleigh, NC

- Soil samples are air-dried and sieved through a #4 sieve to remove stones
- The remaining material is mixed with the appropriate amount of cement until it is well-blended in dry form
- Water is slowly added and mixed until the optimum moisture content is reached
- The stone is mixed back with the soil once the optimum moisture content is approached and reached
- Proctor tests (i.e., Moisture-Density relationships, as per AASHTO T99 or ASTM D698) are <u>not</u> run to determine the optimum moisture content/maximum dry unit weight for a given cement or lime content. Instead the optimum moisture content is judged through a combination of experience and surrogate indicators such as:
    - The “penny print test” which evaluates the imprint left on the first and third surface layer of the compacted sample, i.e., the imprint from the tamper should be about the thickness of a penny
    - The “coolness to the palm” test which represents the reduced temperature and sensation of the material when handled, given sufficient moisture for certain soil types
    - The ability to mold the material in and then break it apart cleanly. Sometimes the compacted sample is tossed into the air to illustrate its ability to maintain its integrity
    - The observation of appropriate color change
    - The extent to which the material sticks to the trowel bar

- The mold is greased with Molytex EP-2, Texaco Brand, Product 01922 L
- Approximately 4000 total grams of material are used for making two Proctor Molds (1/30 ft^3)
- When cement is used, standard Type I is used
- When lime is used, chemical hydrated lime, type N is used, Magnolia Brand, Southern Lime, Calera, Alabama, 866-596-7476.
- The moisture content is determined via an oven at 110 ^oC, +/- 5^oC
- After optimum moisture has been determined by above methods, take 100gram-300gram moisture sample depending on stone content.
- Tap water is used for the moisture content
- Standard hammer is used, 5.5 lb, 12” drop, 3 layers, 25 blows per layer, scarify soil between layers to ensure better contact
- Run two samples at a time
- If cement is used, it is generally 6% cement or 8% cement (i.e., mass of cement per mass of dry soil)
- If lime is used, it is 3% lime (i.e., mass of lime per mass of dry soil)
- Cement samples are cured in the mold
    - Record weight of empty greased mold before making specimen
    - Record weight of mold with sample after making specimen

  - Plexiglass plates are placed on the top and bottom of the mold
  - Curing conditions are 70^{o}F and 100% humidity
  - After 7 days of curing, the sample is extruded and submerged for 4 hours with 1”-2” of water on top
  - Sample is then removed from water and tested for strength

- Lime samples are similar except that
  - The samples are extruded immediately after preparation (i.e., not left in mold)
  - Samples are sealed in a gallon plastic bag
  - Record weight of specimen in gallon bag before placing in oven
  - Bagged samples are placed in oven at 45^{o}C, not to exceed 48°C (120° F), for 2 days
  - Record weight of specimen in gallon bag, after hydration of two days
  - Samples are not soaked, rather tested for strength immediately
  - Remember to subtract the weight of bag when calculating density
- The loading rate is 0.050 in/minute
- Data are recorded every 6 seconds
- Peak strength is recorded

For classification purposes: Sieve #s used are:

20
40
60
140
200
Pan

- Hydrometer 152H used, We are sending a copy of Hydrometer procedures to better explain how test works

Wash over Sieve # 270

Liquid limit and plastic limit as per standards

- We determine L.L. through a NCDOT modified procedure. Using a 1- point determination sample must come together at 25 blows.

Criteria for Additive Selection – Lime versus Cement

Property	**Criteria “A” Use Cement**	**Criteria “B” Use Lime**
% Passing #200 Sieve	35 Max	36 Min
LL	40 Max	41 Min
PI	10 Max	25 Min

*Designer may choose cement or lime for all soils not meeting the criteria in A or B

- Usually designer will choose to run BOTH cement and lime for soils not meeting all criteria in “A” and “B”

Appendix – Chapter 3 – Temperature Modeling of Stabilized Subgrade Soil

List of Appendices – Chapter 3:

(a) (b)

Appendix 3-1: Installation of temperature sensors into subgrade (a) on U.S. 421, near Siler City, before asphalt tacking, (b) installation after asphalt tacking on U.S. 70 near Clayton, NC. The temperature sensor appears as a rubber stopper with a red and black wire at the top.

(a) (b)

Appendix 3-2: (a) Installation of moisture sensor (U.S. 421), (b) After installation and compaction. Notice four wire leads from surface, one for each of three temperature sensors and one for the moisture sensor. The radiation shield and a datalogger are located in the upper right.

(a) (b)

Appendix 3-3: (a) Overall site, U.S. 70 near Clayton, NC, facing Westbound (notice the five gallon bucket initially used to shield the air temperature sensors) (b) facing Eastbound

(a) (b)

Appendix 3-4: (a) Overall site, U.S. 421 near Siler City, NC, facing Northbound, (b) facing Southbound

(a) (b)

Appendix 3-5: (a) Overall site, I-40 near Greensboro, NC, facing Westbound, (b) facing Eastbound

(a) (b)

Appendix 3-6: (a) Overall site, I-485 near Charlotte, NC, facing Northbound with “inner” lane on right, (b), facing Southbound with “outer” lane on right

(a) (b)

Appendix 3-7: Overall site, NC 16 near Weddington, NC, Union County (a) sensor installation (facing North) and (b) field temperature recording

(a) (b)

Appendix 3-8: Temperature sensor installation on NC 16 in Lincoln County (a) drilling holes for temperature sensors (b) wires emanating from surface and toward edge of road.

Appendix – Chapter 4 – Calcium Chloride for Modification of Cement-Stabilized Subgrade Soils

List of Appendices – Chapter 4:

Appendix 4-1: Field Plan – Calcium Chloride/Cement Stabilized Subgrade

- Weather permitting, testing scheduled for the week of June 8-12, 2009
- Target date for calcium chloride site delivery and use: Tuesday, June 9, 2009
- Approximate target location: Station 175+00, near St. James Church Road
- The test sections with calcium chloride will be built within one day
- Comparisons with control sections (built adjacent to the test sections) without calcium chloride ($CaCl_2$), may require sampling of sections built the day before, and the day after.

Dates are given relative to the actual day of $CaCl_2$ dosing.

Date	Task	Responsibility	Comments
Day before	Safety orientation as required by Blythe Construction	TBD-Blythe	~June 8, 2009
Day before	Inspect test section location for instrumentation requirements	J. Daniels	
Day before	Test sieve soil into bucket	T. Walker	
Day before	Collect dry soil for additional laboratory testing (spiked samples, as explained in UNC Charlotte testing plan)	B. Bowers	
Day before	Set up air data logger for temperature measurement	J. Daniels	
Day before	Amount of water placed per unit area with water truck during wet-mixing phase is verified	J. Daniels/J. Cuneo	Done - ~6 gal/m^2
Day of testing	Field Observer/Recorder	Holly Christenbury	Document everything, pictures and notes, etc.
Day of testing	$CaCl_2$ arrives on site via tanker truck at about 9:00 AM	J. Daniels	~June 9, 2009
Day of testing	Several gallons of the source $CaCl_2$ solution are collected.	J. Daniels	The concentration will be checked in the lab to verify that it is 38% and it may also be used for subsequent laboratory testing.
Day of testing	Initial moisture content of soil is measured	E. Connor	As per standard NCDOT practice…any information collected to be shared as appropriate.
Day of testing	Control section (Control 1) is constructed	J. Cuneo	
Day of testing	After wet mixing of Control 1, but before final rolling, collect samples for manual compaction and lab curing	B. Bowers	Note UNC Charlotte Lab Testing Plan
Day of testing	Samples collected for NCDOT Materials and Test Unit	E. Connor	As per standard NCDOT practice
Day of testing	Check for density after final roll	E. Connor	As per standard NCDOT practice
Day of testing	DCP testing after final roll	T. Hearne	Assistance from UNC Charlotte Students

Date	Task	Responsibility	Comments
			Neill Belk Ari Eslaminejad Patrick Cooksey Rob Botzenmayer
Day of testing	Instrument first control section for temperature, noted as Control 1	J. Daniels	Possibly day before depending on anticipated sequencing, idea for Control 1 is to instrument section just before and adjacent to the section to be treated with the Low Dosage of $CaCl_2$ Ben Bowers to assist
Day of testing	Initial moisture content of soil is measured	E. Connor	As per standard NCDOT practice…any information collected to be shared as appropriate. Record moisture content and number of additional water trucks, if necessary, to reach required OMC. It doesn't change dosage during wet-mixing phase – all $CaCl_2$ will be provided during wet mixing phase
Day of testing	$CaCl_2$ is dosed in water truck for wet mixing phase	J. Daniels	Low Dosage added – Note $CaCl_2$ Dosing Plan
Day of testing	Low Dosage section is constructed	J. Cuneo	
Day of testing	Amount of $CaCl_2$ solution placed per unit area with water truck during wet-mixing phase is verified	J. Daniels/J. Cuneo	In addition to another check on the volume per unit area, the solution will be collected and dried to verify the amount of calcium chloride present in the solution. This is also a check on how well the solution is mixed in the water tank. Presumably, the solution has to pass through mixing paddle prior to discharge, so it should be well mixed.
Day of testing	After wet mixing of Low Dosage, but before final rolling, collect samples for manual compaction and lab curing	B. Bowers	Note UNC Charlotte Lab Testing Plan
Day of testing	Samples collected for NCDOT Materials and Test Unit	E. Connor	As per standard NCDOT practice
Day of testing	Check for density after final roll	E. Connor	As per standard NCDOT practice
Day of testing	DCP testing after final roll	T. Hearne	Assistance from UNC Charlotte Students Neill Belk Ari Eslaminejad Patrick Cooksey Rob Botzenmayer
Day of testing	Instrument Low Dosage section for temperature	J. Daniels	Ben Bowers to assist

Date	Task	Responsibility	Comments
Day of testing	$CaCl_2$ is dosed in water truck for wet mixing phase If there is $CaCl_2$ solution remaining in the tank from the Low Dosage section, it is estimated and additional solution is added	J. Daniels	High Dosage added – Note $CaCl_2$ Dosing Plan
Day of testing	High Dosage section is constructed	J. Cuneo	
Day of testing	Amount of $CaCl_2$ solution placed per unit area with water truck during wet-mixing phase is verified	J. Daniels/J. Cuneo	In addition to another check on the volume per unit area, the solution will be collected and dried to verify the amount of calcium chloride present in the solution.
Day of testing	After wet mixing of High Dosage, but before final rolling, collect samples for manual compaction and lab curing	B. Bowers	Note UNC Charlotte Lab Testing Plan
Day of testing	Samples collected for NCDOT Materials and Test Unit	E. Connor	As per standard NCDOT practice
Day of testing	Check for density after final roll	E. Connor	As per standard NCDOT practice
Day of testing	DCP testing after final roll	T. Hearne	Assistance from UNC Charlotte Students Neill Belk Ari Eslaminejad Patrick Cooksey Rob Botzenmayer
Day of testing	Instrument High Dosage section for temperature	J. Daniels	Ben Bowers to assist
Day of testing	Initial moisture content of soil is measured	E. Connor	
Day of testing	Second control section (Control 2) is constructed	J. Cuneo	
Day of testing	After wet mixing of Control 2, but before final rolling, collect samples for manual compaction and lab curing	B. Bowers	UNC Charlotte Lab Testing Plan
Day of testing	Samples collected for NCDOT Materials and Test Unit	E. Connor	As per standard NCDOT practice
Day of testing	Check for density after final roll	E. Connor	As per standard NCDOT practice
Day of testing	DCP testing after final roll	T. Hearne	Assistance from UNC Charlotte Students Neill Belk Ari Eslaminejad Patrick Cooksey Rob Botzenmayer
Day of testing	Instrument second control	J. Daniels	Ben Bowers to assist

Date	Task	Responsibility	Comments
	section for temperature, noted as Control 2		

$CaCl_2$ Dosing Plan

- There are two test sections using calcium chloride, denoted as "Low Dosage" and "High Dosage"
- Each dosage schedule will be tested for a length of 100 m, or in other words, for about one load of cement. Overall dimensions of each test section are anticipated to be as follows:
  - Length: 100 m
  - Width: 9.2 m
  - Depth: 180 mm
- The calcium chloride dosage is based on the cement content. The cement content is expected to be 28kg/m^2
- Concentration, by dry weight of cement, will be 5% and 10% for the Low and High Dosages, respectively, as per concentration schedule below

Concentration Schedule	
Low Dosage	***High Dosage***
5.0 kg $CaCl_2$ / 100 kg cement (5%)	10.0 kg $CaCl_2$ / 100 kg cement (10%)
4.85 g $CaCl_2$ / kg dry soil (at γ_d = 15.7 kN/m^3)	9.70 g $CaCl_2$ / kg dry soil (at γ_d = 15.7 kN/m^3)
1.4 kg $CaCl_2$ / m^2	2.8 kg $CaCl_2$ / m^2
702 gallons of 38% $CaCl_2$ solution diluted in water raised to the total 5000 gallon water truck capacity	1403 gallons of 38% $CaCl_2$ solution diluted in water raised to the total 5000 gallon water truck capacity

- The Low Dosage section will be constructed first, followed by the High Dosage section
- Calcium chloride will be delivered to site by tanker truck equipped with a transfer pump and meter for measuring approximate volumes. If there is any question regarding flowrate, the time required to fill up large containers will be verified. Up to a day of demurrage (i.e., tanker kept on site for use) has been budgeted for on-site use.
- Preliminary information from Jason Cuneo indicates approximately 1 truckload (5000 gallons) of water will be used during the wet mixing phase for each 100 m section.
- After the first Low Dosage section is completed, the volume in the tank will be inspected visually through the manhole. If there is significant (>10% truck filled) calcium chloride solution remaining from the Low Dosage section, this will be accounted for when preparing for the High Dosage section. The following table will be used to guide operations:

Field Planning for Dosage Control

Number of Truckloads	**Volume / Area**		**Volume (gallons) of 38% Solution to add per full truckload to achieve given CaCl2 dosage (wt CaCl2/wt cement)**																
	4000 gal capacity	5000 gal capacity																	
	gal/m2		**0.25%**	**0.50%**	**1%**	**2%**	**3%**	**4%**	**5%**	**6%**	**7%**	**8%**	**9%**	**10%**	**11%**	**12%**	**13%**	**14%**	**15%**
0.5	*2.17*	*2.72*	70	140	281	561	842	1122	1403	1684	1964	2245	2525	2806	3087	3367	3648	3928	4209
1.0	*4.35*	*5.43*	35	70	140	281	421	562	702	842	983	1123	1264	1403	1544	1685	1825	1966	2106
1.5	*6.52*	*8.15*	23	47	94	187	281	374	468	562	655	749	842	935	1030	1123	1217	1310	1404
2.0	*8.70*	*10.87*	18	35	70	140	211	281	351	421	491	562	632	702	772	842	913	983	1053
2.5	*10.87*	*13.59*	14	28	56	112	169	225	281	337	393	450	506	561	618	674	731	787	843
3.0	*13.04*	*16.30*	12	23	47	94	140	187	234	281	328	374	421	468	515	562	608	655	702
3.5	*15.22*	*19.02*	10	20	40	80	120	160	200	240	280	320	360	401	440	480	520	560	600
4.0	*17.39*	*21.74*	9	18	35	70	105	140	175	210	245	280	315	351	385	420	455	490	525
4.5	*19.57*	*24.46*	8	16	31	62	94	125	156	187	218	250	281	312	343	374	406	437	468
5.0	*21.74*	*27.17*	7	14	28	56	84	112	140	168	196	224	252	281	308	336	364	392	420

Note 1: Truck capacity does not influence volume of CaCl2 required. For example, at a 4000 gal capacity, the same volume of 38% solution is required to achieve a given CaCl2 content for a given number of truckloads. However the resulting concentration in a given truckload will be higher as there would be less water. It is more likely that using a lower capacity would require more truckloads (e.g., going from 2.5 truckloads of 5000 gallons to 3 truckloads of 4000 gallons)

Note 2: When filling partial truckloads (that is total of water and solution is less than full capacity), multiply the above value (which is per full truckload) by the percent filling required. For example, if only half a truckload is needed, then it only needs 50% of the above value. If it needs 3/4 of a truckload, then multiply by 0.75.

Note 3: If CaCl2 is already in the truck, then estimate the amount of CaCl2 present as a percent of how full the truck is, and then subtract that from the amount which is required for a given truckload. For example, if a full third truckload was prepared for a 5% low dosage section when only 2.5 truckloads were required and actually used (281 gal added), and the truck is 50% full, then assume that 0.5*281 = 140 gallons of CaCl2 remain. If the next section is, for example, 10%, then you only need to add 561-140 = 421 gallons of 38% solution, the rest is filled with water to capacity for a full truckload

Note 4: If there is a decision to reduce the concentration for a subsequent section or load, and there is already a relatively full load of higher concentration CaCl2, it may be necessary to drain (waste) some to achieve the necessary dilution. For example, if there is half a truckload left over from a 5% section, then there is 0.5*281 = 140 gallons of CaCl2 remaining. If a 2% section is desired, and still 2.5 truckloads are required, then only 112 gallons of CaCl2 are needed. So the water truck should be drained until it is 112/281, or in other words 40% full.

Temperature Measurement Plan

The purpose of measuring air and subgrade temperature will be to evaluate the extent to which accelerated cement hydration due to calcium chloride addition results in changes in the evolved heat, as well as the general response of the subgrade to fluctuations in air temperature.

The two control sections and the two sections treated with calcium chloride will be instrumented with five temperature sensors each, at various depths within the treatment layer and below. Presuming cement stabilization is done to a depth of 180 mm, the planned depths are

Depths from ground surface:

1” (25 mm)
3” (75 mm)
5” (125 mm)
8” (203 mm)
10” (254 mm)

The sensors will be placed 20 meters from the start of a given section and 1.7 meters from the shoulder of the stabilized section.

In addition, air temperature will be measured with two sensors in the general vicinity. Thermal diffusivity will also be measured to calibrate the previously developed temperature model for this project.

Temperatures will be recorded every 15 minutes. Subsurface temperatures will be measured with intelliRock temperature sensors from Engius, LLC. This sensor has a range from -5°C to +85°C and is accurate to ± 1°C. Data were recorded every 60 minutes by the datalogger which is part of the sensor itself.

The location will be selected as appropriate and convenient for the individual test sections. A typical example is given in the following figure - note the wire leads emanating from the surface, these wires are attached to the temperature sensors/dataloggers which are installed in varying depths:

Sensors installed at various depths, propose to do the same for this field test

UNC Charlotte Lab Testing Plan

The purpose of this additional laboratory testing is to evaluate the unconfined compression strength of field-mixed samples as a function of time for both the control in calcium chloride modified test sections. Data from this testing will serve as additional verification to testing conducted at NCDOT Materials and Test Unit. Additionally, compaction data may be collected.

The first priority in guiding sampling location will be where NCDOT is already collecting samples, i.e., to leverage and complement the additional data.

If there are no specifically NCDOT identified sampling locations, samples will be collected at about 16 meters, 50 meters and 84 meters of length for a given section, with enough material collected for 4 Proctor molds each time

At 16 m:
3 days, 35F - S1, S2; 70F- S1, S2
At 50 m:
7 days, 35F - S1, S2; 70F- S1, S2
At 84 m:
28 days, 35F - S1, S2; 70F- S1, S2

	Unconfined Compressive Strength											
	$35^{o}F$						*$70^{o}F$*					
Additive	3		7		28		3		7		28	
Control Section 1	S1	S2	S1	S2	S1	S2	S1	S2	S1	S2	S1	S2
Control Section 2	S1	S2	S1	S2	S1	S2	S1	S2	S1	S2	S1	S2
Low Dosage Section	S1	S2	S1	S2	S1	S2	S1	S2	S1	S2	S1	S2
High Dosage Section	S1	S2	S1	S2	S1	S2	S1	S2	S1	S2	S1	S2
Spiked Intermediate	S1	S2	S1	S2	S1	S2	S1	S2	S1	S2	S1	S2
Spiked Extra High	S1	S2	S1	S2	S1	S2	S1	S2	S1	S2	S1	S2
Spiked Extra Low	S1	S2	S1	S2	S1	S2	S1	S2	S1	S2	S1	S2
Laboratory Controls	S1	S2	S1	S2	S1	S2	S1	S2	S1	S2	S1	S2

The reason for the spiked samples is to enable the preparation of a meaningful plot of UC strength on the y-axis and dosage on the x-axis for field-collected samples. Currently we will have two data points on such a plot, i.e., high and low. The plan is to have the low dosage be 5% $CaCl_2$ (by weight of cement) and 10% for the high dosage.

Spiked samples will be prepared by collecting sufficient soil (just soil, not mixed with cement or calcium chloride, as is the case for the other samples, also we will not compact these samples in the field) from the field and subsequently mixing cement and $CaCl_2$ in the lab according to the following $CaCl_2$ concentrations (by weight of cement):

Spiked Extra Low:	2%
Spiked Extra Intermediate:	7.5%
Spiked Extra High:	15%

All soil will be collected near treatment sections and processed in the laboratory.

The resulting data will vary according to the different mixing conditions (field versus laboratory) but will still yield insight into the strength vs. dosage relationship.

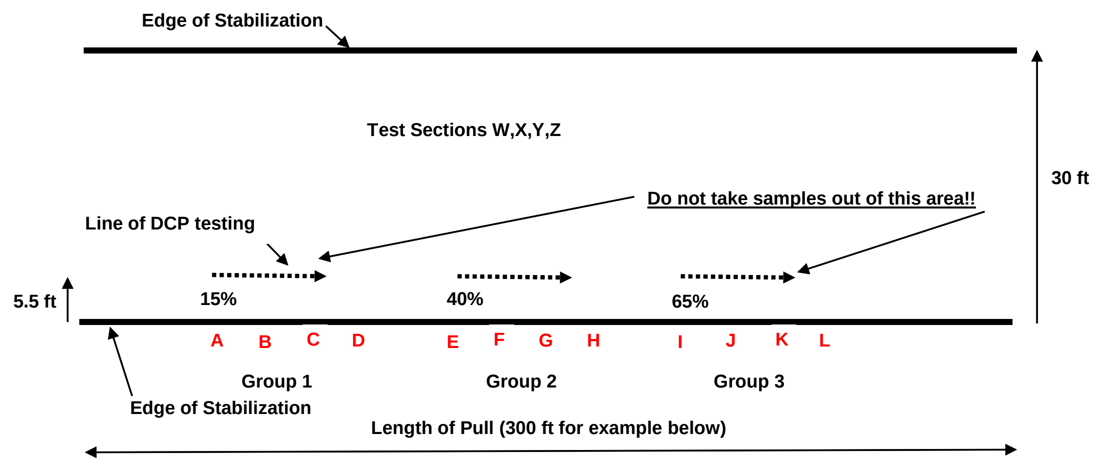
In addition to laboratory testing, we will field-spike a representative set of samples, for purposes of obtaining field-mixed samples with the broader range of $CaCl_2$ concentrations. This will be subject to availability of NCDOT water content data and timing of other priority activities.

Dynamic Cone Penetrometer Plan

- As provided by Tom Hearne, given by:

NC 16 - General Plan for DCP Testing

6/7/09
TMH



3 groups of 3 people

Group 1	Group 2	Group 3
A,B,C,D	E,F,G,H	I,J,K,L
Bob Rogers	Dan Popek	Mike Holmes
*Tom Hearne	*Jon Miller	*Mike Mauldin
UNC Charlotte	UNC Charlotte	UNC Charlotte

*Recorders

Test Section X

Distance (ft)	**45**	**46**	**47**	**48**	**49**	**50**	**51**	**52**	**53**	**54**	**55**	**56**
	●	●	●	●	●	●	●	●	●	●	●	●
		1 ft										
	15% of total pull length											
Group	**1**											
Location	**A**											
Target	**0**	**30 min**	**1 hr**	**2 hr**	**3 hr**	**4 hr**	**8 hr**	**1 day**	**2 days**	**3 days**	**4 days**	**7 days**
Act. Time	**11:09**	**11:39**	**12:09**	**1:09**	→	**etc.**						
Distance (ft)	**65**	**66**	**67**	**68**	**69**	**70**	**71**	**72**	**73**	**74**	**75**	**76**
Group	**1**											
Location	**B**											
Target	**0**	**30 min**	**1 hr**	**2 hr**	**3 hr**	**4 hr**	**8 hr**	**1 day**	**2 days**	**3 days**	**4 days**	**7 days**
↓ **etc.**												
	40% of total pull length											
Distance (ft)	**120**	**121**	**122**	**123**	**124**	**125**	**126**	**127**	**128**	**129**	**130**	**131**
Group	**2**											
Location	**E**											
Target	**0**	**30 min**	**1 hr**	**2 hr**	**3 hr**	**4 hr**	**8 hr**	**1 day**	**2 days**	**3 days**	**4 days**	**7 days**
Distance (ft)	**140**	**141**	**142**	**143**	**144**	**145**	**146**	**147**	**148**	**149**	**150**	**151**
Group	**2**											
Location	**F**											
Target	**0**	**30 min**	**1 hr**	**2 hr**	**3 hr**	**4 hr**	**8 hr**	**1 day**	**2 days**	**3 days**	**4 days**	**7 days**

Some time intervals maybe impossible, e.g., 8 hours for a section completed in the late afternoon. The focus is to keep all the measurements along a relatively narrow longitudinal axis as experience suggests more natural variation in the horizontal direction. And we don't want this variation to obscure comparisons with CaCl2 dosage.

NCDOT Materials and Test Plan

- As per standard practice

Appendix 4-2: Field Notes and Images from Construction

Notes:

- 4500 gallon water truck
- Temp sensors are placed between D and E
- 1 pass = up and back
- Low dosage section approximately 92m – station 170+88 – 171+80
- Sheeps roller – Ingersoll Rand – 100 Pro Pac Series, 175 Pro Pac Series
- Roller – Ingersoll Rand 100 Pro Pac Series
- Dry Mix Machine – TEREX RS-425B
- Grader – CAT 12H
- Wet Mixer – Wirtgen WR 2500 S – used 3500 gals
- High Dosage Section – 92m – 171+80 – 172+72
- Dry mix High Dosage Section was streaky
- Low Dosage set up faster than control, and high dosage even more quickly according to Jason Cuneo and equipment operators and other observers
- Control Section 2 – 92m – 172+72 – 173+64

Timeline:

6:42am – lay cement for section 1
6:55 am – Dry cement mix section 1 – control section 1 roughly 92m – 169+96 – 170+88 – collected samples at A, E, I
7:30 am – Sheeps roller rolls Control Section 1 – Dry Mix
7:45 am – Wet mix Control Section 1 – Sheeps Roller, about 5 passes
8:00 am – Samples at A collected for Control Section 1
8:02 am – Samples at E collected for Control Section 1
8:06 am – Samples at I collected for Control Section 1
8:11 am – Lay cement for Low Dosage Section
8:18 am – Wet mix complete for Control Section 1 – grading began for Control Section 1
8:40 am – Sheeps roller done on Control Section 1
8:43 am – Dry mix Low Dosage Section
8:50 am – Finished laying cement for Low Dosage Section
9:02 am – Sheeps roller over dry mix Low Dosage Section
9:17 am – Final rolling of Control Section 1
9:22 am – Sheeps roller finished for dry mix of Low Dosage Section
9:35 am – Wet mixing for Low Dosage Section
9:36 am – Samples at E collected from Low Dosage Section – Samples at A collected from Low Dosage Section – Samples at I collected from Low Dosage Section
9:37 am – Sheeps roller Low Dosage Section wet mix
9:40 am – Cement High Dosage Section
9:52 am – Roller tire Control Section 1
9:50 am – DCP and Density Testing started on Control Section 1
9:57 am – Roller tire finished Control Section 1
10:00 am – Wet mix Low Dosage Section second pass (?) (Only on side where samples taken)
10:05 am – Dry mix High Dosage Section
10:05 am – Installation of temperature sensors on Control Section 1
10:18 am – Grader Low Dosage Section
10:27 am – Sheeps roller High Dosage Section dry mix
10:31 am – Dry mix complete for High Dosage Section
10:40 am – Steel roller Low Dosage Section – No set passes
11:17 am – Wet mix High Dosage Section
11:19 am – Samples at A collected for High Dosage Section
11:20 am – Sheeps roller over wet mix of High Dosage Section

11:20 am – Samples at E collected for High Dosage Section
11:22 am – Samples at I collected for High Dosage Section
11:24 am – Density and DCP for Low Dosage Section begins
11:32 am – Second pass in center of High Dosage Section wet mix
11:32 am – Complete grading of Low Dosage Section
11:36 am – Lay cement for Control Section 2
11:39 am – Grader for High Dosage Section
11:52 am – Steel roller for High Dosage Section – no set passes
11:58 am – Dry mix for Control Section 2
12:06 pm – Sheeps roller dry mixes Control Section 2 – no set passes
12:09 pm – Cement finished Control Section 2
12:28 pm – Dry mix complete for Control Section 2
12:30 pm – Density and DCP stary for High Dosage Section
12:39 pm – Grading completed for High Dosage Section
1:55 pm – Wet mix for Control Section 2 – grading began
2:00 pm – Sheeps roller for Control Section 2 – no set passes
2:37 pm – Wet mix completed for Control Section 2 – grading began
2:39 pm – Samples at A collected for Control Section 2
2:40 pm – Samples at E collected for Control Section 2
2:41 pm – Samples at I collected for Control Section 2
2:53 pm – Steel roller for Control Section 2 – no set passes
3:37 pm – Paddle pan Control Section 2 – one pass
3:50 pm – Continued steel roller for Control Section 2
4:15 pm – DCP and Density Testing for Control Section 2 began
4:45 pm – Finish final rolling of Control Section 2

Section Mixing Times (Courtesy of Tom Hearne, NCDOT)

6/19/09

NC 16 - Section Mixing Times (06/10/09)

AE,TMH

Location A

	Test Section			
	W	X	Y	Z
Cement on Subgrade	6:42	8:17	9:49	11:39
First Dry Mixer Pass	7:10	8:46	10:08	12:00
Wet Mixer Pass	8:13	9:37**	11:20	2:37
Final Rolling	9:38*	11:05	12:27	4:05

*** completed at 9:57 with rubber tired roller**

**** second mixing with water at 10:02 a.m.**

Location E

	W	X	Y	Z
Cement on Subgrade	6:44	8:21	9:51	11:42
First Dry Mixer Pass	7:12	8:48	10:09	12:02
Wet Mixer Pass	8:14	9:38**	11:21	2:38
Final Rolling	9:38*	11:05	12:27	4:05

*** completed at 9:57 with rubber tired roller**

**** second mixing with water at 10:03 a.m.**

Location I

	W	X	Y	Z
Cement on Subgrade	6:47	8:23	9:53	11:44
First Dry Mixer Pass	7:14	8:50	10:10	12:04
Wet Mixer Pass	8:15	9:39**	11:21	2:39
Final Rolling	9:38*	11:05	12:27	4:05

*** completed at 9:57 with rubber tired roller**

**** second mixing with water at 10:04 a.m.**

06/11/09 - Curing seal applied - completed at 1:30 p.m. (approx. 1 hour)

6/18/09 Stone placed

Calcium chloride tanker (38% solution) (left) pumped into water truck (right)

Samples of source calcium chloride (38% solution). Note that its clear.

View inside water truck while 3 inch diameter pipe delivers calcium chloride solution to left side of water truck tank.

Metal pans use to collect samples of calcium chloride solution delivered to subgrade from mixing truck.

Water truck connected to road mixer

General construction of NC16 test sections

Field sampling after wet mixing of soil cement

In field compaction of field-mixed samples

Appendix 4-3: Effect of Sodium Chloride – UC strength (psi)

Sample Code	Sample1 (psi)	Sample2 (psi)	Average (psi)
Bun-C6-S0.5%w-50F-3D	157.9	152.8	155.4
Bun-C6-S1.0%w-50F-3D	152.5	153.5	153.0
Bun-C6-S1.5%w-50F-3D	149.8	137.0	143.4
Bun-C6-S0.5%w-50F-7D	191.2	192.3	191.8
Bun-C6-S1.0%w-50F-7D	192.8	214.7	203.8
Bun-C6-S1.5%w-50F-7D	197.9	197.9	197.9
Bun-C6-S0.5%w-35F-3D	107.0	108.7	107.9
Bun-C6-S1.0%w-35F-3D	112.8	122.2	117.5
Bun-C6-S1.5%w-35F-3D	104.9	105.1	105.0
Bun-C6-S0.5%w-35F-7D	133.8	129.8	131.8
Bun-C6-S1.0%w-35F-7D	135.2	144.3	139.8
Bun-C6-S1.5%w-35F-7D	132.7	133.8	133.3
Bun-C6%-S1%w-35F-7D	171.3	184.3	177.8
Bun-C6%-S1%w-50F-7D	253.4	246.0	249.7
Bun-C6%-S3%w-35F-7D	177.4	174.9	176.2
Bun-C6%-S3%w-50F-7D	248.4	235.3	241.9
Bun-C6%-S5%w-35F-7D	175.4	186.2	180.8
Bun-C6%-S5%w-50F-7D	222.7	218.2	220.5

Example Legend:
Bun=Buncombe County Soil
C6=6% by weight cement
S1.5%=1.5% sodium chloride by weight of water in sample
50F= 50^0F curing temperature
3D=3 days curing duration

Buncombe Control Comparisons (average values):

Curing Temperature (F)	**Curing Duration (days)**		
	1	**3**	**7**
	UC Strength (psi)		
25	90.9	71.7	94.7
35	99.7	124.0	200.2
50	115.7	147.5	262.3
70	120.7	201.4	217.1

Sample Code	Sample1 (psi)	Sample2 (psi)	Average (psi)
Joh-C6-S0.5%w-50F-3D	214.3	204.9	209.6
Joh-C6-S1%w-50F-3D	222.7	232.8	227.8
Joh-C6-S1.5%w-50F-3D	201.1	221.7	211.4
Joh-C6-S0.5%w-50F-7D	293.5	313.7	303.6
Joh-C6-S1.0%w-50F-7D	331.3	322.2	326.7
Joh-C6-S1.5%w-50F-7D	354.8	332.8	343.8
Joh-C6-S0.5%w-35F-3D	174.4	159.4	166.9
Joh-C6-S1.0%w-35F-3D	170.4	182.1	176.2
Joh-C6-S1.5%w-35F-3D	161.2	153.5	157.4
Joh-C6-S0.5%w-35F-7D	222.9	232.1	227.5
Joh-C6-S1.0%w-35F-7D	207.6	200.6	204.1
Joh-C6-S1.5%w-35F-7D	192.5	204.9	198.7

Example Legend:
Joh=Johnston County Soil
C6=6% by weight cement
S1.5%w=1.5% sodium chloride by weight of water in sample
50F= 50^0F curing temperature
3D=3 days curing duration

Johnston Control Comparisons (average values):

Curing Temperature (F)	**Curing Duration (days)**		
	1	**3**	**7**
	UC Strength (psi)		
25	93.1	108.2	109.5
35	111.8	163.1	177.0
50	128.6	198.1	261.0
70	139.8	212.3	245.2

Sample Code	Sample1 (psi)	Sample2 (psi)	Average (psi)
Gui-C6-S0.5%w-50F-3D	207.0	196.7	201.9
Gui-C6-S1.5%w-50F-3D	218.9	218.9	218.9
Gui-C6-S1.0%w-50F-3D	218.3	220.2	219.2
Gui-C6-S0.5%w-50F-7D	258.9	270.6	264.8
Gui-C6-S1.0%w-50F-7D	256.5	243.6	250.1
Gui-C6-S1.5%w-50F-7D	260.9	252.1	256.5
Gui-C6-S0.5%w-35F-3D	175.8	169.1	172.4
Gui-C6-S1.0%w-35F-3D	164.6	164.8	164.7
Gui-C6-S1.5%w-35F-3D	162.1	163.2	162.7
Gui-C6-S0.5%w-35F-7D	209.3	219.4	214.3
Gui-C6-S1.0%w-35F-7D	207.6	207.4	207.5
Gui-C6-S1.5%w-35F-7D	203.7	207.1	205.4

Example Legend:
Gui=Guilford County Soil
C6=6% by weight cement
S1.5%w=1.5% sodium chloride by weight of water in sample
50F= 50^{0}F curing temperature
3D=3 days curing duration

Guilford Control Comparisons (average values):

Curing Temperature (F)	**Curing Duration (days)**		
	1	**3**	**7**
	UC Strength (psi)		
25	76.4	100.7	46.5
35	98.9	155.5	154.0
50	158.1	162.3	146.1
70	111.8	247.1	293.0

Appendix 4-4: UC Strength (psi), wide range (0-50%), including non-homogenized samples

Buncombe County Soil – 6% Cement, Curing Temperature, Duration and Mass of $CaCl_2$ per Mass of Cement as indicated

Sample Code	**UC Strength (psi)**								
	Sample Number, final number in bold represents average								
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	
25F-3D									
1.5%CaCl2	73	68	70	68	59	75	68	**69**	
2%CaCl2	72	73	82	77	79	86	**78**		
35F-3D									
0.5%CaCl2	93	106	86	**95**					
1.5%CaCl2	88	92	80	91	**88**				
2%CaCl2	99	84	84	84	85	104	87	83	**89**
50%CaCl2	97	95	**96**						
35F-7D									
50%CaCl2	131	137	**134**						
35F-28D									
50%CaCl2	137	136	**137**						
50F-3D									
50%CaCl2	151	130	**141**						
50F-7D									
50%CaCl2	174	192	**183**						
50F-28D									
50%CaCl2	221	206	**214**						
70F-3D									
4%CaCl2	147	140	**144**						
7%CaCl2	131	137	**134**						
10%CaCl2	121	148	**135**						
13%CaCl2	123	132	**128**						
16%CaCl2	111	134	**123**						

Note: Some samples were tested repeatedly (some just twice, others more than five times as shown above) to investigate sample variability

Buncombe Control Comparisons (average values):

Curing Temperature (F)	**Curing Duration (days)**		
	1	**3**	**7**
	UC Strength (psi)		
25	90.9	71.7	94.7
35	99.7	124.0	200.2
50	115.7	147.5	262.3
70	120.7	201.4	217.1

Guilford County Soil – 6% Cement, Curing Temperature, Duration and Mass of $CaCl_2$ per Mass of Cement as indicated

Sample Code	**UC Strength (psi)**									
	Sample Number, final number in bold represents average									
35F-3D	1	2								
2%CaCl2	118	100	**109**							
4%CaCl2	104	115	**110**							
4.7%CaCl2	219	222	**221**							
6%CaCl2	103	98	**101**							
9.3%CaCl2	226	224	**225**							
18.7%CaCl2	223	206	**215**							
23.3%CaCl2	171	181	**176**							
25%CaCl2	149	159	**154**							
35F-7D										
4.7%CaCl2	310	302	**306**							
9.3%CaCl2	284	289	**287**							
18.7%CaCl2	286	277	**282**							
23.3%CaCl2	266	279	**273**							
25%CaCl2	222	214	**218**							
35F-28D										
25%CaCl2	222	214	**218**							
4.7%CaCl2	314	303	**309**							
9.3%CaCl2	285	273	**279**							
18.7%CaCl2	306	297	**302**							
23.3%CaCl2	299	309	**304**							
50F-3D										
4.7%CaCl2	216	238	**227**							
9.3%CaCl2	228	223	**226**							
18.7%CaCl2	235	238	**237**							
23.3%CaCl2	224	231	**228**							
50F-7D										
4.7%CaCl2	283	282	**283**							
9.3%CaCl2	260	281	**271**							
18.7%CaCl2	280	272	**276**							
23.3%CaCl2	264	251	**258**							
25%CaCl2	219	214	**217**							

Sample Code	**UC Strength (psi)**								
	Sample Number, final number in bold represents average								
50F-28D									
25%CaCl2	222	214	**218**						
4.7%CaCl2	402	432	**417**						
9.3%CaCl2	352	397	**375**						
18.7%CaCl2	308	328	**318**						
23.3%CaCl2	349	339	**344**						
70F-3D									
2%CaCl2	217	172	**195**						
4%CaCl2	144	-	**144**						
6%CaCl2	108	153	**131**						
8%CaCl2	67	115	**91**						

Guilford Control Comparisons (average values):

Curing Temperature (F)	**Curing Duration (days)**		
	1	**3**	**7**
	UC Strength (psi)		
25	76.4	100.7	46.5
35	98.9	155.5	154.0
50	158.1	162.3	146.1
70	111.8	247.1	293.0

Johnston County Soil – 6% Cement, Curing Temperature, Duration and Mass of $CaCl_2$ per Mass of Cement as indicated

Sample Code	**UC Strength (psi)**										
	Sample Number, final number in bold represents average										
25F-3D	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	
0.5%CaCl2	118	110	86	126	136	129	101	97	**113**		
1.5%CaCl2	102	96	144	**114**							
2.0%CaCl2	89	99	110	94	87	**96**					
35F-3D											
0.2%CaCl2	135	129	**132**								
0.5%CaCl2	136	138	109	134	149	102	93	133	**124**		
1.5%CaCl2	136	113	108	125	115	131	114	127	145	**124**	
2%CaCl2	130	122	129	109	117	109	130	125	90	97	**116**
6%CaCl2	103	110	**107**								
8%CaCl2	109	115	**112**								
10%CaCl2	106	110	**108**								
12%CaCl2	97	95	**96**								
35F-7D											
6%CaCl2	164	171	157	168	**165**						
8%CaCl2	159	156	162	170	**162**						
10%CaCl2	148	150	**149**								
50%CaCl2	84	96	**90**								
35F-28D											
50%CaCl2	117	120	**119**								
50F-7D											
50%CaCl2	221	217	**219**								
50F-28D											
50%CaCl2	188	167	**178**								
70F-3D											
2%CaCl2	208	218	286	244	**239**						

Note: Some samples were tested repeatedly (some just twice, others more than five times as shown above) to investigate sample variability

Johnston Control Comparisons (average values):

Curing Temperature (F)	**Curing Duration (days)**		
	1	**3**	**7**
	UC Strength (psi)		
25	93.1	108.2	109.5
35	111.8	163.1	177.0
50	128.6	198.1	261.0
70	139.8	212.3	245.2

Appendix 4-5: UC Strength (psi), narrow range (0-2%), homogenized samples

Guilford 35F	S1	S2	S3	Average (psi)
0.0	151	182	195	**176**
0.2	158	142	158	**152.67**
0.5	130	134	152	**138.67**
1.0	147	134	162	**147.67**
1.5	151	136	163	**150**
2.0	150	154	151	**151.67**

Johnston 35F	S1	S2	S3	Average (psi)
0.0	179	147	186	**170.67**
0.2	164	165	166	**165**
0.5	166	165	175	**168.67**
1.0	165	164	175	**168**
1.5	159	135	172	**155.33**
2.0	189	168	165	**174**

Buncombe 35F	S1	S2	S3	Average (psi)
0.0	149	149	146	**148**
0.2	140	149	139	**142.67**
0.5	118	121	129	**122.67**
1.0	126	128	125	**126.33**
1.5	136	143	127	**135.33**
2.0	138	145	129	**137.33**

Guilford 50F	S1	S2	S3	Average (psi)
0.0	157	140	161	**152.67**
0.2	171	176	173	**173.33**
0.5	160	187	170	**172.33**
1.0	184	158	-	**171**
1.5	159	164	159	**160.67**
2.0	117	150	124	**130.33**

Johnston 50F	S1	S2	S3	Average (psi)
0.0	172	186	172	**176.67**
0.2	183	161	168	**170.67**
0.5	175	188	166	**176.33**
1.0	199	163	185	**182.33**
1.5	209	207	241	**219**
2.0	190	208	189	**195.67**

Buncombe 50F	S1	S2	S3	Average (psi)
0.0	165	153	152	**156.67**
0.2	188	176	183	**182.33**
0.5	182	175	171	**176**
1.0	185	184	184	**184.33**
1.5	173	174	-	**173.5**
2.0	174	161	173	**169.33**

Appendix 4-6: UC Strength (psi) of samples collected from the field

35F	**3 Day Tests**			**7 Day Tests**			**28 Day Tests**		
	Average	**1**	**2**	**Average**	**1**	**2**	**Average**	**1**	**2**
Control 1	118	121	115	189	158	220	187	182	192
Low Dosage 2.3%	96.5	95	98	79.5	78	81	77	90	64
High Dosage 8.3%	124.5	124	125	61.5	67	56	95.5	82	109
Control 2	57.5	55	60	101	100	102	127	157	97
70F	**3 Day Tests**			**7 Day Tests**			**28 Day Tests**		
	Average	**1**	**2**	**Average**	**1**	**2**	**Average**	**1**	**2**
Control 1	192	197	187	283.5	295	272	306	308	304
Low Dosage 2.3%	149	132	166	136.5	120	153	77.5	85	70
High Dosage 8.3%	154	173	135	104	94	114	83	69	97
Control 2	70	77	63	173	159	187	186	189	183

Note: Replicate samples were mixed and compacted in the field and then transported to the laboratory where they cured at either 35^0F or 70^0F for 3, 7 or 28 days prior to testing.

Appendix 4-7: UC Strength (psi) of samples mixed in the lab

35F	3 Day Tests			7 Day Tests			28 Day Tests		
	Average	**1**	**2**	**Average**	**1**	**2**	**Average**	**1**	**2**
Control 1	170.5	173	168	200.5	200	201	234.7549	218.01	251.50
1% Dosage	158	153	163	195.5	193	198	271.1776	277.77	264.59
5% Dosage	135.5	144	127	192	198	186	233.5	230	237
70F	**3 Day Tests**			**7 Day Tests**			**28 Day Tests**		
	Average	**1**	**2**	**Average**	**1**	**2**	**Average**	**1**	**2**
Control 1	180.5	180	181	234.5	236	233	278	270	286
1% Dosage	192.5	202	183	247.5	240	255	294.5	295	294
5% Dosage	187	183	191	250	247	253	338.5	342	335

Note: Replicate samples were mixed and compacted in the laboratory where they cured at either 35^0F or 70^0F for 3, 7 or 28 days prior to testing.

Appendix 4-8: DCP Data collected from NC16 Construction (Summary)

Courtesy of Tom Hearne, NCDOT

Summary NC 16 DCP Data

Section		W	W	W	W	W	W	W	W	W	W
Location		Median CBR Values									
Target Time	Depth (inches)	0	30 min	1 hour	2 hours	4 hours	8 hours	1 day	2 days	5 days	7 days
	0-1	8	8	9	10	14	16	20	23	31	36
	1-2	8	9	10	11	15	18	24	33	41	47
	2-3	9	10	12	12	17	23	33	35	52	58
	3-4	10	11	14	15	18	22	38	47	64	63
	4-5	10	12	16	16	20	25	42	52	73	74
	5-6	11	13	17	19	22	27	49	59	83	85
	6-7	12	14	17	18	21	28	45	61	79	82
	7-8	12	14	17	18	22	25	40	48	68	72
	8-9	12	13	14	16	19	19	27	41	53	53
	9-10	10	11	11	13	15	14	18	26	30	28
	10-11	9	10	10	12	11	12	13	16	18	16
	11-12	8	9	9	10	10	10	11	12	14	11

Section		X	X	X	X	X	X	X	X	X	X
		Median CBR Values									
Target Time	Depth (inches)	0	30 min	1 hour	2 hours	4 hours	8 hours	1 day	2 days	5 days	7 days
	0-1	11	11	12	14	18	16	21	23	24	25
	1-2	11	12	15	16	19	23	27	28	36	37
	2-3	16	15	16	18	21	30	33	37	42	51
	3-4	16	16	18	20	22	34	38	46	51	59
	4-5	18	18	18	21	25	36	43	53	65	69
	5-6	15	18	18	22	26	35	42	53	55	59
	6-7	14	15	18	21	23	29	36	50	51	52
	7-8	13	15	15	18	21	26	30	40	42	41
	8-9	12	13	15	17	17	19	21	27	28	24
	9-10	12	11	13	15	15	16	18	18	19	19
	10-11	12	11	12	13	13	14	14	15	15	14
	11-12	12	9	10	11	10	11	12		12	12

Section		Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
		Median CBR Values									
Target Time	Depth (inches)	0	30 min	1 hour	2 hours	4 hours	8 hours	1 day	2 days	5 days	7 days
	0-1	11	11	11	13	19	14	18	23	30	28
	1-2	12	13	15	16	19	21	23	27	32	32
	2-3	13	14	16	18	21	28	27	31	30	35
	3-4	15	16	18	21	24	29	33	39	39	41
	4-5	16	17	19	22	26	33	42	51	48	50
	5-6	16	17	20	23	28	34	42	51	57	59
	6-7	15	18	20	22	26	32	38	45	56	59
	7-8	15	17	18	20	25	28	35	41	48	45
	8-9	14	15	17	17	21	27	26	33	35	39
	9-10	14	15	16	15	17	17	20	22	24	23
	10-11	14	15	15	14	15	16	15	18	17	17
	11-12	13	12	14	13	14	13	13		14	14

Section		Z	Z	Z	Z	Z	Z	Z	Z	Z	Z
						Median CBR Values					
Target Time	Depth (inches)	0	30 min	1 hour	2 hours	4 hours	8 hours	1 day	2 days	5 days	7 days
	0-1	13	15	12	13	13		19	24	26	28
	1-2	14	16	17	18	18		25	32	41	43
	2-3	16	17	19	21	22		29	41	56	52
	3-4	17	19	21	23	27		33	47	67	67
	4-5	17	20	21	24	26		38	49	77	78
	5-6	17	19	20	21	26		34	47	65	69
	6-7	16	16	18	21	24		30	38	51	55
	7-8	15	15	16	17	19		23	29	35	40
	8-9	14	15	15	15	15		17	20	23	25
	9-10	13	13	13	13	14		15	16	16	17
	10-11	13	13	13	13	13		13	15	14	14
	11-12	12	13	13	12	13		13	15	13	13

Variation in All DCP Test Results by Section

0-7" Average CBR Coefficients of Variation

	Section			
Time	W	X	Y	Z
0	26	32	18	24
30 min	30	31	21	24
1 hour	43	31	18	22
2 hours	26	58	21	21
4 hours	30	26	21	24
8 hours	29	25	27	-
1 day	26	22	29	25
2 days	23	20	36	22
5 days	23	25	42	28
7 days	26	27	37	31