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Civil & Environmental Engineering

SOIL MIXING DESIGN METHODS AND CONSTRUCTION TECHNIQUES FOR USE IN HIGH ORGANIC SOILS

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FINAL REPORT

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
AREA				
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		TEMPERATURE (exact degrees)		
ºF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	ºC

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fL	foot-Lamberts	3.426	candela/m^2	cd/m^2

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa
kip	kilopound	4.45	kilonewtons	kN

APPROXIMATE CONVERSIONS TO SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
TEMPERATURE (exact degrees)				
°C	Celsius	1.8C+32	Fahrenheit	°F

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2
kN	kilonewtons	0.225	kilopound	kip

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

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16. Abstract

Organic soils present a difficult challenge for roadway designers and construction due to the high compressibility of the soil structure and the often associated high water table and moisture content. For other soft or loose inorganic soils, stabilization via cement or similar binders (a method called soil mixing) has proven to be an effective and predictable solutions; the FHWA has published a comprehensive design manual for these techniques. Organic soils, however, are not addressed therein to a level of confidence for design as organic soils do not follow the trends of inorganic soils.

In short, the high porosity, high water content, and high levels of humic acids differentiate organic soils from all other soils where soil mixing has been proven successful. To combat these effects, more cement content is required to bring the water/cement ratio down to acceptable levels and even more cement is required to offset the acidity.

Extending the observations of past researchers, a threshold cement content was defined below which no strength gain was achieved. This threshold was then defined as a cement content offset above which the measured strengths matched well with other soil types.

This report presents the findings from a thorough literature search, laboratory bench tests, large scale laboratory tests, field evaluation of past and on-going projects and concludes with recommendations for designing for soil mixing applications in highly organic soils.

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Executive Summary

Organic soils present a difficult challenge for roadway designers and construction due to the high compressibility of the soil structure and the often associated high water table and high moisture content. For other soft or loose soils (inorganic soils), stabilization via cement or similar binders (a method called soil mixing) has proven to be an effective solution, and to this end, the Federal Highway Administration has published a comprehensive design manual for these techniques. Organic soils, however, are not addressed therein to a level of confidence for design as organic soils do not follow the trends of inorganic soils. This has been attributed to the high porosity, high water content, and high levels of humic acids common to organic soils.

This report presents the findings from a thorough literature search, laboratory bench tests, large-scale laboratory tests, field evaluation of past and on-going projects, and concludes with recommendations for designing soil mixing applications in highly organic soils.

Laboratory tests (bench tests) were performed to assess the effect of cementitious binder type, binder content, mixing method, organic content, and curing time on strength gain. This phase of the study involved over 700 tests where in all cases, specimens with organic content higher than approximately 10% required disproportionally more cement for the same strength gain when compared to inorganic or low organic content samples.

Using the findings of the bench tests, a 1/10 scale test bed was built in which soil containing approximately 44% organics was placed and conditioned with rain water. The dimensions of the bed accommodated three side-by-side tests wherein dry and wet soil mixing was performed each on one third of the bed. The remaining third of the bed was left untreated. Load tests were then performed on the three portions of the bed where the load for a simulated roadway was placed. These loads were left in place for several weeks and monitored for movement. Results showed marked improvement for the treated portions relative to no treatment, with virtually identical response coming from dry or wet methods (both used identical amounts of cement per volume).

Concurrent to the bench tests and 1/10 scale load tests, field evaluation of past and on-going soil mixing programs were conducted. These showed in all cases that soil mixing has been largely successful. Both wet and dry mixing programs were reviewed. Two problematic sites where continued subsidence of a rural road and bridge over organic and/or soft soil were also investigated to provide a comparative reference.

The findings of this study suggest that the adverse effects of organic soils can be combatted where more cement content is required to bring the water/cement ratio down to acceptable levels and even more cement is required to offset the acidity. While this has been a recurring observation of past researchers, a cement factor threshold was defined by the study findings below which no strength gain was achieved. This threshold was then defined as a cement factor offset above which the measured strengths matched well with other soil types. As a result, a recommended approach for designing soil mixing applications in organic soils was developed.

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Chapter 1: Introduction

Florida is well-known for its sunny beaches and year-round moderate temperatures that attract tourists from around the world. The annual rainfall, climate, and relatively flat land make Florida well suited to produce wetlands, promote plant life, and foster the associated organic decay. These wetlands are almost as well-known as the beaches, for example, the Everglades and it's aptly named I-75 corridor through it, the Alligator Alley, or the Green Swamp with its lush plant life and recreational areas. These areas contain over two million acres of organic deposits often termed "muck", and when the organic content exceeds 75%, it can be classified as "peat" (Stinnette, 1997). However, organic deposits are not isolated in wildlife preserves; rather, they are commonly encountered throughout the state in existing or proposed new roadway alignments. Organic deposits is an undesirable subgrade material for construction of highways due to their high compressibility and low strength.

Historically, there have been many approaches to handling soft compressible soils that include organic deposits. From these experiences, the FDOT *Soils and Foundations Handbook* lists several options in Section 8.4.1.3:

1. *Reduce fill height. This is seldom practical except in planning phase.*
2. *Provide waiting period to allow for the majority of consolidation to occur.*
3. *Increase surcharge height.*
4. *Use a lightweight fill.*
5. *Install wick drains within the compressible material to be surcharged.*
6. *Excavate soft compressible material and backfill with granular soil.*
7. *Ground modification such as stone columns, dynamic compaction, etc.*
8. *Deep soil mixing.*
9. *Combinations of some of the above.*

The first four options pertain to techniques used to compensate for load/settlement properties while the last four are soil modification approaches. Most of the options are aimed at new construction, but existing roads and bridge embankments can be vulnerable to long-term settlement that leads to recurring maintenance costs. This study focuses on ground modifications that make use of cement-stabilized soil mixing methods.

Soil mixing is basically categorized into two methods: dry soil mixing or wet soil mixing. Dry soil mixing is best suited for saturated soils with high moisture contents like clay or organics. Therein, the natural moisture in the soil is sufficient to hydrate cement products that are introduced in a *dry* powder form, hence the name. In contrast, wet soil mixing uses cement from pre-*wetted* cement slurry. As the slurry is already hydrated, it does not depend on the natural moisture content to ensure strength gain is realized. As a result, it is well suited for dry loose soils or soils that do not contain enough moisture to activate the dry mixed alternative. Figures 1.1 and 1.2 show dry and wet mixing equipment, respectively.


Figure 1.1. Soil mixing using powdered cement injected while tilling soil.


Figure 1.2. Soil mixing using fluid cement slurry pumped through ports in auger blades.

This report is organized into five ensuing chapters that address the use of soil mixing with particular focus on organic soil applications. Chapter 2 explains some of the concerns of organic soils including: (1) discussion of unique compressibility characteristics of organic material (2) an overview outlining the applicability of each of the FDOT *Soils and Foundations Handbook* suggested alternatives as they pertain to organic soils, (3) soil mixing and other stabilization techniques, (4) lessons learned from applicable case studies, and (5) an interpretation of the literature search findings.

Chapter 3 deals with laboratory testing where bench tests were conducted using varied: binder types, binder contents, and organic contents to test the effects on the cement stabilized soil capacity.

Chapter 4 discusses 1/10th scale outdoor testing that was performed to verify the findings of the bench tests. These tests included long-term compression tests of wet and dry mixed organic soil along with a control sample of identical soil composition but untreated. The simulated roadway scenario was intended to withstand a 5ft thick new roadway base over an organic deposit (or 600psf).

Chapter 5 presents the findings from five sites with various ground treatment or maintenance programs. These include long-term monitoring of previously performed organic soil stabilization, settlement data from a roadway crossing an untreated deep organic deposit, a soil mixing project that ran concurrent to and was tracked by the research team, and a comparative study of a project where the ground remediation used no binders.

Finally, Chapter 6 concludes by summarizing the project findings and provides recommendations for designing / anticipating the required binder content for stabilizing organic soils.

Chapter 2: Literature Review

This chapter discusses the historical approaches used to address the presence of organic or compressible materials in roadway alignments.

2.1 Compressibility Characteristics of Organic Soils

When a soil is subjected to a compressive stress the resulting volume change or consolidation may be attributed to some or all of the following factors:

1. The relocation of solids
2. The deformation of solids
3. The deformation of pore water and air
4. The expulsion of pore water and air

In the case of clays, factors 3 and 4 cause a time lag in settlement. A static load produces a pressure gradient in the pore water which causes movement to the drained surfaces. The movement is slow and a function of permeability. The time lag in settlement caused by this phenomenon is referred to as the hydrodynamic lag. In order for clay particles to move closer together under a static load, the structured double layer of water surrounding the clay particles must deform. The deformation may be caused by loads that tend to expel the water or by shear loads that cause a shear deformation in the water surrounding the clay particles. Both actions have a viscous nature and are a function of the magnitude of the load that causes the deformation. The time associated with this viscous resistance is called the viscous lag. Terzaghi's one dimensional consolidation theory only takes into account the effects of the hydrodynamic lag as being responsible for the time delay in settlement.

In organic soils, on the other hand, the major factors of volume change include rapid expulsion of pore water and air (if unsaturated) and long-term deformation of the solids. The latter (secondary) consolidation stage is due to the continued movement of particles as the weak soil structure readjusts to the increased effective stress. Both empirical and theoretical methods have been developed for predicting settlement due to secondary compression. Although the processes of primary and secondary consolidation are treated separately for convenience of analysis, they actually occur simultaneously (Edil and Dhowian 1979).

Two major assumptions forming the framework for Terzaghi's theory are not valid for organic soil, due to: (1) the compressibility of organic solids and (2) the change in permeability under any one increment of applied stress. These two variations are believed to account for the significant differences in consolidation behavior between organic and mineral soils. Organic soils undergo a rapid dissipation of pore pressure leading to a short-term primary stage and a long-term secondary stage of consolidation. The large magnitude and short duration of the primary stage and the continuous long-term secondary compression are the major departures from mineral soil behavior. Based on these deviations the conventional consolidation theory is not applicable for organic soils.

There have been both empirical and theoretical models developed to predict settlements due to secondary compression. In Buisman's (1936) method, the coefficient of secondary consolidation $\mathbf{C_a}$ is defined as the slope of the linear portion of a vertical strain-logarithm time plot. It is implied that the coefficient of secondary consolidation is determined over the appropriate range of stress increase and that the compression follows a linear relationship in a plot of strain versus the logarithm of time. A major disadvantage of the Buisman's approach is the assumption that compression continues indefinitely; hence, a sample could disappear in a laterally confined condition. Another drawback of the Buisman approach is that the time at which secondary consolidation begins is not clearly defined (Gibson and Lo 1961). Nevertheless, the Buisman theory is widely used in the analysis of clays as the constant rate of secondary compression is frequently observed for most clays within the range of times considered (Edil and Dhowian 1979). This same behavior has also been observed and reported for organic soils by Berry and Vickers (1975).

A rheological approach which is based on the assumption that structural viscosity is linear was suggested by Gibson and Lo (1961). Their approach was based on the rheological scheme consisting of a Hookean spring, ***a***, connected in series with a Kelvin element comprised of a spring, ***b,*** and dashpot **λ**. Limitations of the model include the assumption that the structural viscosity of the soil is linear. The model also does not take into account certain non-linearities which may result from finite strains, stress level, and strain rate. It has also been reported that **λ** is non-linear over time. In spite of these drawbacks, the Gibson & Lo theory yields reasonable results in predicting the rate and magnitude of organic soil settlement when compared to field measurements (Tan et al, 1971).

When these approaches were used by the PIs to analyze the laboratory consolidation test results of Florida organic soils, remarkable agreement was observed in the ultimate settlement predictions, thereby evolving into a definitive method for predicting the ultimate settlement of organic soil (Gunaratne et al 1998). The following expressions to predict the ultimate strain and hence, settlement of organic soil are based on the organic content (OC) and the applied stress.

$$\varepsilon_{ult} = \Delta\sigma[a + b] \tag{1}$$

where

$$a = \frac{\dfrac{97.8}{(1.27\sigma + 97.8)^2} + \dfrac{23.1(OC)}{(0.16\sigma + 23.1)^2}}{F(OC,\sigma)} \tag{2a}$$

$$b = \frac{\dfrac{360.1}{(1.86\sigma + 360.1)^2} + \dfrac{40.2(OC)}{(0.52\sigma + 40.2)^2}}{F(OC,\sigma)} \tag{2b}$$

$$F(OC,\sigma) = \left[2.79 - \frac{\sigma}{(0.78\sigma + 74.3)}\right] + OC\left[9.72 - \frac{\sigma}{(0.12\sigma + 15.3}\right] \quad (2c)$$

and

OC = organic content
$\Delta\sigma$ = applied stress
σ = in situ stress

Another significant unfolding of the above study was the development of valuable correlations between the void ratio, pressure and the organic content of natural organic soils. It was illustrated how this can be employed in the prediction of ultimate ground subsidence on organic deposits.

2.2 Applicability of FDOT Recommendations for Compressible Soils

2.2.1 Wick Drains

Use of wick drains are an effective means to reduce the consolidation time by shortening the drainage path of compressible materials which typically have poor drainage properties (e.g. clayey soils). The drains are installed prior to surcharging throughout an entire treatment area on a repeating pattern as shown in Figure 2.1. Although typically considered to be prefabricated vertical drains, the same rate of drainage can be brought about by way of stone or sand columns installed in any fashion. The efficiency of the drainage is dependent on the spacing, **s**, and diameter of the drains, **d,** and can be assessed using the following well established theoretical relationships:

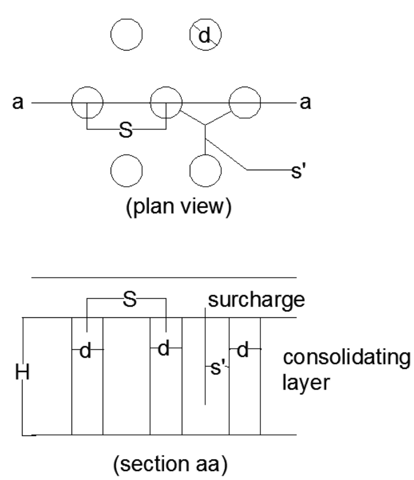

Figure 2.1. Wick drain configuration (Mullins, 1996).

Horizontal drainage paths typically dominate the consolidation process. Based on Biot and Savaart's 3-D consolidation theory, the degree of radial consolidation, $\mathbf{U_r}$, can be determined by:

$$U_r = 1 - \exp(-\frac{8T_r}{F(n)}) \quad (3)$$

where

$$F(n) = \frac{n^2}{n^2-1}\ln(n) - \frac{3n^2-1}{4n^2} \quad (4a)$$

and

$$n = \frac{s}{d} \quad (4b)$$

$$T_r = \frac{C_h t}{s'^2} \quad (4c)$$

where $\mathbf{C_h}$ is the coefficient of horizontal consolidation that depends primarily on the horizontal permeability of the soil.

Similarly, the contribution from vertical drainage is determined using the conventional Terzaghi's expression for rate of consolidation where

$$T_v = \frac{C_v t}{H^2} \quad (4d)$$

and $\mathbf{C_v}$ is the coefficient of vertical consolidation, $\boldsymbol{T_v}$ is determined to find the degree of vertical consolidation, $\boldsymbol{U_v}$. ***H*** and ***s'*** are the distance of the longest vertical drainage path vertically and radially, respectively. The total degree of consolidation from both combined radial and vertical drainage is determined using:

$$U = 1 - (1 - U_v)(1 - U_r) \quad (5)$$

Figure 2.2 shows the effect of the spacing to diameter ratio, ***n***, and was prepared using a layer thickness of 10 ft, a $\mathbf{C_h}$ value of 3.84 in^2/day and $\mathbf{C_v}$ of 11.52 in^2/day. With no wick drains, the degree of consolidation is about 90% after one year. By using wick drains this level of consolidation occurs in as little as 2 weeks depending on the spacing ratio (**n** = 2, 2.5, 4, 8, 16).

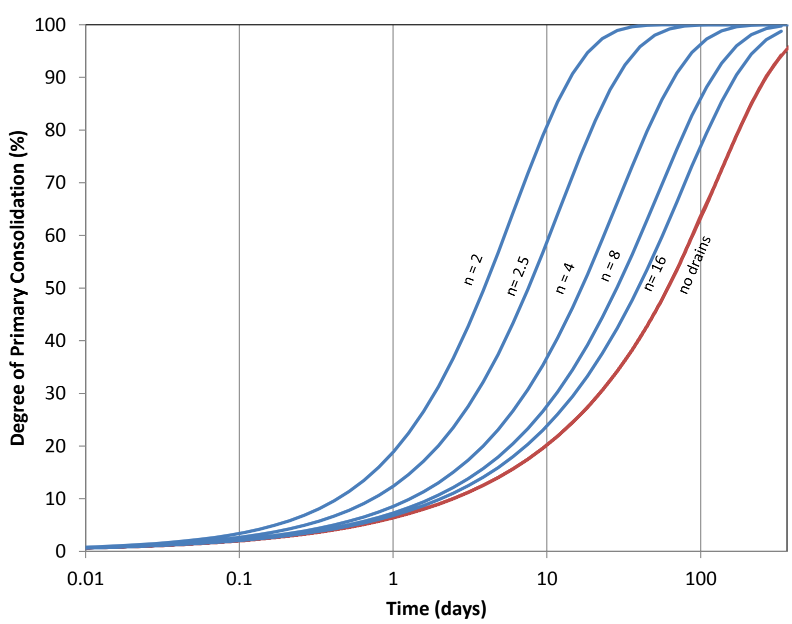

Figure 2.2. Consolidation rate as function of n based on both radial and vertical drainage paths.

A wide selection is available in terms of wick drain manufacturers, products and contractors. For stabilization programs involving soil mixing and other viable construction techniques in organic soils, wick drains and the associated theory can be very useful. However, it must be noted that this becomes an effective method of treatment only when the primary consolidation component of the organic soil settlement dominates the secondary component, which can only be expected in *organic clays*.

2.2.2 Excavation and Replacement of Soft Soils

When soft compressible materials are encountered near the surface, it is also possible to excavate and replace these soils with suitable backfill material. This can be an expensive alternative which may require dewatering, sheet pile support, or both. If the removed material is contaminated, disposal of the material adds additional complexity. Figure 2.3 shows an example of a demucking program along Interstate I-4 near Plant City that extended for over a mile (Mullins, 1996).

Figure 2.3. Excavation and replacement of organic soils along Interstate I-4 (Plant City).

For deeper deposits this is not practical and the expense of this approach is generally prohibitive with the exception in projects where the organic material could be reused as top soil elsewhere in the project. This option is irrelevant to the specific objectives of this project, but provides a baseline for economic comparison when evaluating soil mixing cost efficiency.

2.2.3 Ground Modification

Ground modification encompasses a broad range of techniques including stone columns, sand columns, dynamic compaction, dynamic replacement and soil mixing. Of these, soil mixing is discussed separately in an ensuing section as a viable treatment technique for treating Florida's organic soils.

2.2.3.1 Stone Columns

Stone columns are semi-rigid inclusions installed by progressively packing sand or stone into a borehole formed by a vibrating electric motor within a probe at the end of a long steel stem. The length and diameter of the probe can be tailored to suit the site geology. Stone columns can be installed using dry or wet methods. Dry methods are usually exclusively used in clayey/poorly draining soils to avoid excessive site disturbance and extended delays in dissipating pore pressure. Wet methods are generally considered easier to use, less expensive and only used in free draining soils. In cohesive soils, the vibration has minimal effects on consolidating the material but the overall stiffness of the treatment area is improved proportional to the area of columns installed relative to the overall area. The column diameter is not expected to be much larger than the probe when installed in cohesive soils. In granular soils, the probe vibration can densify the surrounding soils with a radial influence as much as 2-3 diameters away from the probe. This technique is commonly used to stabilize sinkhole prone areas by pre-collapsing weak karst strata. Stone columns are not considered to be a viable approach to stabilizing organic deposits due to the progressive loss of lateral support that is necessary to assure column stiffness.

2.2.3.2 Dynamic Compaction

Dynamic compaction (DC) is a method of densifying loose deposits up to depths of 30 ft by dropping a heavy weight (6 to 40 tons) from heights as much as 100 ft (Figure 2.4). The depth of improvement (in meters) has been empirically given by:

$$D = e\sqrt{WH} \qquad (6)$$

where $\boldsymbol{W}$ is the weight of the pounder (in tonnes), $\boldsymbol{H}$ is the drop height (in meters), and e is an energy efficiency factor (ranges from 0.3 to 0.8) that lumps the effects of frictional losses of the dropping system and variations in soil type (Menard and Broise 1975). Normally these pounders are wider than tall with heights ranging from 1.5 to 2 ft. The impact produces a strong compression wave without excessive penetration, however care must be taken to test the site's initial soil strength to ensure that the pounder is not lost with the first few drops, particularly in organic soils (Lukas 1986). Alternately, semi-empirical energy methods can be used to predict the crater depth prior to the initial impact and set an appropriate drop height (Mullins et al 2000).

Figure 2.4. Twenty-five-ton (25-ton) DC pounder dropped 50 ft to densify landfill contents.

When using DC, a predetermined print pattern is laid out across the treatment area where multiple impacts on a given location and multiple passes across the site are used. Although dynamic compaction is reported to be effective both above and below the ground water table, construction difficulties can arise if the water table is not maintained at least 6-7 ft below the ground surface (Lukas 1986). This in many cases requires the grade to be raised which also provides a convenient working platform. For organic soils dynamic compaction has been adapted and is called dynamic replacement using a modified hammer.

2.2.3.3 Dynamic Replacement

Dynamic replacement and mixing (DRM) is an innovative ground modification technique developed in Singapore as an extension to more common dynamic compaction. Therein, consolidation can be accelerated by dynamic replacement (DR) and DRM of the compressible deposits with sand columns. This process takes place in two phases: (1) DR using low energy punching through a sand surcharge to form columns shown in Figure 2.5, followed by (2) high energy impacts on the columns that burst the columns and issue jets of sand from the installed DR columns into the surrounding deposits to create a stiffer dual layer composite of the two material components (Lo et al, 1989).

Figure 2.5. Four-ton pounder lifted to 40 ft (left); during impact (right).

Field and laboratory investigations have shown that DRM can transform in situ peaty clay deposits into an upper sand raft with pockets of peaty sand underlain by a fairly uniform layer of mixed sand and peat. Mixing between the columns further aids in radial drainage especially in organic clays. In this way, DR is considered to be an in situ *mechanical soil mixing* method that uses no binder.

DRM has been shown to minimize secondary compression of peaty clays while expediting the primary compression (Lo, Ooi and Lee 1990). Significant strength increases in peaty material that accompanies DRM is an additional benefit. Other related studies (Lee et al., 1980, Kruger et al., 1980 and Terashi et al., 1981) demonstrated that methodical application of high energy is not only effective but also is at least 30% less expensive for peaty clays over the other common stabilization procedures.

Inspired by the published advantages of DRM, such as financial savings and preclusion of needless construction time delays, a team of University of South Florida (USF) researchers lead by the PIs collaborated with FDOT on a research project to examine the effectiveness of DRM on Florida organic material (Gunaratne et al, 1997). This study demonstrated that dynamic replacement (DR) is an effective technique to improve both the settlement and strength properties of shallow to moderately deep organic soil deposits. The resulting diameter of the sand column and grid spacing are used to predict consolidation rate using wick drain theory presented earlier and the strength of the treatment area (discussed later).

More recently, DR was used to stabilize soft clays near the Alafia River where the SR-37 Bridge crossing the river was widened. The resulting improvements show startling success, especially given the ease and cost effectiveness of the approach. Full documentation of this project is presented in ensuing chapters.

2.2.4 Soil Mixing

There are numerous proprietary methods of soil mixing wherein a binder such as lime, cement, or slag is mixed with the in situ material to improve its strength characteristics. The relevant equipment range from full length multi-auger systems to huge blenders with vertically or horizontally oriented paddles. Soil-cement is perhaps the most rudimentary method which has been used for decades as a construction material to increase the strength of roadway subgrades or serve as bedding for pipes where compaction becomes difficult after installation. Soil-cement preparation is typically an above-ground mixing process that is not the focus of this study; instead, methods to strengthen/mix soil in place are addressed herein. These will include jet grouting, wet soil mixing, and dry soil mixing.

Like stone column installation methods, soil mixing can be categorized as dry or wet soil mixing depending on whether the binder is added to the soil as a dry powder or as a pre-wetted slurry. The decision between the applications of the wet or dry methods, is based on a threshold moisture content of the soil; when the moisture content is greater than 60%, enough natural moisture exists to use dry methods and for in situ soils with moisture contents below 40%, wet methods are used. Dual applicability exists between moisture contents of 40 and 60%.

2.2.4.1 Jet Grouting

Jet grouting is one of the earliest of the modern soil mixing techniques which could be ideal for stabilizing existing roadways as it uses a small-diameter drill stem to inject high pressure grout out the side of a cutting tip (up to 650 ft/sec). This produces a column of improved soil with radius proportional to the imparted pressure and inversely proportional to the in situ soil strength. When used on an appropriate spacing and grid pattern, it is conceivable that a large section of existing road could be improved, leaving only a series of small-diameter core holes in the pavement.

Four basic versions of jet grouting are readily available: single fluid jetting (grout slurry only), double fluid jetting (grout and air), triple fluid jetting (grout, air and water), and “superjet” grouting (Hayward Baker Inc. 2009). Past applications for soil stabilization, such as that intended for this project, have used the double fluid method. Generally, the four methods can produce reinforced soil columns with diameters of 2 to 4 ft; 3 to 6 ft; 3 to 5 ft; and 10 to16 ft, respectively, depending on the in situ soil strength.

2.2.4.2 Wet Soil Mixing

Technically jet grouting can be identified as a wet soil mixing technique, but a common version of wet soil mixing makes use of a mechanical mixing method that injects a wet binder slurry. The equipment used is similar to drilled shaft rigs where a large diameter multi-paddle tool slowly spins and advances into the soil while injecting grout slurry (Figure 2.6). This process produces spoils up to 30 or 40% of the final column volume which may be a problem if the soil is contaminated.

Figure 2.6. Wet soil mixing equipment (courtesy of Hayward Baker).

The resulting column strength is dependent on the cement content which is injected on a per linear foot basis to achieve the desired strength. The zone of improvement is limited to the diameter of the paddles and like all soil stabilization methods it is performed on a pattern that provides sufficient coverage to achieve the design strength of the entire treatment area. This method is most effective in soils with moisture content less than 60%. For existing roadways, the significant level of disturbance to the entire road bed would make this a less appealing option.

2.2.4.3 Dry Soil Mixing

Dry soil mixing (DSM) is a technique relatively new to the U.S. but it has been used for several decades abroad. It was first used in the U.S. in 2006 on the Jewfish Creek Project along the US-1 corridor in south Florida (Garbin and Mann 2010). This method can use equipment similar to Figure 2.6 on the same type of pattern layout or it can be performed on shallow soil deposits using a horizontal axis, tilling-type tool head (Figure 2.7). The latter treatment produces complete coverage of side by side rectangular areas which then can be considered *mass stabilization*. This ground improvement/soil mixing method blends a dry binder into the soil (using either blade type) by means of high pressure air. Figure 2.7 shows the horizontal tilling tool recently used at the Marco Island Airport. Both the Jewfish Creek and Marco Island case studies are discussed later.

DSM can be used to stabilize contaminated soils where spoil removal is problematic. But, like most soil mixing systems discussed, this requires highly specialized equipment.

Figure 2.7. Tilling type mixing tool for DSM mass stabilization (2- to 3-ft diameter and 5 ft wide).

DSM is ideal for weak soils where the undrained shear strength is less than 200 psf. It is well suited for organic soils as it requires the higher moisture content to fully activate the binder and can be performed on both shallow and deep deposits. The high moisture content of organic soil results in high w/c ratios unless large amounts of cement are used. Contractors purport that use of a combination of cement and slag tends to give better results.

On the onset of this study, there had been only 35 dry mixing projects in the U.S. of which several have been in organic deposits in Florida.

2.2.4.4 Different Binders as Cement Replacement

As an overview, it is understood that the type of binder (cement, fly ash, slag, etc.), amount of binder, and water-to-cement ratio all affect the strength of concretes or other cemented matrices like that found in soil mixing. Therein, the time to obtain full strength can also be a side effect. Historically, blending cement with alternate binders was used as a means to lower cost where coal fired power plants produce waste products well suited for use as a cement replacement. For materials such as fly ash, this was mostly true where the cost was about half that of cement; however, costs today are similar to cement (Mindess, Young, & Darwin, 2003). This is because over time it was discovered that these materials held a greater advantage than just cost efficiency. Benefits include increased strength, durability, workability, and reduced permeability. Fly ash for example allows a reduction in the amount of water needed for the mixture, thus leading to a lower water-to-cement ratio for the same slump and in turn an increase in strength. Chloride diffusivity was shown to be markedly reduced in flyash mixes where diffusivity values dropped by orders of magnitude. As a result, most FDOT concrete mixes now require at least 20% flyash.

Pozzolans (such as fly ash, silica fume, rice husk ash), however, are limited by the amount of calcium hydroxide within the paste to react (Mindess, Young, & Darwin, 2003) which in general slows the strength maturation. Figure 2.8 shows the effect of various binders on strength development up to 60 days. In all cases, the supplanted mixes surpass the control (100% cement) in strength, even at 28 days. It should be noted that although it was not explicitly stated in the cited source the assumption was made that the control represented 100% cement and the percentage of binder was the amount replaced. It was also assumed that the same water-to-cement ratio was used for all mixes and the overall binder content was also the same (total binder weight per volume of concrete). Therein, silica fume (SF) can be used for high early strength (due to high fineness), but does not gain significantly later; rice husk ash (RHA) has an even more dramatic effect on early strength. While silica fume and rice husk ash mixes gain the majority of their strength within 28 days, fly ash (class C and F) requires more time and still gain strength after 56 day. Slag mixes, similar to fly ash, are also known for slow strength development (>56 days), which correlates to low heat production rates, advantageous for mass concrete applications. Regardless of the ultimate strength of a given mix, construction schedules often dictate the constituent proportions so delays are not imposed. Therein, ultimate strengths are often higher than needed.

Figure 2.9 shows an interpretation of Figure 2.8 and helps to show the speed of each admixture reaching their total strength. This graph shows a different perspective in terms of time whereas from Figure 2.8 the pure cement control looks ineffective in comparison (strength wise), but in Figure 2.9 the control reaches its full strength before any of the other mixtures, even the rice husk ash. It is a clear indicator of which materials to use for early strength or later strength. The effect of pure cement fineness (such as Type I, II or III) cements are not addressed.

These same trends are applicable to soil mixing applications using cementitious binders.

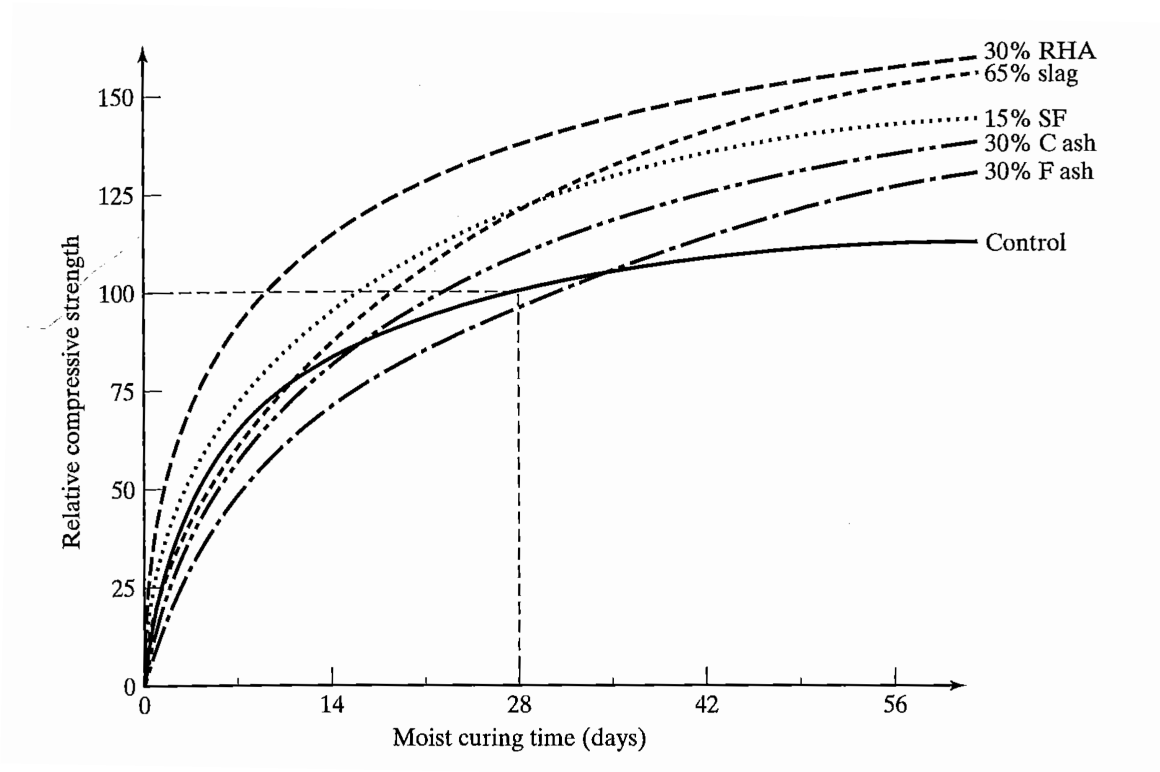

Figure 2.8. Effect of binder type and time on compressive strength (Mindess et al., 2003).

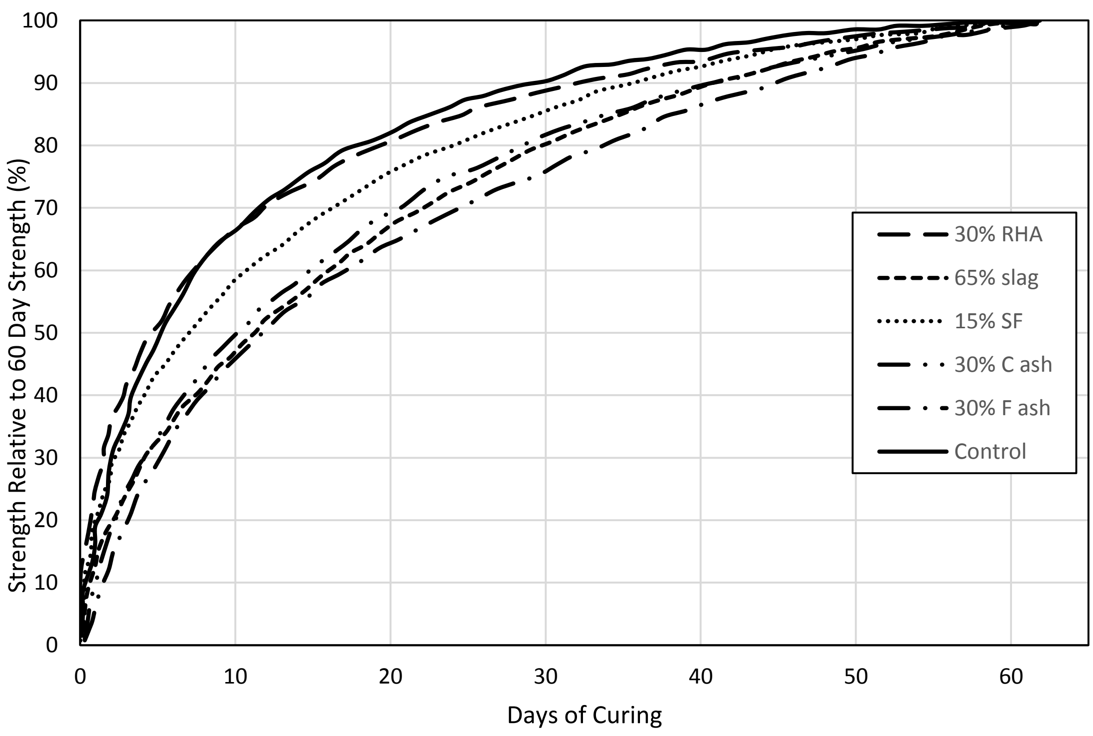

Figure 2.9. Relative effect of binder type and time on percent ultimate strength (adapted from Figure 2.8).

2.2.4.5 Strength of Treatment Area

The strength of mass stabilized soils which is intended by design to be uniform can be verified by sample checks throughout the treated site. For this, specialized penetrometers are used that are calibrated with plate load tests or lab results.

Heterogeneous mixing methods that produce columns of higher strength material (e.g. stone columns, DR, WSM or DSM) use a proportional strength approach dependent on the replacement ratio to predict the combined strength of the treatment area as follows:

$$q_{comb} = a_s(q_{col} - q_{sur}) + q_{sur} \tag{7}$$

q_{comb} = design strength of the treatment area
a_s = replacement area ratio
q_{col} = strength of the column
q_{sur} = strength of the surrounding untreated soil

The replacement area ratio, $\mathbf{a_s}$, is determined by the ratio of the column area to unit cell area.

$$a_s = \frac{a_{col}}{a_{unitcell}} \tag{8}$$

The unit cell area is dependent on the treatment pattern and spacing as shown in Figure 2.10.

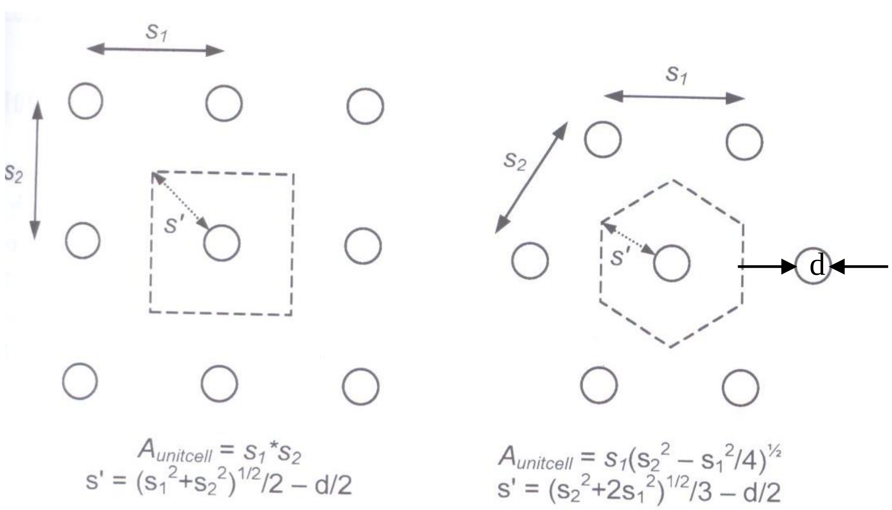

Figure 2.10. Determination of unit cell and farthest distance to untreated soil (adapted from Filz 2012)

In order to achieve a combined strength of 1,000 psf by using a sand column strength of 5,000 psf (based on Mullins 1996) with a surrounding soil strength of 100 psf, the required replacement ratio needed can be computed by simply solving equation (7) for $\boldsymbol{a_s}$, as:

$$a_s = \frac{1{,}000psf - 100psf}{5000psf - 100psf} = 0.18\ (or\ 18\%\ replacement\ required).$$

The spacing to diameter ratio, ***n***, (Eqn. 4b) of the treatment area would then be 2.0 or 2.2 for square or triangular pattern, respectively, which in turn is used to adjust the pattern spacing based on the planned column dimension. The same parameters can be used to predict the consolidation rate when the columns are free draining and act as wick drains (not for cement based soil mixing).

2.2.4.6 Transfer Platform

The transfer platform is the layer of competent fill placed above the columns to both form the required embankment and transfer the roadway loads to the supporting columns. This layer can be combined with a geo-fabric to reduce the required height. A minimum height of material denoted by $\mathbf{H_{crit}}$ is needed to assure no differential displacement at the top of the layer if no geo-fabric is used. The latest method of determining this depth uses the following equation (Filz et al 2012):

$$H_{crit} = 1.15s' + 1.44d_{col} \tag{9}$$

For values of ***n*** less than 3, translates to $\mathbf{H_{crit}}$ values approximately equal to the center to center column spacing, S.

2.3 Case Studies Related to the Remediation of Florida Organic Soils

Throughout the state of Florida there have been several cases of successful modification of organic soils as well as cases of continuous maintenance caused by underlying organics. Several have been selected to highlight the types of scenarios that will be investigated by this study.

2.3.1 US-1 Jewfish Creek Mass Stabilization

Use of dry soil mixing in the form of mass stabilization was successfully used in 2006 to stabilize a roadway over organic soils along US-1, which is the main route to the Florida Keys. This project widened the roadway by 40ft along the 18 mile stretch which contained 10 to 15 ft of organic silts with organic contents and moisture contents ranging from 40 to 60% and 85 to 650%, respectively (Garbin and Mann, 2012).

Prior to construction, bulk samples from 10 different locations were used in an extensive laboratory testing test program with trial binder mix ratios to establish the mix design which resulted in 200 to 300 pcy (75% slag/25% cement). A specialized penetrometer, known as a Kalkpelarsonden, *from the Swedish kolumn – penetration – sonde* or KPS (Fransson, 2011), was used which provided a wider surface area (26 x ¾ in) for nonhomogeneous materials since the standard CPT was prone to hitting isolated hard or soft spots that were not representative of the entire mix (Figure 2.11). Although other test methods were also used, KPS testing was the primary source of day to day quality assurance (one test every 2,500 ft^2). In its entirety, the soil mixing program took 13 months to treat 360,000 cubic yards of organic soil.

Figure 2.11. KPS penetrometer with load distribution wings (left); and KPS thrusting unit (right).

Post treatment testing included long term monitoring of a surcharged section of treated soil. Settlement plates were installed just above the treated soil and the surcharge was increased over a two week period to a height of 25 ft. Settlement of the mixed soil slowed to a stop about two weeks after the full load had been applied. Figure 2.12 and Figure 2.13 show the treatment process and surcharge test results, respectively.

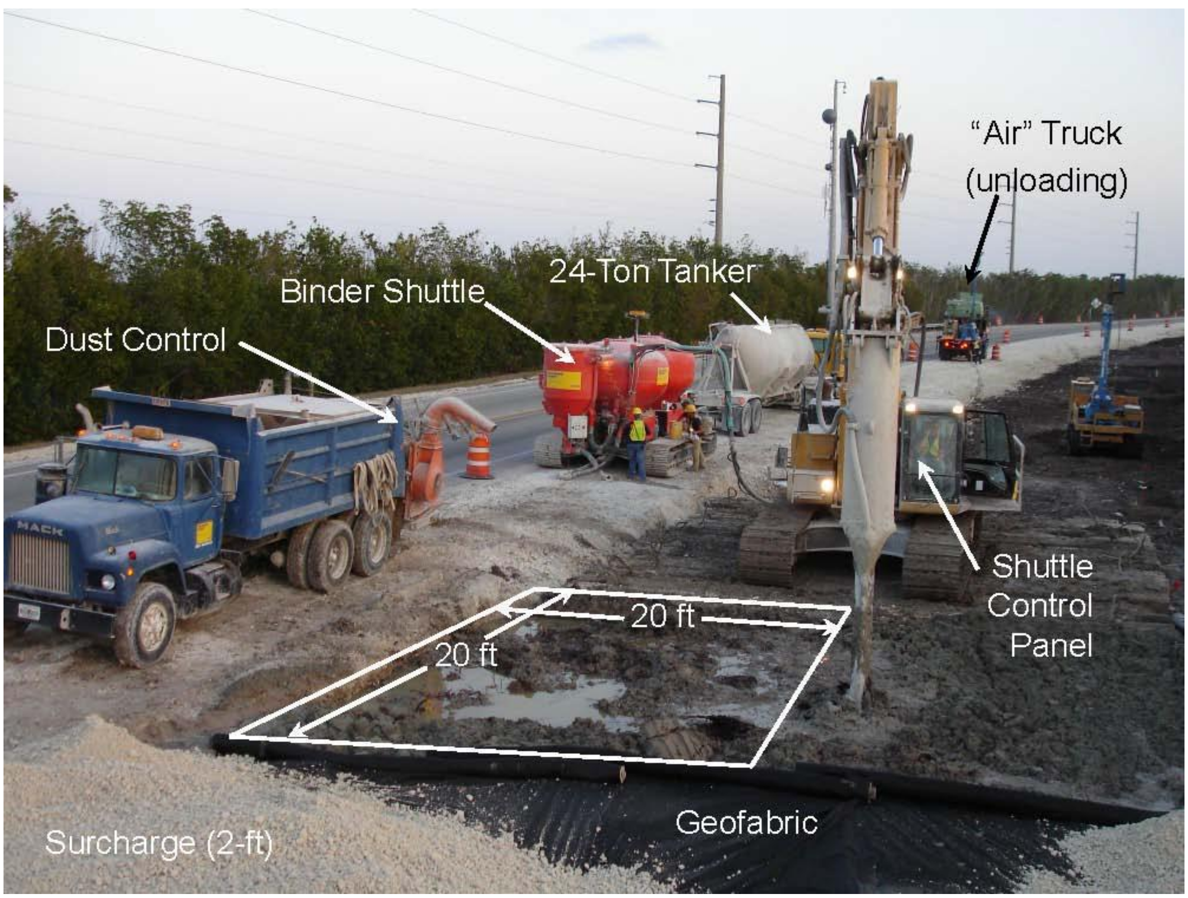

Figure 2.12. Mass stabilization equipment used along US-1 (courtesy of Hayward Baker).

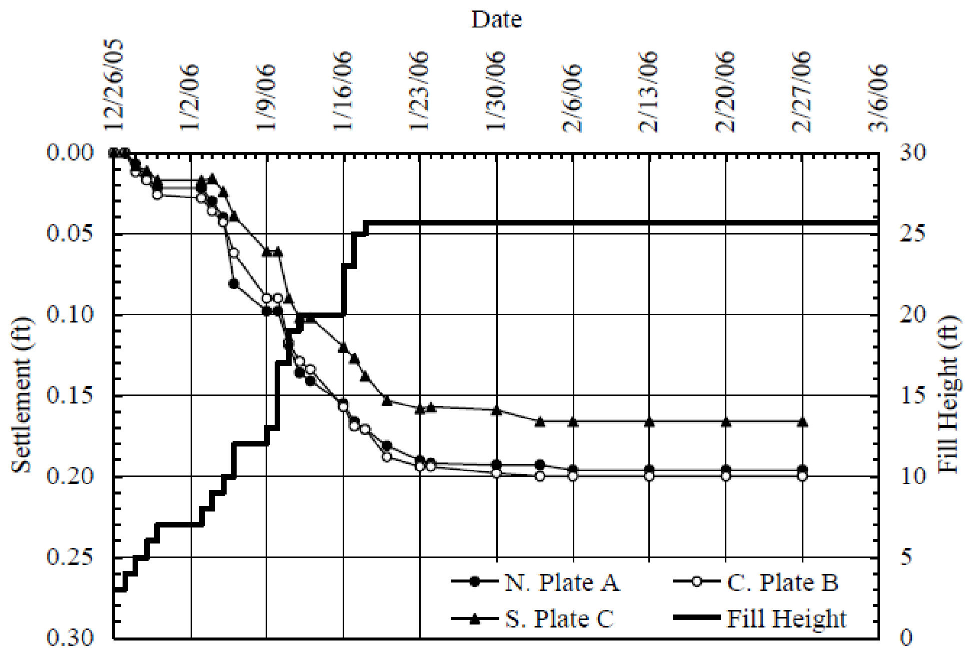

Figure 2.13. Long-term monitoring of mass-stabilized soil along US-1 (courtesy of Hayward Baker).

2.3.2 Dry Soil Mixing at Marco Island Airport

Based on the success of DSM at the Jewfish Creek project, DSM was also chosen for the Marco Island Taxiway Project. This program stabilized the soil beneath the new taxiway and apron area at the existing Marco Island Airport.

The subsurface profile consisted of 2 to 3 ft of loose sandy fill underlain by layers of highly organic peat extending to depths of 18 ft beneath the peat was a loose sandy soil with traces of silt. Due to the proximity to the coast, groundwater depths from 0.5 to 4 ft were recorded. Laboratory results of the site soil indicated moisture contents ranging from 145 to 425% with organic contents ranging from 17.5 to 58%.

To achieve the required level of stabilization, and eliminate any secondary consolidation of the organic layer, the 28 day design shear strength of 15 psi (2,160 psf) for the soil/cement mixture was established for the treatment areas. Pre-construction laboratory testing was performed in an effort to establish initial binder mix proportions of both cement and slag. Results from 14 day strength testing yielded a preliminary binder dosage of 125 pcy in areas where the peat layer was minimal (approximately 1 to 3 ft) and 275 pcy in areas where the peat layer extended to depths of 18 ft (lower average pH).

The chosen treatment pattern consisted of side-by-side 5 X 20 ft rectangular areas that slightly overlapped. During the initial construction, daily KPS strength testing was performed to closely monitor the shear strength increase of the soil mixed areas. KPS test results yielded shear strength values of 15 psi or greater after two to three days in areas where the peat layer reached depths of 3 to 6 ft.

Given the greater than expected strengths, cement was reduced to 125 pcy in all areas. However, it was later found that the KPS rod was bent enough to register deceivingly high resistance readings (larger blade surface area). Subsequent testing showed that the soil mixture did not meet design standards, and that section of the taxiway was re-treated. Additional complications at this site included high water table from rain events, which ultimately resulted in increased cement content to 400 pcy to achieve the desired strengths.

The above study illustrated that means of assuring proper soil mixing performance are a vital component of any program which will be closely scrutinized during this study. The long-term performance of this stretch of road is discussed in Chapter 5.

2.3.3 Roadway Subsidence State Road 33, Polk City

State Road 33 on the northern outskirts of Polk City runs just across the southeast edge of the Green Swamp in Polk County. For over 70 years, a 1,000-ft section of the road has experienced incessant settlement and constant attention from the district maintenance office. Figure 2.14, shows an aerial as well as the north-looking and south-looking views of this section of road.

In Figure 2.14 the newly repaved section of road can be identified clearly along with distress and subsidence in the encircled views. A boring conducted in 2006 within the northbound lane at the lowest point revealed 43 in of asphalt used in correcting the surface subsidence, underlain by 5 to 6 ft of sand and 72 ft of organic material. The boring was terminated without finding the bottom of the organic layer due to MOT concerns, but conclusively identified the cause.

(a)

(b)

(c)

Figure 2.14. (a) Aerial view of a 1,000-ft section of SR 33-north of Polk City, FL, that has been continuously repaired to combat subsidence. Encircled region represents worst area. (b) Visible distress along SR-33 approaching subsidence zone from the south. (c) Visible subsidence along SR-33 approaching from the north.

As with many of the existing roads in this condition, there is no one stabilization method that will address all scenarios. In this case continued maintenance exacerbates the problem because of the regularly added overburden to assure serviceability and safety.

2.3.4 Dynamic Replacement Interstate I-4 Ramp L

In a previous study conducted by the PIs (Gunaratne et al, 1997 and Mullins, 1996), a large portion of the I-4 exit interchange in Plant City was identified to have a surficial organic deposit (10-15 ft) and was used to demonstrate the effectiveness of dynamic replacement of Florida organic soils (Figure 2.15).

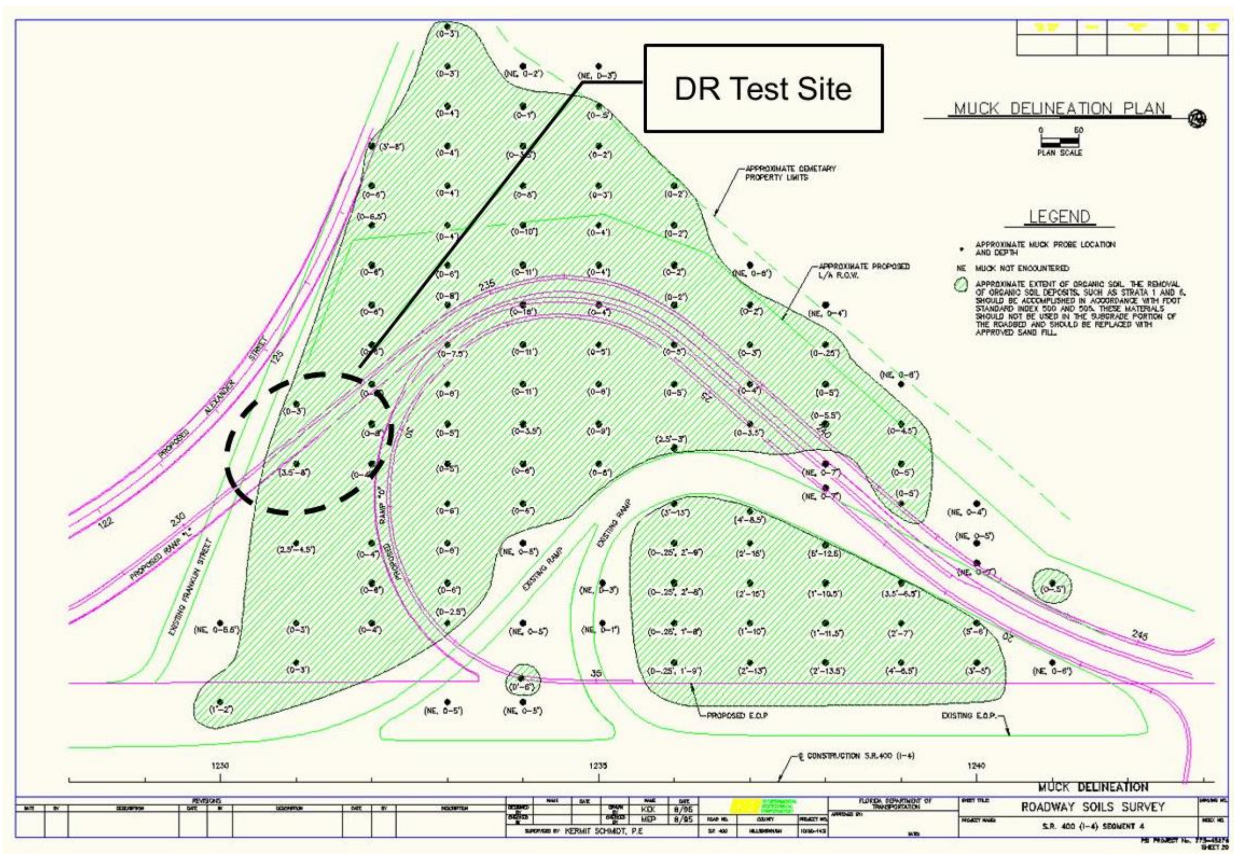

Figure 2.15. Muck delineation survey of I-4 Ramp L construction area (Mullins, 1996).

The DR approach discussed earlier was followed where a blanket of clean sand was placed over a 10 ft thick organic deposit and a specially designed pounder (Figure 2.5) was dropped to systematically create a pattern of sand columns through the organic layer (Figure 2.16). Post treatment testing verified the depth of sand columns using CPT soundings and improved settlement response after surcharging. Figure 2.16 shows the pattern created by the DRM craters at predetermined impact locations. Figure 2.17 shows a cross section of the columns as well as the final soil types and locations as determined by CPT soundings performed on a 1 ft spacing radially outward from the center of the pattern.

The effectiveness of the DR method was evaluated on the basis of surcharge induced settlement measurements in two 50 ft diameter surcharged areas: one DR modified site and one control (untreated) site. In the treated area an additional 5 ft layer of fill was placed over the previously placed 5 ft blanket. The control site was surcharged in three stages consisting of two layers of 2.5 ft, and one layer of 5 ft. Figure 2.18 shows the plan and profile view of the surcharge program as well as the settlement results. Strain was computed based on the original 10 ft layer thickness.

Figure 2.16. DR pattern used on I-4 Ramp L project in Plant City, FL (Mullins, 1996).

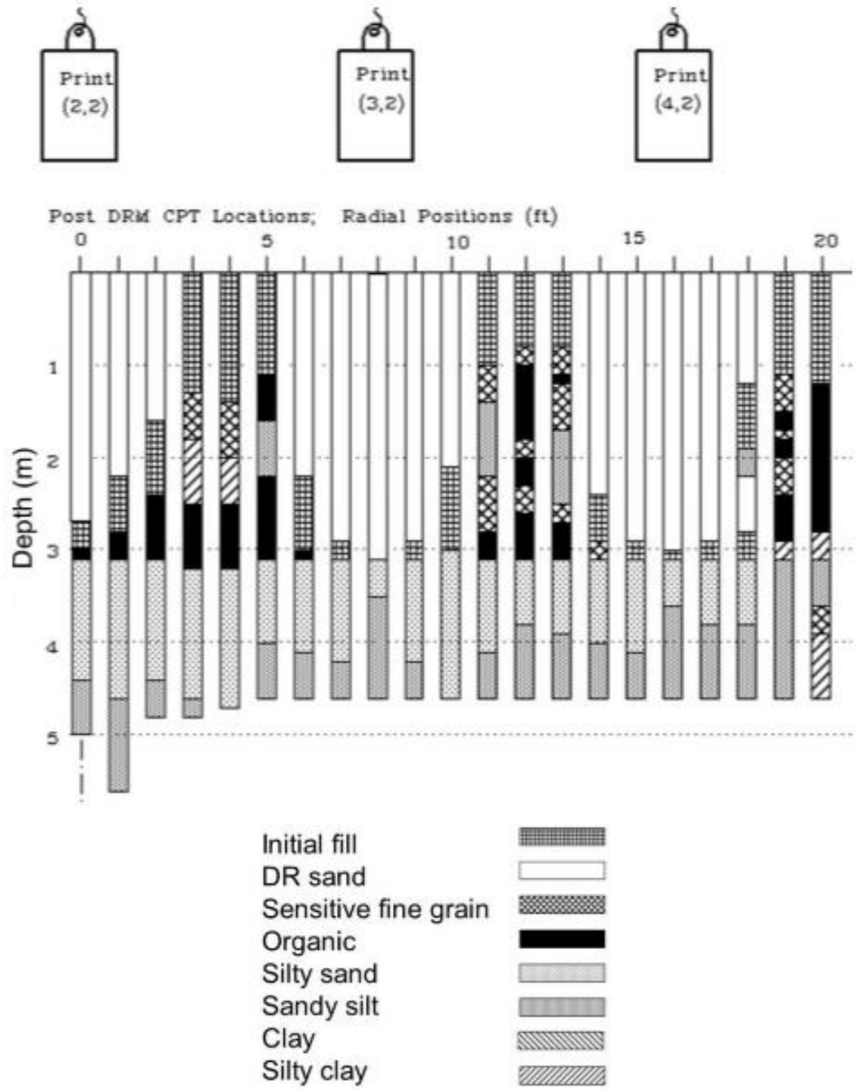

Figure 2.17. CPT results showing sand columns and mixed zone between columns.

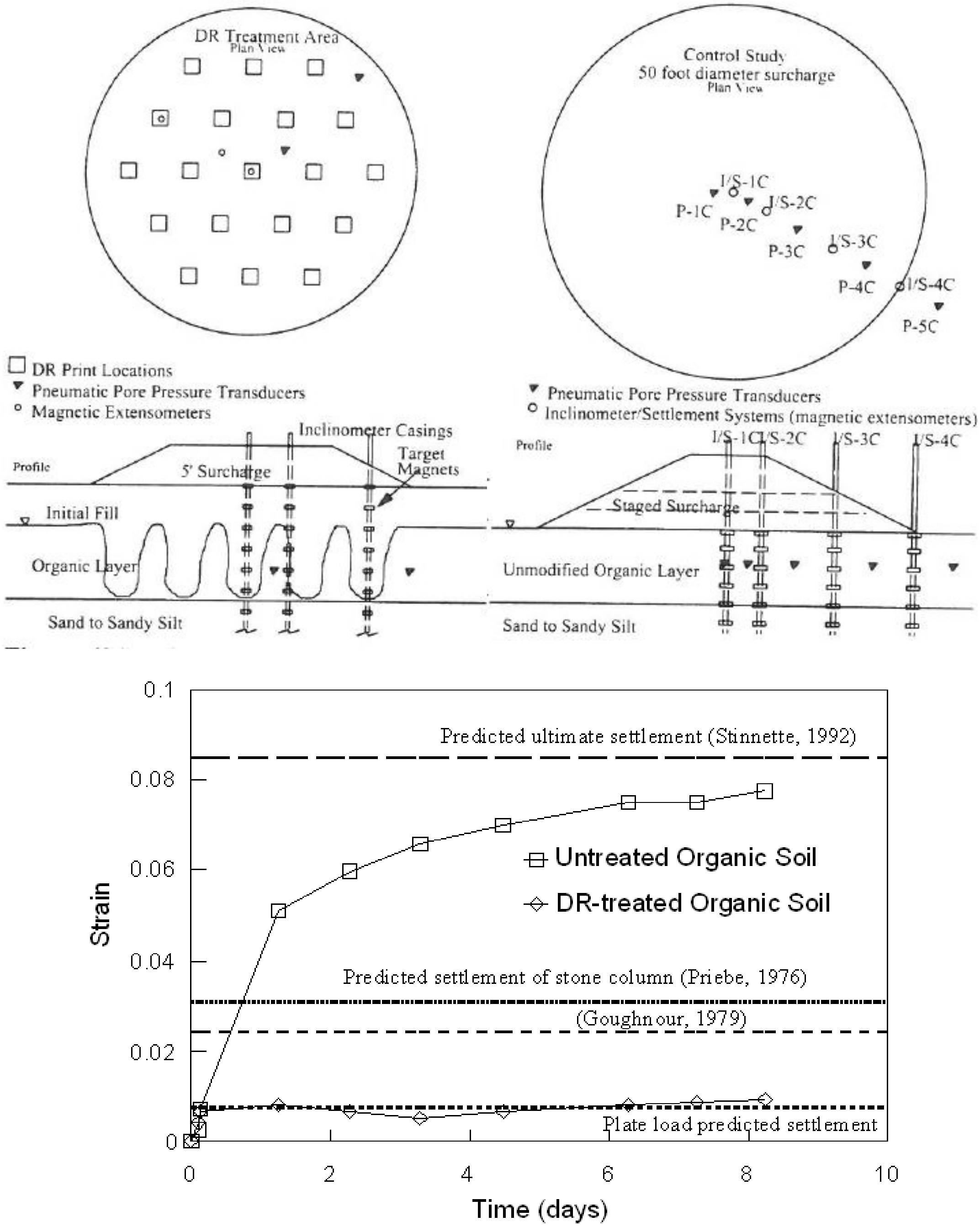

Figure 2.18. Results of surcharge-induced settlement of DR treated and untreated organic deposit.

This study demonstrated that DR is an effective technique to improve the settlement and strength properties of organic soil deposits, but is limited to shallow deposits.

2.4 Laboratory Soil Mixing Case Studies

Literature cited laboratory studies have been provided that also form the basis of the latest FHWA Manual for soil mixing applications. Different types of soils are presented in this compilation as well as the utilization of different mixing methods (wet or dry), mixing procedures, tamping style, and curing conditions.

2.4.1 "Factors affecting strength gain in lime-cement columns and development of a laboratory testing procedure." By: J.R. Jacobson, G.M. Filz, and J.K. Mitchell (2003)

This project was initiated to test lime-cement columns with the soil from the I-95/Route 1 interchange site and two other soils from State Route 33 in West Point, Virginia.

I-95/Route 1 Interchange

The soil from I-95/Route 1 interchange had a range of organic content varying from 1.8% to 46.4% with an average of 10.5%. By USGS classification the soil is organic silt (OH). The average moisture content was 65%, the organic content showed to be less than the average for samples with an average of about 6%, and the average pH was 6.6. The average liquid limit of the samples was 67. Table 2.1 below shows results of this soil when mixed with 100% cement.

Table 2.1. Results from I-95/Route 1 unconfined compressive tests.

Batch No.	Initial Moisture%	W/C	28-day strength kPa (psi)	% Organics	USGS Classification	From
26	67	2.51	965 (140.0)	6%	OH	I-95/ Route 1
22	75	3.33	938 (136.0)	6%	OH	I-95/ Route 1
16	67	4.26	414 (60.0)	6%	OH	I-95/ Route 1

State Road 33 in West Point, VA

The soil from State Road 33 in West Point, VA consists mostly of marsh deposits of soft, organic clays with moisture contents varying from 15% to 200%, as well as organic contents of 0% to 40%. Overlaying the soft clay is a variable amount of fill material, below is 3 to 6 m of loose to firm sand, and below that is moderately stiff silty clay.

State Route 33 in West Point, VA (Zone 1)

Zone 1 was taken at a depth of 4.5 to 7.5 m and contained fairly uniform material. By USGS classification the soil was determined to be organic silt (OH). Its average moisture content was 92%, average organic content was 7%, average pH was 4.8, and its average plasticity index was 57. Table 2.2 below shows results of this soil when mixed with 100% cement.

Table 2.2. Results from Zone 2 unconfined compression tests.

Batch No.	Initial Moisture %	W/C	28-day strength kPa (psi)	% Organics	USGS Classification	From
1	91	7.06	450 (65.3)	7%	OH	SR-33
5	95	4.77	625 (90.6)	7%	OH	SR-33
9	86	3.47	790 (114.6)	7%	OH	SR-33

State Route 33 in West Point, VA (Zone 2)

Zone 2 was taken at a depth of 11.0 to 14.5 m and contained material that had greater variance. By USGS classification the soil was determined to be an organic silt (OH). Its average moisture content was 120%, average organic content was 15%, average pH was 3.7, and its average plasticity index was 80. Table 2.3 below shows results of this soil when mixed with 100% cement.

Table 2.3. Results from Zone 2 unconfined compression tests.

Batch No.	Initial Moisture %	W/C	28 day strength kPa (psi)	% Organics	USGS Classification	From
17	150	7.99	250 (36.3)	15%	OH	SR-33
21	150	5.32	450 (65.3)	15%	OH	SR-33
25	138	3.92	640 (92.8)	15%	OH	SR-33

These batches were mixed using the dry mixing method. A 4-liter capacity KitchenAid™ stand mixer, using the dough hook attachment, was used to do this. This capacity yielded eight treated samples. During production, the soil was first homogenized then the weight of the batch was taken along with two moisture samples. Using a microwave they were able to accelerate the time needed for the moisture samples to dry. Once the moisture content was recorded the amount of lime, cement, and water required was calculated. If water was to be added, it was added to the mix first, followed by the lime and cement which was then sprinkled on top of the soil over the first minute of mixing. The lowest speed on the mixer was used and the batch was mixed for five minutes. Over the five minutes, in three equal intervals, the mixer was stopped and the soil was scraped from the sides and bottom of the bowl using a spatula. Specimens were made using plastic molds 50mm (1.97in.) diameter by 100mm (3.94in.) tall.

The main findings from this case study were:

1. If the soil is allowed to dry out and an attempt is made to reinstate the previous soil moisture, the strength of the mixture will decrease.
2. If employing the addition of lime, mixture strength can increase or decrease depending on soil type.
3. As the soil water to cement ratio increases, strength of the mixture decreases (for 100% cement soil mixtures).

2.4.2 "Engineering behavior of cement stabilized clay at high water content." By: N. Miura, S. Horpibulsuk, and T.S. Nagaraj (2003)

This study was done to analyze the results of cement treated soft marine deposits. The soil (marine deposits) came from a seabed in a coastal region near Tai Kowk Tsui Harbour in Hong Kong. For uniformity of the sample, the marine deposits were sieved through a 150 μm size sieve after being diluted with water. Available soil properties are presented in Table 2.4. Typically, marine deposits from this area are clayey silt or silty clay with undrained shear strength of below 30 kPa (4.4 psi). (Yin & Lai, 1998)

Table 2.4. Soil Characteristics of marine deposits used (Yin and Lai, 1998).

Liquid Limit (LL) (%)	62
Plastic Limit (PL) (%)	30
Plasticity Index (PI) (%)	32
Water Content, w (%)	60, 80
Initial Void Ratio, e	1.6, 2.1
Specific Gravity, G_s	2.67
pH	8
Grain Size Distribution	
Clay (%)	28
Silt (%)	46
Fine Sand (%)	26

The water content of the samples before mixing was controlled at 60% and 80%. Mixing was done utilizing the dry mixing method by adding dry Portland cement powder to the sieved and preconsolidated soil. This mixture was formed using a laboratory size conventional concrete mixer. Samples were placed into cylindrical pipe molds, vibrated on a laboratory size vibration table for void reduction, and lastly a palette knife was used to trim, compress, and expel air bubbles when necessary. The pipes were placed on a smooth glass plate and covered with a piece of plastic membrane. After being air cured for 1 to 2 days samples were then placed in a water tank and cured for 28 days at a constant temperature of 25°C. (Yin & Lai, 1998)

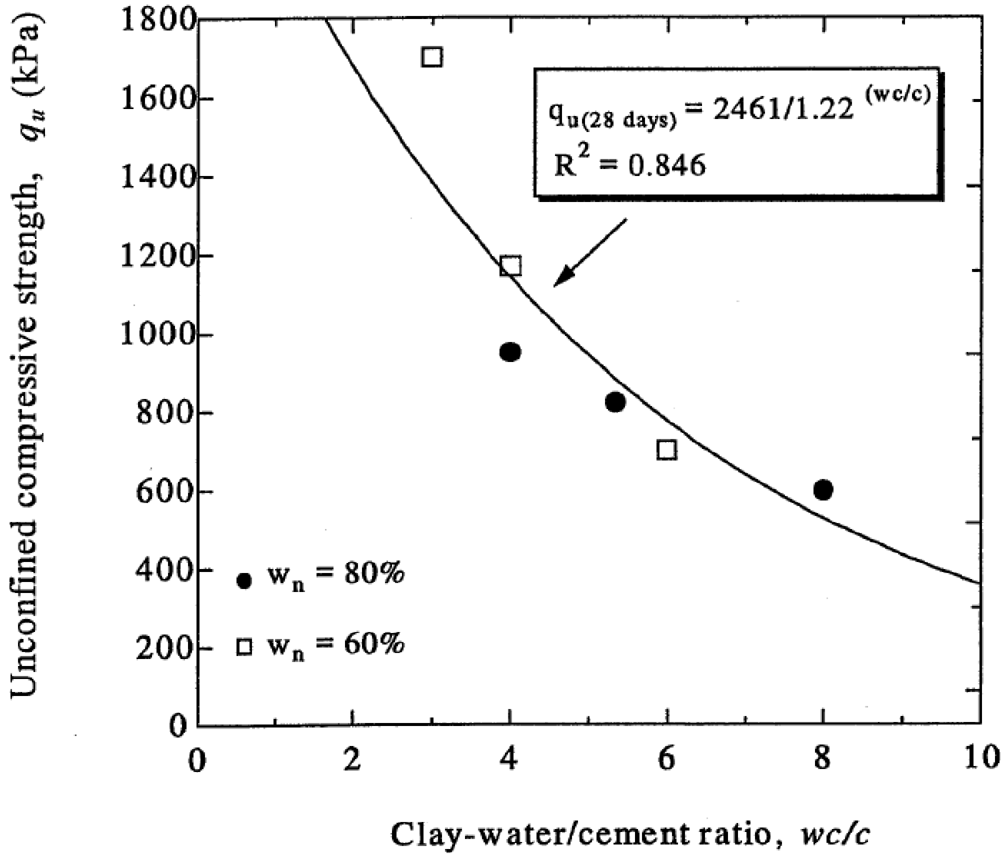

Figure 2.19. 28-day results from Yin and Lai's study (Miura et al., 2002).

Figure 2.19 is taken from *Engineering Behavior of Cement Stabilized Clay at High Water Content* and is based on the data from Yin and Lai in *Strength and Stiffness of Hong Kong Marine Deposits Mixed with Cement*. It shows the results from the study for a 28-day unconfined compression test for both the 60% and 80% water content.

2.4.3 "Assessment of strength development in cement-admixed high water content clays with Abrams' law as a basis." By: S. Horpibulsuk, N. Miura and T.S. Nagara (2003)

The basis for this case study was to investigate the engineering behavior of cement treated Bangkok clay, whose soil properties can be seen in Table 2.5.

Table 2.5. Characteristics of soft Bangkok clay (Uddin et al., 1997).

Characteristics Values of the Physical Properties of the Base Clay	
Properties	Characteristic Values
Liquid Limit, LL (%)	103
Plastic Limit, PL (%)	43
Plasticity Index, PI (%)	60
Water Content, w (%)	76-84
Liquidity Index, LI	0.62
Total Unit Weight (kN/m^3)	14.3
Dry Unit Weight (kN/m^3)	7.73
Initial Void Ratio, e	2.2
Color	Dark Gray
Activity	0.87
Sensitivity	7.3
Soil pH	6.1
Grain Size Distribution:	
Clay (%)	69
Silt (%)	28
Sand (%)	3

This project is an example of the wet mixing method as the samples were prepared by mixing the base clay with cement slurry. The mixing process was achieved by gloved hands until the mixture was homogenous. Regarding slurry preparation, it was produced using a 0.25 water and hardening agent ratio. Table 2.6 shows the properties of the Type I Portland cement used for the slurry. (Uddin, A.S., & D.T, 1997)

Table 2.6. Properties of Type I Portland Cement Used in the Study (Uddin, A.S., & D.T, 1997).

Properties of Type I Portland Cement Used in Study	
Chemical Composition	By Weight (%)
Silicon Dioxide (SiO_2)	21.63
Aluminum Oxide (Al_2O_3)	5.09
Ferric Oxide (Fe_2O_3)	2.92
Magnesium Oxide (MgO)	0.91
Sulfur Trioxide (SO_3)	1.68
Loss on Ignition	0.82
Insoluble Residue	0.11
Tricalcium Silicate ($3CaO \cdot SiO_2$)	58
Tricalcium Aluminate ($3CaO \cdot Al_2O_3$)	8.6
Physical Properties	Rate
Fineness, Specific Surface (Blaine)	3,000 cm^2

After mixing, the product was put into steel molds with dimensions of a 75 mm (2.95 in.) diameter and 90 mm (3.54 in.) height. Samples were compacted using 30 blows per layer for five equal layers. The compaction process was accomplished using a one-inch diameter steel rod which fell from a height of 200 mm (7.87 in.). Curing of the samples was then done in a humid room. (Uddin, A.S., & D.T, 1997)

The graph shown below is featured in *Assessment of Strength Development in Cement-Admixed High Water Content Clays with Abrams' Law as a Basis* and shows some results seen from the unconfined compression tests done. Its data is based on K. Uddin's Thesis *Strength and Deformation Behavior of Cement-Treated Bangkok Clay*.

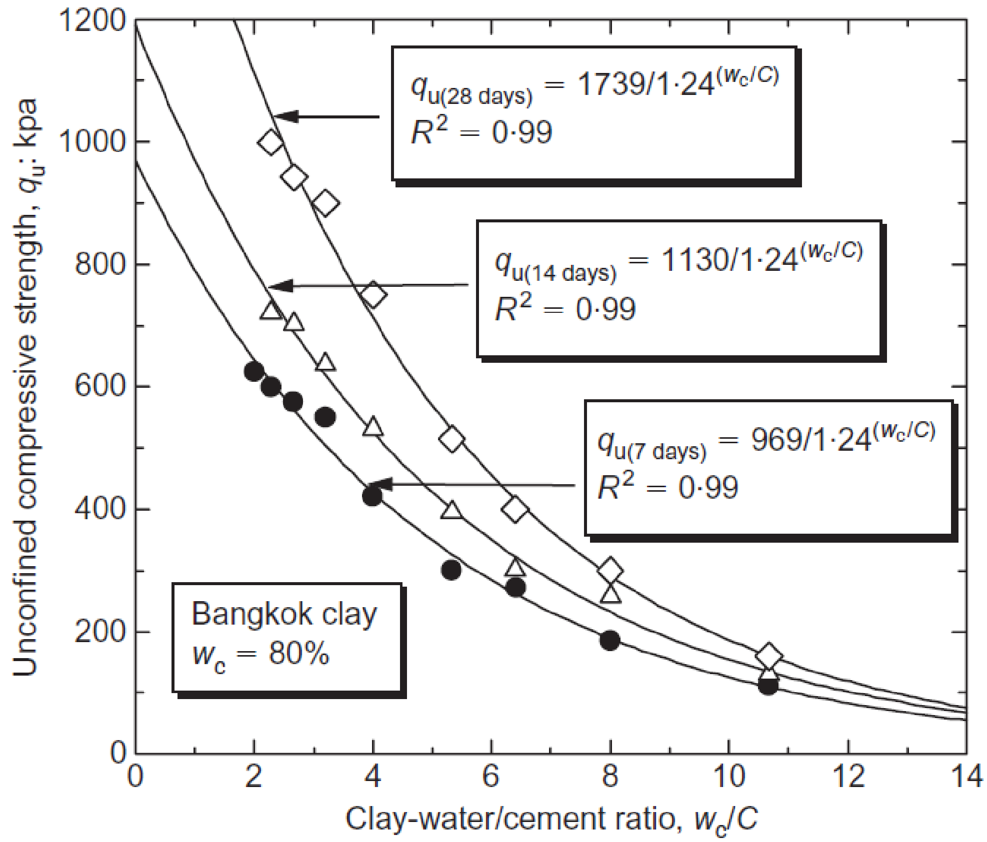

Figure 2.20. Unconfined compressive strength results of the study (Horpibulsuk et al., 2003) (690 kpa = 100 psi).

2.4.4 "Fundamental characteristics of cement-admixed clay in deep mixing." By: G.A. Lorenzo, and D.T. Bergado (2006)

The purpose of this study was to test compressibility and strength properties for cement-admixed clay with a high water content in the application of deep mixing. The soil tested in this study is typical soft Bangkok clay. The sample was taken from a depth of 4-5 m (13.1-16.4 ft.) at the Asian Institute of Technology (AIT) campus in Klong Luang, Pathumthani, Thailand and contains the properties as shown in Table 2.7.

Table 2.7. Soil Characteristics of typical soft Bangkok clay used in study.

Liquid Limit (LL) (%)	103
Plastic Limit (PL) (%)	43
Plasticity Index (PI) (%)	60
Water Content, w (%)	76 - 84
Liquidity Index (LI)	0.62
Total Unit Weight, (kN/m^3)	14.3
Dry Unit Weight, (kN/m^3)	7.73
Initial Void Ratio, e	2.31
Specific Gravity, G_s	2.68
Color	Dark Gray
Activity	0.87
Sensitivity	7.4
Grain Size Distribution	
Clay (%)	69
Silt (%)	28
Sand (%)	3

Applying the wet mixing method, samples were mixed with a cement slurry at a water-cement ratio of 0.6, using Type I Portland cement. Samples were mixed using a portable mechanical mixer until a homogenous paste was reached. Molds used were PVC and had a diameter of 50 mm with a height of 100 mm. The temperature and humidity of the curing room were 25°C and 97%, respectively. Figure 2.21 shows the results of test program where both the effects of w/c ratio and time on the unconfined compression strength follow expected trends (690kPa = 100psi).

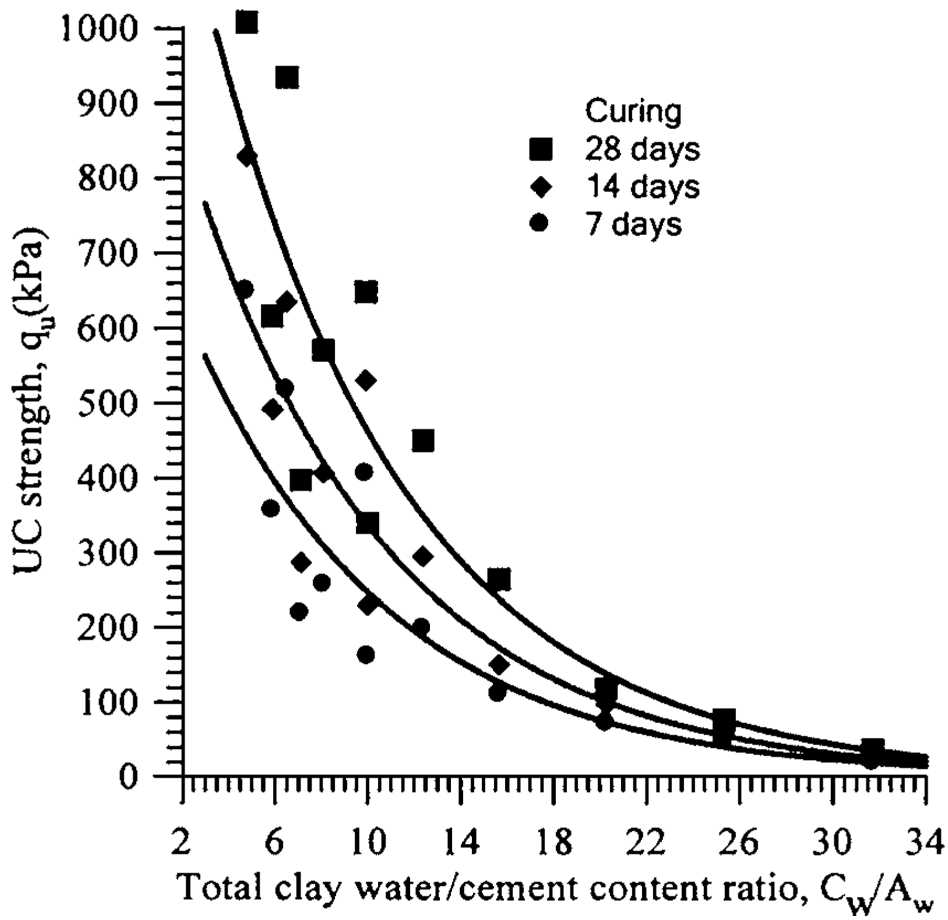

Figure 2.21. Unconfined compressive strength versus total clay water-to-cement ratio.

2.4.5 "Laboratory mixing, curing, and strength testing of soil-cement specimens applicable to the wet method of deep mixing." By: D.K. Hodges, G.M. Filz, and D.E. Weatherby (2008)

Completed in 2008, this study analyzes a laboratory method for testing deep soil mixtures similar to field practices. The main variables taken into consideration were "the characteristics of the binding agent, the nature of the untreated soil, the mixing procedure, and the curing conditions" (Hodges, Filz, & Weatherby, 2008). Five different soils were tested, all falling into the category of a relatively easy to mix soil, seeing as the research was limited this way.

Table 2.8. Summary of Soil Properties.

	USCS Classification	AASHTO Classification	G_s	Atterberg Limits			% Fines	Saturation Moisture Content (%)
				LL	PL	PI		
Light Castle Sand	SP	A-3	2.66	NP	NP	---	<1.0	23.0
Northern Virginia Sandy Clay	CL	A-6	2.80	32	22	10	66	18.4
P2 Silty Sand	SM/SP to SC/SP	A-2 to A-6	2.78	29 to 38	23 to 34	4 to 6	7	35.9
Vicksburg Silt	ML	A-6	2.71	27.4	22.1	5.3	100	26.3
Washed Yatesville Silty Sand	SP	A-1-b	2.67	NP	NP	---	<1.0	20.3

Each of the five soil types presented in Table 2.8 were passed through a No. 4 sieve before testing. Moisture contents of the soils were taken to be in saturated condition, as if the soil were acquired from beneath the ground water table. For testing, the soils were first oven dried, then the amount of water needed to attain saturation was calculated and added in.

For this study, two main factors were used to control mix designs; an "in-place cement factor ($\alpha_{in\ place}$) and water-to-cement ratio of the binder slurry (w/c)" (Hodges, Filz, & Weatherby, 2008). For the binder slurry, a range of water-to-cement ratios from 0.6 to 1.5 was chosen. Soils containing little or no fines (Light Castle Sand, Yatesville Silty Sand) used the lower end of the range (0.6, 0.8, 1.0), while the other soils with higher fine contents used almost the full range (0.75, 1.0, 1.25, 1.5). In place cement factors included 150, 250, and 350 kg/m^3 (252.8, 421.4, 590 pcy, respectively).

For mixing the binder slurry, a "450-watt Oster® 12-speed blender with a 5-cup capacity glass jar" was used (Hodges, Filz, & Weatherby, 2008). Two other methods, a kitchen stand mixer and hand mixing were attempted, however, they were found to be unsuccessful. The measured amount of cement was placed in the blender, then the necessary amount of slurry water was slowly added. After this the blend was pulsed for roughly 15 seconds, allowing the water to infiltrate the bottom of the jar. Once this was accomplished, the Oster® was set to a medium speed and run for about 3 minutes.

The actual soil mixing was done in a "250-watt Kitchen Aid^{TM} stand mixer with a 4-liter-capacity mixing bowl" (Hodges, Filz, & Weatherby, 2008). Multiple attachments were used for the mixing. The dough hook performed the best when dealing with cohesive soils and higher fine contents (meaning a thicker consistency), and the flat beater best mixed the soils with a lower fine content. Homogenization of the soil was done first by mixing it alone for 3 minutes. The binder slurry was then transferred into the soil with continuous mixing. After all the binder slurry was added, the combination was mixed for 10 minutes.

When the 10 minutes of mixing time was completed, the bowl was removed from the mixer and stirred by hand using a small ladle. Upon nearly every third pass, a ladle-full of the mixture was placed into a mold. All "molds were filled one ladle-full at a time", and all molds also received "one-ladle full of mixture before the first mold received a second" (Hodges, Filz, & Weatherby, 2008). For the removal of air bubbles, "light tapping of the mold" was used if the mixture was fluid-like or the mixture was rodded if it was on the stiffer side (Hodges, Filz, & Weatherby, 2008). Once filled, the overflow on the tops of the samples was scraped off, the outsides of the molds were cleaned, and the samples were then capped. For curing, the tightly sealed samples were labeled and submerged into a water bath with constant room temperature.

As the time approached for the sample to be tested, an occurrence of bleed water was seen. This led to uneven and/or sloped ends at the tops of the specimens. Sanding the specimen was attempted, however unsuccessful, therefore a rock saw was utilized to remove both ends of the specimen (it also made extraction simpler as the samples could be removed from the bottom of the mold). Testing of samples was performed by unconfined compressive strength tests, with a "displacement rate of approximately one percent of initial specimen length per minute" (Hodges, Filz, & Weatherby, 2008). ASTM D2166 was used to make area corrections to adjust for sample strain, and ASTM C39-86 was applied for a correction factor when the sample had a length to diameter ratio under 1.8.

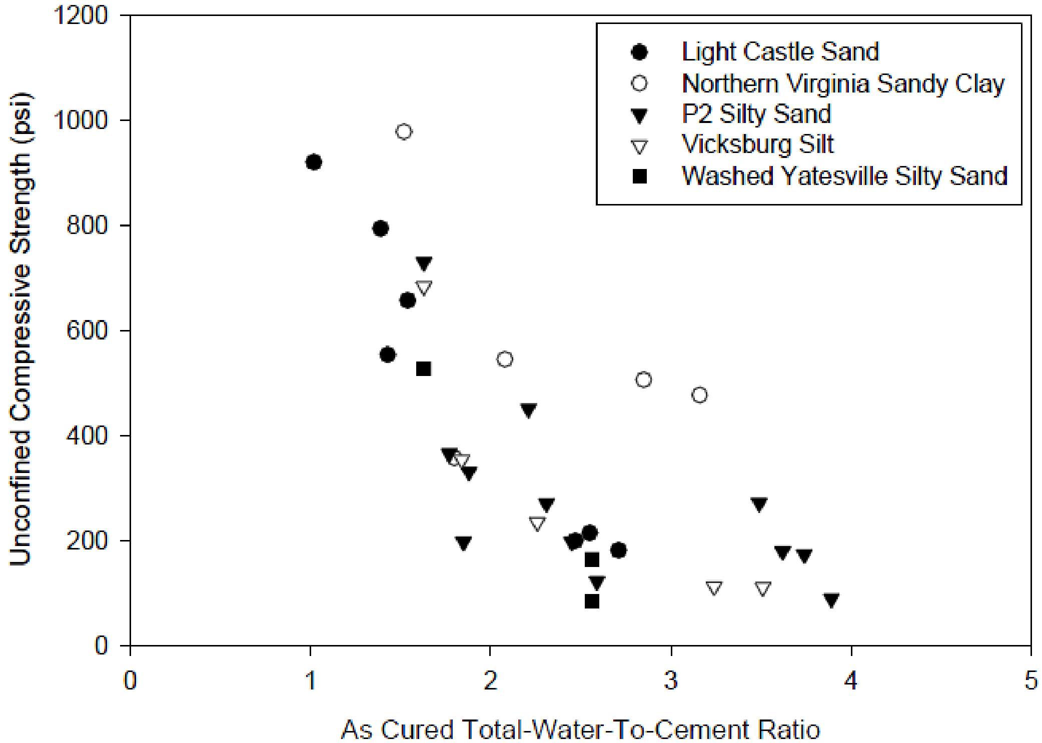

Figure 2.22. 28-day strength versus as-cured total water-to-cement ratio.

Figure 2.22 illustrates the results seen from this research. A general trend can be seen correlating a lower water to cement ratio with higher strength, and a higher water to cement ratio with lower strength.

2.4.6 Interpretation of Literature Findings

Figure 2.23 compiles results from all the above case studies. It strongly demonstrates the trend of decreasing strength with increasing water to cement ratio. Seen in Figure 2.23 there is a general trend of increasing strength with an increasing cement content (which can be defined as the weight of the cement divided by the weight of the soil), however it is very scattered and seems to depend highly on soil type. Figure 2.24 shows the same relationship as Figure 2.23, but with cement factor (weight of the cement divided by the volume of the mix). The trend of this graph better displays the increasing strength with an increasing amount of cement in the mixture.

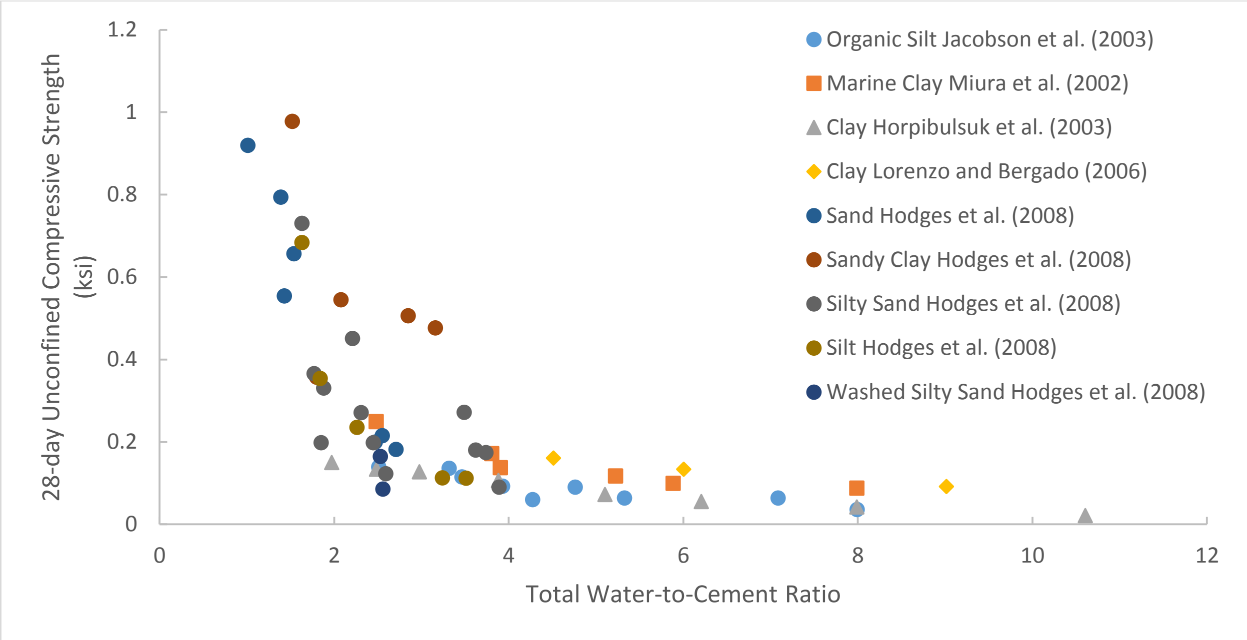

Figure 2.23. Overall results seen from case studies (Hodges et al., 2008) (Horpibulsuk et al., 2003) (Jacobson et al., 2003) (Lorenzo and Bergado, 2006) (Miura et al., 2002).

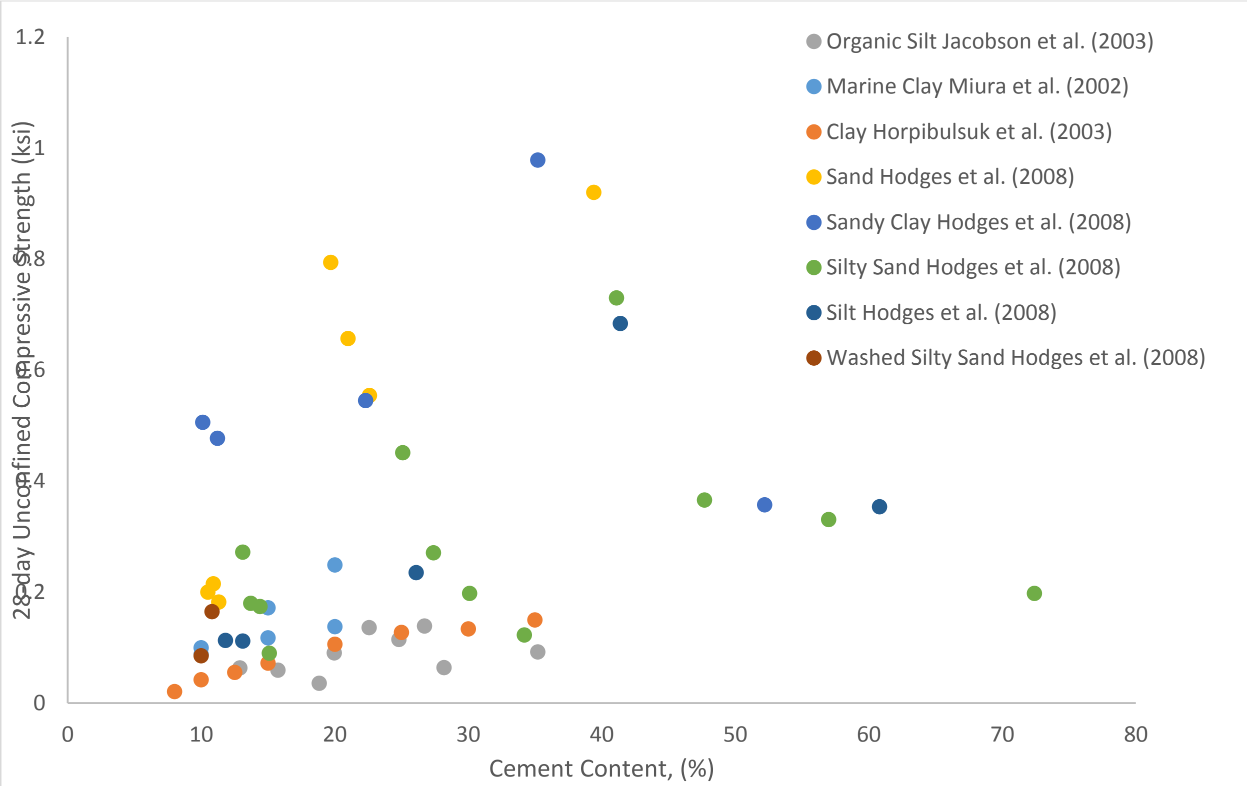

Figure 2.24. Overall results seen from case studies in terms of cement content, excluding Lorenzo and Bergado, 2006 (Hodges et al., 2008) (Horpibulsuk et al., 2003) (Jacobson et al., 2003) (Miura, et al., 2002).

Considering cement factor is typically used instead of cement content Figure 2.24 was converted to represent this by the following process:

$$CF = \frac{W_c}{V_{mix}} = \frac{W_c}{V_{cement} + V_{water} + V_{dry\ soil}} = \frac{W_c}{\frac{W_c}{G_{s,cement}\gamma_{water}} + \frac{(^{w}/_{c})W_c}{\gamma_{water}} + \frac{^{W_c}/_{A_w}}{G_{s,soil}\gamma_{water}}}$$

Where

$W_c = weight\ of\ cement$

$^{w}/_{c} = water\ to\ cement\ ratio\ of\ the\ mix$

$A_w = cement\ content, \%$

$CF = cement\ factor$

If the cited case study provided a cement factor, it was directly used else the cement factor was computed using the equation above. The 28 day strength vs cement factor is shown in Figure 2.25.

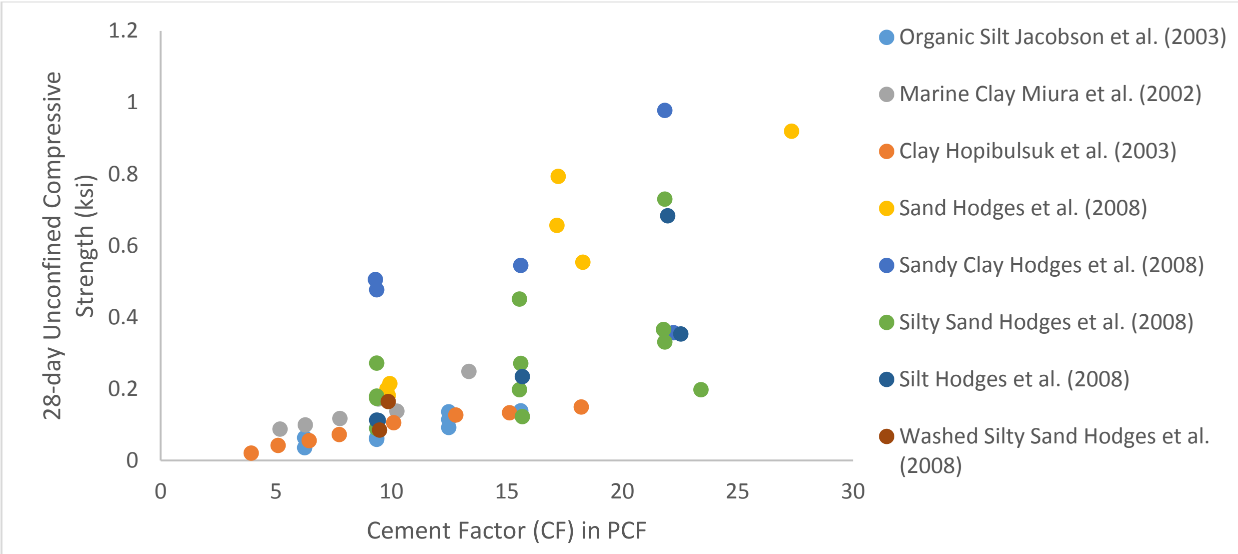

Figure 2.25. Overall results seen from case studies in terms of cement factor, excluding Lorenzo and Bergado (Hodges et al., 2008) (Horpibulsuk et al., 2003) (Jacobson et al., 2003) (Miura, et al., 2002).

2.4.6.1 Breakdown of Literature Findings by Water-to-Cement Ratio

Due to the case study results yielding wide-ranging results, Figures 2.23 and 2.25 are shown again (now as Figure 2.26 and 2.27 respectively). This time instead of distinguished by case study, they are shown in terms of water-to-cement ratio and cement factor (1-2.49, 2.5-3.99, 4-11). When presented in this manner, clear distinguished trends can be seen between unconfined compression strength and both water-to-cement ratio and cement factor. Figures 2.26 and 2.27 also shows a regression line plotted for all of the literature data, along with its accompanying equation. These power equations present the possibility of predicting an unconfined compressive strength from a known water-to-cement ratio or cement factor.

Figure 2.28 shows how the plot of water-to-cement ratio vs. cement factor for all the literature data compares to that of the theoretical plots for the 0-blow count saturated sands and 30-blow count saturated sands. The 0- and 30-blow count saturated sands respectively represent the loosest (void ratio of 1) and densest sands (void ratio of 0.6) that can be treated through soil mixing. When adjusting either of the theoretical plots to represent a void ratio of 1.6, the resulting plot falls directly on the regression line of the literature data. Since only clays can have void ratios greater than 1, it is suggested that the literature material contained substantial amounts of clays.

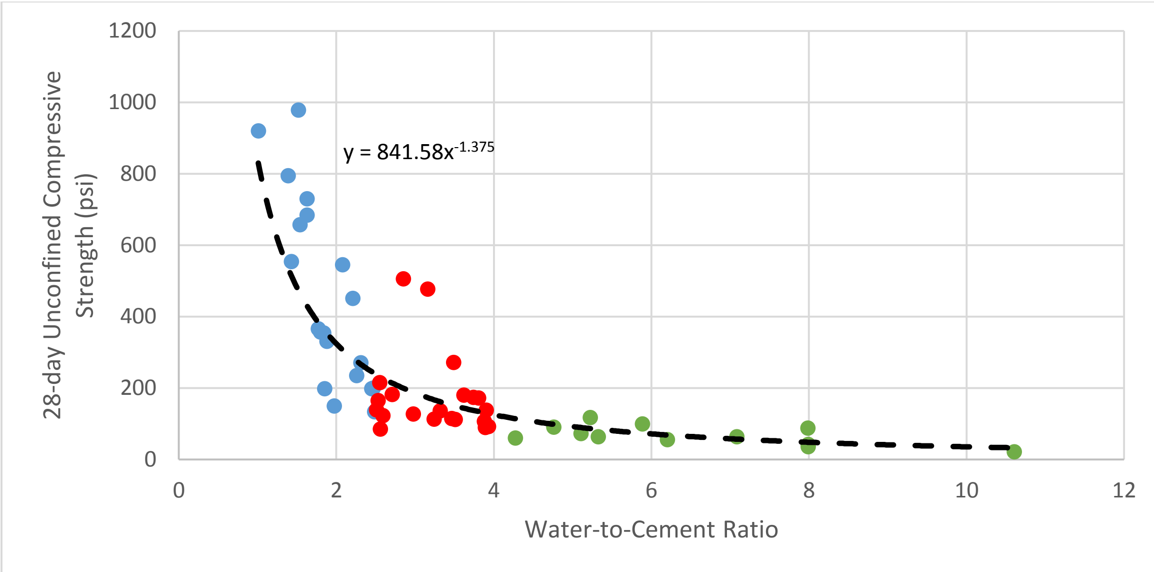

Figure 2.26. Overall results seen from case studies grouped by color into different water-to-cement ratio categories. Blue is 1-2.49; Red is 2.5-3.9; Green is 4-11. (Hodges et al., 2008) (Horpibulsuk et al., 2003) (Jacobson et al., 2003) (Miura, Horpibulsuk, & Nagaraj, 2002).

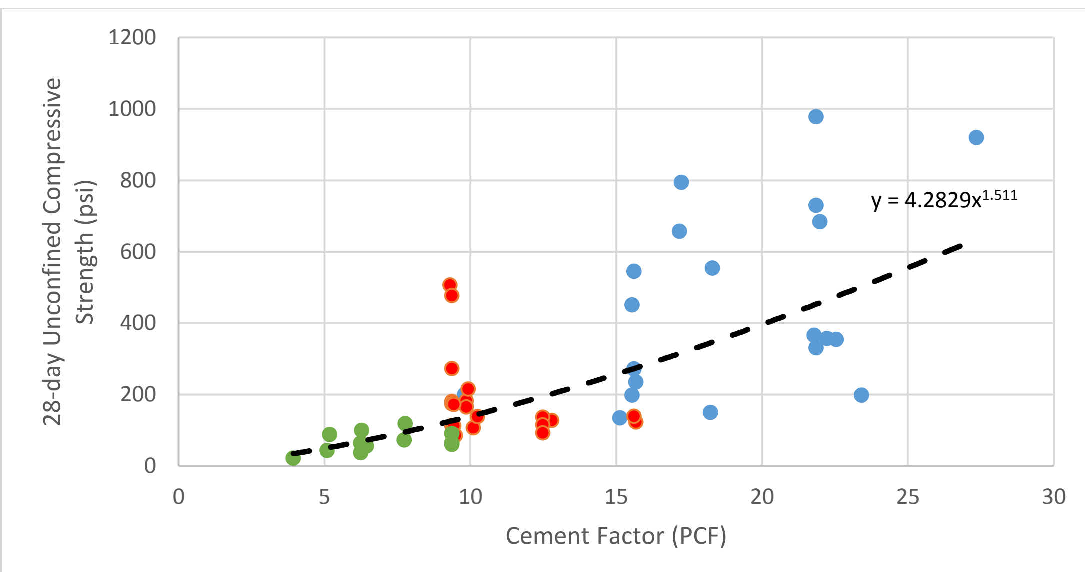

Figure 2.27. Overall results seen from case studies in terms of cement factor, separated by water-to-cement ratio. The grouping is the same as explained in Figure 2.26. (Hodges et al., 2008) (Horpibulsuk et al., 2003) (Jacobson et al., 2003) (Miura et al., 2002)

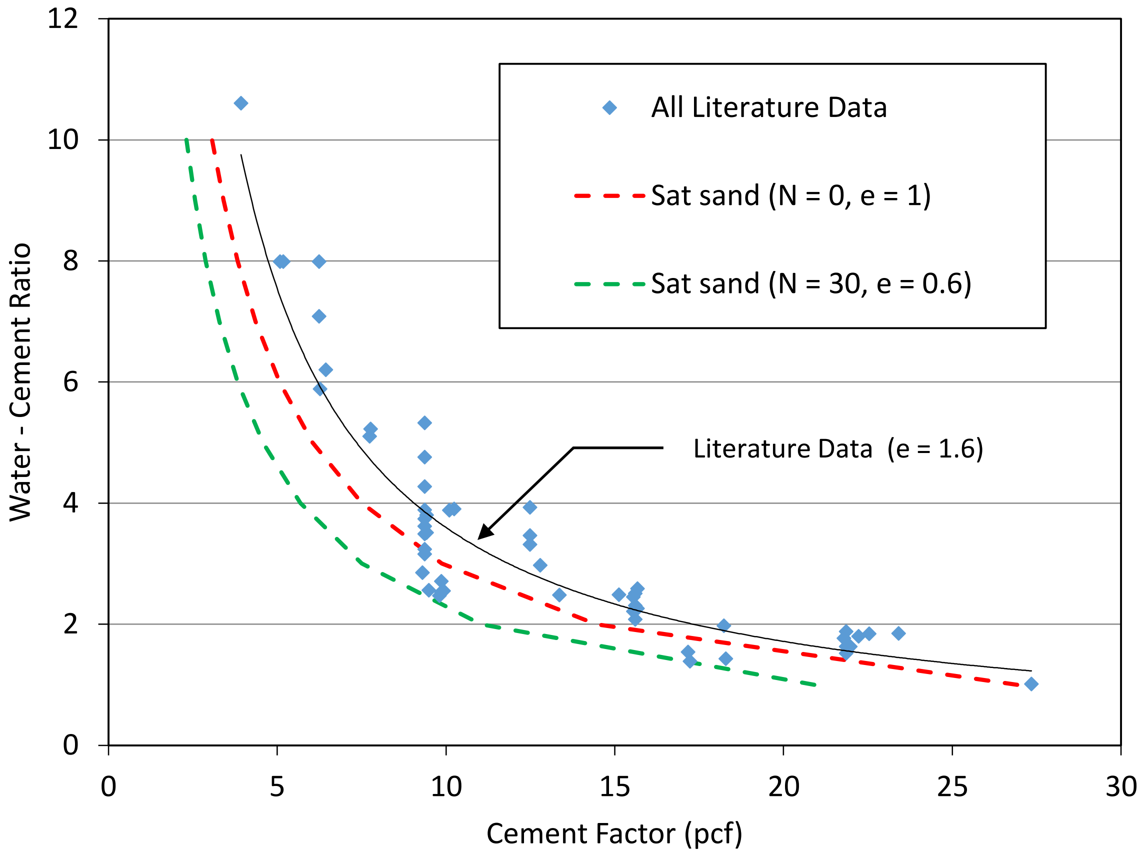

Figure 2.28. Water-to-cement ratio versus cement factor.

Figures 2.29-2.35 shows plots of each case study against the developed regression curve. With the exception of Hodges Northern Virginia sandy clay, each case study falls within an acceptable range to the developed regression curves. Though there is a larger variation within the cement factor regression, the adherence of these various soil types to the developed regression suggests that these trends are independent of soil type.

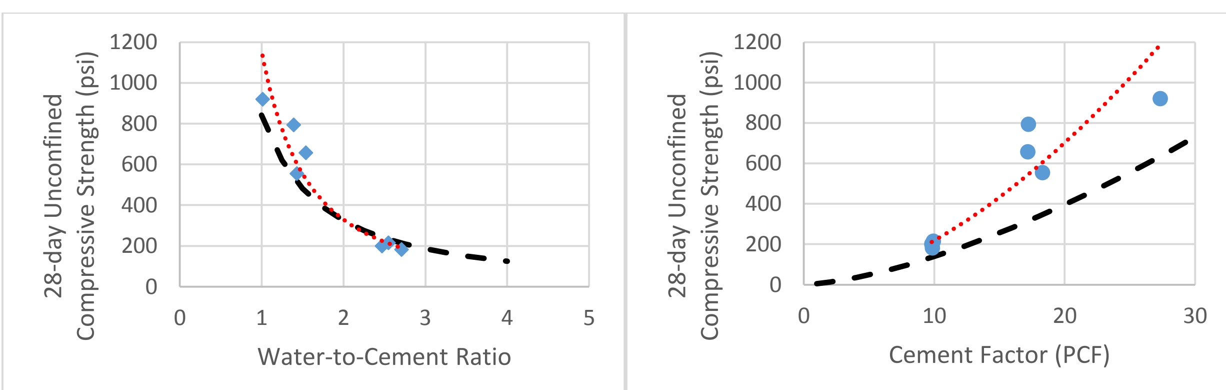

Figure 2.29. Hodges Light Castle sand. (Hodges, Filz, & Weatherby, 2008).

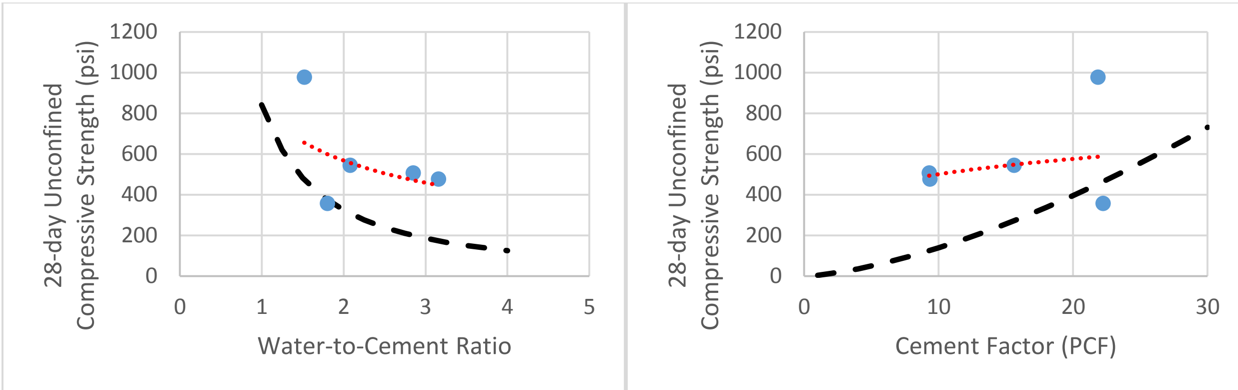

Figure 2.30. Hodges Northern Virginia sandy clay. (Hodges, Filz, & Weatherby, 2008).

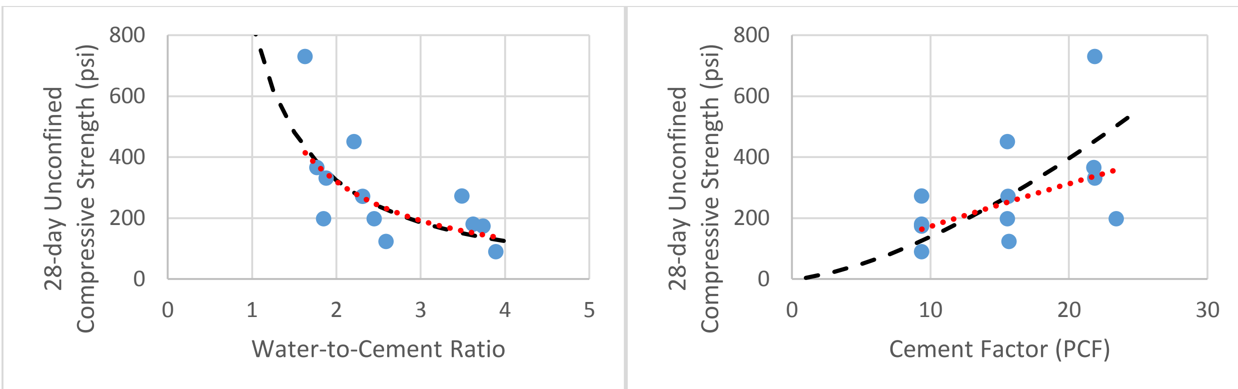

Figure 2.31. Hodges P2 Silty Sand. (Hodges, Filz, & Weatherby, 2008).

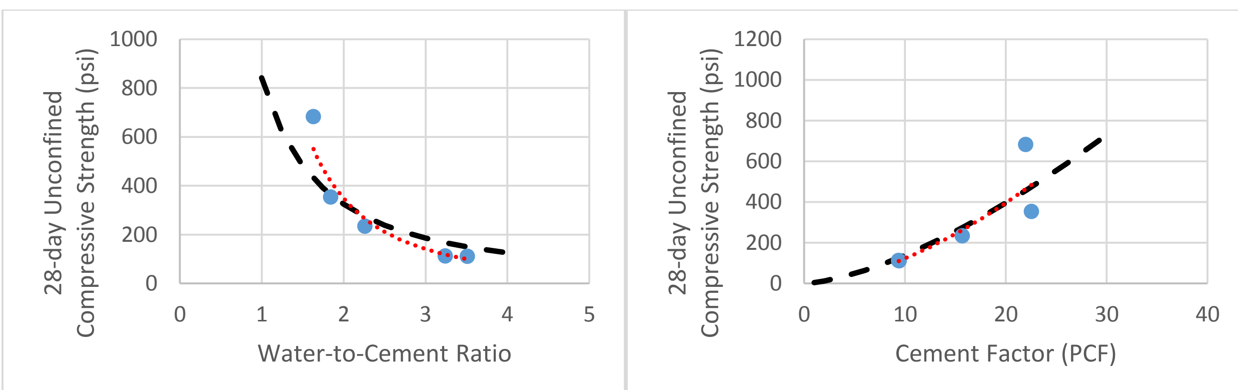

Figure 2.32. Hodges Vicksburg silt. (Hodges, Filz, & Weatherby, 2008).

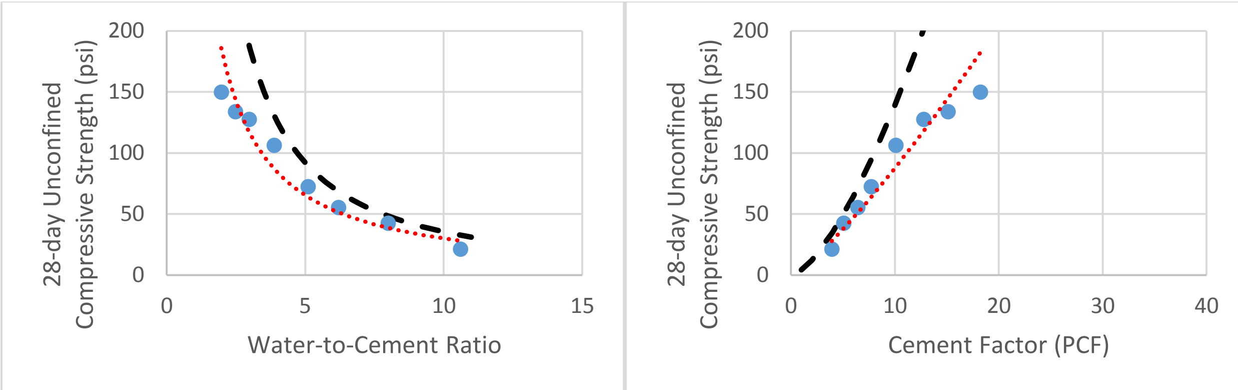

Figure 2.33. Horpibulsuk soft Bangkok clay. (Horpibulsuk, Miura, & Nagara, 2003).

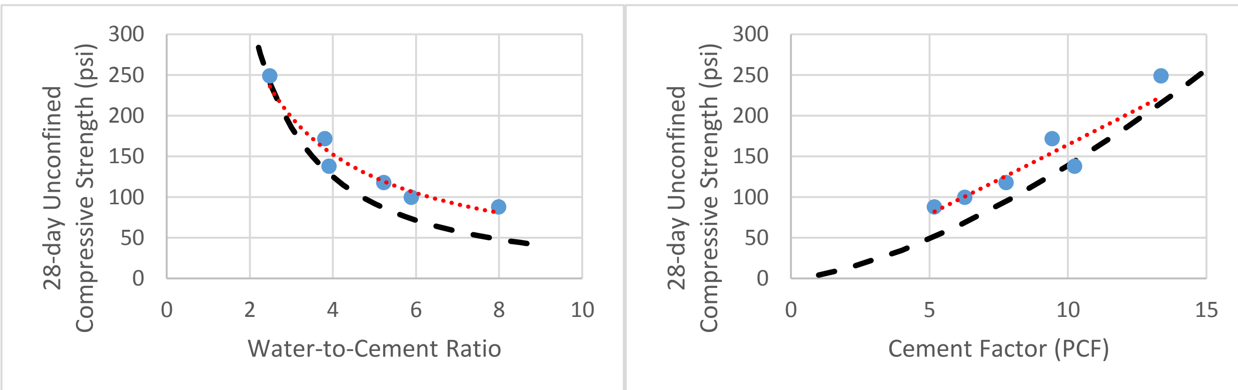

Figure 2.34. Miura soft marine deposit of silty clay. (Miura, Horpibulsuk, & Nagaraj, 2002).

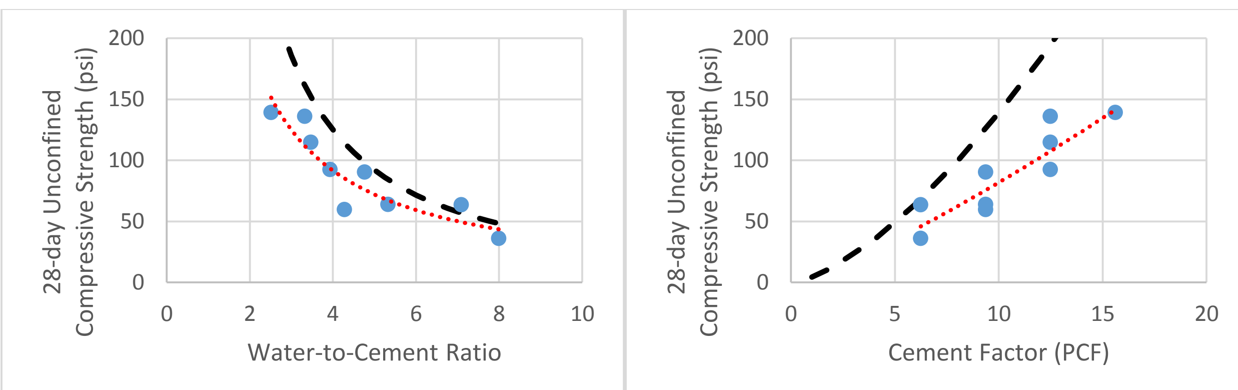

Figure 2.35. Jacobson organic silt. (Jacobson, Filz, & Mitchell, 2003).

2.4.6.2 Breakdown of Literature Findings (Dry vs. Wet)

For completeness, the above literature findings were further sorted by wet or dry mixing methods shown in Figures 2.36 and 2.37, respectively. Most literature and the FHWA Manual (discussed next), make no distinction between wet or dry with regards to design.

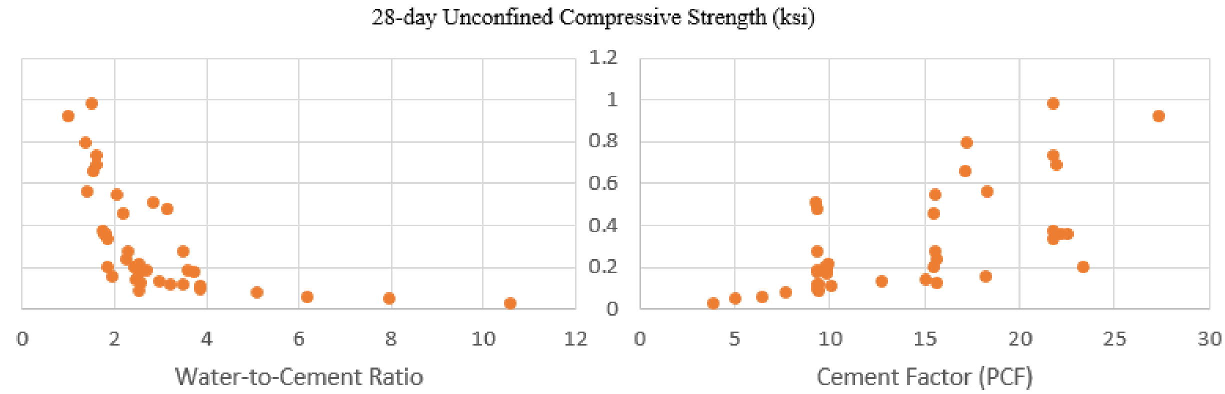

Figure 2.36. Separation of Figures 2.23 and 2.25 by wet mixing method.

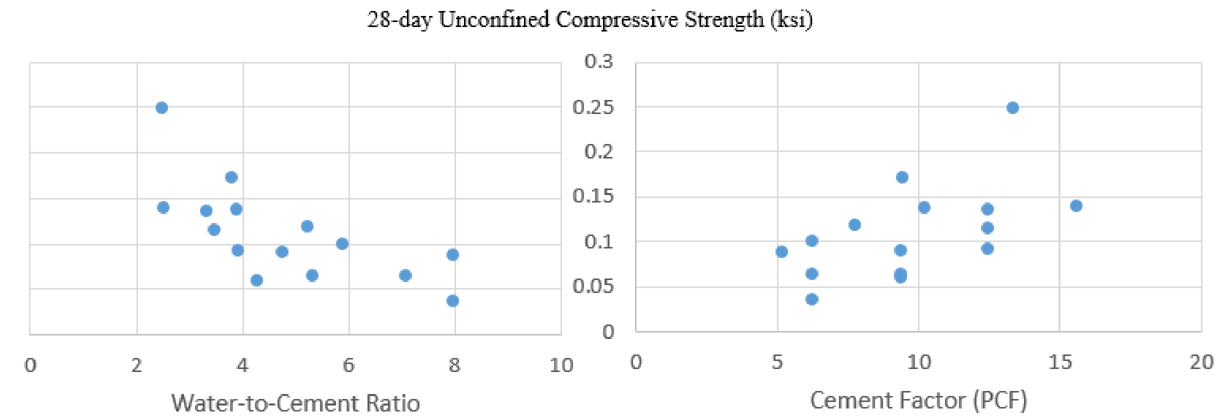

Figure 2.37. Separation of Figures 2.21 and 2.23 by dry mixing method.

2.5 FHWA Design Manual for Deep Soil Mixing

The Federal Highway Administration Design Manual for deep soil mixing (Bruce, et al, 2013) provides a comprehensive design and quality assurance guideline for deep soil mixing using both wet or dry methods. Therein, equipment types, mix methods, binder types, design procedures, site characterization, binder content, etc. are all discussed at length along with design examples and quality control protocols. Much of literature review provided mirrors that previously presented herein and culminates with a strength versus w/c (w/b, water to binder ratio) for the purposes of estimating the required binder (Figure 2.38). However, this curve is defined for inorganic soils. In fact, the manual acknowledges that organic soils do not adhere to any predictive methodologies and extra care and review should be exercised when dealing with organic soil. Several cautionary excerpts are presented as examples.

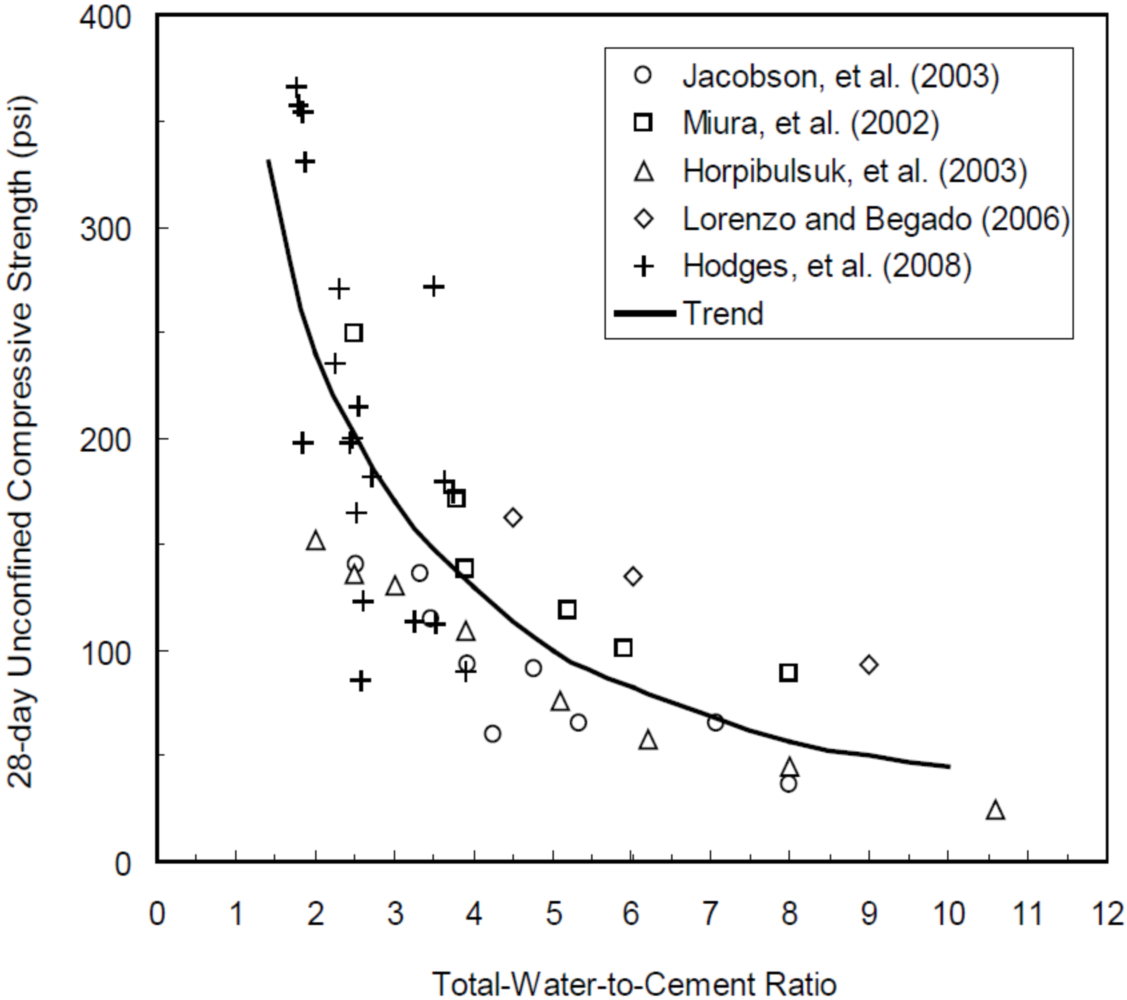

Figure 2.38. Strength vs. W/C ratio (Bruce et al., 2013).

Planning. When planning for a deep mixing project, the manual advises. . .*increasing organic content often requires higher cement content, and organic contents greater than about 10 percent may produce significant interference with cementation. Humus, which is finely divided and decomposed organic matter in soil, has more potential to interfere with cementation than fibrous organic material that is not as decomposed.* No recommendations are provided to address/predict the effect.

Important Site Exploration Details. Site investigations should define and characterize the moisture content and organic content as *more binder is needed to achieve the same strength.* Organic soils having both high moisture content and humus content are therefore particularly problematic. The manual further states. . . *soils with high organic content may require large amounts of binder to achieve suitable strength. Organics may interfere with cementation because organic colloids can attract the calcium in cement or lime and prevent it from participating in the chemical reactions that stabilize the mixture. Humus is more detrimental to cementation than*

fibrous organics because organic colloids from humus can become more widely dispersed in the mixture than intact fibers from fibrous organic material. Consequently, the amount and type of organic material are key parameters that should be well characterized for a deposit.
Again, caution is advised but no recommendations are provided.

Effect of Water-to-Binder Ratio. Given the design curve in Figure 2.38, the manual wisely states that a starting cement content or w/c ratio can be estimated where decreasing w/c ratios tend to increase the strength recognizing that there is a significant amount of data scatter. However, this curve is only applicable to inorganic soils. It notes: *organic soils tend to require more binder than inorganic soils, and sandy soils require less binder than clay soils. Slag-cement binders can be more effective than pure cement for treating organic soils.* As the curve was developed based on laboratory results it further notes that laboratory test results often vary from field results stating. . . *mix design is not an exact science, and site-specific testing is necessary. . .the strengths of laboratory-mixed specimens can be higher than the strengths of field-mixed specimens*.

The strength of field-mixed samples was shown to be as low as 20% of laboratory specimens making field verification an important part of quality control / assurance. This percentage is independent of soil type.

Effect of Curing Time. The effect of curing time is to increase mixture strength. Based on a review of data by researchers, the generally accepted equation estimating the fraction of total strength developed, ***f***, in ***t*** days:

$$f = 0.187\ln(t) + 0.375$$

provides a conservative estimate of the strength increase with time for cement and cement-slag treatment, ***except for some highly organic soils****.*

Establishing Project-Specific Unconfined Compression Strengths. This section of the manual provides a step by step procedure for estimating the required binder content starting by identifying the soil type or site characterization. The first step is to *determine representative values of water content and organic content for each stratum to be treated. . . Use the plot [Figure 2.38] of total water-to-cement ratio versus unconfined compressive strength to estimate the range of unconfined compressive strength that can be achieved for each stratum. Recall that this plot is for 100 percent cement binder and that slag-cement mixtures may be more appropriate for organic soils.* Like other notations addressing organic soils, no rationale is provided to better explain how to better combat the presence of organic soil.

Establishing Trial Deep Mixed Ground Properties. The design of a treated soil matrix is associated with an unconfined compression strength at 28 days of age. Typical values are noted to range between 75 and 150psi for soft soils. The maturation factor, f_c, should be established based on the estimated time (*t*) between mixing and application of 75 percent design load of the proposed embankment height. The manual notes that the maturation equation is *based on a conservative interpretation of a wide range of data including cement and cement-slag blends with fine- and coarse-grained soil.(34) Site-specific testing could be used to justify larger values of* f_c*.* ***One exception*** *to the conservative trend provided by figure 30* ***is for highly organic soils or***

peat *treated with 100 percent PCC* [Portland cement concrete]*, for which the value of f_c should be limited to 1.0 unless site-specific testing permits a higher value. However, cement-slag blends with a high proportion of slag are often used for organic soils and peat; in which case* [the maturation equation] *provides a conservative estimate of strength gain with time.*

Costs. With regards to costs, the FHWA Design Manual states: *higher binder/soil ratios to mix* ***organic soils*** *or meet higher strength QC/QA criteria increases costs.* And . . .*stiffer/denser cohesive soils and* ***soils containing organics/peat*** *are more costly to mix.*

It further states the cost to use wet soil mixing is estimated to be $100 per cubic meter ($77/yard) if . . .*soils can be relatively easily mixed* ***without*** *obstructions, cobbles, or* ***significant peat or organic content.*** This means soils including loose to medium dense cohesionless soils, soft and wet clays and silts, and soft marine clays near the liquid limit; NOT organic soils. Soils that are . . . *stiff or more difficult to mix and may* ***contain organics or peat . . .*** are more likely to cost on the order of $140 per cubic meter ($107/yard).

2.6 Swedish Deep Stabilization Research Centre

The Swedish Deep Stabilization Research Centre and the U.S. National Deep Mixing programs collaborated in translating a Swedish Geotechnical Institute publication (1999) which was published in 2002 (Axelsson et al., 2002). The mission of both organizations was the dissemination of international experience where the Swedish experiences with dry soil mixing were far beyond that of the rest of the world. Organic soils were a focus of this effort.

The study identified organic soils, called *mud and peat*, as problematic, stating:

> *Mud and peat, unlike clay, have high organic content. The organic material may include retarding substances such as humus and humic acids. During stabilization the humic acids react with $Ca(OH)_2$ to form insoluble reaction products which precipitate out on the clay particles. The acids may also cause the soil pH to drop. This negatively affects the reaction rate of the binders, resulting in a slower strength gain in mud and peat than in clay.*
>
> *Studies in Finland indicate that in soils with high organic contents, such as mud and peat, the quantity of binder needs to exceed a "threshold." As long as the quantity of binder is below the threshold the soil will remain unstabilized. A reason for this may be that the humic acids are neutralized when sufficient binder is added.*
>
> *A recent study at the University of Oulu, Finland, shows the negative effect of humus and humic acids on the effectiveness of soil stabilization. However, the results of the study indicate that the humus and humic acid content of the soil is only one of several factors affecting stabilization effectiveness. Hence the stabilization outcome of a binder cannot at present be definitely predicted merely by determining the organic content and humus content of the soil.*

> *Cement is often a* ***more effective stabilizer*** *than lime* ***in mud and peat soils****. This is probably due to the effect of humic acids as discussed above and to the inhibition of one of the most important strength-enhancing mechanism of the lime (pozzolanic reactions). In pozzolanic reactions the lime reacts with clay particles in the soil to form binding materials. In peat and mud the organic material occupies so much of the soil volume that the stabilizer fails to come in contact with the few clay particles that are present, with the result that pozzolanic reactions do not take place. Cement gives a more robust strength gain as the cement forms binding materials with water and clay particles play no role.*

While the FHWA manual is thorough in all areas excepting organic soils, this study pointed out possible explanations for the effects of organic soils on cement stabilization performance: (1) the concept of a required binder threshold that is required to offset the acidity of organic soils below which no improvement is achieved and (2) the possibility that pure cement works better for organic sands and perhaps that slag/cement mixes are better suited for organic clays.

These concepts were scrutinized and entertained during this research project.

Chapter 3: Small-Scale Laboratory Testing

In order to better understand the effects of cement stabilization in organic soil mixing, an extensive laboratory testing program was undertaken. This was performed in two phases: Phase 1 focused on chemically treating soils prior to mixing with pH modifiers; and Phase 2 focuses on binder and organic content variations. The Phase 1 discussion includes general testing information used in both phases.

3.1 Phase 1: Chemical Approach

The acidic composition of organic soils has been speculated to be a primary mechanism leading to extremely poor strengths relative to inorganic soils when both are treated with cementitious binders. Phase 1 focused on raising the pH of a soil prior to mixing. Overall, Phase 1 involved the following: acquiring a suitable sample of organic soil, testing the chemistry or organic soils in general, soil mixing, compressibility test, and results.

3.1.1 Acquiring a Sample

Three sites were identified early as potential sources of organic soils. The first site was a de-mucking project of a drainage canal in Hillsborough County (Figure 3.1). Samples were pulled and tested from this site and showed organic contents of only 6%. Interestingly, many low organic content soils are thought to be problematic by visual inspection when in fact they are not. The second site was at the Crosstown/I-4 Connector Project (Figure 3.2). This site was a retention pond modification, and samples showed organic contents of only 10%. A third site identified by the State Materials Office was also deemed unacceptable for the study due to low organic content.

Figure 3.1. Potential organic source from a Hillsborough County de-mucking of a canal.


Figure 3.2. Potential organic source from the Crosstown / I-4 connector retention pond modification.

Although no construction was planned for the SR-33 location discussed in Section 2.3.3, this area was a known site where organic materials were the cause of an ongoing settlement/maintenance issue. Therefore, special provisions were afforded by the FDOT District I maintenance office to extract material from the right-a-way along that stretch of problematic stretch of road. Approximately 7 yards of soil was excavated from the SR-33 right-a-way near Polk City. The soil sample was obtained using a Gradall excavator (Figure 3.3). The samples were taken from near the point 8 survey location (discussed in Chapter 5). The area excavated was approximately 6 x 8 x 4 ft. The sample was then transported back to USF, where it was held in a covered fiberglass tank (Figure 3.4). The soil excavated on 4/16/2013 was used for Phase 1 and a portion of Phase 2; this soil had an organic content of 66%. Additionally, on 12/4/2014, approximately 8 more yards of soil was retrieved from the same site within close proximity of the first location. The second sample had an organic content of 44%.

Figure 3.3. SR-33 Soil Excavation using Gradall provided by FDOT.

Figure 3.4. Fiberglass holding tank.

3.1.2 Testing the Chemistry of Organic Soils

After obtaining the sample from SR-33, the moisture content, organic matter, resistivity, pH, chloride and sulfate levels were determined.

The pH of organics is known to be in the acidic range due to the presence of tannic and/or humic acids. The acidic environment is detrimental to the reactiveness of Portland cement which lab tests have shown to require a pH of 12 or higher to commence. The mass stabilization process used at Jewfish Creek and Marco Island Airport applied the blended binder simultaneously which did not allow the pH to be neutralized prior to introduction of cement. This is essentially similar to mixing cement with acid instead of water where much of the binder is consumed to neutralize the acid with the hope that the cement (or slag) will eventually raise the pH to a suitable level. However, increasing the pH prior to mixing was conceived to have a dramatic effect on the cement reactions as observed in mineral or polymer slurry preparation. At first glance, it seemed impractical to pretreat the soil, but the present practice for shallow mass mixing already makes an initial pass with a backhoe to loosen the soil which is then followed by the mixing/tilling process. Further, discussions with personnel from Hayward Baker Inc. indicated that pretreating the soil had not been attempted in the laboratory testing phase and if pretreating can successfully raise the pH, the act of pretreating would not add time or cost to the process. In fact, it could reduce the total amount of binder and the associated cost.

Although blended cement/slag binders have been successfully used by contractors to combat the low pH issue, it is also conceivable that other products such as soda ash, pot ash, lye, lime or pearl ash could also be used to either pretreat or compliment the pH stabilization. These additives have varying effectiveness but in general require less than one pound per cubic yard to raise the pH to 7 which is far less than the increase in binder (125 to 400 pcy) required at Marco Island to assure initiation of hydration. The selection of pH adjusting materials is cost driven. Presently, slag and cement are roughly of the same price for the purposes of soil mixing, but mixes using slag generally require higher binder concentrations. This Task addressed pH pretreating options, different mix ratios, binder materials, and order of mixing with the goal of increasing the reliability of the completion of the cement reaction.

Although organic soils are often classified into a few basic categories (e.g. organic sands, organic clay, or organic silts) the type of organic material can also have varied effects on binder enhanced mixtures. In addition to reviewing chemical proportioning and order of mixing, these materials were tested for texture, pH, organic content and moisture content as is customary in classifying organic soils. Example lab results are presented below, but many other tests were performed over the course of the study.

3.1.2.1 Moisture Content Determination

The moisture content was determined in accordance with ASTM D 2974-00, test method A. Two samples of approximately 50 g. were oven dried for 16h at a temperature of 105°C (~220°F). Results found below. (Table 3.1)

Table 3.1 Moisture content determination

MOISTURE CONTENT	
wt of wet soil + pan (g)	82.98
wt of dry soil + pan (g)	45.15
wt of water (g)	37.83
wt of pan (g)	31.39
wt of dry soil (g)	13.76
% moisture (average)	**275%**

3.1.2.2 Organic Matter Determination

The oven-dried samples from the moisture content determination were transferred to two porcelain crucibles and then placed in a muffle furnace. The temperature was gradually increased to 440°C (~824°F). The samples remained in the furnace until no change of mass occurred after a further period of heating, approximately 8 h. Organic content results below (Table 3.2).

Table 3.2 Organic matter determination

ORGANIC CONTENT	
wt of soil + pan before burn (g)	76.84
wt of soil + pan after burn (g)	64.70
wt of pan (g)	51.07
wt of soil before burn (g)	25.77
wt of soil after burn (g)	13.64
wt of remaining material (g)	12.14
% organics (average)	**47%**

3.1.2.3 Soil Resistivity

The soil resistivity was determined according to FM 5-551. The (as received) soil was placed in the soil box (Figure 3.5), taking care to fill any voids. The soil box was then connected to the resistivity meter (Figure 3.6) and the results were recorded. The soil was removed from the soil box and 10 mL of de-ionized water was added. The soil was returned to the soil box and another reading was recorded. This process was repeated until resistivity of the soil began to increase. Results found below (Table 3.3). The soil resistivity was taken to be the lowest value recorded, 5k Ω-cm.

Table 3.3 Soil resistivity results

	RESISTIVITY (Ω-cm)
Soil (as received)	5,500
10 mL	5,400
20 mL	5,200
30 mL	5,000
40 mL	5,300

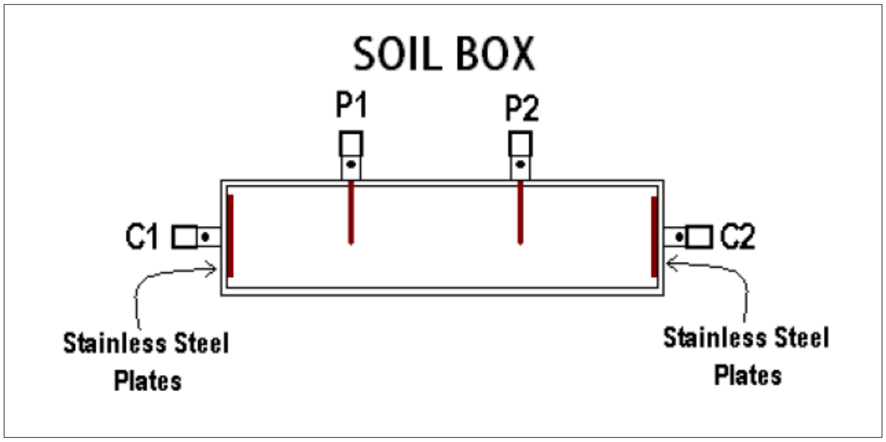

Figure 3.5. Soil box.

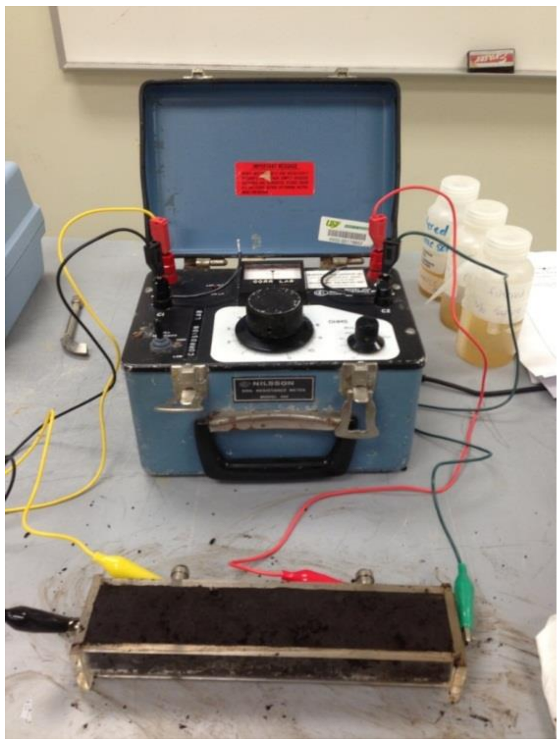
Figure 3.6. Soil box connected to resistivity meter.

3.1.2.4 Soil pH Determination

ASTM D 4972 test method A was used to determine the soil pH. Procedure 10.3 *pH in Distilled Water* was used. Approximately 10g of dried soil was placed in a beaker along with 10mL of de-ionized water. It was mixed and allowed to sit for an hour, then the pH was measured. The pH of the soil ranged from 5.5 to 7 (Figure 3.7). However, for soil mixing applications, it should be noted that this test method effectively dilutes the pore solution to produce a lower concentration (more neutral) than the in situ pH condition. Litmus paper was used for all soil mixing samples.

Figure 3.7. Soil pH determination.

3.1.2.5 Chloride and Sulfate Determination

To prepare for these tests, a sample was taken from the middle of the storage bed and oven dried at 110°C (~230°F) overnight. The sample was then sieved using a No. 10 sieve. 100 g. of the oven dried soil was placed in an Erlenmeyer flask along with 300 mL of de-ionized water, and allowed to sit for at least 12 hours. The soil and DI water mixture was filtered, the soil was discarded and the remaining liquid was used for testing.

In accordance with FM 5-552 the average chloride level was 590 ppm. The Hach low range chloride test kit was used. It took 5 drops of silver nitrate titrant to get the sample to the orange-red rust color, as seen in Figure 3.8.

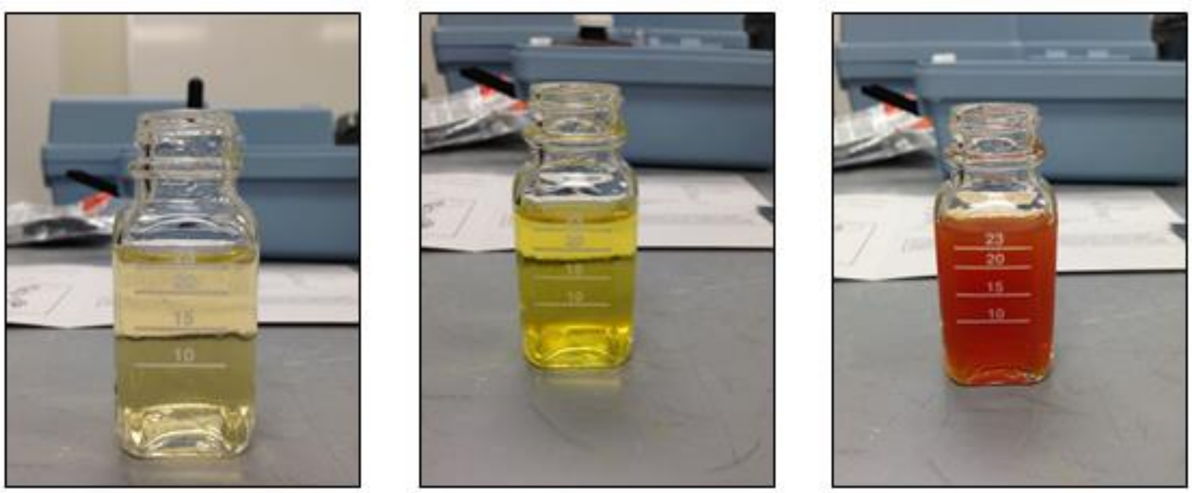
Figure 3.8. Chloride testing

In accordance with FM 5-553 the average sulfate level was found to be 35ppm. Hach Sulfate, Pocket Colorimeter II Test Kit was used (Figure 3.9).

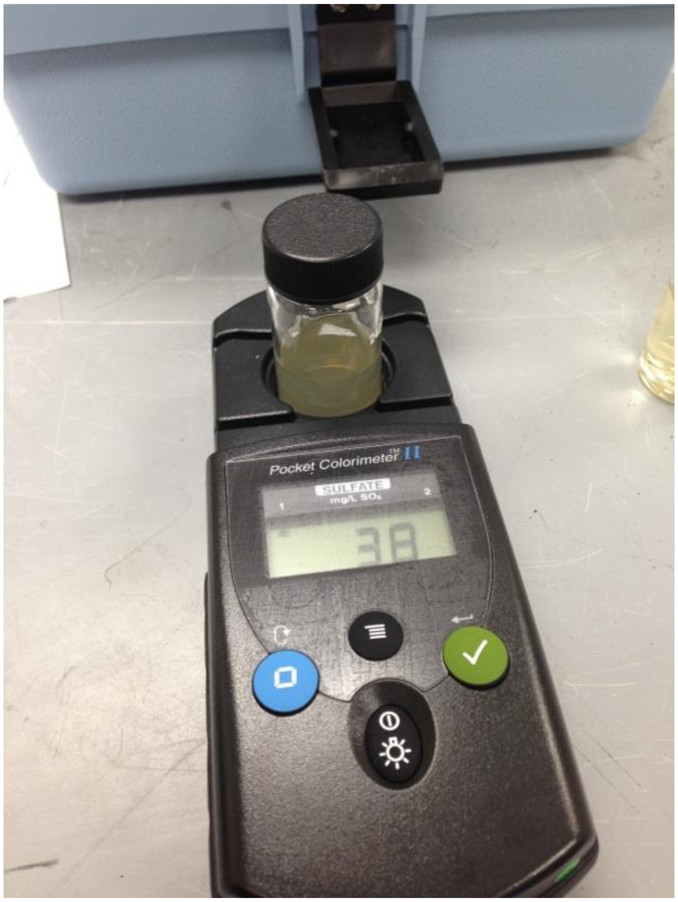

Figure 3.9. Sulfate testing.

3.1.2.6 Soil Properties Summary

Summarizing the lab findings:

- 260-300 % moisture content range
- 50-65 % organic content range
- Resistivity of 5kΩ-cm
- 5.5 to 7 pH range
- Chloride level of 590 ppm
- Sulfate level of 35 ppm

The environmental classification of the SR-33 soil sample was determined to be moderately aggressive according to FDOT Standard Specification for Bridge Construction 2013, Chapter 2 Site and Material Criteria.

3.1.3 Soil Mixing

After reviewing literature and projects on soil remediation, it was decided that several samples would be created with varying parameters. The following parameters were changed and/or monitored with each sample: pH, moisture content, mixing method, and cement content. The pH of the soil was adjusted using soda ash (sodium carbonate), increasing the pH. The moisture content (MC) was varied from 280% (in situ) to 400%. For each MC both dry and wet mixing methods were performed. Dry mixing, where the cement is added directly to the sample, and wet mixing, where a water/cement slurry is added to the sample. Type II cement was used in the amounts of 100pcy, 200pcy, and 300pcy. The same criteria were tested with lye (sodium hydroxide), lime (calcium oxide), and pot ash (potassium carbonate) as the pH modifier.

In order to produce a complete range of data, attempts to lower the pH of the soil were also undertaken. As tannic acid is the predominate source of acidity in the organic soil, efforts to exacerbate the conditions were undertaken where samples of varying moisture contents were dosed with tannic acid in attempt to bring the pH down to 3 or 4 (from approximately 6). This was achieved using a 50g sample with 0.82 M $C_{72}H_{52}O_{46}$.

In addition to the above parameters, tests to find the optimal mixing energy were performed. The effects of mixing energy were tested using samples with a single moisture content of 350%, and no pH modifiers were added. Both wet and dry mixing methods were used for 200pcy and 300pcy of cement. Mixing times were set at 30, 60, 120, 240 and 480 seconds (0.5, 1, 2, 4 and 8 min).

The following steps represent the soil mixing procedure. Figure 3.10 contains pictures associated with the mixing procedure.

- Batch approximately 2ft.^{3} of soil and obtain the moisture content; adjust as needed.
- Gather a 2000 gram sample soil from batch.
- Determine the amount of soda ash to add.
- Determine the amount of cement to add.
- If wet mixing, determine the amount of water to create a 0.6 w/c paste
- Place soil sample onto mixing stand, set mixer speed to gear 1
- Record pH of soil
- Begin mixing in soda ash and continue until thoroughly distributed; stop mixing
- Record pH of soil
- If wet mixing, combine cement and water
- Begin mixing and add the cement or water/cement paste
    - For 100 pcy mix for approximately 40 seconds
    - For 200 pcy mix for approximately 50 seconds
    - For 300 pcy mix for approximately 60 seconds
- Place the soil mixture in the 4 x 8 in. cylinder, in 3 layers, tamping between each layer
- Record the pH of the soil mixture using litmus paper (right)
- Cap the cylinder and store in climate controlled area

Figure 3.10. Mixing procedure pictures (read from left to right).

3.1.4 Unconfined Compression Tests

After a 7 day cure time the samples were removed from the 4 x 8in cylinders and placed in the Material Testing System (MTS) machine to determine the unconfined compression strength. Figures 3.11 and 3.12 show a test cylinder during and after testing, respectively.

Figure 3.11. Unconfined compression testing.

Figure 3.12. Cylinder after failure.

3.1.5 Soil Mixing Test Results

Much of the preliminary work was based on the possibility or practicality of pretreating low pH soils to achieve a more neutral pH, better-suited for cement activation. To this end, the organic soil obtained, with pH ranging from 5.5 to 7 (ASTM test method), was also tested using litmus paper directly applied to the wet soil prior to the addition of cement. As expected, the introduction of soda ash as a pH modifier increased pH with increased amounts of soda ash as shown in Figure 3.13. Although soil mixing specimens were prepared with soda ash contents up to 17.2 pcy, it is apparent from Figure 3.13 that only 4 pcy or less is necessary to achieve a pH of 7. Table 3.4, Table 3.5 and Table 3.6 show the overall laboratory test results for soil samples with initial moisture contents of 280, 350, and 400%, respectively.

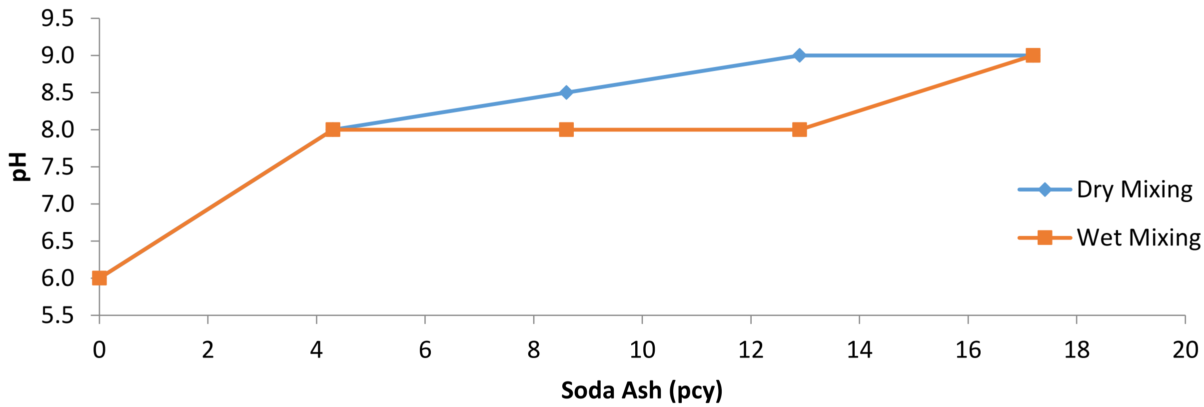

Figure 3.13. pH vs. Soda Ash Dosage.

Several observations have been made in regards to the results. For all moisture contents, 100 pcy of cement was not sufficient. After the third batch, the 100 pcy series was discontinued. Figure 3.14 shows the 100 series with no soda ash, and Figure 3.15 shows the 100 series with the maximum soda ash, 17.2 pcy (Figure 3.15), neither reached levels above 1 psi of unconfined compressive strength. Figure 3.16 shows that the dry mixing samples reached almost 20 psi of strength while the wet mixing samples only obtained 12 psi of strength. This is not surprising given the increase in the w/c ratio. Additionally, soda ash levels above what was required to achieve a neutral condition appeared to decrease the 7-day strength.

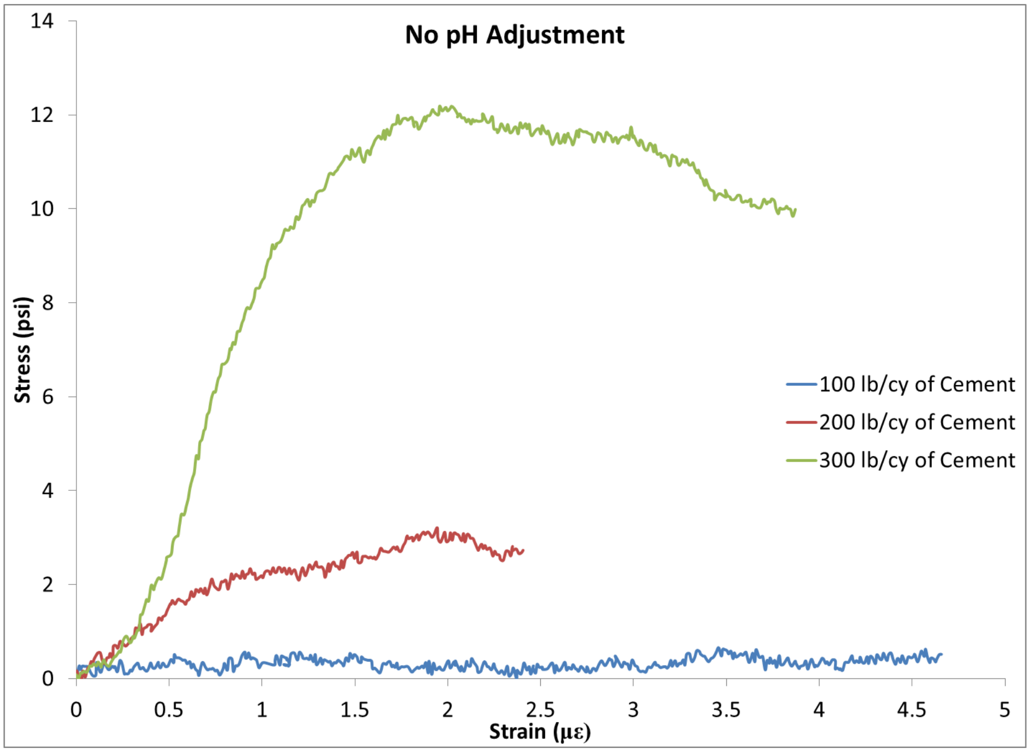

Figure 3.14. Stress vs. Strain: MC=400%; No pH Adjustment (0 pcy Soda Ash); Dry Mixing Method.

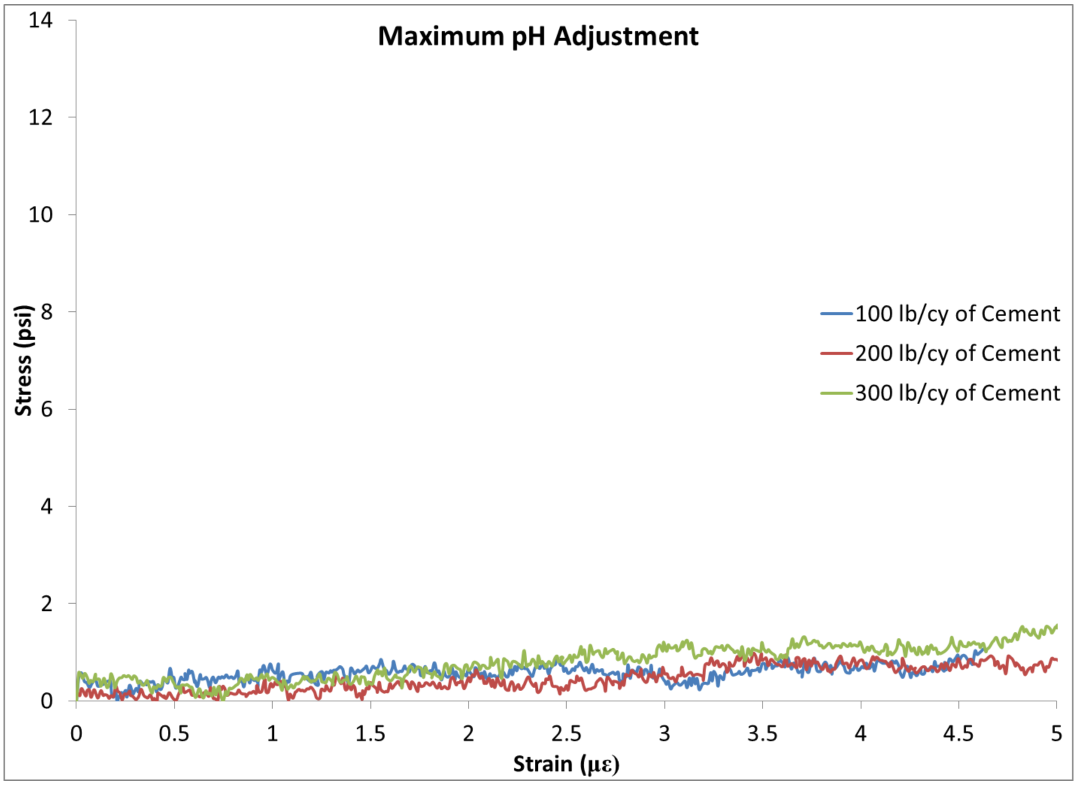

Figure 3.15. Stress vs. Strain: MC=400%; Maximum pH Adjustment (17.2 pcy Soda Ash); Dry Mixing Method.

As only a very small amount of soda ash is required to bring the soil pH above neutral (7), it is conceivable that no more than 1 to 4 pcy will ever be required (for similar soil). A slight increase in capacity can be seen in Figure 3.16 (left), where the pH had increased above 7, but not when pretreated to 9 or above as with the higher soda ash specimens. Similar to concrete, this again goes back whether or not the soda ash is killing or retarding the cementitious reactions.

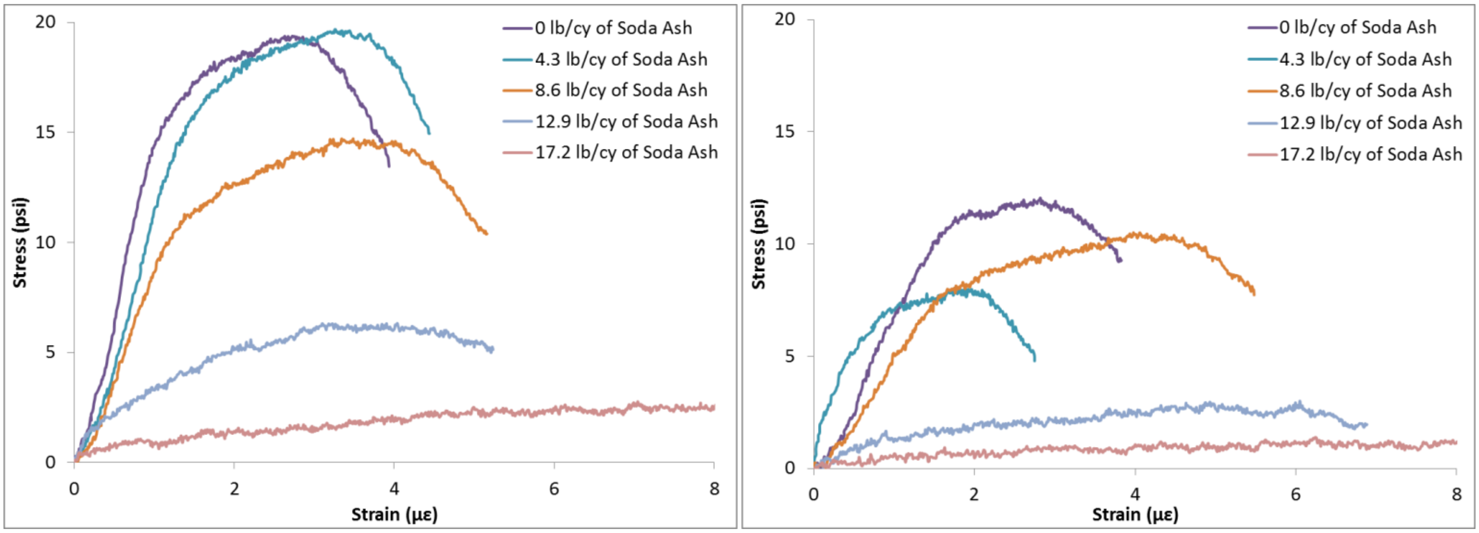

Figure 3.16. Stress Strain diagram comparing the dry mixing method (left) and the wet mixing method (right) at 300 pcy cement.

Table 3.4 Unconfined Compression Test Results for a MC of 280% (in situ)

280% Moisture Content							
Cement (pcy)	**Soda Ash (pcy)**	**Dry Mixing Method**			**Wet Mixing Method**		
		UC (psi)	**pH Before Cement**	**pH After Cement**	**UC (psi)**	**pH Before Cement**	**pH After Cement**
200	0	3.7841	6.0	11.0	3.6120	6.0	11.0
	4.3	3.4795	7.0	10.0	3.0393	7.5	11.0
	8.6	3.3622	7.0	11.0	2.0864	8.0	12.0
	12.9	3.7353	8.0	10.0	2.3486	8.0	10.0
	17.2	3.7055	8.0	10.5	1.7504	8.0	10.0
300	0	11.4227	6.0	12.0	11.8167	6.0	11.0
	4.3	17.8700	7.0	12.0	12.2209	6.0	12.0
	8.6	12.0234	8.0	12.0	10.6893	8.0	11.0
	12.9	12.4718	8.0	12.0	6.2285	7.0	11.0
	17.2	7.0646	8.0	12.0	2.6593	8.0	11.0

Table 3.5 Unconfined Compression Test Results for a MC of 350%

350% Moisture Content							
Cement (pcy)	**Soda Ash (pcy)**	**Dry Mixing Method**			**Wet Mixing Method**		
		UC (psi)	**pH before Cement**	**pH after Cement**	**UC (psi)**	**pH before Cement**	**pH after Cement**
200	0	3.9833	6.0	10.0	4.9630	6.0	12.0
	4.3	4.0451	6.0	10.0	2.2142	7.0	12.0
	8.6	5.3361	8.0	10.0	3.0269	8.0	10.0
	12.9	2.3062	8.5	10.0	0.5674	8.5	10.0
	17.2	1.8431	9.0	11.0	0.3903	9.0	10.0
300	0	19.3803	6.0	11.0	11.9462	6.0	12.0
	4.3	19.5942	8.0	11.0	8.0477	8.0	12.0
	8.6	14.6581	8.5	11.0	10.2768	8.0	11.0
	12.9	6.3263	9.0	11.0	2.8304	8.0	10.0
	17.2	2.5774	9.0	10.0	1.2968	9.0	10.0

Table 3.6 Unconfined Compression Test Results for a MC of 400%

400% Moisture Content							
Cement (pcy)	**Soda Ash (pcy)**	**Dry Mixing Method**			**Wet Mixing Method**		
		UC (psi)	**pH before Cement**	**pH after Cement**	**UC (psi)**	**pH before Cement**	**pH after Cement**
200	0	3.0846	6.0	10.0	4.0356	6.0	10.0
	4.3	1.7315	8.0	9.5	1.8355	7.0	11.0
	8.6	2.3576	8.0	10.0	2.5048	7.0	10.0
	12.9	1.1431	9.0	10.0	0.5050	8.0	9.0
	17.2	1.0524	9.5	11.0	0.2557	9.0	9.5
300	0	12.0503	6.0	11.0	10.0814	6.0	10.0
	4.3	5.3353	8.0	11.0	5.7114	7.0	10.0
	8.6	4.7991	8.0	11.0	3.2810	8.0	10.0
	12.9	2.4388	9.0	11.0	3.8182	8.0	9.5
	17.2	1.4152	9.0	12.0	0.6663	8.5	10.0

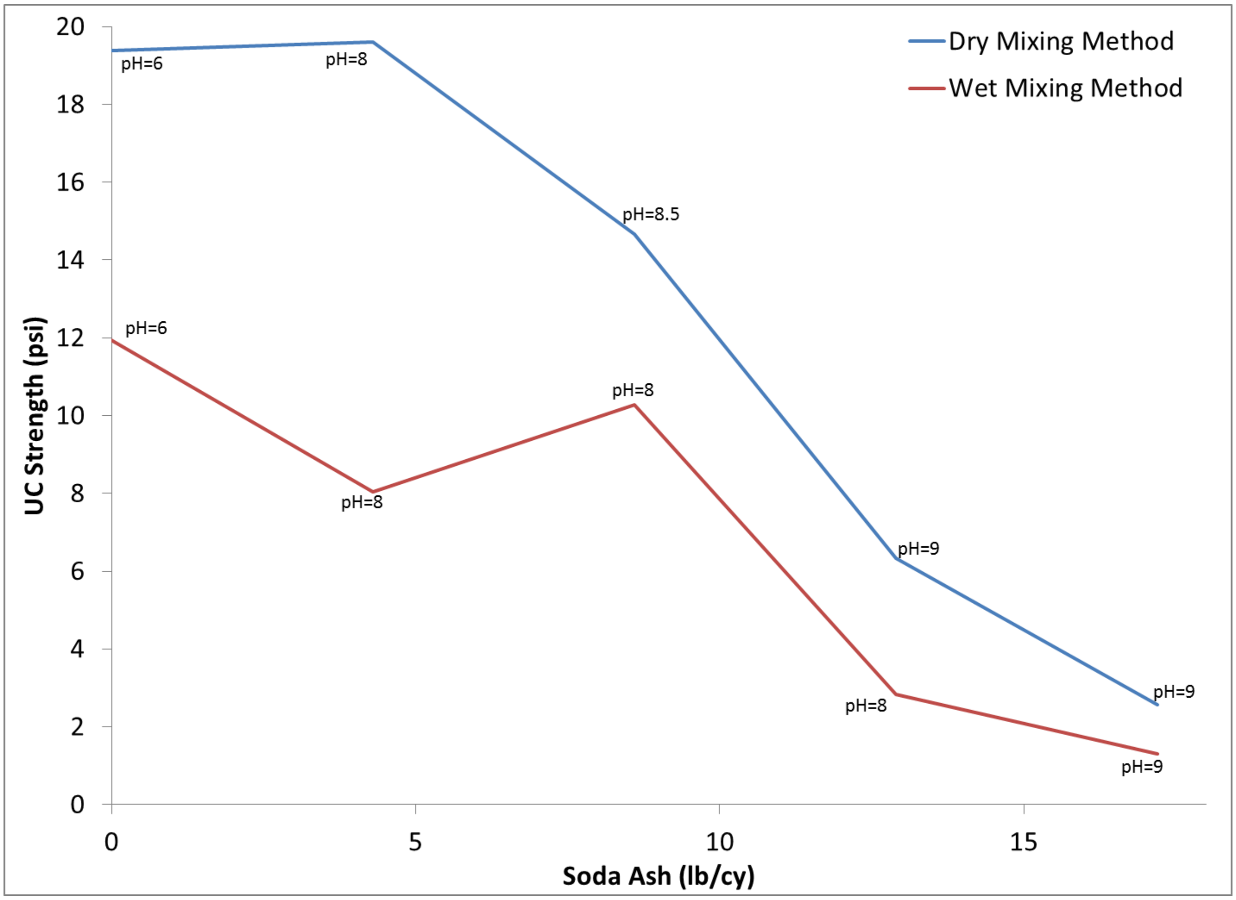

Figure 3.17. Unconfined Compressive Strength vs. Soda Ash content for a moisture content of 350%.

While some positive effect may be noted at very low doses, the addition of soda ash was overall detrimental to the soil mixed unconfined compression strengths.

3.1.6 Time Dependency

As mentioned above, further investigation was undertaken to determine whether or not the soda ash was retarding or simply killing the cementitious reactions. For this test, the samples were tested at 7, 14, and 28 days. A cement content of 300 pcy and a moisture content of 350% with varying soda ash content (0 to 17.2 pcy) were chosen to conduct this test. Both wet and dry mixing methods were used. Of the 30 samples only one exceeded the 7 day strength. The additional soda ash appears to show little to no change between 7 to 28 day cure times (Figure 3.18). No positive increase in capacity was noted from prolonged curing times.

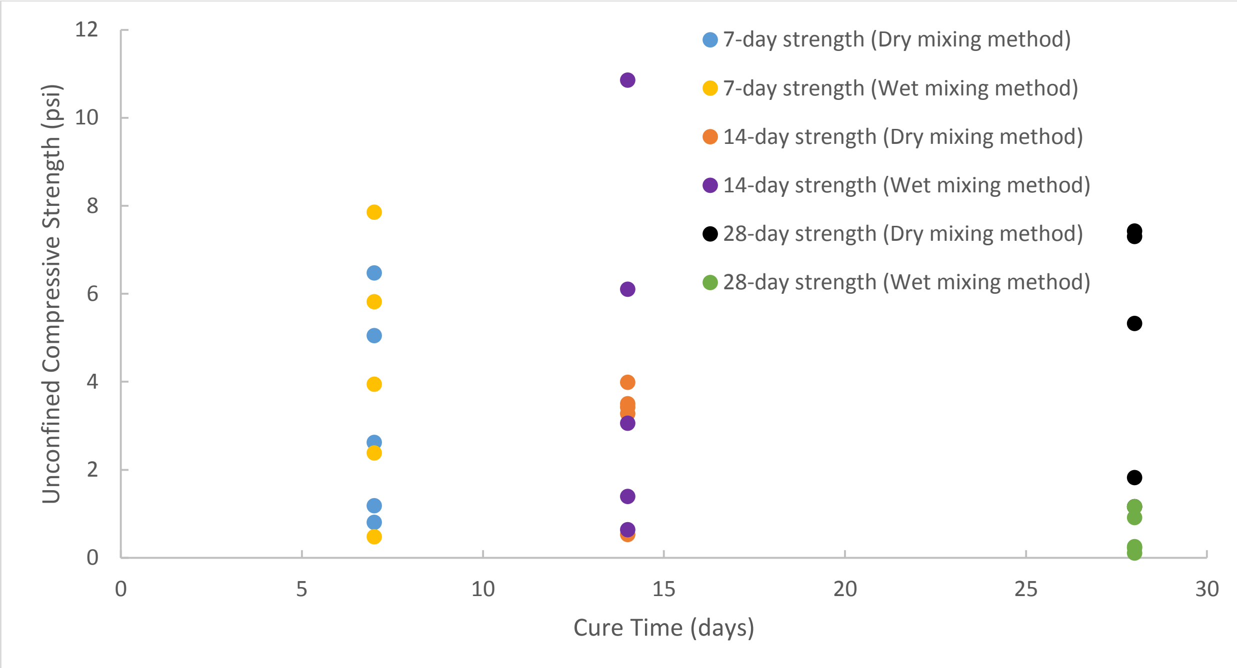

Figure 3.18. No strength gain in soda-ash-treated specimens.

3.1.7 Testing of the Optimum Mixing Energy

When setting up the laboratory testing program, matching the mixing energy per volume of treated soil with that used in the field was difficult. For DSM, the rate of revolution of the field tool is generally 50 to 100 rpm, and the paddle diameter can vary drastically depending on the equipment. As a result, the mixing energy associated with the two pieces of equipment shown in Figures 2.6 and 2.7 varied depending the amount of time spent in a given region of the soil matrix. This is further complicated by the diameter of the blade and the variation in energy imparted to the soil at the center and edges of the blades; the local velocity and mixing energy is proportional to the radius of the blade at that location. Use of the KPS discussed in Chapter 2 is one way in which variations within the treated soil have been tested in field testing. Laboratory tests were designed to address this and/or provide corrections for unrealistic/better-than-field mixing efficiencies. However, in practice, calibration of a lab/field performance ratio (e.g., 2 to 5) was required to compensate for the large difference in mixing techniques used in the lab and field. Regardless of whether in the field or lab, a measure of acceptable mixing effort should be provided.

Mixing energy was varied for two different mix designs involving 200 and 300 pcy for both wet and dry mixing methods. Figures 3.19 and 3.20 show mixing times ranging from 30 sec to 8 min. The time versus strength graphs show an optimal mixing time of at least 4 minutes. This is also presented in Table 3.7.

While the 200-pcy test series appeared to develop no additional capacity from mixing times greater than 60 seconds, the 300-pcy test series showed increased benefit from additional mixing times up to 240 seconds. For the purpose of consistency, a four-minute mixing time was adopted.

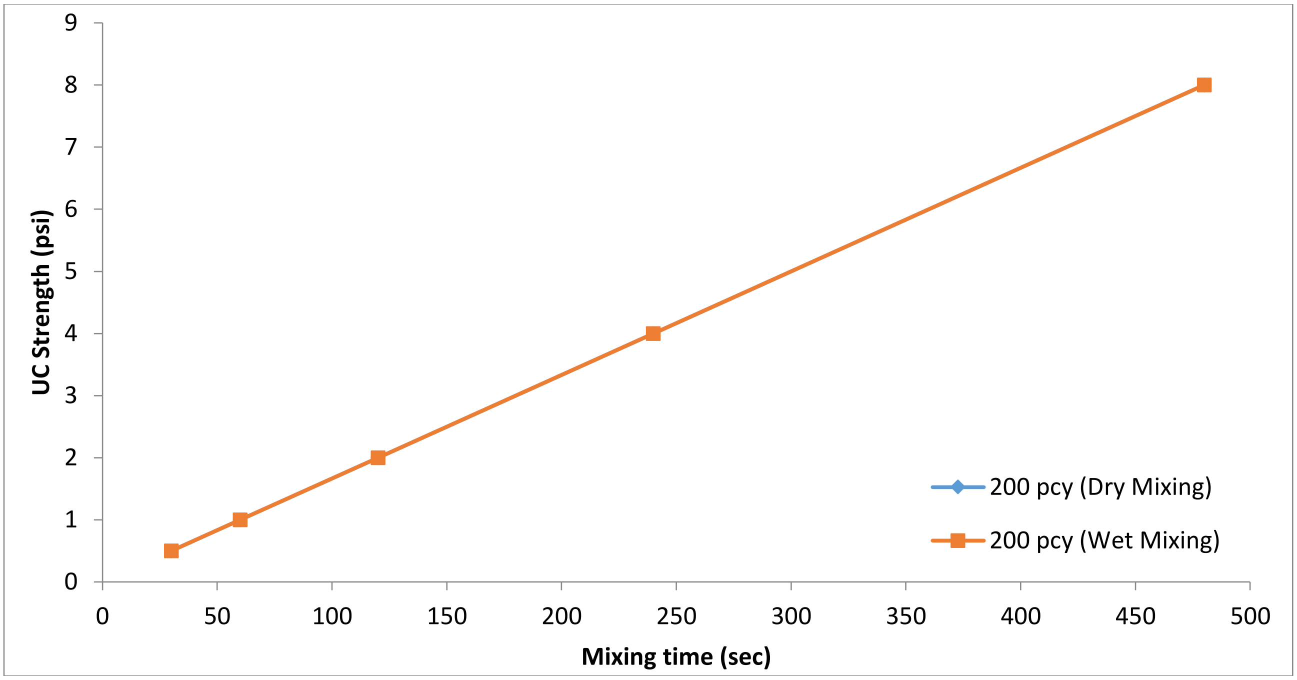

Figure 3.19. Mixing Energy – MC of 350%; 200 pcy; Dry Mixing Method and Wet Mixing Method

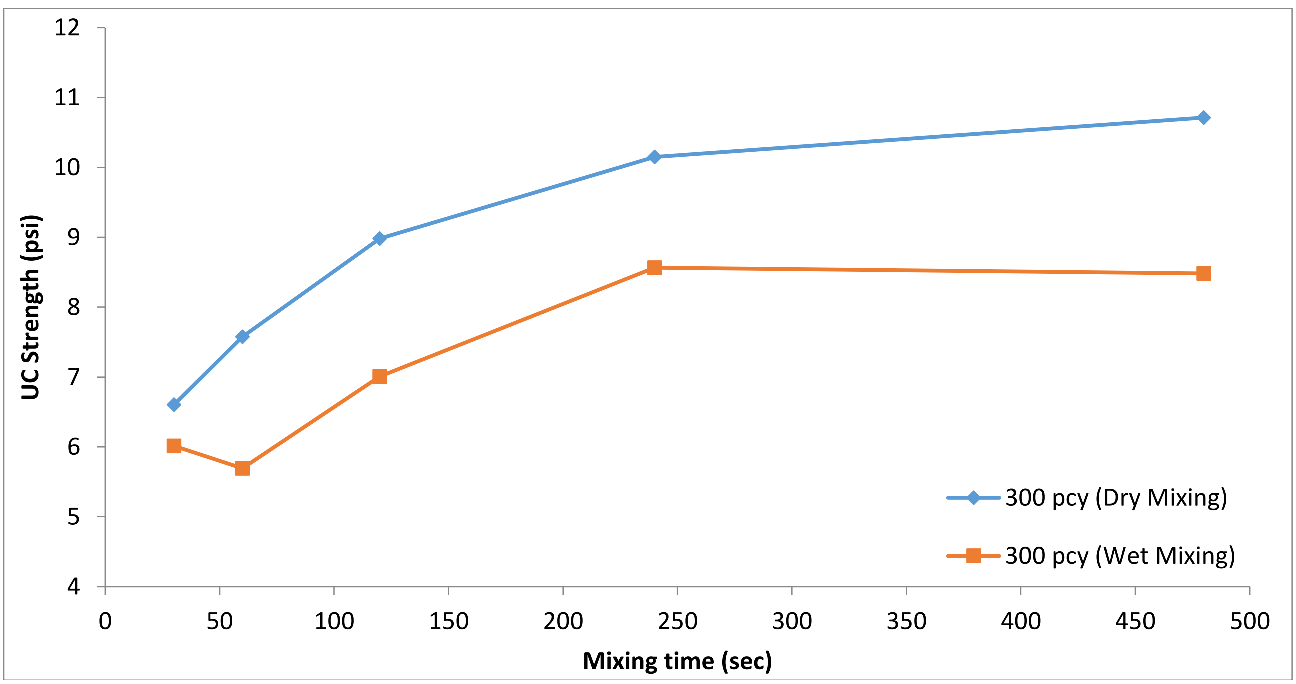

Figure 3.20. Mixing Energy – MC of 350%; 300 pcy; Dry Mixing Method and Wet Mixing Method

Table 3.7. Mixing Energy Data

Cement (pcy)	Sample	Mixing Time (sec)	Unconfined Compressive Strength (psi)
200 (Dry mixing method)	1	30	1.893
	2	60	3.130
	3	120	2.568
	4	240	3.886
	5	480	3.638
200 (Wet mixing method)	1	30	2.207
	2	60	3.620
	3	120	2.327
	4	240	3.763
	5	480	3.420

3.1.8 Varied pH Modifiers

As noted in the earlier tests with soda ash, slight increases in capacity was observed when the pH was increased above 7, but not when drastically pretreated to 9 or above. Therefore, more subtle doses of pH modifiers were used, 0 to 4 pcy and more modifiers were tried. Likewise, as the soil in situ state is difficult to maintain due to sample transport, all samples were returned to a standard moisture content established at 350% and the dry mixing method was chosen for all pH modifier trials. Figure 3.21 shows the results from the calcium oxide (lime) tests with the respective pH values and Figure 3.22 shows the use of potassium carbonate (pot ash) with the respective pH value.

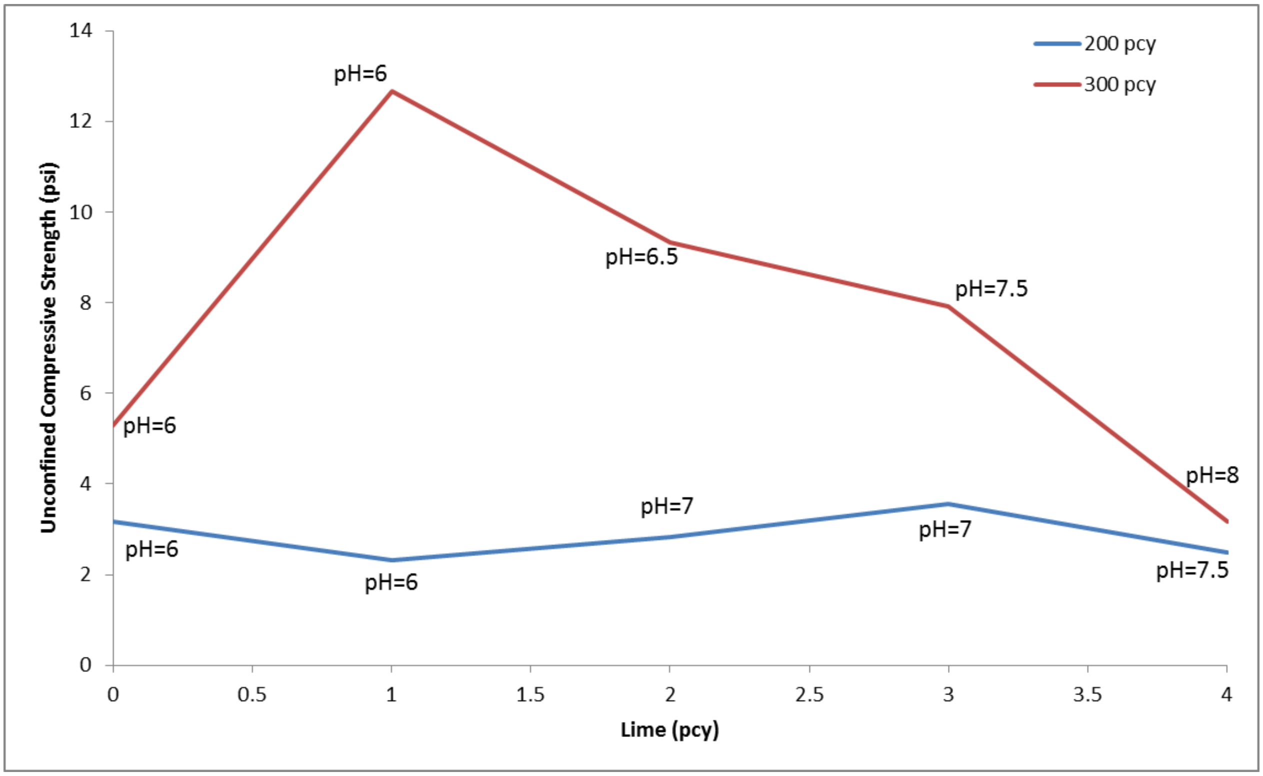

Figure 3.21. pH Modifier, Lime – MC of 350%, 200 pcy and 300 pcy of cement

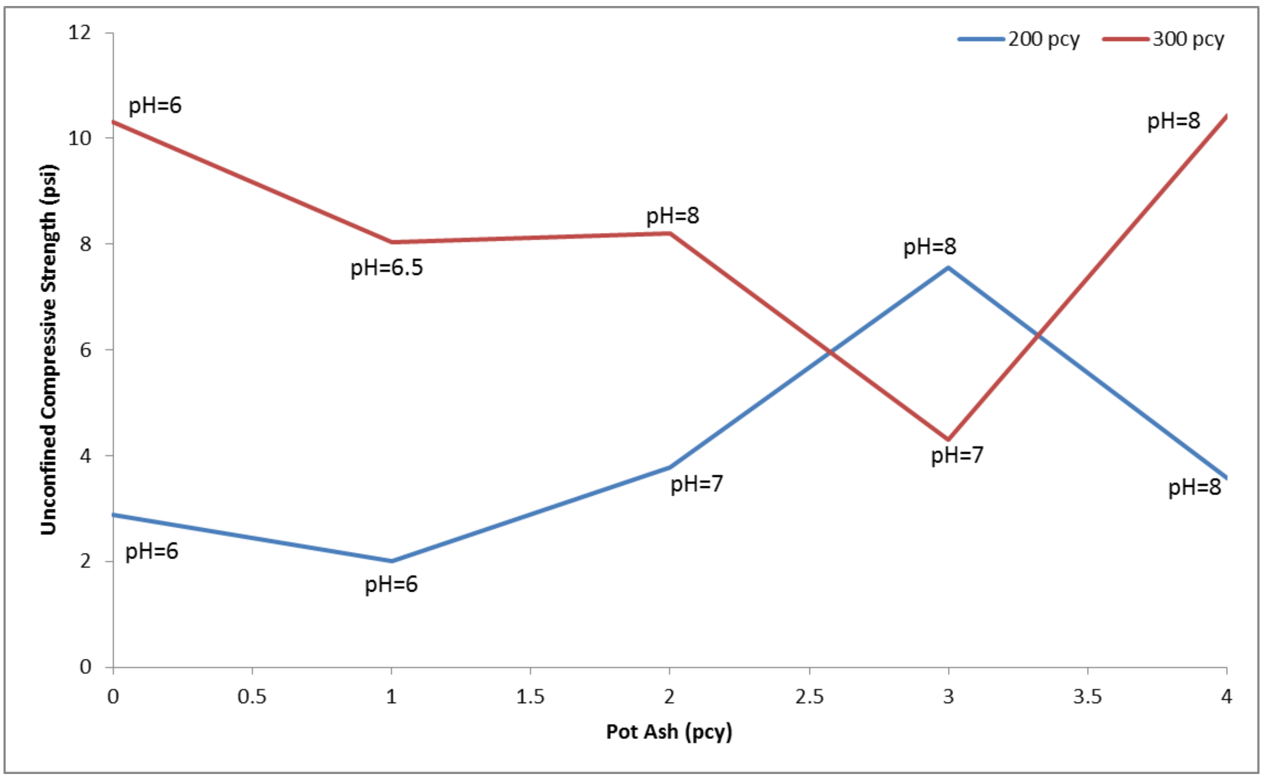

Figure 3.22. pH Modifier, Pot Ash – MC of 350%, 200 pcy and 300pcy of Cement

The addition of different pH modifiers resulted in varied results but none startlingly effective. Figure 3.23 (200 pcy of cement) shows a strength of approximately 7.5 psi with the addition of 3 pcy of pot ash. Almost the opposite holds true for the addition of 3 pcy of lime, which only reached a strength of 3.5 psi. The varying results continue in Figure 3.24 (300 pcy of cement). The addition of 1 pcy of lime gave the highest strength at approximately 13 psi, where 1 pcy of pot ash yielded approximately 8 psi of strength.

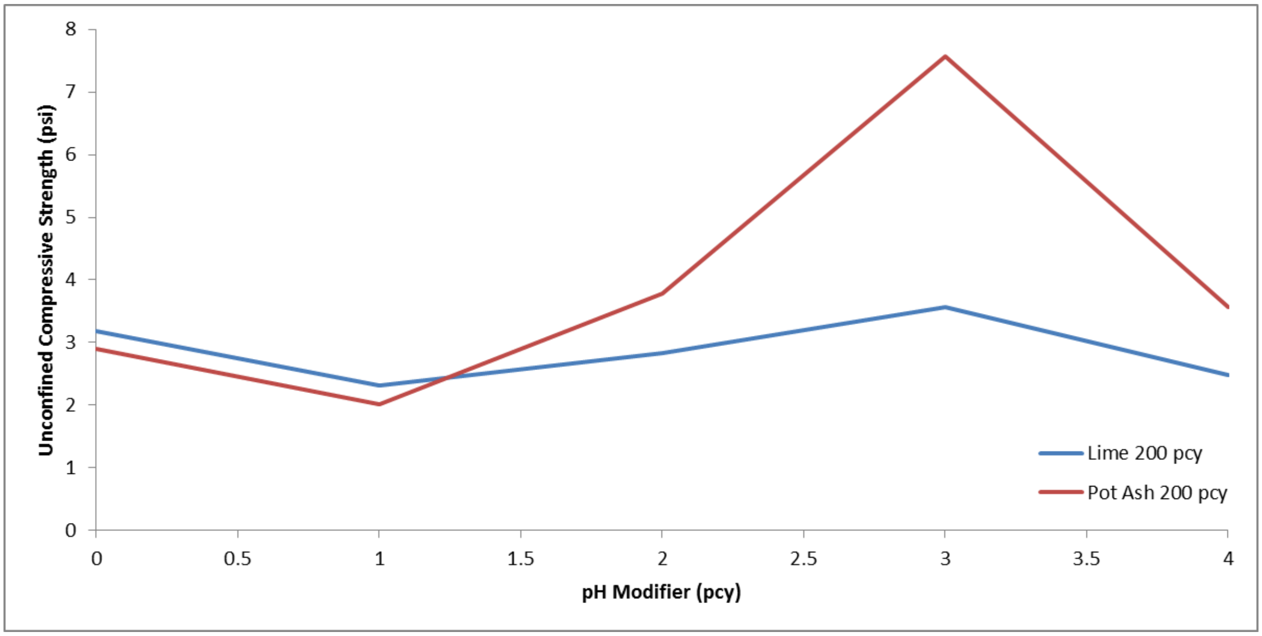

Figure 3.23. pH vs. Strength of Different pH Modifiers - 200 pcy of Cement

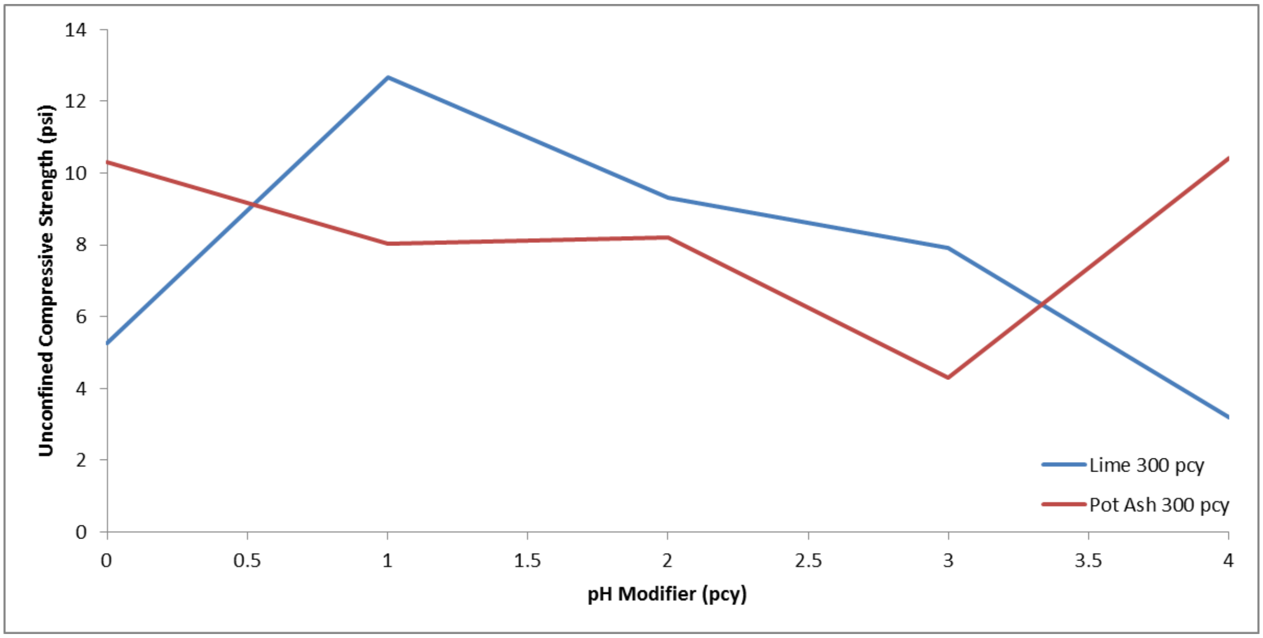

Figure 3.24. pH vs. Strength of Different pH Modifiers - 300 pcy of Cement

3.1.9 Effects of W/C Ratio

The data from the unconfined compression tests were revisited concentrating on the water to cement ratio versus strength relationship. From this observation it is hypothesized that pH may play a far lesser role than originally thought relative to the effects of water/cement ratio. Therein, simple means of increasing the solids content and thus decreasing void volume may significantly reduce the amount of cement needed.

The w/c ratio was calculated using the moisture content of the soil and the cement added for dry mixing samples. Figure 3.25 includes all data collected to date and shows the trend generated between strength and w/c ratio. The samples follow the general trend of higher strengths for lower water to cement ratios. When compared to historical data from other sources (Filz, 2010) the presence of the organics clearly has an adverse effect when compared to inorganic soils at similar w/c ratios.

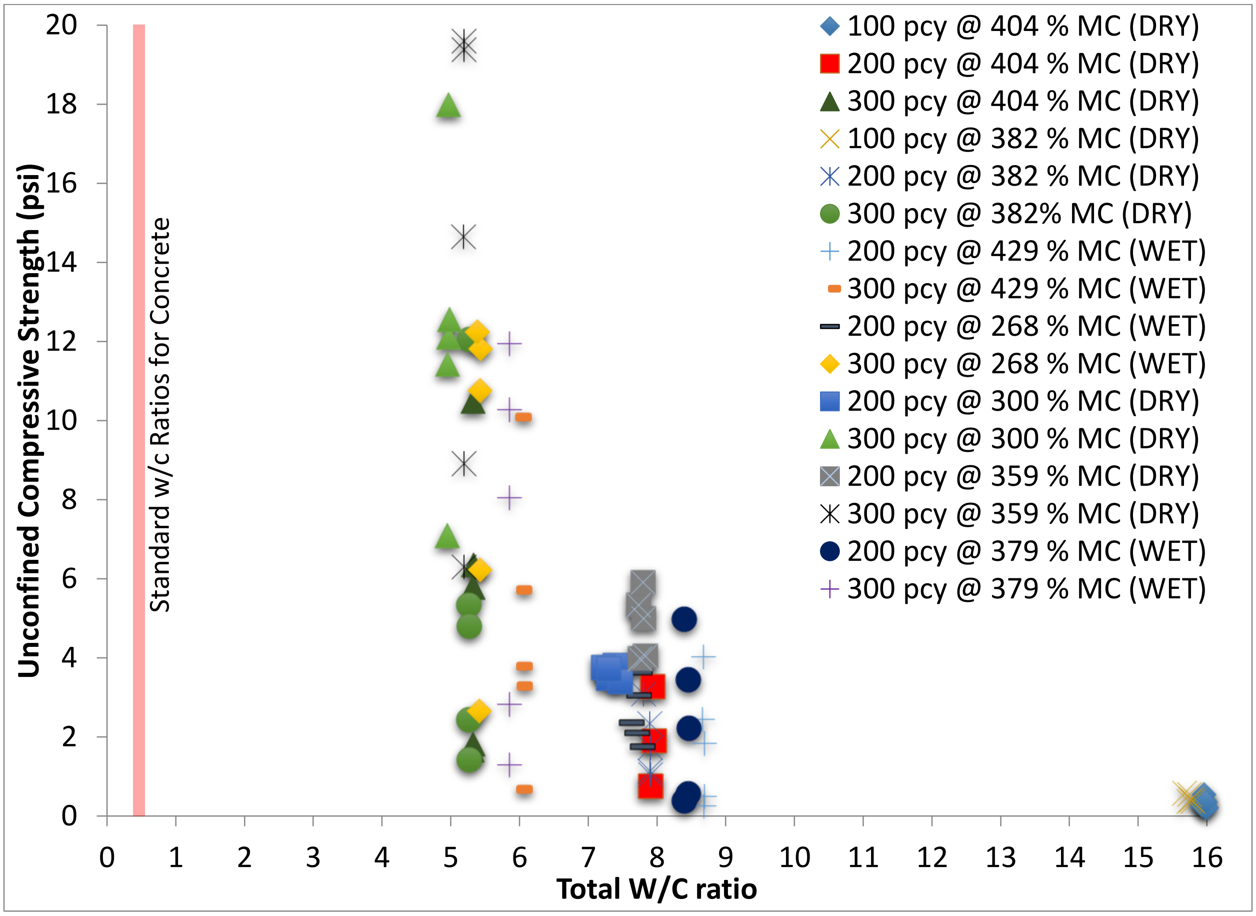

Figure 3.25. Relationship between strength and water-to-cement ratio for lab samples.

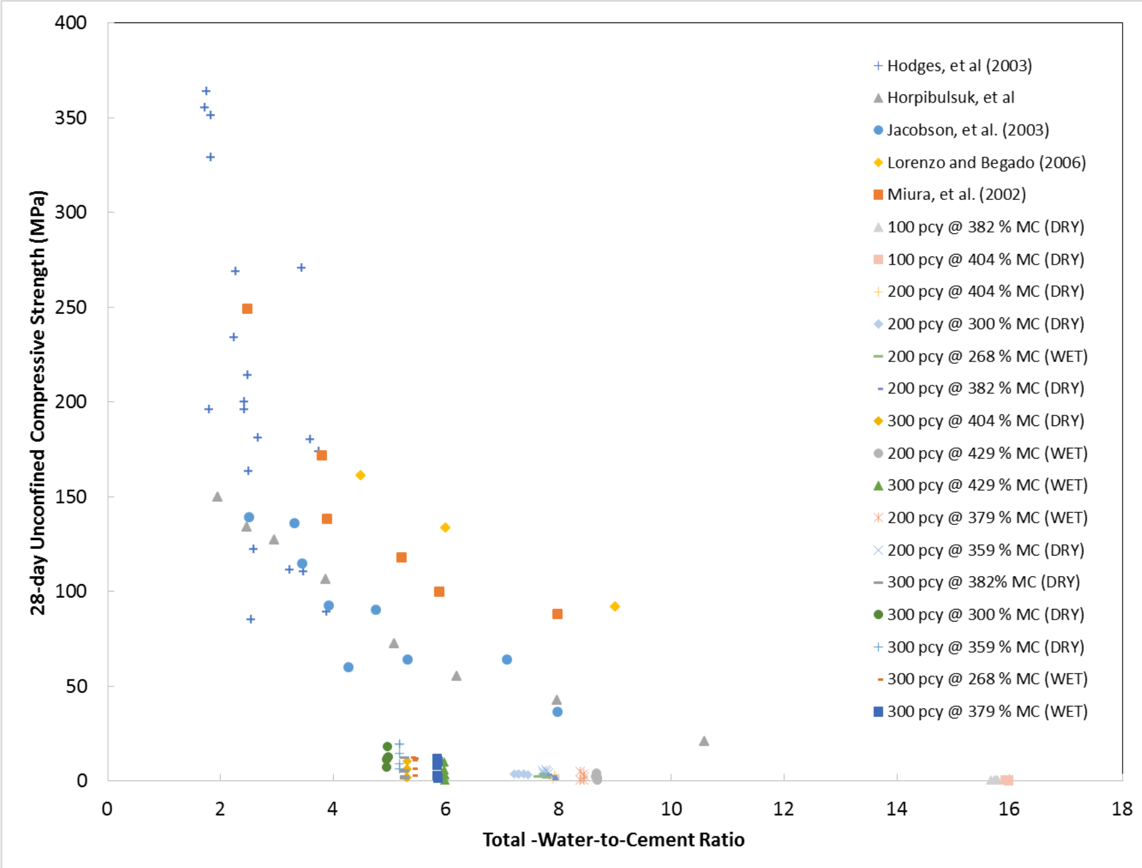

Figure 3.26. Relationship between strength and total water-to-cement ratio (inorganics and Phase 1 organics shown along the bottom of the curve less than 25 psi).

As a point of comparison, the unit volume diagrams for loose and medium dense sand (N=0 and N=30) are shown in Figure 3.27 along with the cement volume required to achieve 100 psi soil mix. These stabilization values are based on FHWA design curve (Figure 2.38). Also shown is the required cement to stabilize an organic soil to the same 100 psi strength based on lab results (OC=40%, MC=176%). Interestingly, the unit volume diagram for 4000 psi concrete (also shown) contains roughly the same amount of inorganic material (coarse and fine aggregate) as the 30 blow count sand stabilized to only 100 psi.

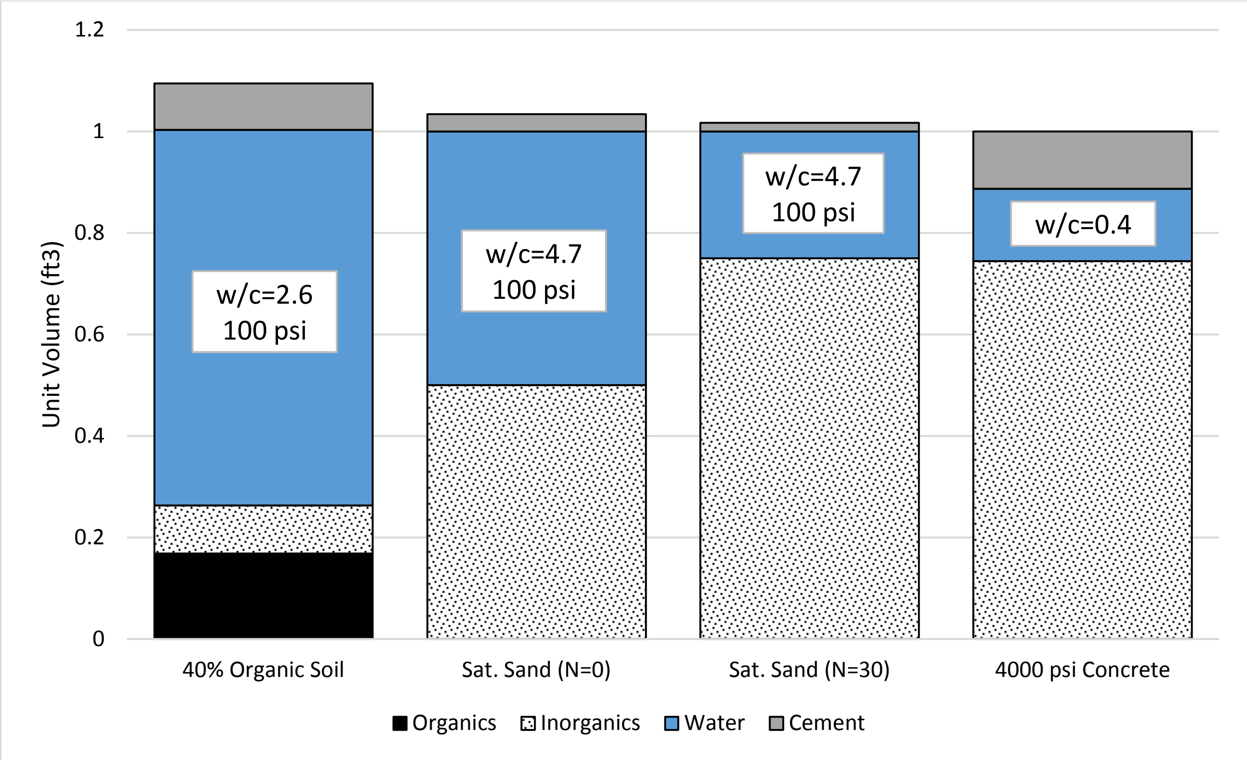

Figure 3.27. Unit volume diagram for soil and concrete (stabilization cement shown as extra volume).

Note in all cases dealing with organic soils, the tremendous water volume must be overcome to develop a reasonable w/c ratio.

3.2 Phase 2: Binder and Organic Content Variations

Phase 2 studied only the use of different binder types, binder contents, organic content and curing time on the mixed soil strength. The two binders studied were cement and slag. Based on literature citations, slag can have a favorable result in organic soil mixing. Many speculate this to be attributed to slag being more tolerant to lower pH conditions.

In addition to binder variations, the effects of organic content were investigated as well. This was done by adding back sand as an inorganic component to the 66% organic soil collected. A more detailed explanation of this is presented in the Varying Organic Content section.

With several variables involved in organic soil mixing, the testing matrix became enormous. Knowing this, the goal of Phase 2 was to create a number of mix designs that would give some insight to how the several variables involved relate to strength. Phase 2 consisted of a total of 56 different mix designs. These mix designs are occasionally referred to as batches within this report. Each mix design produced nine testing cylinders; this accounts for a total of 504 cylinders. The nine cylinders in each batch made it possible to test three at a time at three different curing durations. In general, curing durations were 14, 28, and 61 days long. These cylinders varied in cement, slag, water, and organic content as well as cure time. The cement used was Portland Type I/II. The ground granulated blast furnace slag (simply referred to as "slag" in this document) was obtained from Argos, a local concrete supplier in Tampa. In reality, even the type of cement and slag could be further subdivided but was not. Figure 3.28 shows the overall mixing matrix. Furthermore, Figure 3.29 extends this mixing matrix to include curing time; these form the overall test matrix. Each section of this matrix is discussed in detail in its respective section.

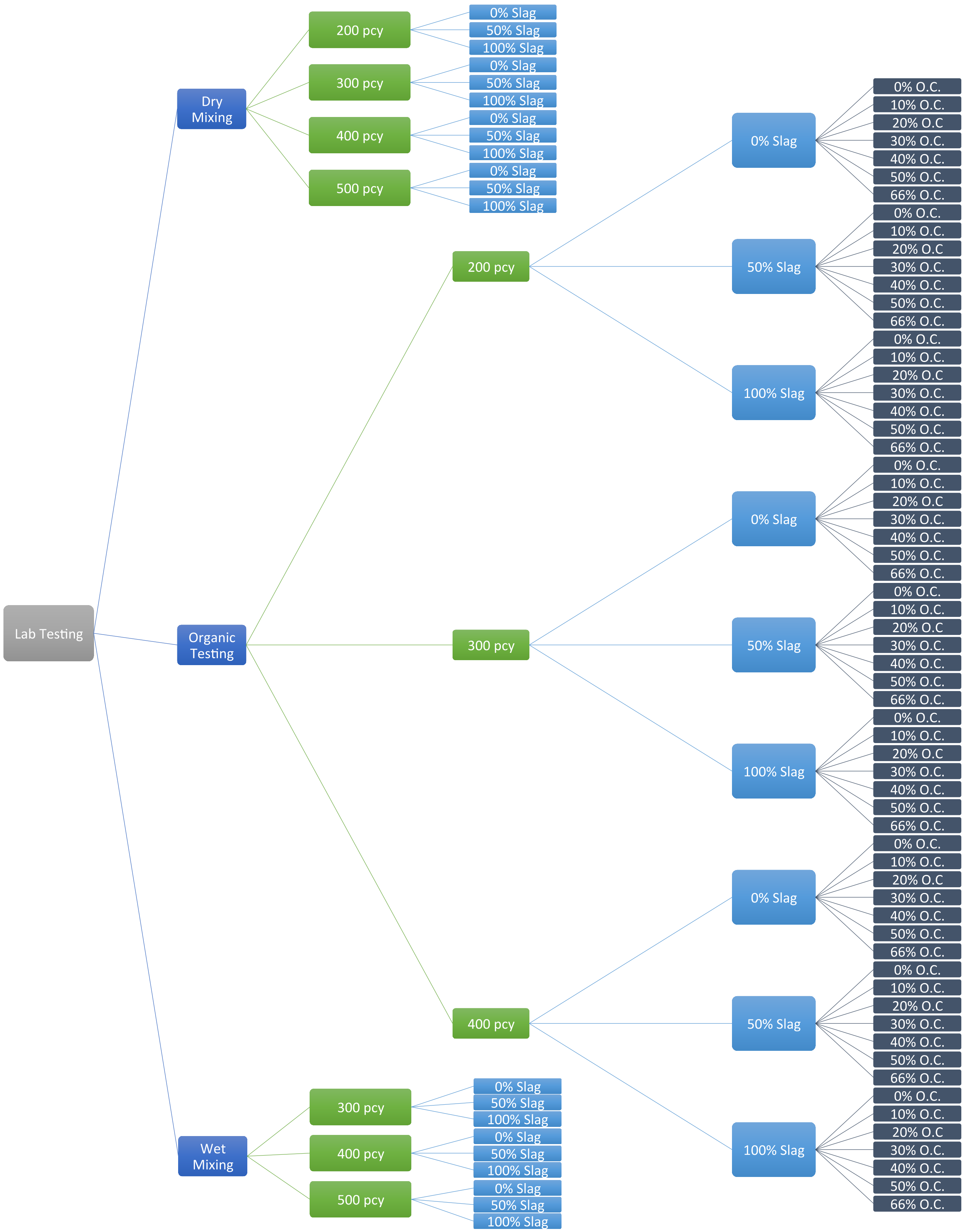

Figure 3.28. Overall mixing matrix.

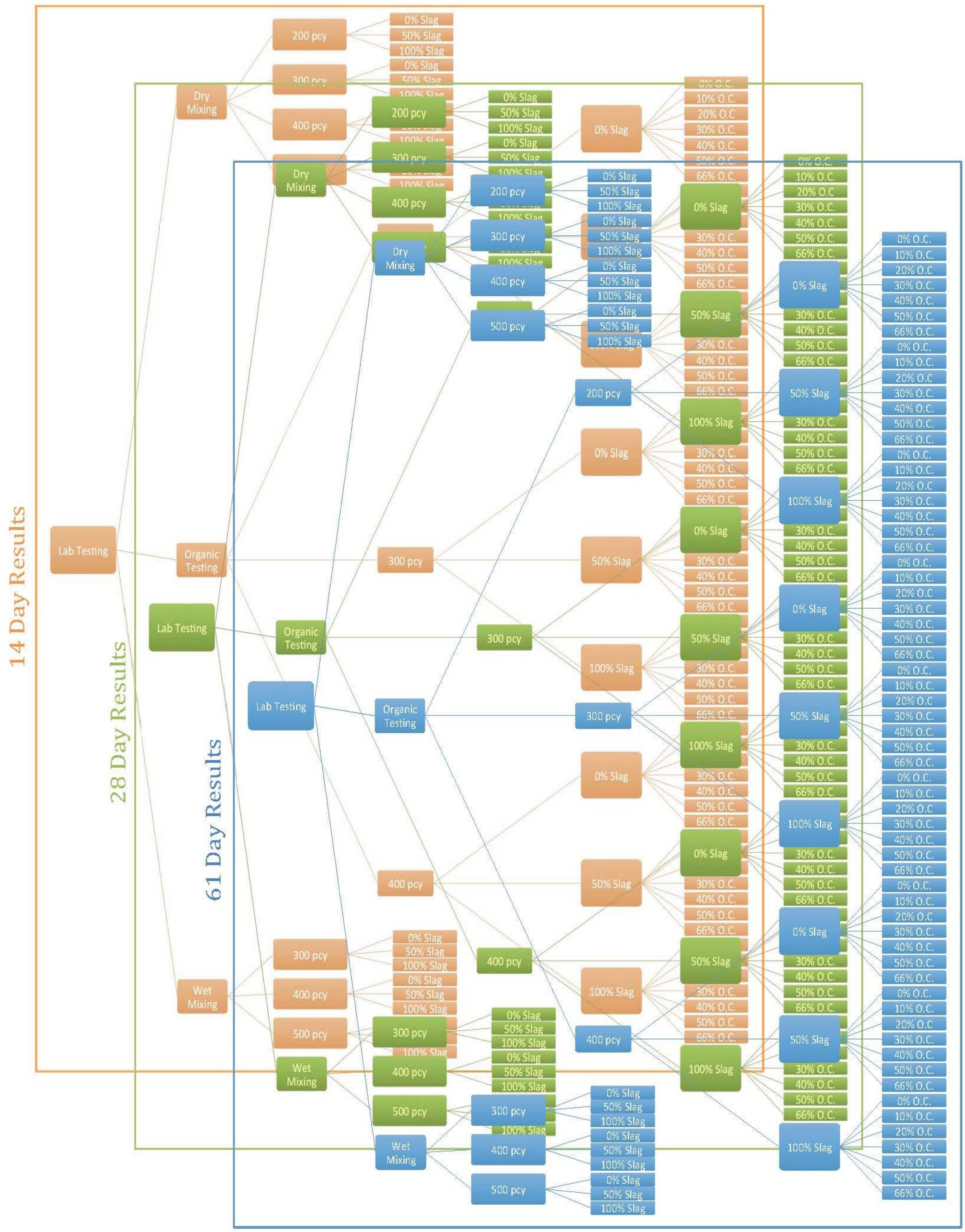

Figure 3.29. Overall test matrix.

3.2.1 Cement and Slag in Highly Organic Soils

The effects of blended binders were the first item of interest. In these experiments, the organic content was not adjusted. The variables adjusted were cement and slag content. As stated in the literature review section of this report, replacing a portion of cement with slag in concrete mixes will have some level of strength increases. The purpose of these tests was to investigate how slag replacement effects strength in highly organic soils. The soils used in this section had organic contents ranging from 42-66%.

Eleven different mix designs were used in this investigation. These eleven mixes can be broken into essentially three groups. The first group used only cement as the binder, the second group used a combination of cement and slag as the binder, and the third group used only slag as the binder. In the second group, slag accounted for 50% of the binder by mass; this is denoted as 50% slag replacement. The mixes varied in binder amount from 200 to 500 pcy. Figure 3.30 displays this series of tests as a branch off the overall test matrix.

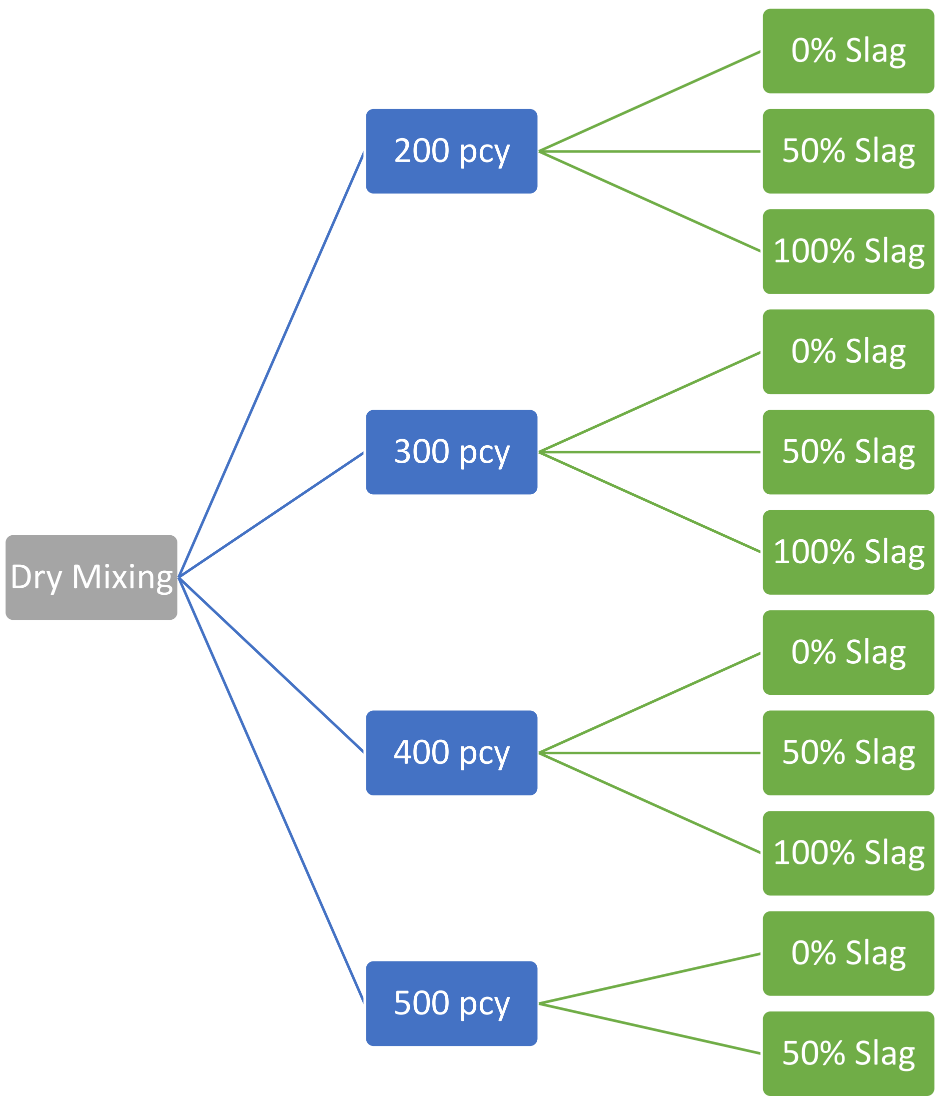

Figure 3.30. Test matrix for dry mixing.

It should be noted that a 100% slag replacement mix design in missing from Figure 3.30. This is due to the fact that the 100% slag replacement mixes created before the 500 pcy mix was added proved to have no strength and could not be removed from the cylinder mold. Therefore only 0% and 50% slag replacement batches were prepared for the 500 pcy dry mixing section.

3.2.2 Varying Organic Content

In addition to the effects of blended binders, the effect the organic content of a soil on strength also evaluated. Data from 45 different mix designs were used in this series of tests. This included 39 additional mix designs created specificity for this investigation. These mix designs primarily varied in organic content, but they also varied in cement and slag content. This series made up a majority of the test matrix, and a branch of test matrix is provided in Figure 3.31 below for reference. While Figure 3.31 represents the 300 pcy branch, it should be known that there also exist similar branches for 200, 400 and 500pcy.

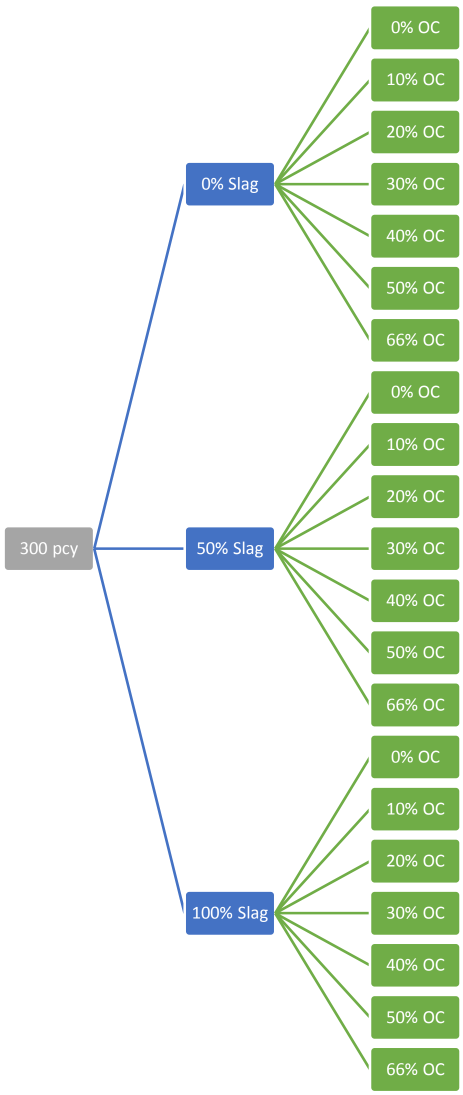

Figure 3.31. Example test matrix for 300-pcy binder content showing further subsets based on organic content, % OC

Referring to Figure 3.31, 0% OC mix designs used only sand and no organic soil. The 66% OC was used as the upper limit due to the fact that it was the highest organic content available. OC values of 10, 20, 30 40, and 50% OC were the target values between the upper and lower limits of 0% and 66%, where the organic content of the soil was adjusted by adding sand. The actual organic content of each mix was calculated and documented.

3.2.3 Wet Mixing

The main difference between wet mixing and previously described dry mixing is that the binder was pre-mixed with water, or hydrated, before being mixed with the soil. The water present within organic soils was typically more acidic due to the chemical effects of the organic soil. Mixing the binder with pH-7 water may then be less prone to adverse effects. However, wet mixing involved adding additional water to a system that already had a high amount of water. This resulted in very high w/c ratios, which yielded weaker strengths.

Six mix designs were created for this series of tests. It was decided to use binder contents of 300, 400, and 500 pcy as opposed to 200, 300, 400, and 500 pcy, used in dry mixing. The 200-pcy mixes were dropped in the wet mixing investigation due to their dry mixing counterpart producing very little strength. Figure 3.32 provides the wet mixing section of the test matrix.

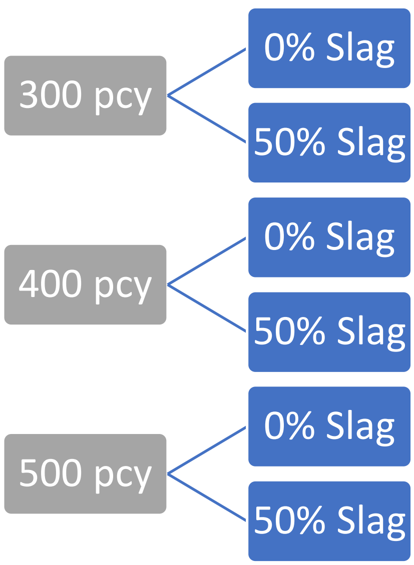

Figure 3.32. Test matrix for wet mixing showing binder variations.

3.2.4 Mixing Procedures

While the mixing procedure in Phase 2 is similar to that of Phase 1, it is necessary to address the differences. Additionally, it is helpful to have a standalone section in this chapter dealing with each mixing procedure. In Phase 2, there are three mixing procedures: Dry Mixing, Varying Organic Content Mixing, and Wet Mixing. While they are similar, they are each distinct. Therefore, for completeness and clarity, all three procedures are provided.

General modifications from the mixing process in Phase 1 are as follows:

1. A larger mixer was used. This was done to easily create enough soil for nine cylinders in each batch. See Figure 3.33.
2. 3 x 6 in. cylinders were used instead of 4 x 8 in. This was also done for the purpose of being able to obtain more samples out of each batch. See Figure 3.34.

Figure 3.33. Larger mixer used in Phase 2.

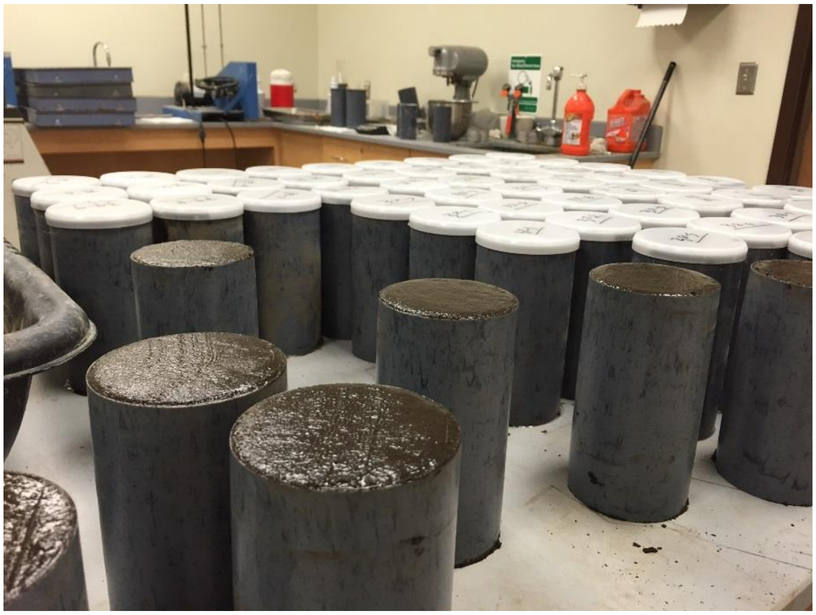

Figure 3.34. Prepared 3 x 6 in. cylinders.

3.2.4.1 Dry Mixing Procedure

The dry mixing procedure applies for the eleven mix designs used in the initial Cement and Slag in Highly Organic Soils section. This is also known as batches 1-9 and 55-56. Dry mixing means that a dry binder was added directly to the soil.

1. Calculate the amount of materials needed for nine 3 inch by 6 inch cylinders: soil, cement, and slag.
2. Mix the raw soil alone for approximately 4 minutes in the large mixer.
3. Measure pH.
4. Take small samples to calculate moisture content. See Section 3.1.2.1.
    - These samples may then be used the following day to calculate the organic content. See Section 3.1.2.2.
5. Add dry binder. This is either cement or slag or both. Then mix together for 4 minutes.
6. Measure pH.
7. Place mixed soil into nine 3 x 6 in. cylinders. This was done in three layers. As opposed to traditional tamping, the cylinders were moderately taped on the table. This was done to remove air voids while avoiding over compacting the soil.

3.2.4.2 Varying Organic Content Mixing Procedure

This procedure applied to the 39 additional mix designs that were used in the Varying Organic Content section. This accounts for batches 10-48.

1. Calculate the amount of materials needed for nine 3 inch by 6 inch cylinders: soil, cement, slag, water, and sand.
2. Mix the raw soil alone for approximately 4 minutes in the large mixer.
3. Measure pH.
4. Add calculated amounts of water and sand and mix for approximately 4 minutes.
5. Measure pH.
6. Take small samples to calculate moisture content. See Section 3.1.2.1.
    - These samples may then be used the following day to calculate the organic content. See Section 3.1.2.2.
7. Add dry binder. This is either cement or slag or both. Then mix together for 4 minutes.
8. Measure pH.
9. Place mixed soil into nine 3 x 6 in. cylinders. This was done in three layers. As opposed to traditional tamping, the cylinders were moderately taped on the table. This was done to remove air voids while avoiding over compacting the soil.

3.2.4.3 Wet Mixing Procedure

This procedure applied to the six mix designs used in the wet mixing section. This accounts for batches 49-54.

1. Calculate the amount of materials needed for nine 3 inch by 6 inch cylinders: soil, cement, slag, and water.
2. Mix the raw soil alone for approximately 4 minutes in the large mixer.
3. Measure pH.
4. Take small samples to calculate moisture content. See Section 3.1.2.1.
    - These samples may then be used the following day to calculate the organic content. See Section 3.1.2.2.
5. In a separate container, mix the calculated amounts of binder and water with a high energy mixer for approximately 4 minutes, or until thoroughly mixed.
6. Introduce the mixed binder and water to the soil and mix for together for 4 minutes.
7. Measure pH.
8. Place mixed soil into nine 3 x 6 in. cylinders. This was done in three layers. As opposed to traditional tamping, the cylinders were moderately taped on the table. This was done to remove air voids while avoiding over compacting the soil.

3.2.5 Results

As performed in Phase 1, the Material Testing System (MTS) machine was used to determine the unconfined compressive strength of each cylinder. A sample being tested is shown in Figure 3.35. Cylinders were tested three different curing durations. The first two were typically 14 and 28 day curing times. The third curing duration, as opposed to 56 days, was 62, or in some cases 61, due to schedule conflicts. In cases were cylinders were deemed untestable due to low strengths, some cylinders were withheld from testing for testing at a longer cure time.

Figure 3.35. Unconfined compression testing of 3 x 6 in. cylinders.

To account for changes in mass, specifically water losses, cylinders were weighed twice. They were weighed once directly after mixing and again right before testing. At largest, the drop in mass was 1.06%, and the average loss in mass was only 0.42%.

3.2.5.1 Cement vs. Slag in Highly Organic Soils

In this chapter, highly organic soils were considered anything with an organic content higher than 40%. It was intended that the 0, 50, and 100% slag replacement mix designs would produce enough data to calculate an optimum slag replacement value at a given binder content. However, cylinders containing 100% slag replacement were extremely weak and considered untestable. In other words, they were not able to be taken out of the cylinder molds without falling apart.

While in contrast with literature, the data from the 0 and 50% slag replacement revealed that mixes with only cement as the binder clearly outperformed those with slag replacement. This trend may be seen in Figure 3.36 and Figure 3.37. In these two figures, each data point represents the average of three tests.

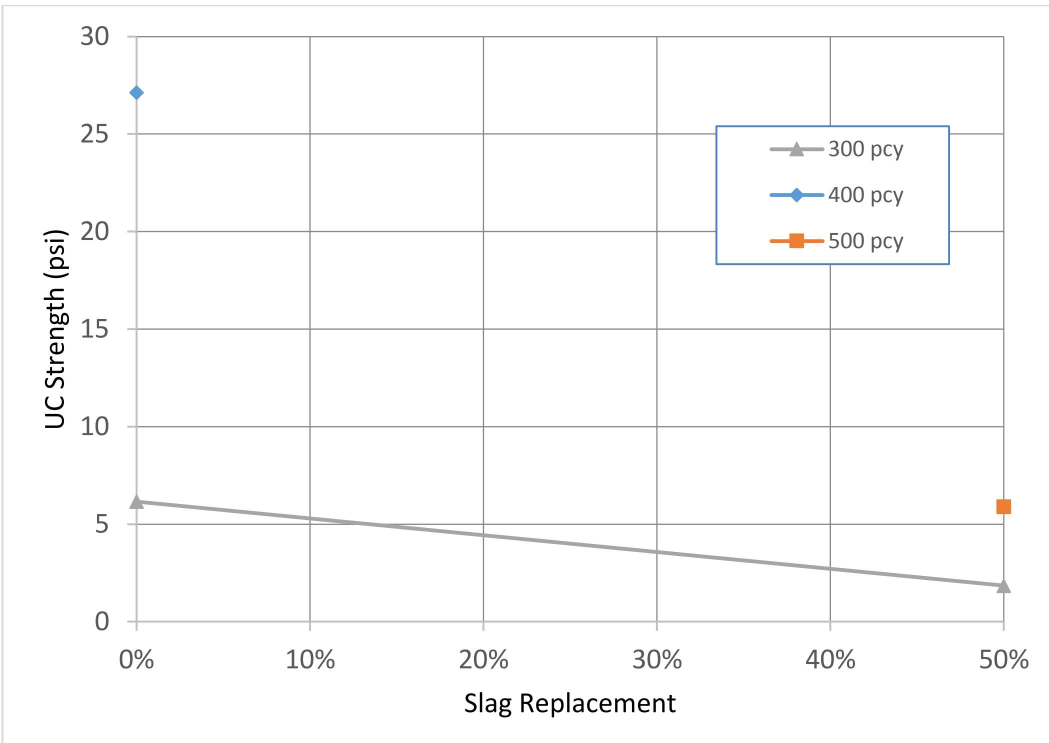

Figure 3.36. Slag replacement vs. 28-day strength in highly organic soils.

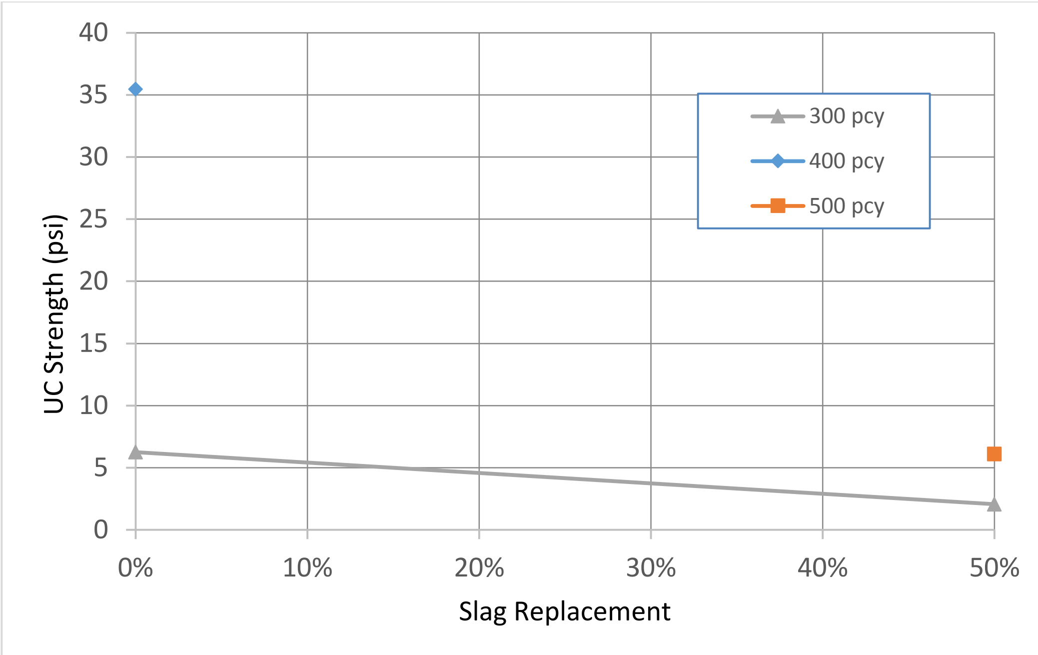

Figure 3.37. Slag replacement vs. 61- to 62-day Strength in highly organic soils.

This data may also be presented in terms of binder content vs strength. Figure 3.38 displays this using the raw data points, and Figure 3.39 uses the average of three raw data points.

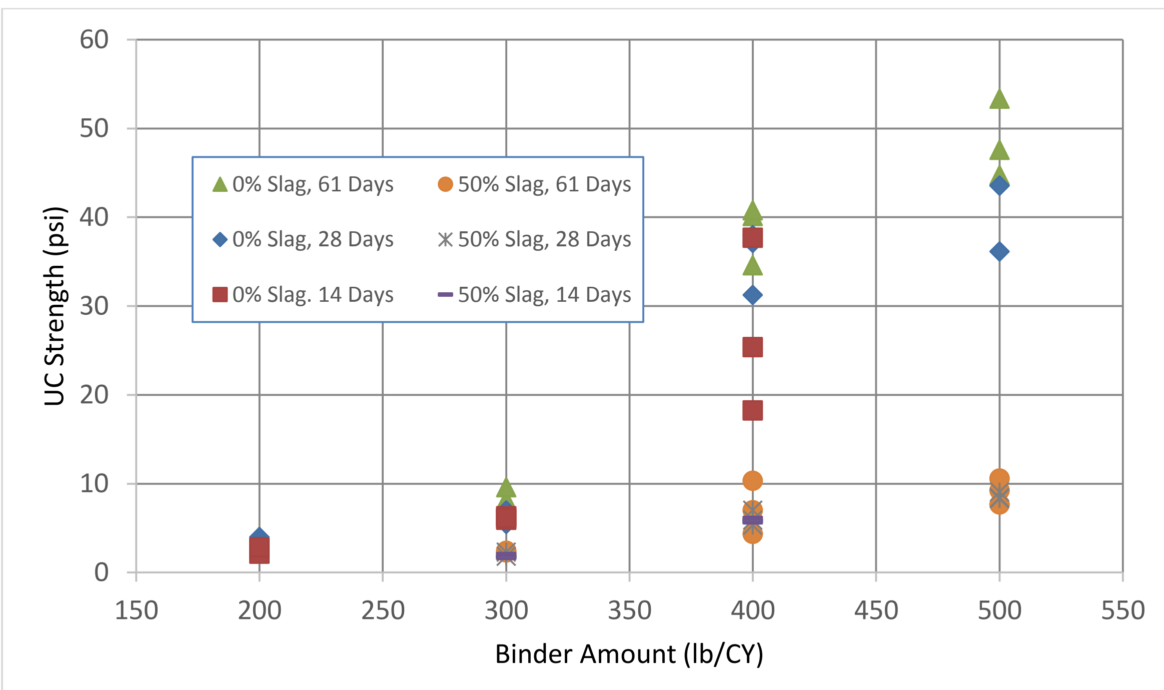

Figure 3.38. Binder amount & type vs. strength in highly organic soil (raw data).

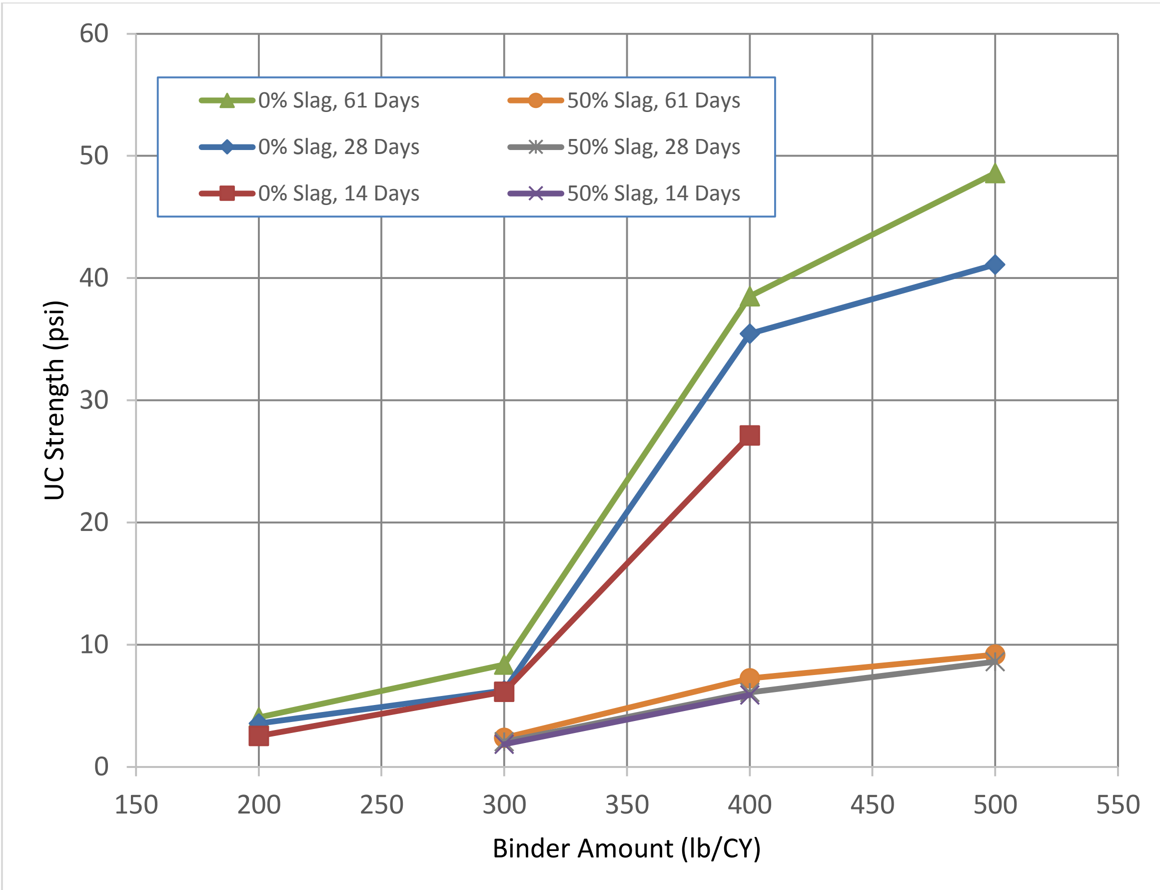

Figure 3.39. Binder amount & type vs. strength in highly organic soil (average).

Many observations can be drawn from Figure 3.39. First of all, mixes with 50% slag replacement produced considerably lower strengths than their 0% counterpart. This begs the question, "how much of an effect does slag actually have on strength?" The data suggest that slag may contribute to strength in some degree, just not more than cement.

As an example, the 400 pcy mix with 50% slag replacement contains 200 pcy of cement. If the slag did not contribute to strength at all, it would be expected that the results of the 400 pcy mix with 50% replacement would have similar results to that of the 200 pcy mix with 0% slag replacement. However at 28 and 61 days of cure time, the strength increased from 3.55 psi to 6.10 psi and from 4.06 psi to 7.25 psi, respectively. These are gains of 72% and 79%, respectively. These gains are the product of some binding effect in the slag, or they may just purely be a product of less organic soil per unit of volume.

While adding slag to a mix in addition to a fixed amount cement increases strength, the strength is considerably higher the same amount of cement is added instead. As an example and referring to 0% slag replacement mixes, there is an 899% increase in unconfined compressive strength from 200 pcy to 400 pcy. Once again focusing on the 0% slag replacement mixes, qualitatively, there is very little gains in strength from 200 pcy to 300 pcy; however there is a considerable increase in strength from 300 pcy to 400 pcy. Then there is once again a smaller increase from 400 pcy to 500 pcy. This leaves Figure 3.39 with somewhat of an "S" shape. Considering the fact that the 500 pcy samples had lower moisture contents than the 400 pcy mixes, one might expect the 500 pcy to be significantly stronger than the 400 pcy mixes; however this is not the case. While there are gains in strength, they are not as significant as expected. This information is presented quantitatively in Table 3.8 and Table 3.9. Table 3.9 also contains the same data that is plotted in Figure 3.39.

Table 3.8. Increases in 28-Day strength in dry mixing of highly organic soils.

Amount of Cement	Increase in 28-Day Strength (psi)	Increase in Strength (%)
200 → 300	2.71	76%
300 → 400	29.20	467%
400 → 500	5.65	16%

Table 3.9 Binder amount and type vs. strength in highly organic soil.

Slag Replacement	Batch	lb/yd³	w/c	Moisture	% OC	14 Stress (psi)	28 Stress (psi)	61 Stress (psi)
0%	4	200	6.90	362%	66.4%	2.55	3.55	4.06
	8	300	4.22	221%	65.9%	6.15	6.26	8.37
	9	400	3.08	216%	65.9%	27.12	35.46	38.52
	55	500	2.11	154%	42.1%	N/A	41.11	48.59
50%	1	200	6.88	362%	66.4%	N/A	N/A	N/A
	2	300	4.49	362%	66.4%	1.85	2.07	2.37
	3	400	3.29	362%	66.4%	5.90	6.10	7.25
	57	500	2.09	154%	42.1%	N/A	8.61	9.18

It should be noted that in Table 3.9, or anywhere else within this chapter, the w/c ratio provided is considered the w/c of the entire system. In other words, it is the ratio of all of the water present in the system divided by all of the binder present within the system. This includes all of the water present in the soil before introducing any binder.

3.2.5.2 Varying Organic Content

Testing compressive strengths in soils with various organic contents proved to produce some of the most significant findings of this report. As expected, strength increased as organic content decreased. Fourteen day compressive strength data taken at various organic contents are shown in Figure 3.40. Similarly, the 28 day and 61-62 day compressive strength data are shown in Figure 3.41 and 3.42, respectively. In order to more easily observe any trends, Figures 3.43, 3.44, and 3.45 present the same data but exclude the 0% OC data points; this provides a zoomed in version of their counterpart graph. This data is also provide in a tabular form in Table 3.10. Every data point shown in Figures 3.40-3.45 and listed in Table 3.10 represents the average of 2 or 3 data points.

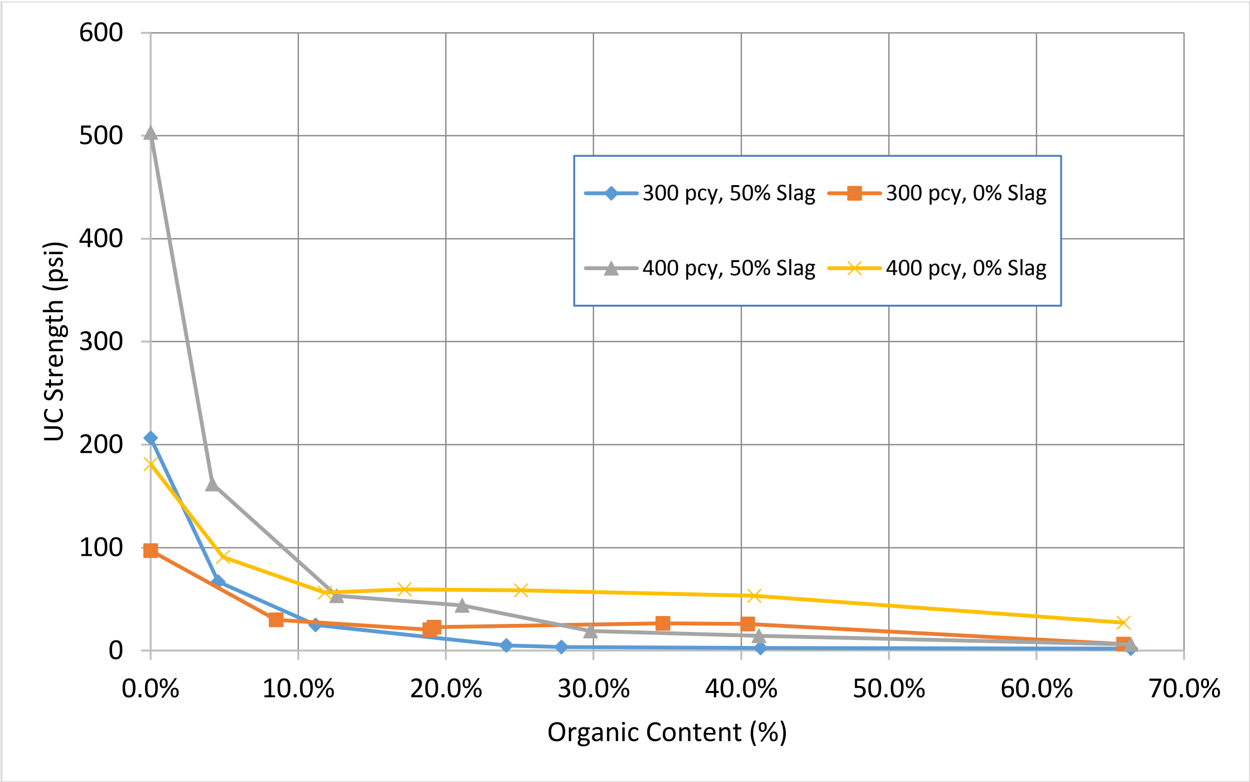

Figure 3.40. Organic content vs. 14 day strength.

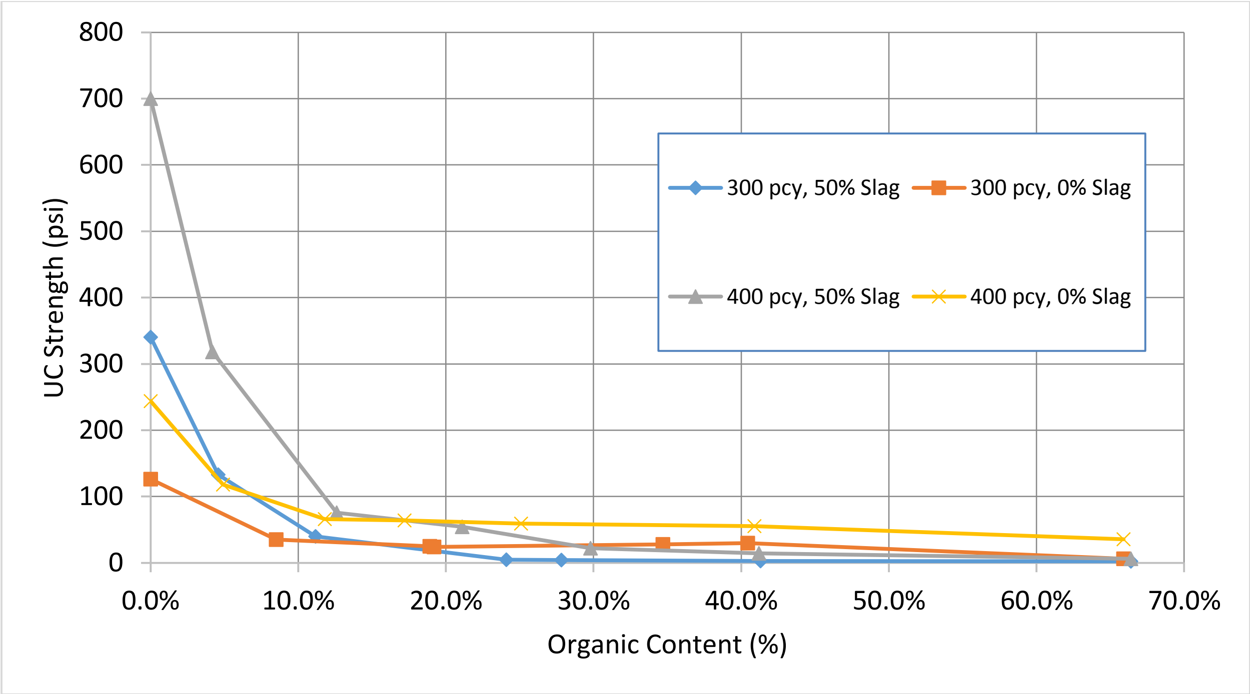

Figure 3.41. Organic content vs. 28 day strength.

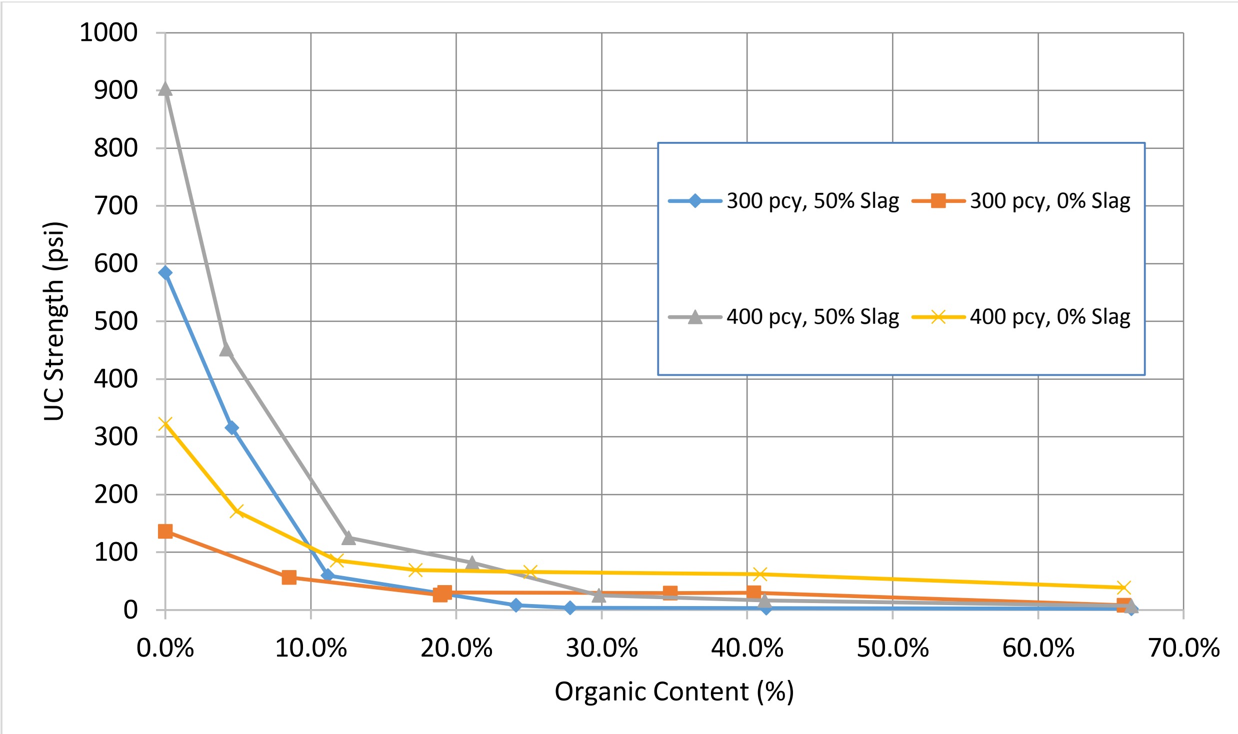

Figure 3.42. Organic content vs. 61- to 62-day strength.

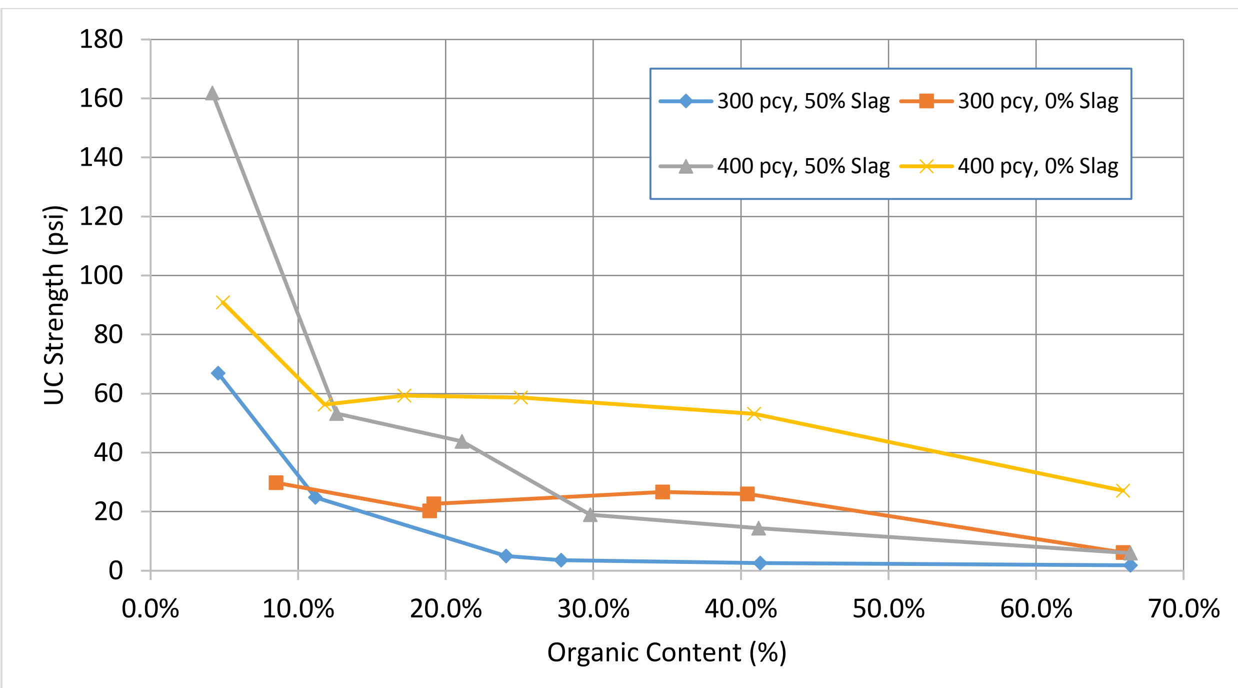

Figure 3.43. Organic content vs. 14-day strength (zoom).

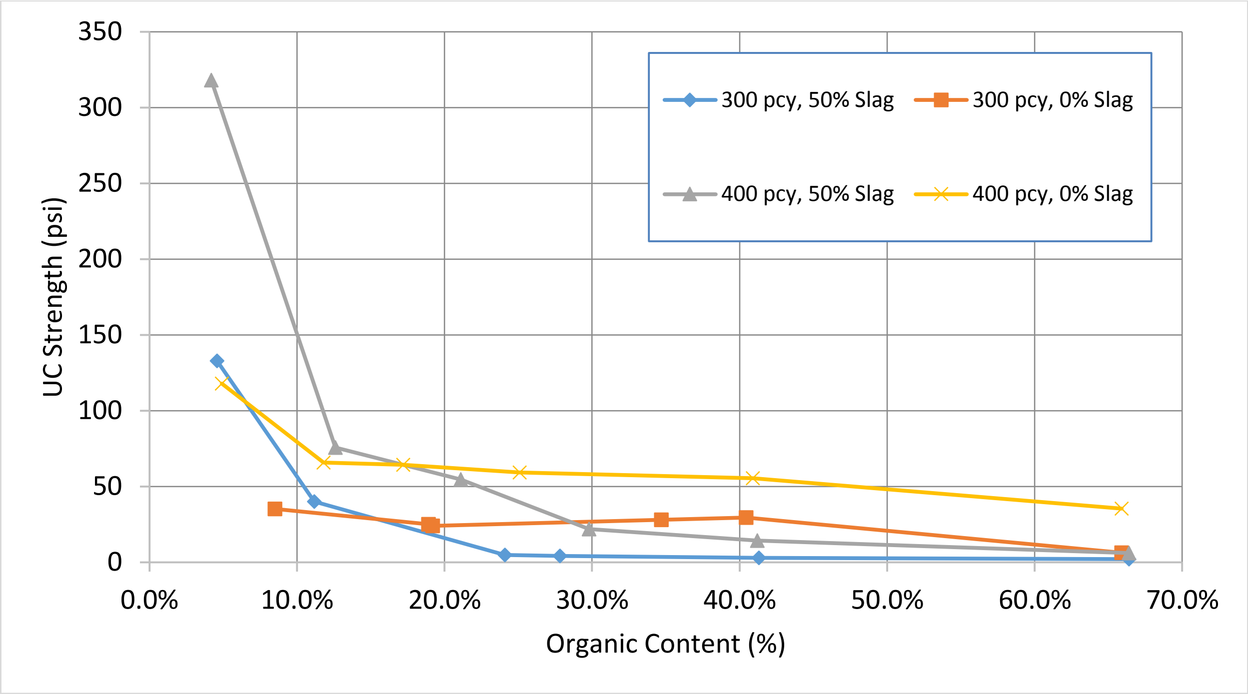

Figure 3.44. Organic content vs. 28-day strength (zoom).

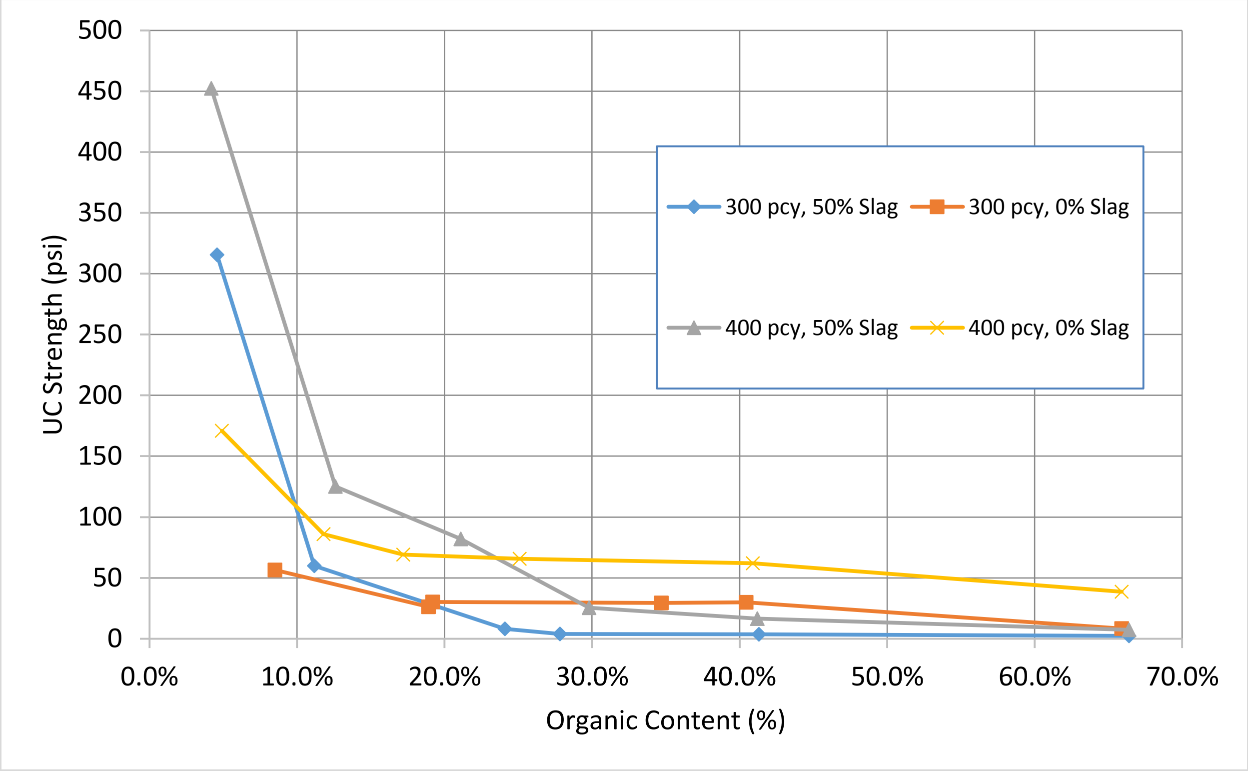

Figure 3.45. Organic content vs. 61- to 62-day strength (zoom).

Table 3.10 Batch information and strengths at various organic contents.

Batch	OC	Binder Content	Slag Replace	w/c	14 Day Strength (psi)	28 Day Strength (psi)	61-62 Day Strength (psi)
2	66.4%	300	50%	4.49	1.85	2.07	2.37
10	41.3%			3.86	2.53	2.90	3.57
11	27.8%			3.52	3.57	4.26	3.8
12	24.1%			3.38	4.96	4.86	8.06
13	11.2%			3.00	24.80	40.06	59.94
14	4.6%			2.50	66.88	132.80	315.56
48	0.0%			2.50	206.51	340.28	584.24
8	65.9%	300	0%	4.22	6.15	6.26	8.37
15	40.5%			3.84	25.99	29.57	29.9
16	34.7%			3.70	26.61	27.93	29.32
17	19.2%			3.42	22.62	23.99	30.19
18	18.9%			3.26	20.26	25.10	26.18
19	8.5%			2.90	29.85	35.17	56.53
46	0.0%			2.49	96.96	126.42	136.31
3	66.4%	400	50%	3.29	5.90	6.10	7.25
25	41.2%			2.88	14.40	14.32	16.59
26	29.8%			2.70	19.00	21.82	25.44
27	21.1%			2.54	43.84	54.53	82.08
28	12.6%			2.30	53.21	75.59	125.02
29	4.2%			1.97	161.89	318.12	452.23
47	0.0%			1.86	503.18	699.98	903.41
9	65.9%	400	0%	3.08	27.12	35.46	38.52
34	40.9%			2.96	53.15	55.33	62.05
33	25.1%			2.70	58.63	59.31	65.68
32	17.2%			2.52	59.33	64.18	69.2
31	11.8%			2.35	56.25	65.75	85.87
30	4.9%			2.09	90.86	117.89	170.97
45	0.0%			1.83	181.06	243.64	322.2

Observation #1: The Point at which Slag Replacement is Beneficial. As discovered in section 4.5.1, mixes with 0% slag replacement out-performed the 50% slag replacement mixes in soils with organic contents greater than 42%. In fact Figure 3.44 suggests that in 400 pcy at 28 days, slag replacement only increases strength if the organic content is less than 18%. However when cure time was increased to 61 days, 50% slag replacement out performed 0% slag replacement if the organic content was less than 25%. This point is shown where the lines for 0% slag replacement and 50% slag replacement for a given binder content intersect. Therefore as cure time increased, the benefits of slag replacement applied to a higher range of organics. This is shown in Table 3.11 and Figure 3.46. It could be concluded that at any given cure time, slag replacement was most beneficial in soils with lower organic contents, and binders made of only cement were more beneficial in soils with higher organic contents.

Table 3.11 When slag replacement is beneficial in organic soils.

Cure Time (Days)	Point Where Slag Replacement Was Beneficial in 300 pcy Mixes	Point Where Slag Replacement Was Beneficial in 400 pcy Mixes
14	< 11% OC	< 12% OC
28	< 16% OC	< 18% OC
61-62	< 19% OC	< 25% OC

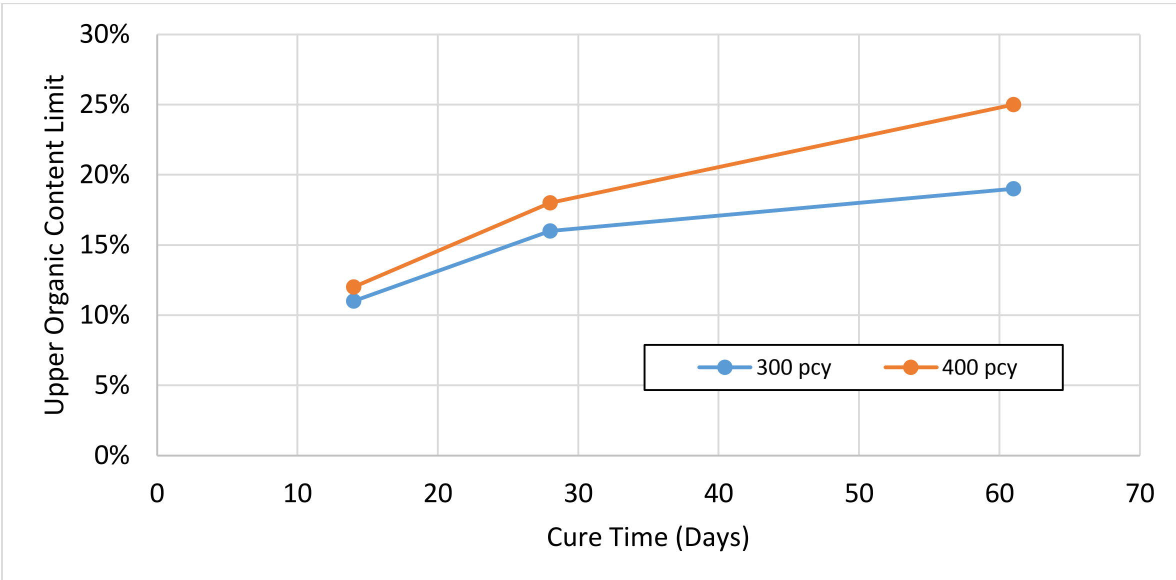

Figure 3.46. When slag replacement is beneficial in organic soils.

The strength vs. time plot is shown for this series of tests in Figure 3.47. In this figure, the point where a 50% slag replacement line intersects with a 0% slag replacement line of similar %OC reveals at what cure time slag replacement is beneficial. If the slag is helpful, it occurs later on.

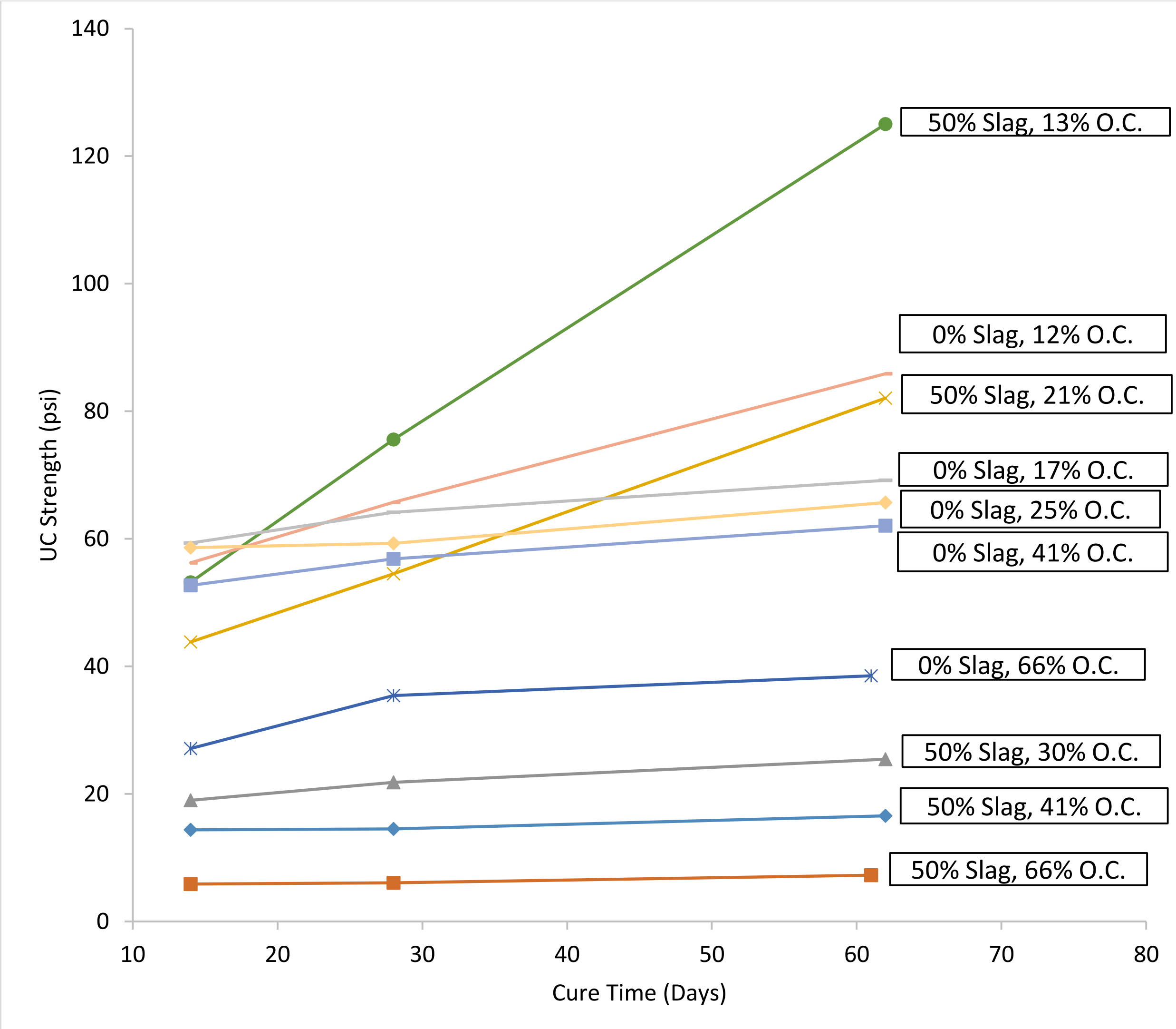

Figure 3.47. Strength vs. time in 400 pcy mixes.

As seen in Figure 3.47, longer cure times are needed for slag replacement to be beneficial at higher organic contents. However at organic contents over 25%, 50% slag replacement batches never had higher strengths than the 0% replacement batches. It should be noted that this is based on the longest cure time of 62 days. Additional testing at longer cure times may reveal cure times where slag replacement will be beneficial for higher organic contents.

<u>Observation #2: Organic Content is Irrelevant for a Particular Range.</u> This trend is only appears for mixes without slag. Considering Figures 3.40 – 3.42, there is a range where strength appears to be somewhat independent of organic content. This is seen in the sections of the graphs where strength vs. OC curves become relatively horizontal. At 61-62 days of cure time, this range is shown between 19 and 41% OC Strengths only change by 7.15 psi, or 11.5%, within this range for 400 pcy mixes and by 0.87 psi, or 2.97%, for 300 pcy mixes. Even though w/c ratios within this range vary by approximately 0.5, the data suggest this variation has little impact on strength.

3.2.5.3 Wet Mixing

Out of the six wet mixes created, only the 0% slag replacement set was deemed testable. Figure 3.48 shows the results obtained from wet mixing. The markers in Figure 3.48 represent each cylinder test and the lines represent an average; in all other figures, both markers and lines refer to averages of two or three data points. Notable information about this data may be found in Table 3.12. Figure 3.49 compares these results to their dry mixing counterpart.

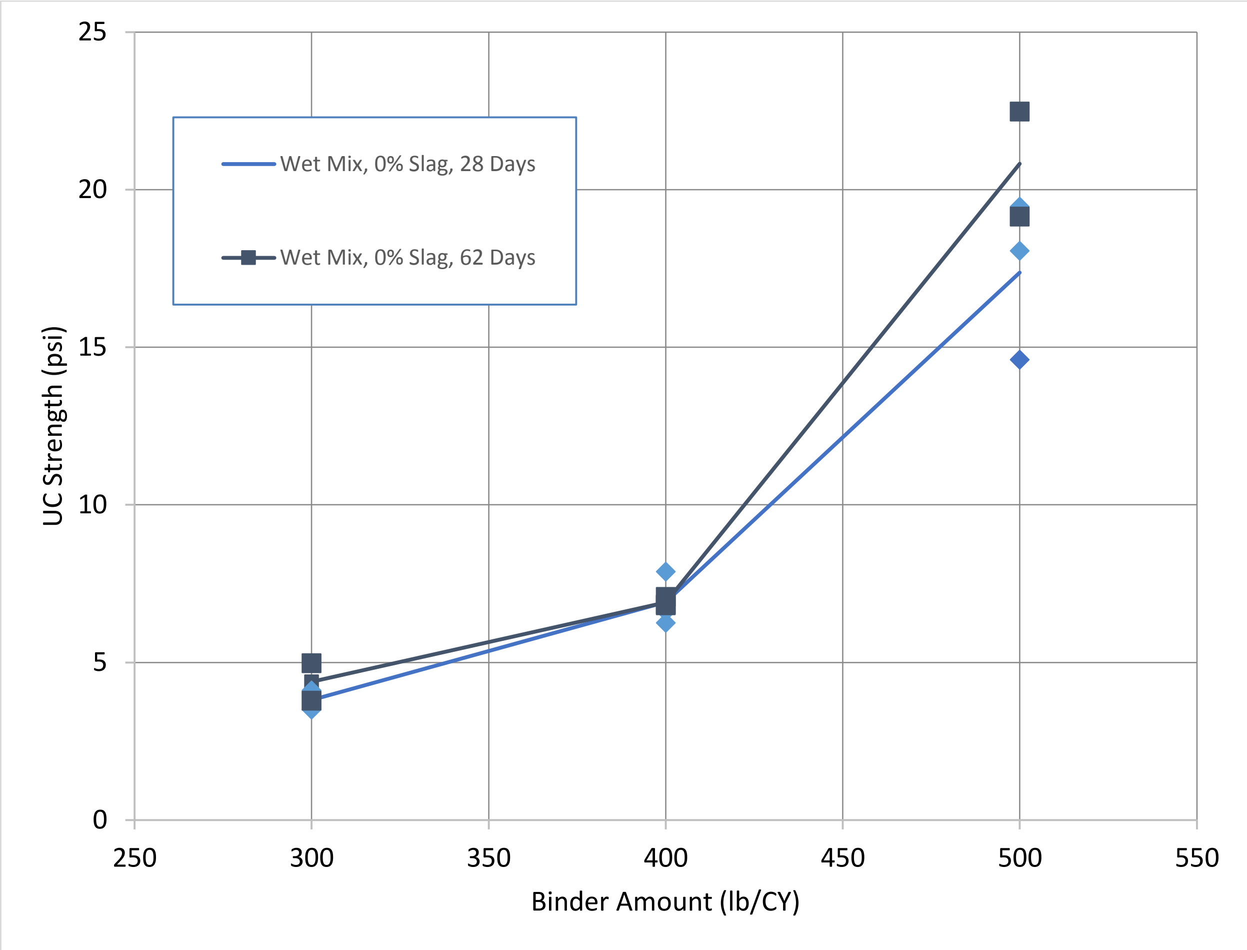

Figure 3.48. Wet mixing test results.

Table 3.12 Wet mixing results and information.

Slag Replacement	Batch	PCY	W/C	Moisture	% OC	28 Stress (psi)	61 Stress (psi)
0%	49	300	4.23	154%	43.8%	3.81	4.39
	50	400	3.17	154%	43.8%	6.92	6.91
	51	500	2.53	154%	43.8%	17.37	20.82

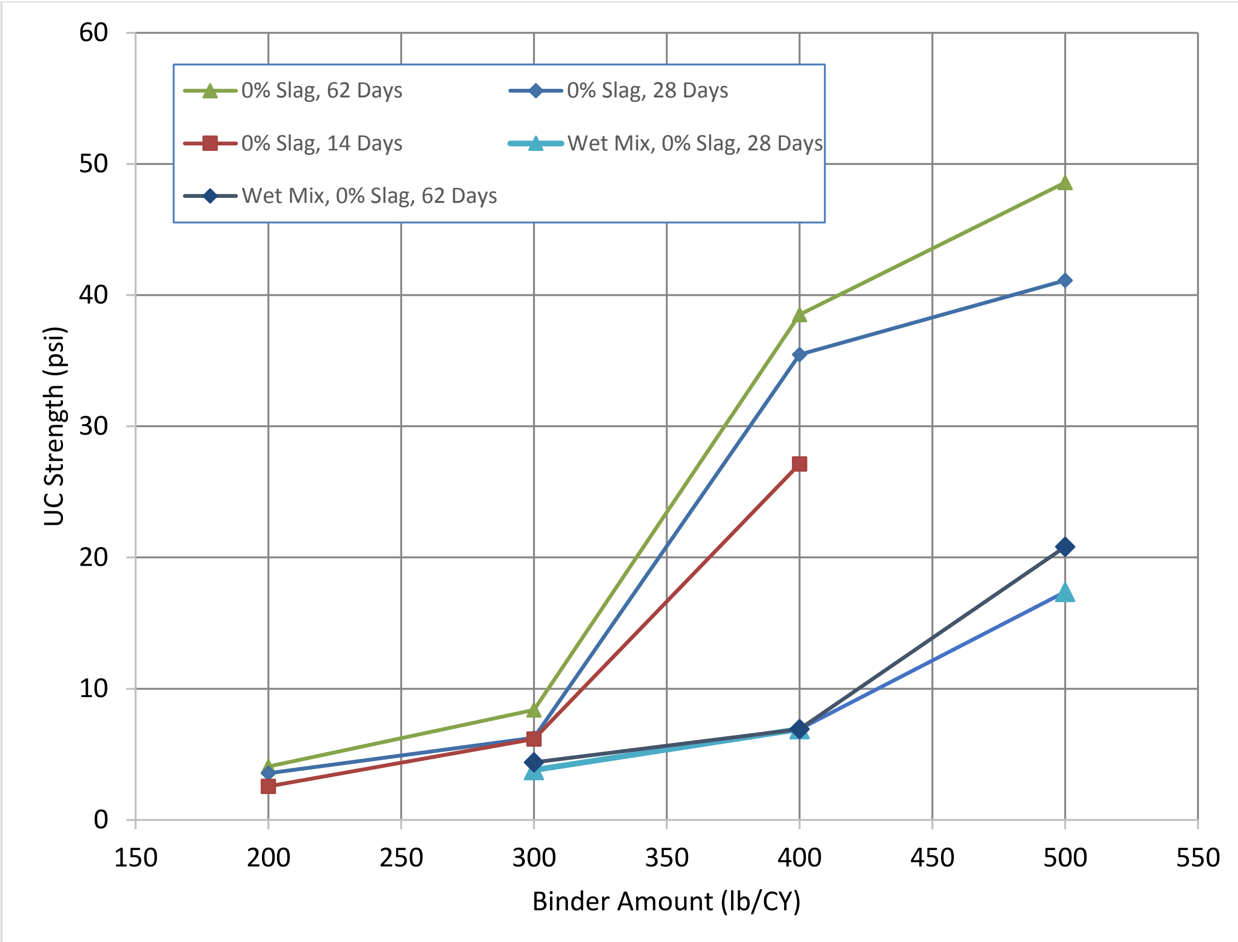

Figure 3.49. Dry mixing and wet mixing test results.

No clear explanation is available as to the poorer performance of the wet mixes relative to the dry mix counterparts. This is not supported by literature findings.

Chapter 4: Large-Scale Laboratory Testing

In order to confirm the findings in Chapter 3, 1/10th scale testing was performed using simulated field conditions where both wet and dry mixing methods were employed. Both of these methods were then compared to an untreated control of the same organic soil composition. Using a test bed divided into three sections, these tests were performed side-by-side: the outer two portions of the test bed were treated and the middle represented the control soil. After treatment, all three portions were individually loaded by adding water tanks placed on each test bed section. Water was added in increments over time. Both load and displacement were monitored throughout the testing.

Both wet and dry mixes were designed to simulate a roadway subjected to 5 ft. of fill. This is equivalent to a design load of approximately 600 psf. To closely investigate potential differences between wet and dry mixes, the same binder content was used in both treatments.

This chapter presents all of the events involved in large-scale laboratory testing in chronological order. This includes the fabrication of the test bed, soil preparation, wet mixing, dry mixing, loading of the system, and the test results.

4.1 Fabrication of Test Bed

The test bed was designed to be 12 ft. long, 4 ft. wide, and 3 ft. tall and to be a structurally sound confining bed that resisted any deformations of the walls due to lateral soil pressures. The test bed was fabricated using sheet metal for the floor and walls of the bed. Additionally, I-sections were used as base supports, and C-channel sections were used as wall reinforcement. This added significant stiffness to the bed. The 12 ft. length of the bed allowed the bed to be partitioned into three 4 ft. x 4 ft. sections; these sections featured a wet mix design, a dry mix design, and an untreated control. Figure 4.1 shows the fabrication of the bed, and Figure 4.2 shows the partitions. Figure 4.3 shows the finished bed.

Figure 4.1. Welding of the bed (left) and the bed upside-down during fabrication.

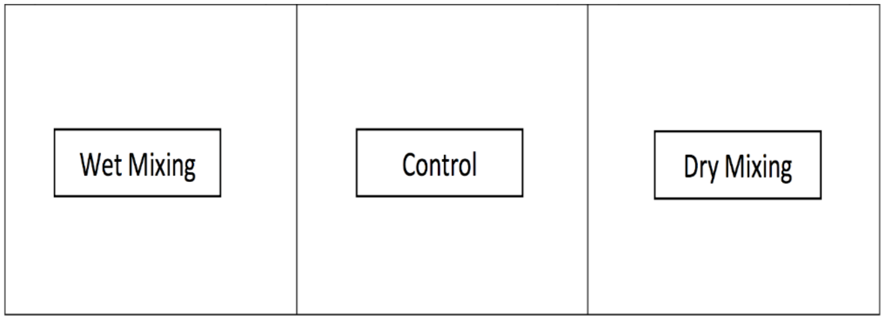

Figure 4.2. Partitions of the steel testing bed.

Figure 4.3. Testing bed with tent.

4.2 Organic Soil Placement and Properties

After the bed was fabricated, organic soil was delivered from SR-33. This soil was evenly placed in the bed until the bet was 2 ft deep. Once in the bed, the moisture content was 167% and the organic content was 40%. Figure 4.4 shows the excavation of the soil, it being delivered to the laboratory, and being placed in the bed.

Figure 4.4. Excavation and delivery of organic soil to bed.

4.3 Wet Mixing Concept and Equipment Testing

In wet soil mixing, it is typical to create columns of mixed soil rather than treating the entire volume. This is known as area replacement and was discussed in Section 2.2.3.5. This method was found to be practical for large scale laboratory testing. This section explains the mix design to be used, the concept for the mixer, and the testing of the concept.

4.3.1 Mix Design

Using an area replacement ratio of 20%, a system strength of 600psf (5ft embankment) would require a column strength of 3000 psf or 20.83 psi. The highest strength tested with wet mixing obtained was 17.37 psi using 500 pcy of cement with OC = 40%. Both the binder content and w/c ratio data was extrapolated to find the necessary binder content. As the soil to be mixed was similar to the lab soil, lab data was directly applicable to selecting binder content. Figure 4.5 shows the extrapolated data.

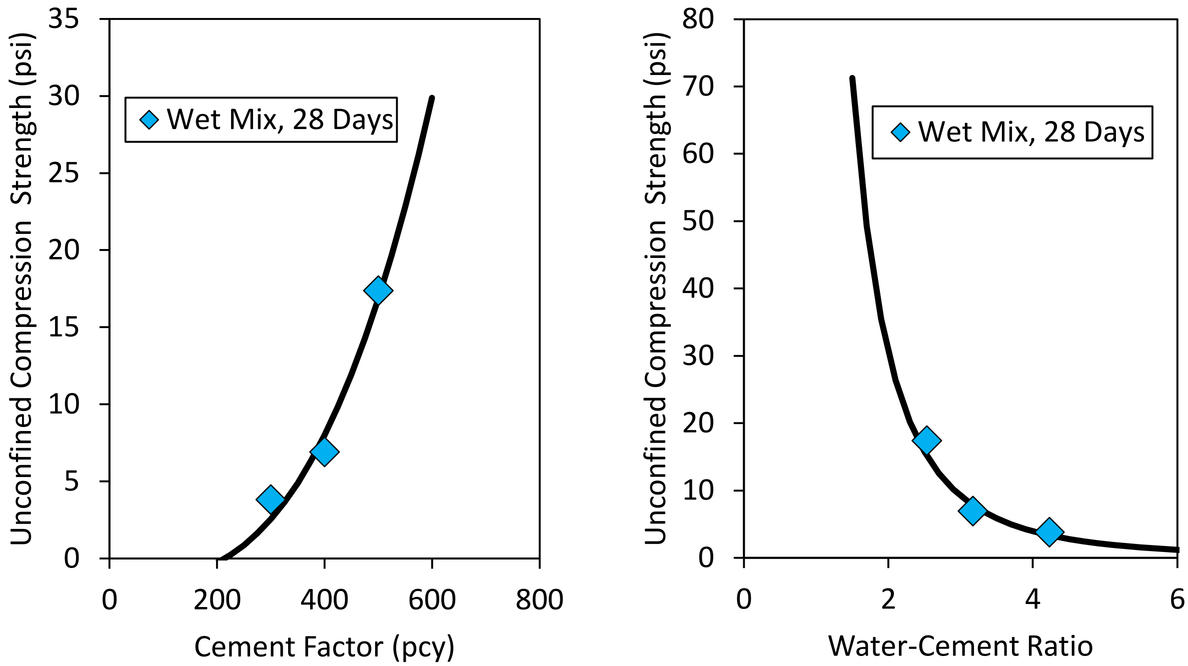

Figure 4.5. Extrapolated wet mixing data for a grout w/c ratio of 0.8.

Using Figure 4.5, the cement factor curve suggest 535 pcy of cement was needed. However, using an assumed moisture content of 200% and the w/c ratio relationship this resulted in a predicted cement factor of 590pcy would be needed. This translates into about 13lbs of 0.8 w/c grout per 4.25in diameter, 24in deep column.

4.3.2 Mixer Concept and Final Design

In order to simulate field mixing practices, a mixing machine was designed to follow the conceptual diagram in Figure 4.6. The machine concept involved an auger with a hollow core, through which the 0.8 w/c grout could be pumped while being spun/mixed with a engine attachment. Before drilling, a fixed volume of grout was mixed externally and placed in the pressure pot that was connected via grout hoses to the auger swivel head. As the drilling occurred, pressurized air would force the grout to flow out of the pressure pot, through the hoses, into the hollow core of the auger and discharged into the untreated soil as it spun. Both the engine and the pressure pot were suspended from crane hoists that were capable of translating in three dimensions to accommodate vertical and spatial positioning.

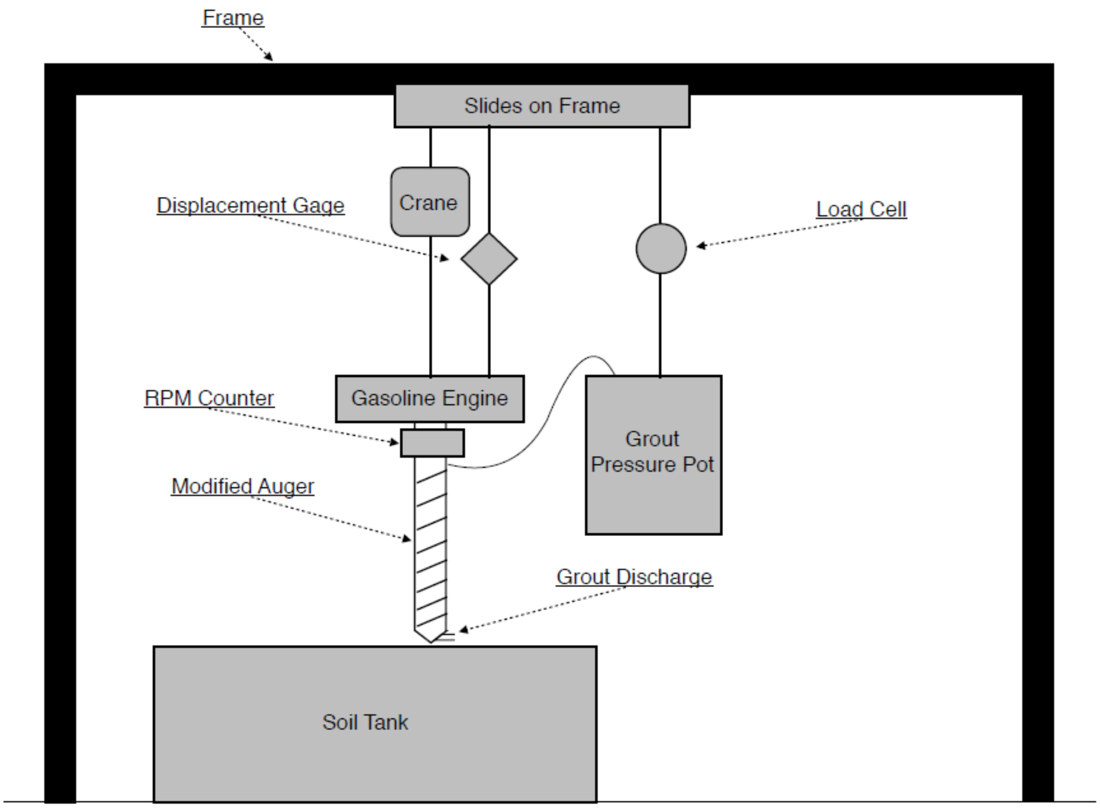

Figure 4.6. Wet mixing machine concept.

The mixing machine equipment consisted of a 2.75 ft long, 4 in diameter auger with a hollow core, attached to a 5 hp two man hole digger, henceforth called the mixer. The auger was modified such that every ¼ turn of the auger flights were removed and a ¼ in. diameter hole was drilled near base to allow for grout discharge. By removing half of the flight area the soil would be mixed and not lifted/mined from the bed. Figure 4.7 shows the modified auger.


Figure 4.7. Modified auger.

The mixer was suspended from a manual chain hoist which hung from a geared trolley attached to the frame. The manual chain hoist allowed for vertical translation of the mixer while the geared trolley allowed for horizontal translation of the mixer. Attached to the geared trolley was a wheeled electric hoist, also situated on the frame that allowed for both the horizontal and vertical translation of the grout pressure pot.

The mixer was instrumented with a string line vertical displacement transducer and an actuating magnetic switch allowing for the monitoring of the vertical translation of the mixer and rotations of the auger (Figure 4.8). Similarly, the volume of grout pumped was monitored by suspending the grout pressure pot from a load cell. The load cell measured the weight of the pressure pot and using the measured density of the grout, the load measurement was converted to grout volume. The final design also included a flowmeter as a backup for measuring the amount of grout pumped. The actual mixing machine can be seen in Figure 4.9.


Figure 4.8. Magnetic rotation counter.

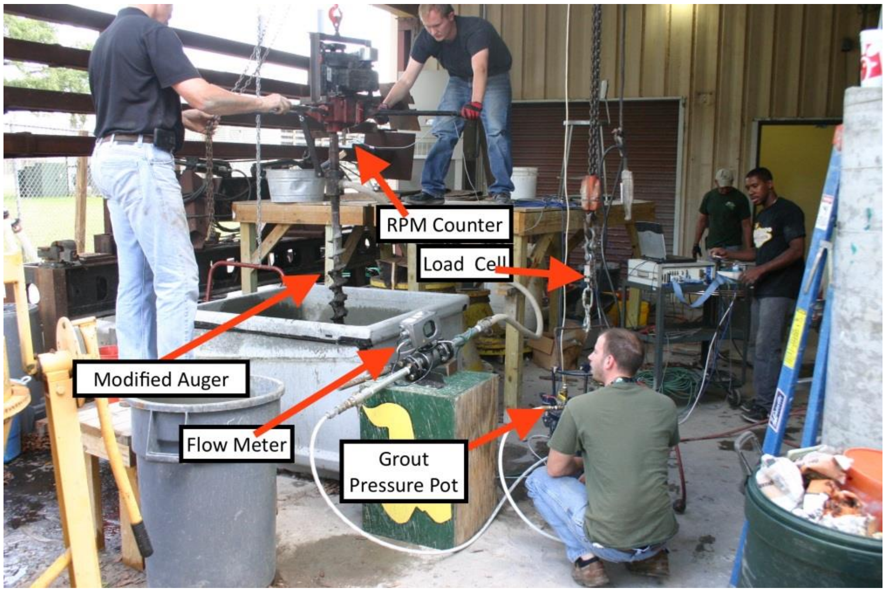

Figure 4.9. Wet mixing system.

4.3.3 Calibration of the Wet Mixing System

Before mixing, it was important to measure the relationship between grout pressure and flow rate. A calibration curve was obtained by measuring the weight loss of the pressure pot at various pressures. The weight loss was then converted into volume. A water - cement ratio of 0.8 was chosen for the grout. Figure 4.10 below shows the calibration test setup overview.

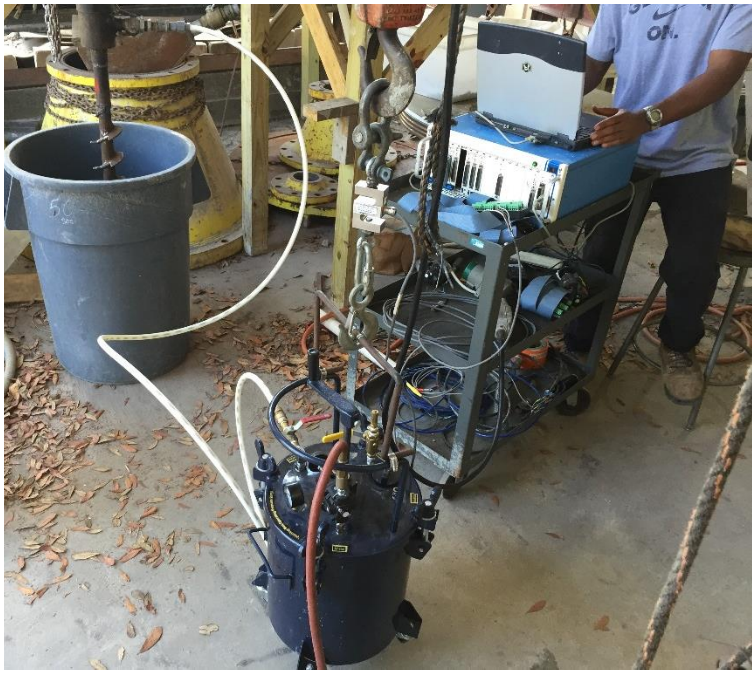

Figure 4.10. Grout calibration setup.

For calibration testing, the modified auger was placed in an open barrel (Figure 4.11). Note that care was taken to ensure that the bottom of the auger was never submerged in grout. This allows grout to flow through the system with no soil resistance. However it was expected than once the auger was in the soil, there would be some level of back pressure. However, due to the loose nature of organic soils, it was also expected that this back pressure will be minimal. Therefore the results from this test should provide a reasonable prediction for grout flowrates and the required grout pot pressure. While a grout flow meter was also used, these calibrations confirmed the capability of the grout delivery system over a wide range of flow rates.

Figure 4.11. Modified auger in test barrel.

Flow rates were measured at five different grout pressures. The weight of the pressure pot was monitored electronically while the grout pressure was manually read. Figure 4.12 shows the analog pressure gauge.

Figure 4.12. Analog pressure gauge (15psi).

The data collected is presented in Figure 4.13. The slope of each line in Figure 4.13 represents the flowrate at each of the five grout pressures. The flowrate was then plotted as a function of grout pressure. This became the calibration curve. Figure 4.13 shows the calibration curve in terms of gallons per second, and potentially more helpful, Figure 4.15 is the same calibration curve but in terms of gallons per minute.

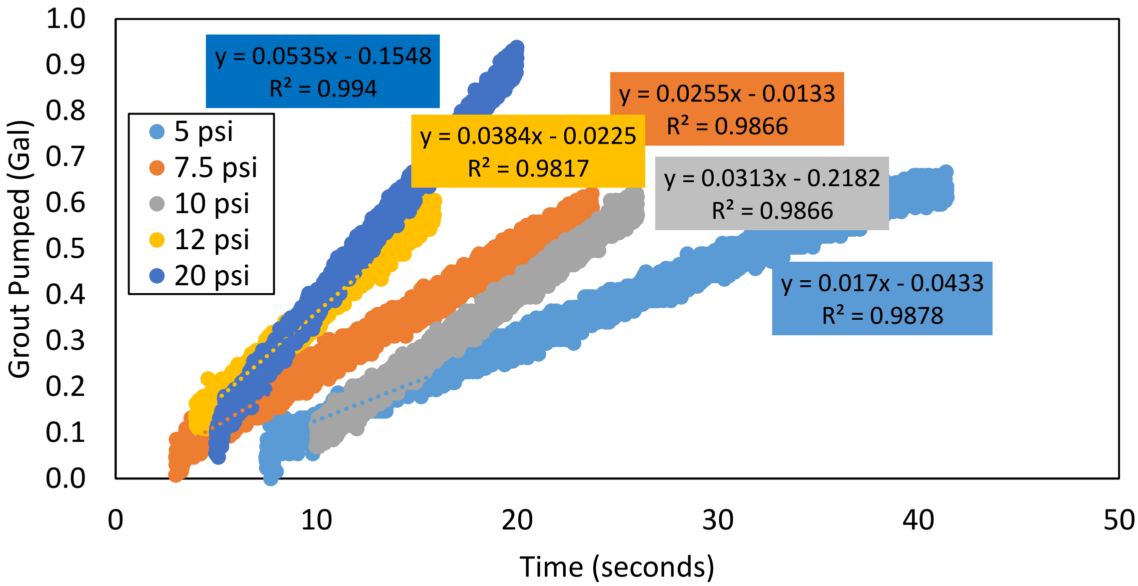

Figure 4.13. Grout volume pumped vs time (cropped).

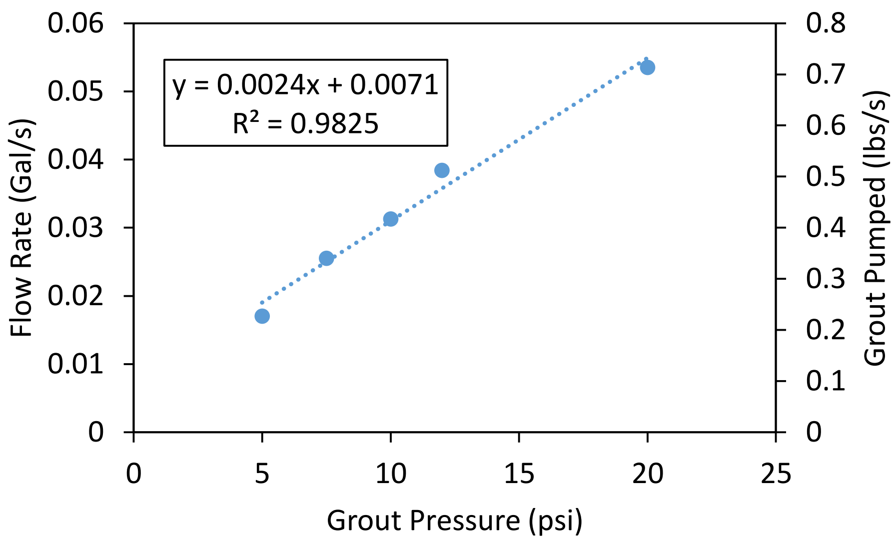

Figure 4.14. Calibration curve in gallons per second.

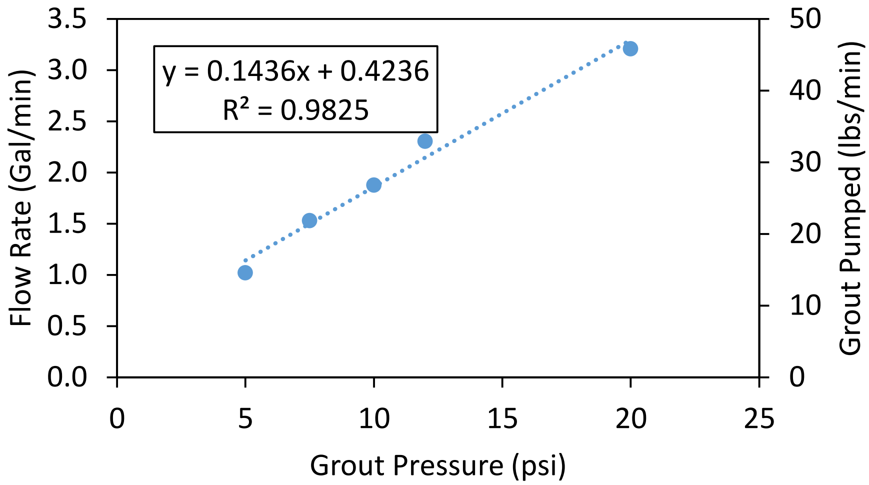

Figure 4.15. Calibration curve in gallons per minute.

These results proved to be linear as predicted. It should be noted that these calibration curves are only valid for grout pressures within the range tested as the linear trend line has a y intercept not equal to 0.

4.3.4 Preliminary Equipment Tests

Before being used on the large steel bed, the wet mixing system was tested on soil in a separate container filled with the same soil to the same depth (Figure 4.9). Figure 4.16 shows the mixing of a test column.

When running the system, grout was monitored in terms of weight. 16lbs of 0.8 w/c grout was chosen as the target amount of grout to place in a column. This consisted of 8.9lbs of cement and 7.1lbs of water per column. Additionally, a flow rate of 1gpm was used. 8.9lbs of cement in a final column volume of 0.36ft^3 generates a cement factor of 674pcy. This was 14% more than the 590pcy previous prescribed, but proved to be more achievable with the rates of penetration and extraction that could be performed.

Figure 4.16. Test column being wet mixed (left); close up (right).

After a test column was created, a threaded rod was placed in the middle of the column so that it could later be removed. Figure 4.17 shows the column being removed after 28-days. Figure 4.18 shows close ups of the column before and after it was rinsed off.

Figure 4.17. Wet mixed column being removed from soil after 28 days.

Figure 4.18. Wet mixed column (left) and after washed (right).

Figures 4.17 and 4.18 demonstrated that this wet mixing system could create the cementitious columns that were needed and therefore was acceptable for the large scale laboratory mixing bed. The measured diameter of the column was 4.25 in.

4.4 Dry Mixing Concept and Equipment Testing

To stay consistent with the column approach, the first dry mixing concept was to mix dry powder cement into the soil. However the dry powder cannot be pumped in the same manner that grout was for the wet mixing. Therefore dry powder was inserted into a PVC pipe. This pipe had a cap at the bottom that could be removed once the pipe was in the proper location. The pipe was designed to hold the same amount of cement per column as the wet mix, 8.9 lb. Once the cap was removed, the pipe was carefully removed. This left the dry powder cement vertically distributed within the target area. The modified auger used in wet mixing was then placed to the side of the cement column. It was then vertically raised and lowed to mix the cement with the soil. This process is shown conceptually in Figure 4.19. This method was tested several times in the smaller prototyping bed. Unfortunately each time a dry mixed column was created, it showed zero evidence of strength. Therefore a different approach was developed. Figure 4.20 shows this method being tested.

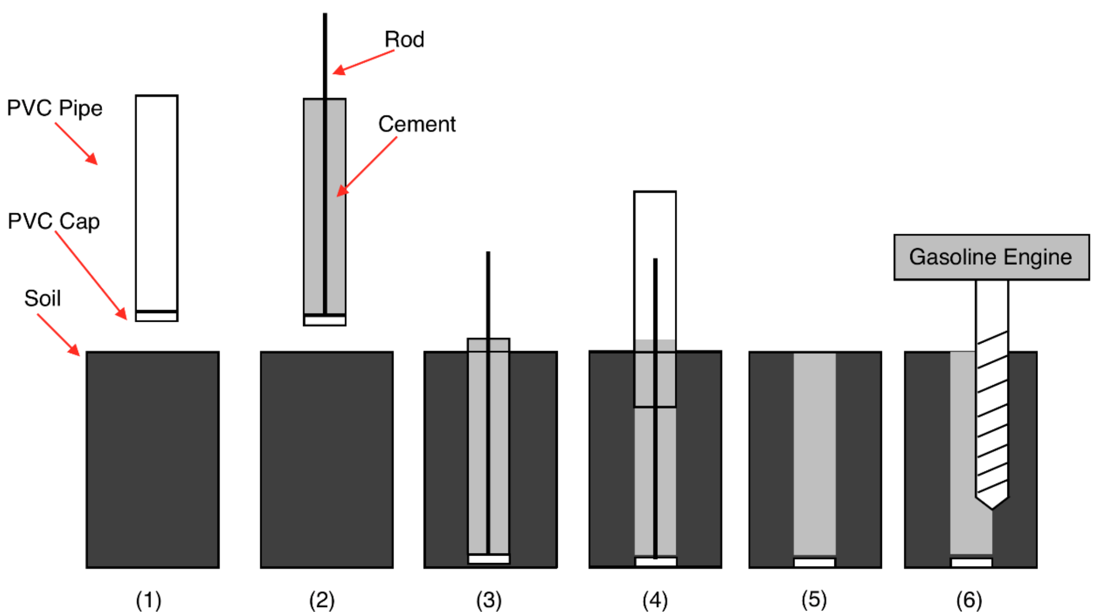

Figure 4.19. Stages of the dry mixing concept.

Figure 4.20. Testing of dry mixing concept.

Mass dry mixing was the second concept and was used in the large scale laboratory mixing bed. In general, the amount of cement needed was simply placed on top of the soil in the form of a dry powder. This was then mixed using a tiller (see section 4.6).

4.5 Wet Mix Column Installation

Using an area replacement ratio of 20%, a hexagonal column pattern, and an effective column diameter of 4.25 in., the center-to-center spacing of columns was 9 in. The wet mixing partition of the steel bed used flag markers to designate column locations. Figure 4.21 shows the column layout and the numbering. Since the loading area (discussed later) consisted of a 2 ft. diameter bearing plate, the two "rings" of columns shown in Figure 4.21 were chosen to provide an adequate loading area. This pattern consisted of 19 columns. The loaded areas would only be the central seven columns while the peripheral columns were intended to provide later confinement. The mix design called for 16 lb of grout in each column. Figure 4.22 provides a summary of the actual grout placed in each column. It should be noted that these amounts do not include the grout discharged on the surface of the soil. Figures 4.23 and 4.24 provide an overview of the mixing and a close-up view, respectively.

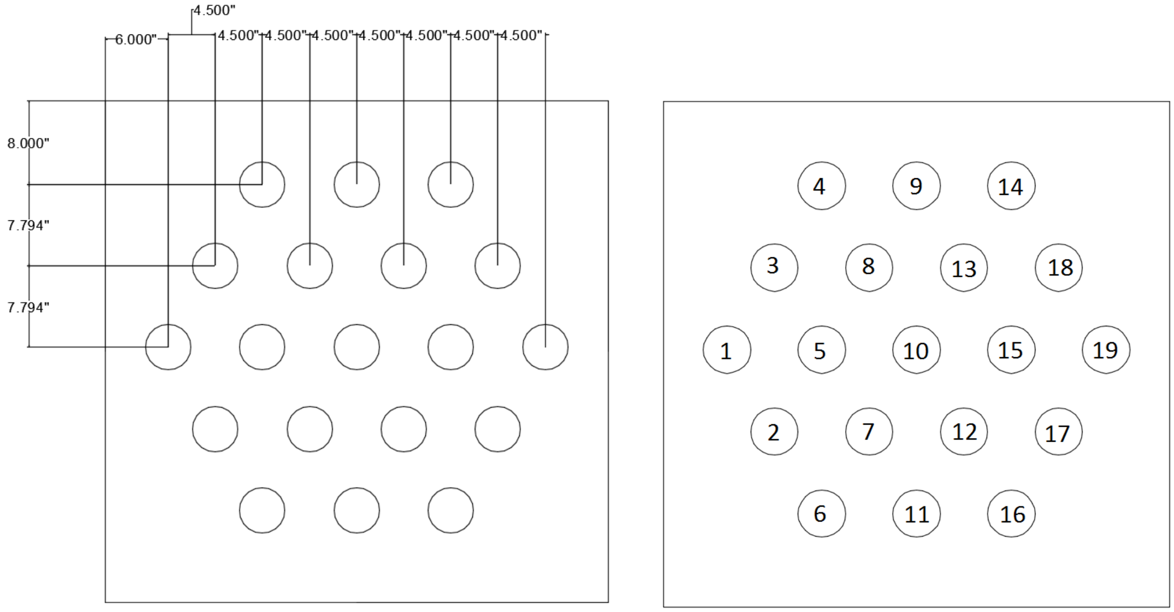

Figure 4.21. Hexagonal column pattern and numbering.

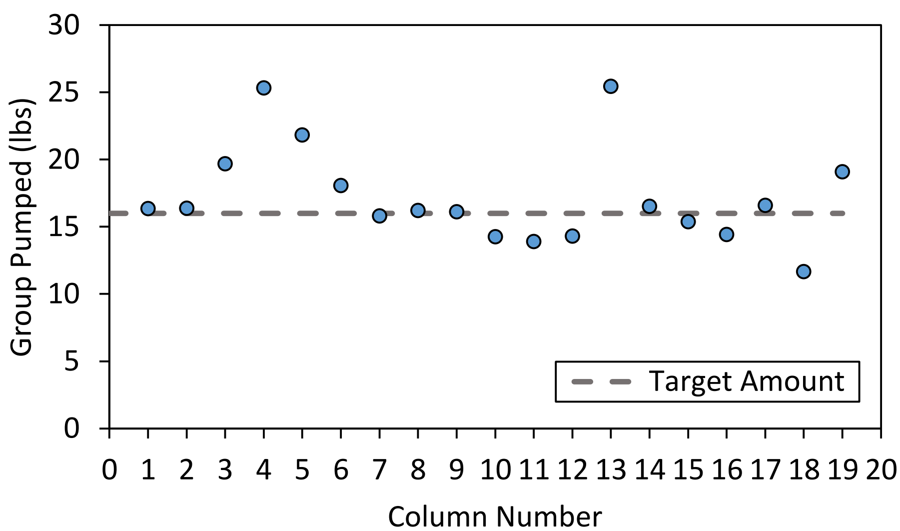

Figure 4.22. Grout injected into each column.

Figure 4.23. Wet mixing in steel bed overview.

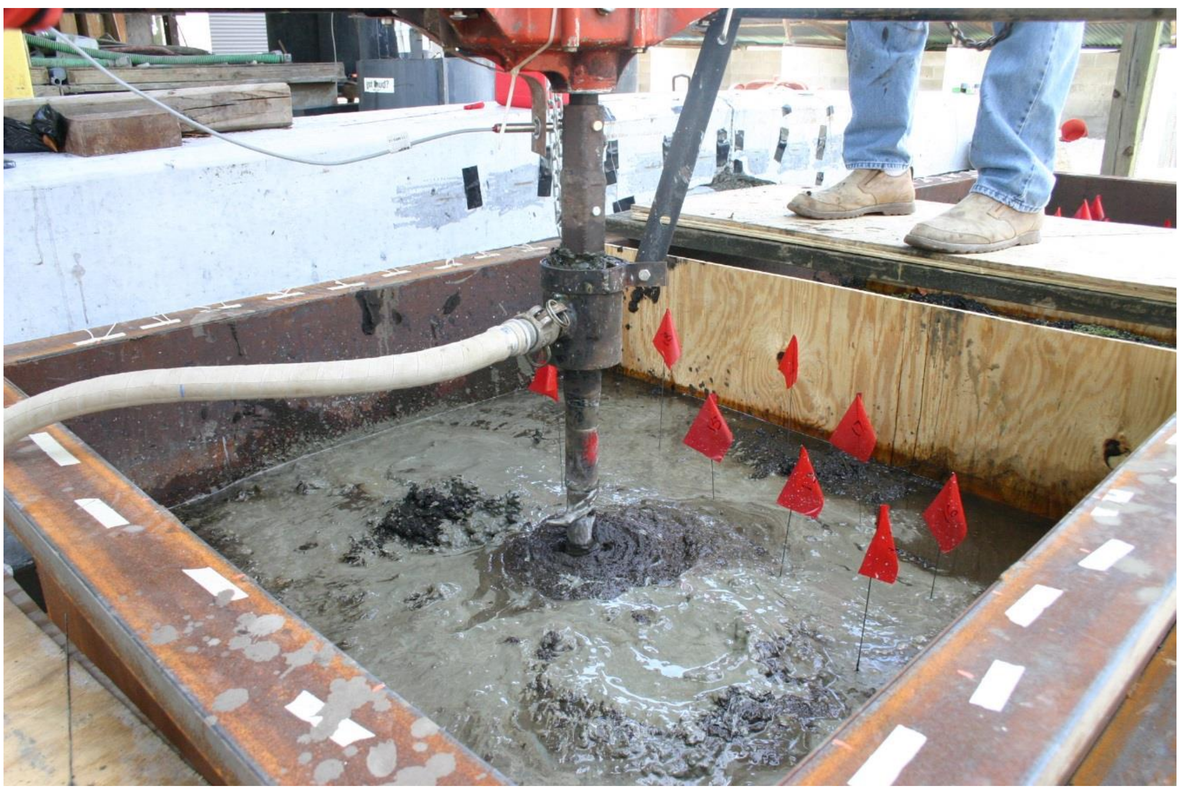

Figure 4.24. Close up of wet mixing.

4.6 Dry Mixing

The dry mixing method added dry powder cement directly on the surface of the soil. Then the soil was mixed in place. In order to closely compare wet and dry mixing methods, the same global cement factor used in wet mixing was used for the dry. The wet mixing method used a total of 168.9 lb of cement for a total treatment volume of 19.6 ft^3. Note that this treatment volume is not the volume of the tank partition, but rather the volume of the soil effected by the treatment plus the volume of the grout added. See the calculation below.

$$Treatment\ Area = \frac{Area\ of\ Columns}{\%\ Replacement} = \frac{19 * 12.57\ in^2}{20\%} = 1193.8\ in^2$$

$$Treatment\ Volume = Treatment\ Area * Soil\ Depth + Volume\ of\ Grout$$

$$Treatment\ Volume = 1193.8\ in^2 * 24\ in^2 + 19\ Columns * \frac{275.1\ in^3}{Column} = 33877.6\ in^3$$

$$Treatment\ Volume = 19.6\ ft^3$$

This equated to a cement factor of 8.6 pcf or 232.6 pcy. Since the entire dry mixing partition of the tank will be treated, the equation below was used to determine the amount of cement needed. This equation is further explained in Appendix B.

$$C = \frac{V_s * CF}{1 - \frac{CF}{62.4}\left(\frac{1}{3.15} + w/c\right)}$$

$$C = \frac{(4\ ft * 4\ ft * 2\ ft) * 8.6\ lb/ft^3}{1 - \frac{8.6\ lb/ft^3}{62.4}\left(\frac{1}{3.15} + 0\right)} = 288\ lbs$$

A tiller, shown in Figure 4.25, was used for pre-mixing the soil and mixing the cement with the soil. Pre-mixing was performed to simply breaking up the soil without adding any binder. This was done to not only loosen the soil but also to test the mixing capabilities of the tiller.

Figure 4.25. Dry mixing tiller.

During the pre-mixing stage, the tiller had trouble mixing once the tiller was below the surface. At all times the soil condition was maintained in a fully saturated state whereby captured rainwater was used to fill any water that was lost to evaporation (Figure 4.26). Additionally, two smaller mixing paddles used vertically assisted in breaking up the soil (Figure 4.27).

Figure 4.26. Rain water added to maintain saturation condition.

Figure 4.27. Two mixing paddles breaking up soil.

Rain water was used to simulate field conditions and to avoid any chemical issues that would arise from using tap water. Additionally, the two outer blades of the mixing tiller assembly were removed to increase the depth to which the tiller could operate. After modifications, the tiller performed far better under the surface of the soil (Figure 4.28).

Figure 4.28. Tiller in soil prior to cement introduction.

Once the entire dry mixing portion of the bed could be tilled, moisture tins were taken, the top was leveled (Figure 4.29), and then cement was introduced.

Figure 4.29. Leveled soil prior to cement introduction.

The entire weight of cement was uniformly added to the surface of the soil; then the cement was leveled to help consistently mix once mixing began (Figure 4.30).

Figure 4.30. Introducing dry cement to soil.

The addition of dry powder cement reduced workability which required more mixing time using both the tiller and mixing paddles. Figure 4.31 shows the progression of mixing with the last picture displaying the soil smoothed out at the end of treatment.

Figure 4.31. Dry mixing progression.

4.7 Loading

In order to observe the effects of long term loading, the loading system was designed to gradually load the soil over time. Loading took place after the treated soils gained sufficient strength. Water was chosen as the load because it is simple to gradually change and monitor. The water was stored in 300-gallon 3-ft. diameter plastic tanks. Beneath the water tanks were 2ft diameter steel bearing plates. The ratio between the diameter of the water tank and bearing plate allowed the soil to be subjected to greater pressure with less height of water 140 psf/ft. of water. The three tanks are shown below in Figure 4.32.

Figure 4.32. Water tanks placed on top of soil.

A steel frame of angle sections was constructed on top of the bed to keep the water tanks vertically aligned. Vertical movement of the water tanks was permitted, translation or tipping was restricted. A second frame was used to mount instrumentation.

A small amount of bedding sand was placed directly on the soil with the bearing plate on top of the sand. The sand helped to seat the plate and more uniformly apply the load. Due to displacements that will occur once loading began, the water tanks were not set directly on the bearing plate, but rather supported 6 in. above the bearing plate using struts welded to the plates. An intermediate layer of plywood was used to minimize local stress from the struts directly below and supporting the water tanks. This is shown in Figure 4.33 and 4.34.

Figure 4.33. Bearing plate assembly on top of sand (wet mix in background with tank in place, control in middle, and dry mix in foreground).

Figure 4.34. Bearing plate assembly with plywood.

An additional frame spanned the entire tank on which string line transducers were mounted to continuously monitor displacements using a field data logger. Figure 4.35 shows a string line transducer connected to the top of a water tank.

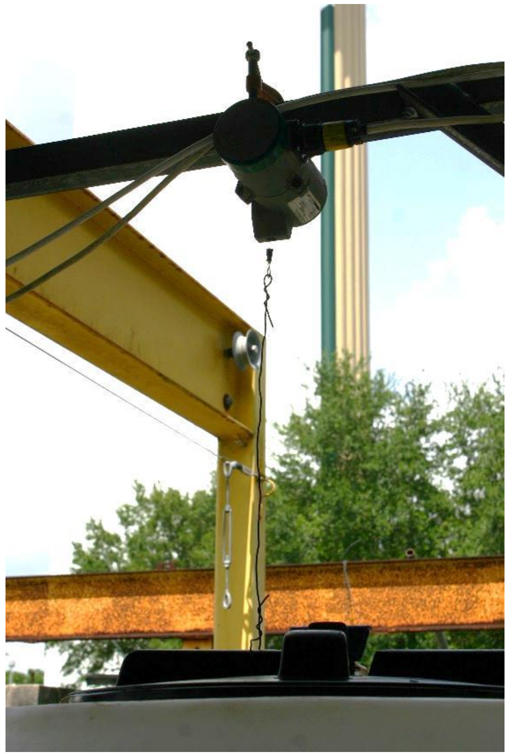

Figure 4.35. String line transducer mounted above water tank.

Using weight to volume relationships of water, and the diameter of the tanks, and the diameter of the bearing plate, the water height within the tank was converted to pressure. As seen in Figure 4.32, the marks on the side of the tank report pressure applied to the soil. The water levels were checked daily to ensure that there were no changes in load. Due to the tanks being capped, the only changes in load occurred when the load was intentionally raised to the next loading step. Each loading step was 50 psf. ASTM D1143 criteria for increasing to the next load step was used whereby the displacement per hour had to be less than 0.01 in/hr.

Displacement data collected by the string line transducers was remotely sent to an office computer for analysis. In addition to the computer collected data, daily survey measurements were manually taken as a backup.

4.8 Results

Overall the two treated tanks supported the design load of 600 psf and as expected far outperformed the control soil. The schedule of loading is shown in Figure 4.36. During the beginning of the loading process, the soil displacement would be low enough to perform multiple steps in one day. Note that in Figure 4.36, day 1 represents the first day that load was applied. Figure 4.37 shows the displacements of the soil over time. Note that the wet mixed and control soils began loading on the same day and the dry mixed soil began loading a few days later. Figure 4.38 goes on to show applied pressure vs displacement.

In these figures, the string lines collected the data for both wet mixed and control soils. Issues with the string line attached to the dry mixed soil led to unreliable data. Therefore the survey data is used in its place. In the wet mix and control soils, the survey data closely matched the string line data (Figure 4.38). Therefore the survey data for all three partitions may be considered reliable.

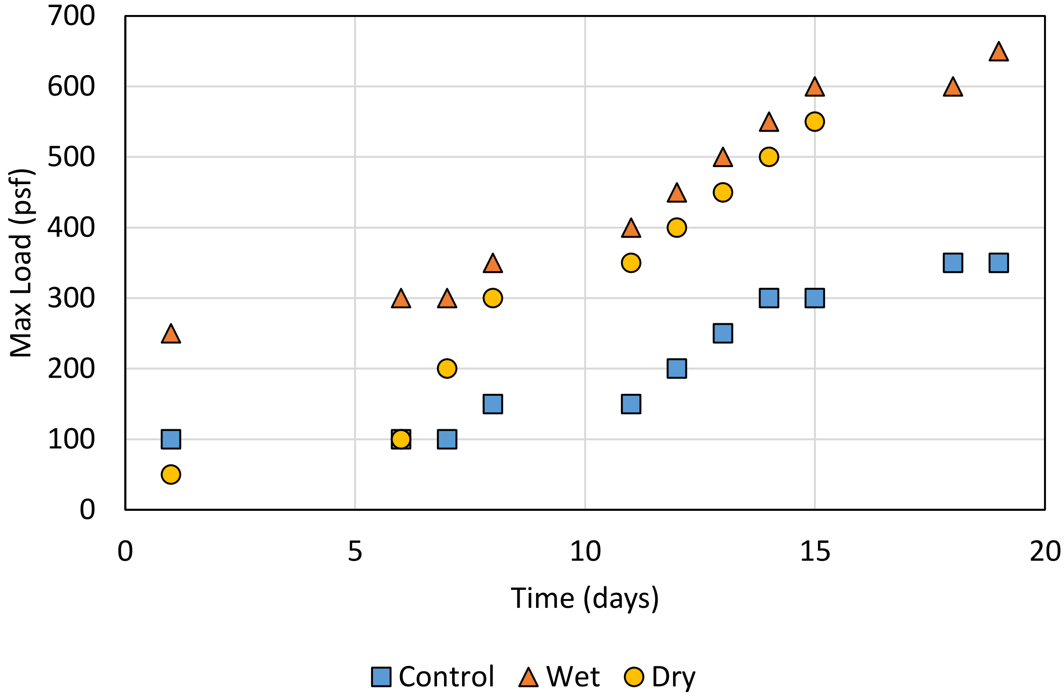

Figure 4.36. Schedule of loading.

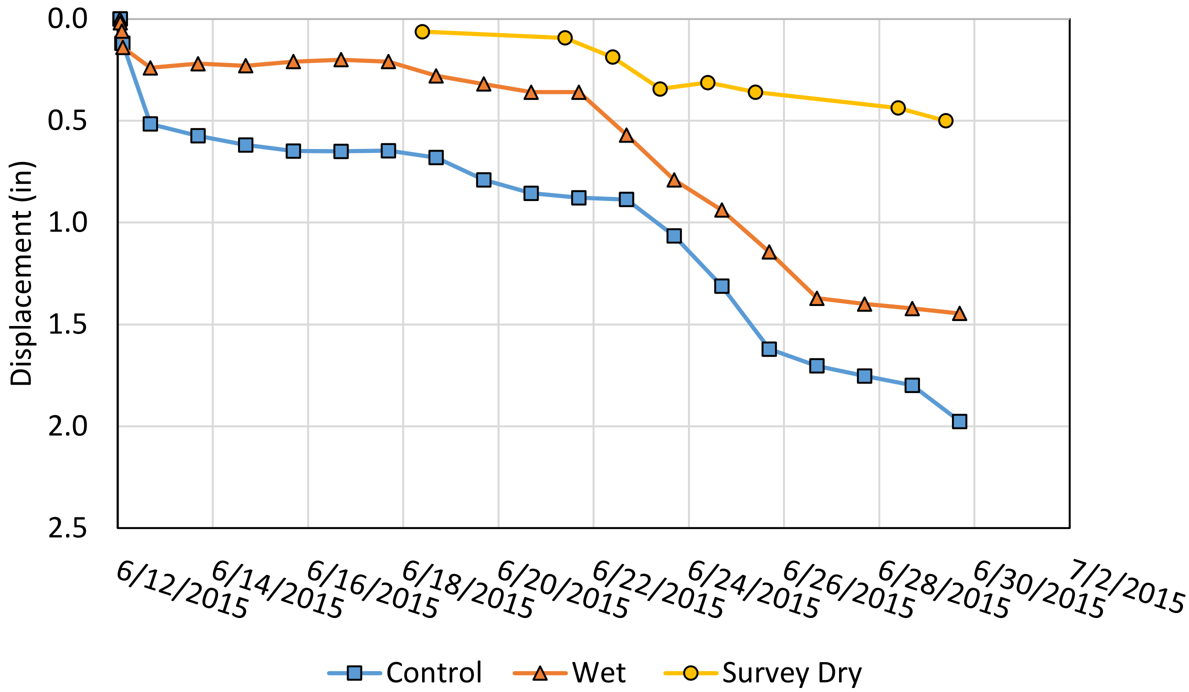

Figure 4.37. Displacement vs date.

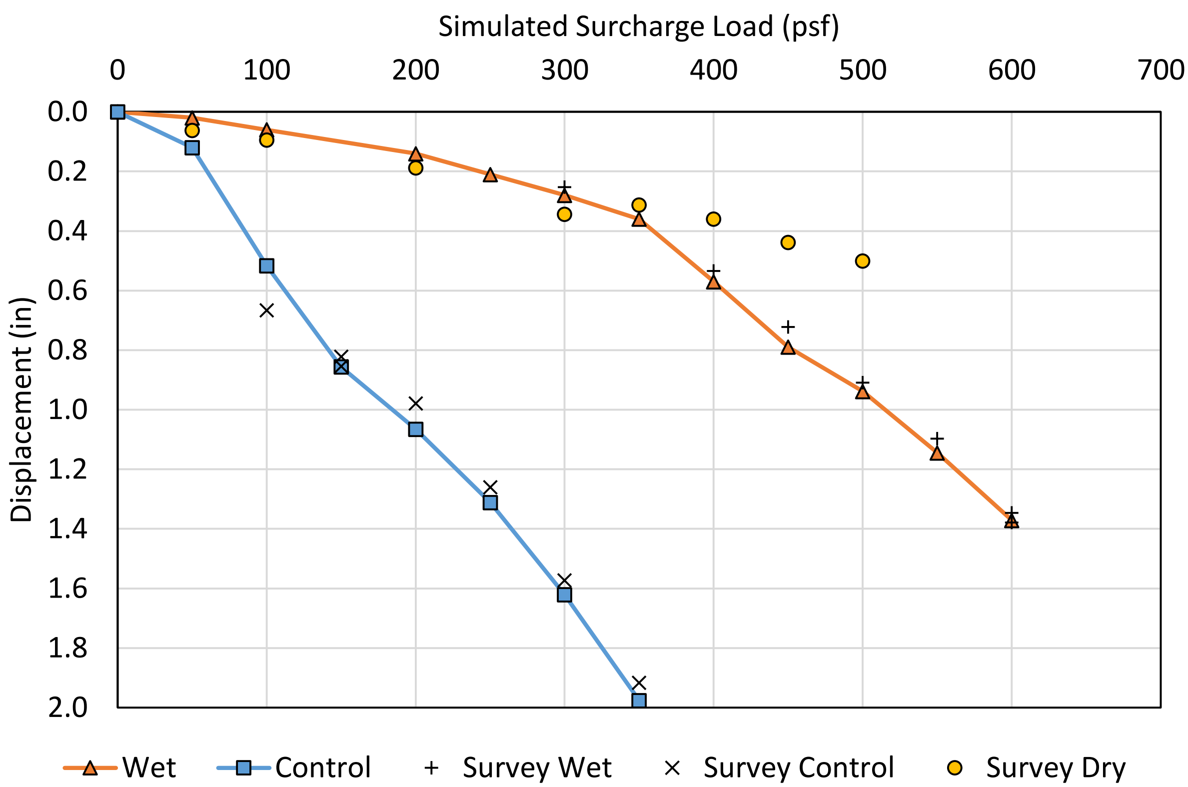

Figure 4.38. Pressure vs displacement.

Chapter 5: Field Evaluations

At the onset of the project it was difficult to predict the number and types of sites that might become available during this study. The preliminary visit and review of the SR-33 site at that time indicated that it might be a fitting candidate site for study with the aim of recommending possible remedial/soil mixing alternatives for the existing roadways. Likewise, data from the Jewfish Creek and Marco Island projects were similarly promising. All sites were checked quarterly for settlements.

A test program was established for the SR-33 corridor whereby baseline settlement surveys of the roadway (as-is) were followed by quarterly measurements. CPT profiling along one side of the road were used to delineate the extents and depths of the organic deposit. Organic samples were also extracted from this site and used for laboratory testing discussed earlier. Given the relatively short stretch of affected roadway (relative to Jewfish Creek) this site was denoted as a *relatively small deposit* per the original RFRP designations.

This chapter provides an overview of full scale soil mixing programs that were either previous performed or conducted concurrent to the project timeline. These include: SR-33, Jewfish Creek, Marco Island Airport, US-331 over Choctawhatchee Bay, and SR-37 over the Alafia River.

5.1 State Road 33, Polk City

Field survey measurements were performed along SR-33 just North of Polk City. The initial surveying was done on Friday, October 19, 2012 and included 11 points (approximately 100 feet separation) along the West side of the roadway (Figure 5.1). Figure 5.2 shows the baseline measurements referencing a concrete culvert just north of the problem area. These locations were re-used throughout the life of the project and also used for CPT location references.

In cooperation with the FDOT District 1 geotechnical group, eleven cone penetration tests (CPT) where performed along SR-33 on November 20 & 21, 2012. The soundings were done at the survey locations reported earlier. Figure 5.3 shows the first of eleven CPT soundings; all CPT data are shown in the Appendix. From this data a soil profile along the roadway was created (Figure 5.4). During the CPT testing, a second set of survey measurements were also taken. The survey showed relatively no change from the first survey; surveys were continued over the three year duration of the study (Figure 5.5).

Figure 5.1. Survey measurement location IDs along SR-33 just North of Polk City.

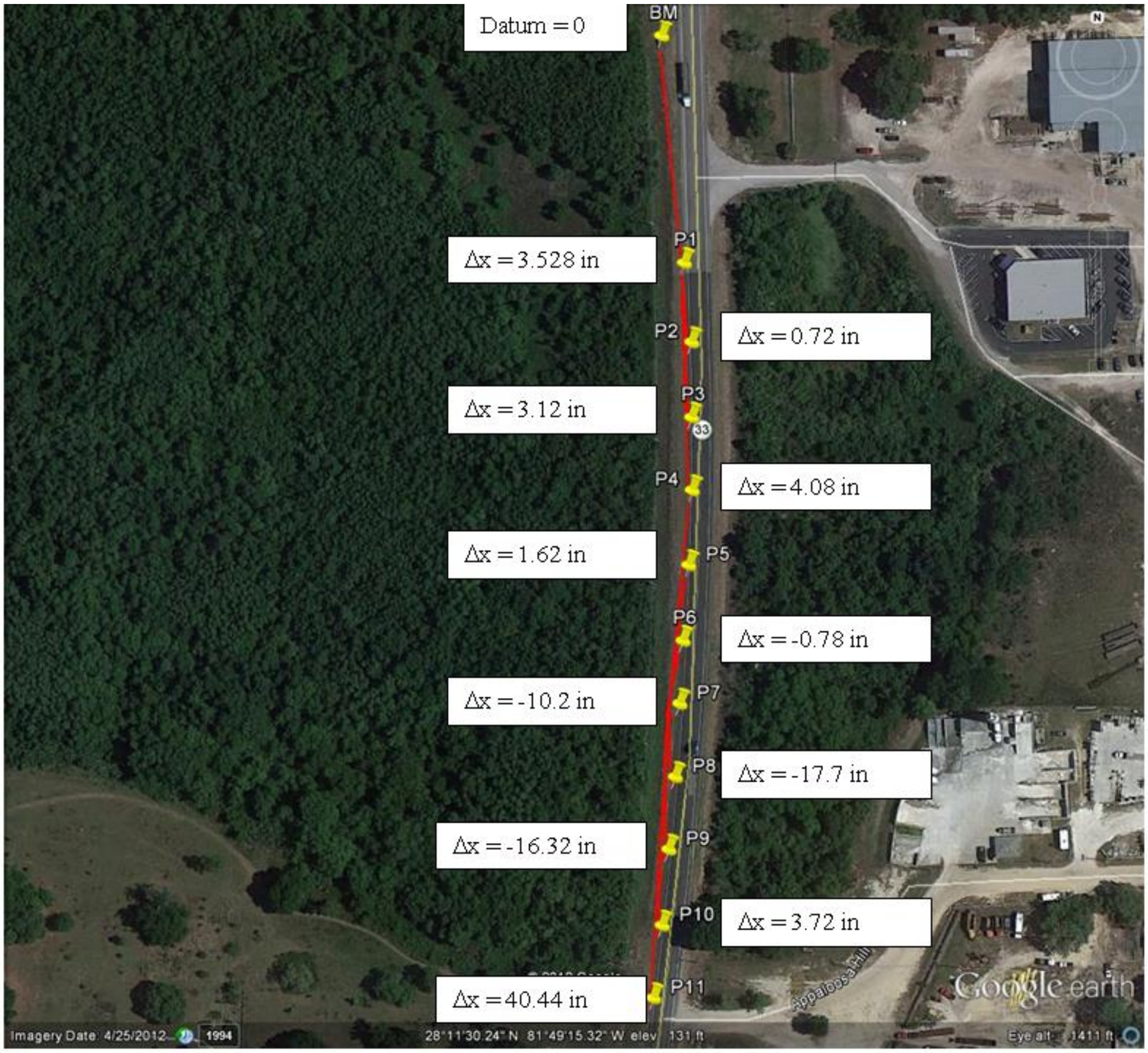

Figure 5.2. Initial Survey measurements along SR-33 just North of Polk City.

A third survey of SR-33 just north of Polk City was conducted on Monday, July 8, 2013. Subtle variations were noted that appeared to be small and within the tolerance of the survey equipment (Figure 4.3).

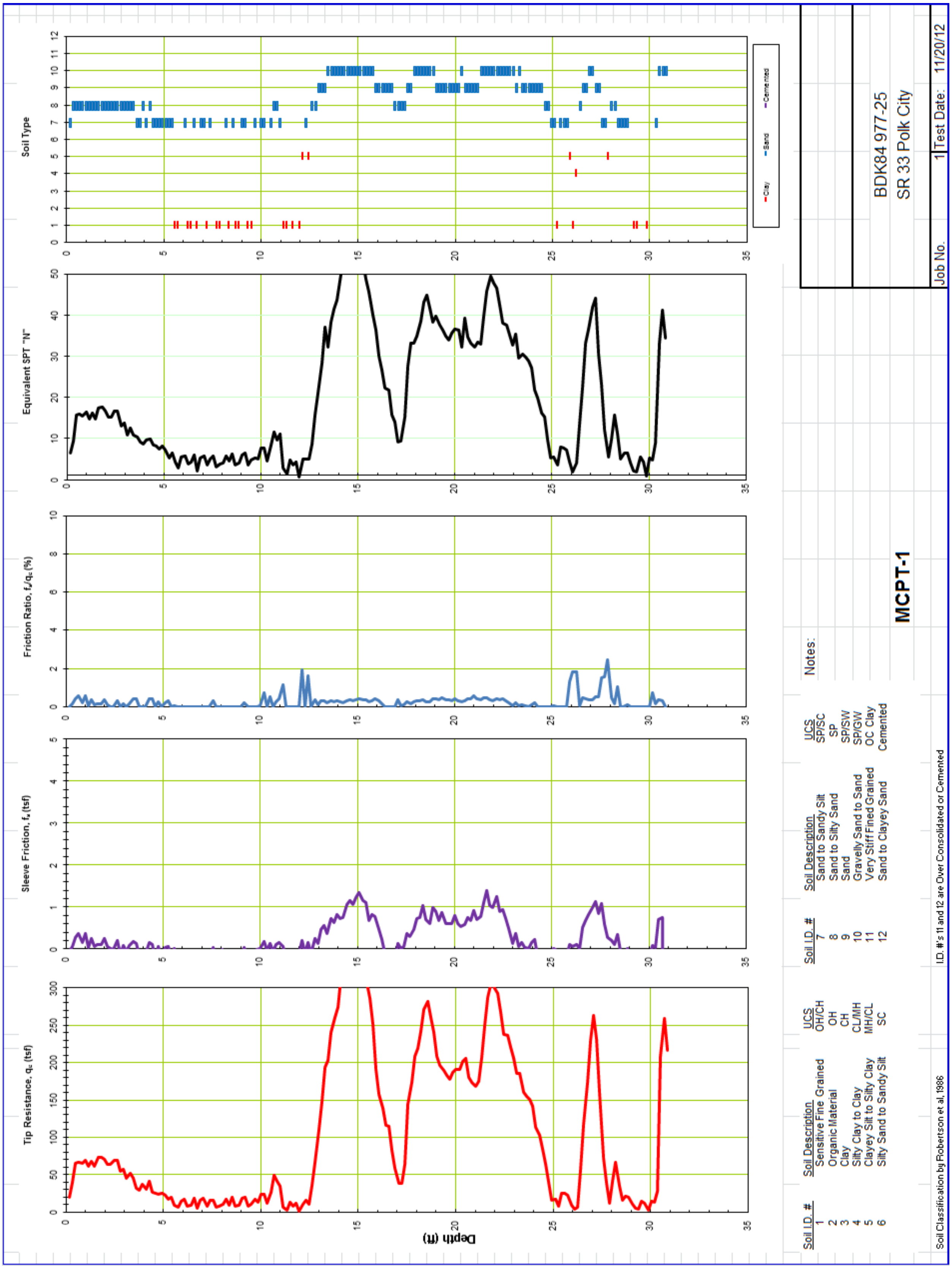

Figure 5.3. CPT 1 along SR-33.

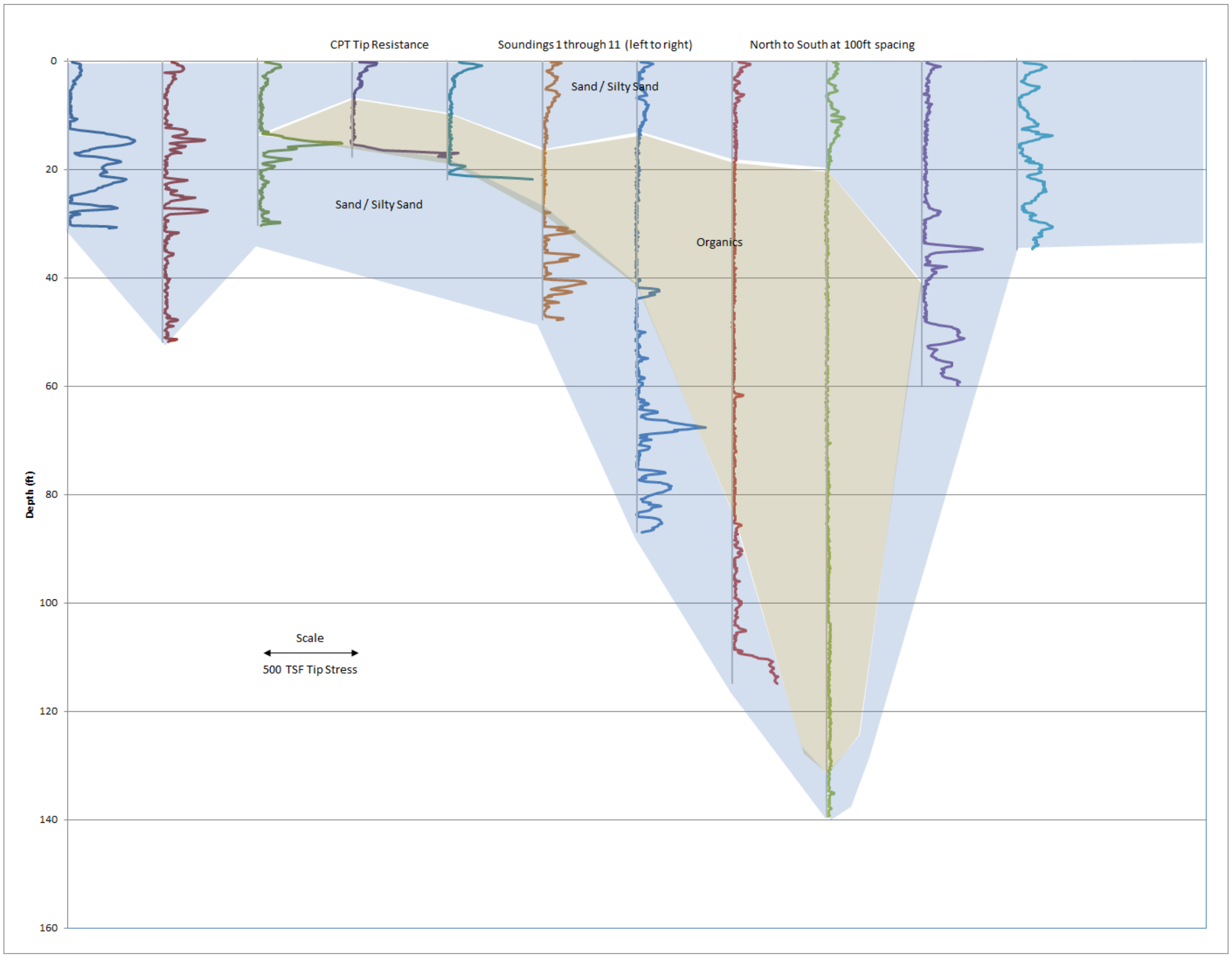

Figure 5.4. Soil profile created from individual CPT soundings along SR-33 corridor in Polk City.

Coincidentally, or not, the location and thickness of the organic material (shown as tan/brown) corresponds directly to the top of roadway surface elevation shown in Figure 5.5.

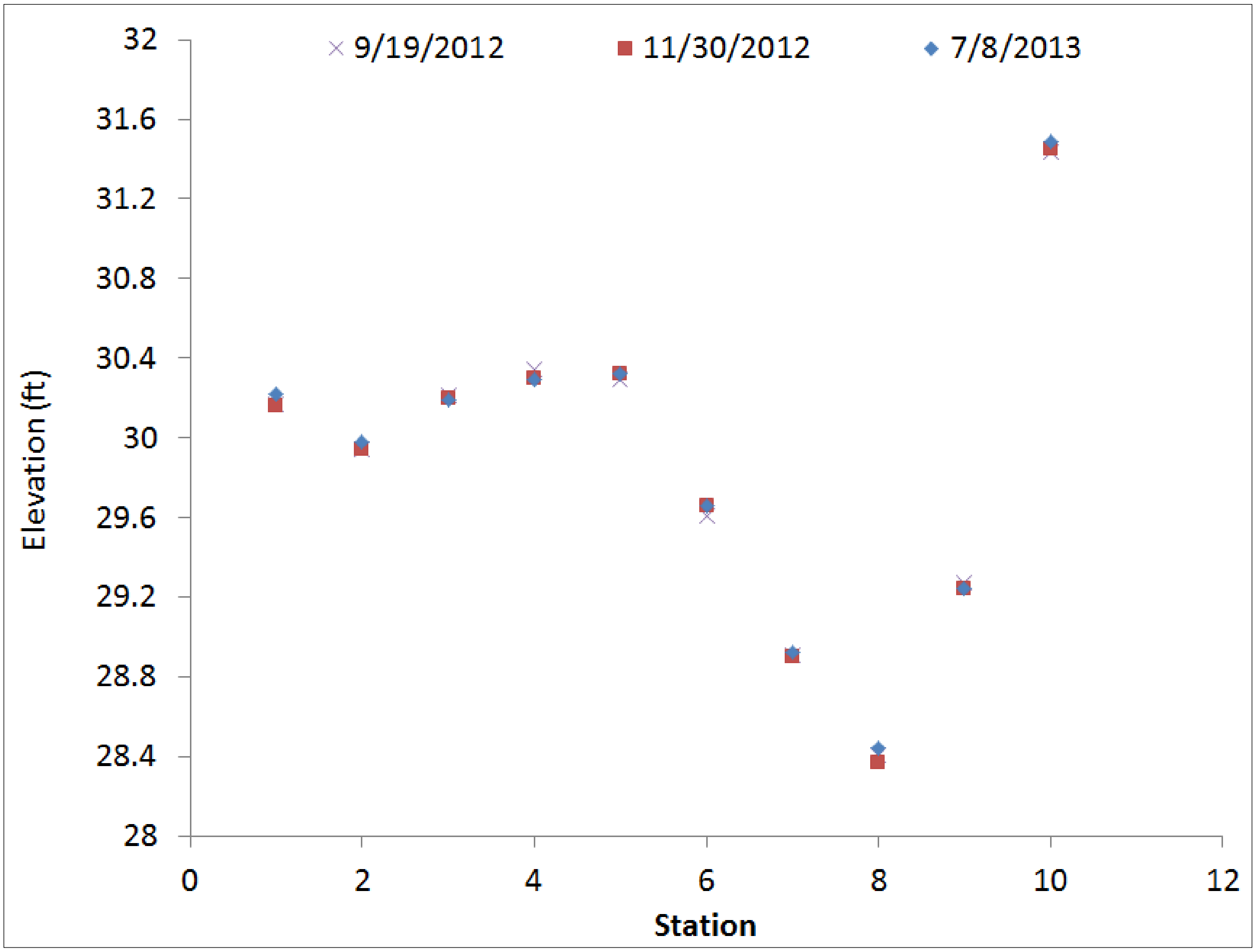

Figure 5.5. Survey data from SR-33 north of Polk City (SB Roadway).

5.2 Jewfish Creek US-1

As a mechanism to monitor the performance of the dry mixing performed along US-1 near the Jewfish Creek area, 25 survey points were established along the shoulder of the Southbound roadway from Station 1326 to 1350. This points closely coincided with previously taken locations by the state. Table 5.1 summarizes the historical survey points along the roadway.

Table 5.1. Summary of Survey Elevations along Jewfish Creek Southbound Roadway.

Date	**4/1/2009**	**2/23/2010**	**3/12/2013**
Point	**EL (ft)**	**EL (ft)**	**EL (ft)***
1325	6.03	6.04	
1326	6.64	6.65	7.27
1327	7.11	7.09	7.06
1328	7.74	7.72	6.62
1329	8.36	8.33	6.27
1330	8.76	8.72	6.37
1331	8.96	8.93	6.61
1332	9.06	9.03	6.75
1333	9.30	9.27	7.00
1334	9.49	9.45	7.22
1335	9.67	9.61	7.34
1336	9.88	9.85	7.64
1337	9.96	9.95	6.78
1338	9.97	9.95	6.82
1339	9.90	9.87	6.68
1340	9.80	9.77	6.47
1341	9.58	9.56	6.29
1342	9.35	9.33	6.05
1343	8.72	8.71	6.26
1344	8.10	8.09	6.50
1345	7.51	7.50	6.80
1346	6.82	6.81	6.72
1347	6.25	6.26	6.46
1348	5.89	5.90	6.20
1349	5.72	5.71	6.00
1350	5.70	5.71	5.97

*Elevation of the benchmark was set at +10 ft until information on the benchmark is obtained.

The locations were again surveyed between the historical data and the recent surveys are on the magnitude of 1.55 feet. More information was gathered about past surveys to compare the two survey results and to identify the apparent difference in benchmarks. The roadway showed little to no change since the previous survey as shown in Figure 5.6. However, one reading near STA 1343-1344 may be experiencing continued settlement.

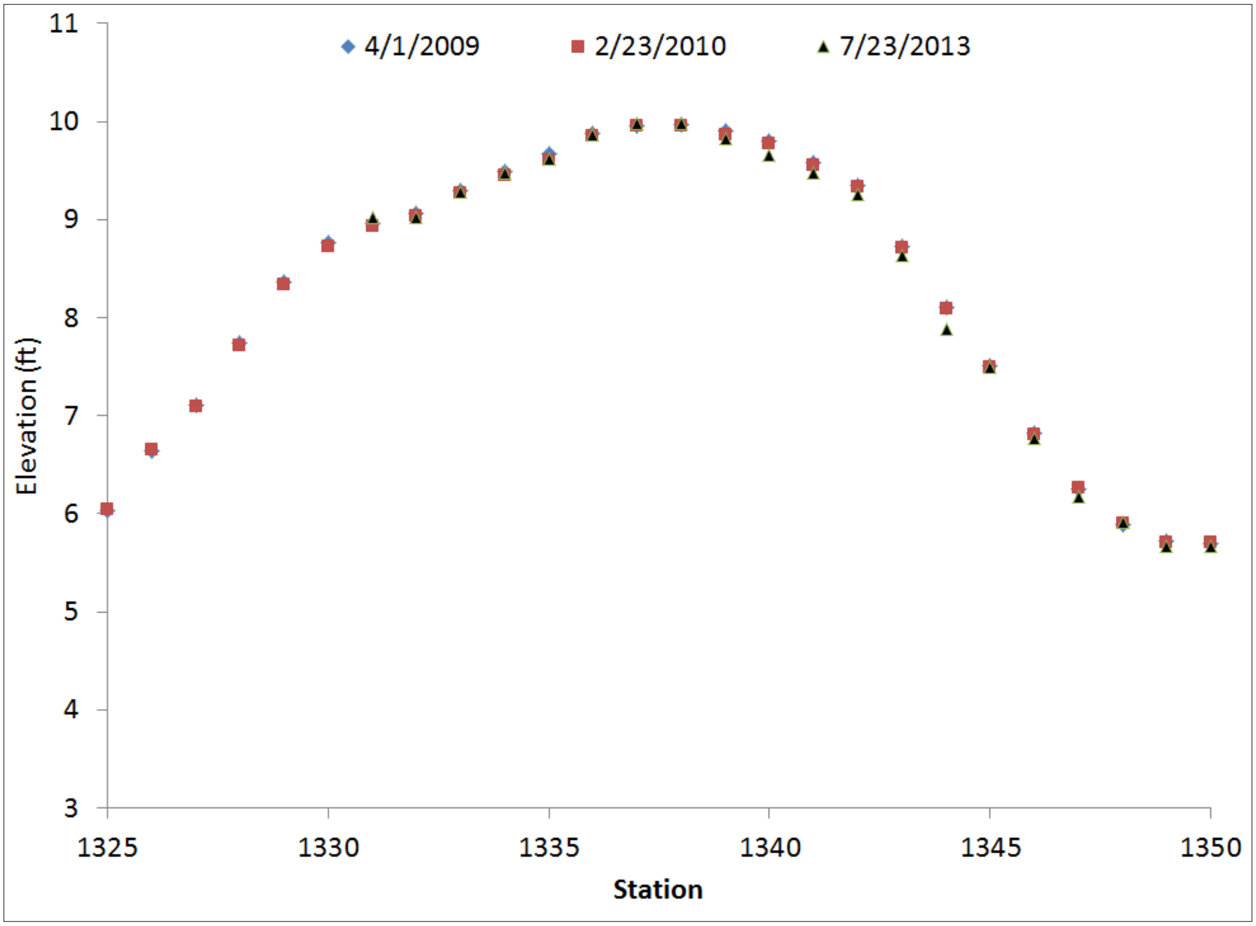

Figure 5.6. Assumed Benchmark Correction for US-1 at Jewfish Creek (High side of SB Roadway)

5.3 Marco Island Executive Airport

Field survey measurements were performed along the taxiway of the Marco Island Executive Airport. The surveying included 8 points along the taxiway and one point on the corner of the South aircraft hangar. Figure 5.7 shows the survey points along the taxiway. Table 5.2 and 5.3 summarize the survey points along the taxiway. Figure 5.8 shows a graph of the top of ground profile for the various surveys performed.

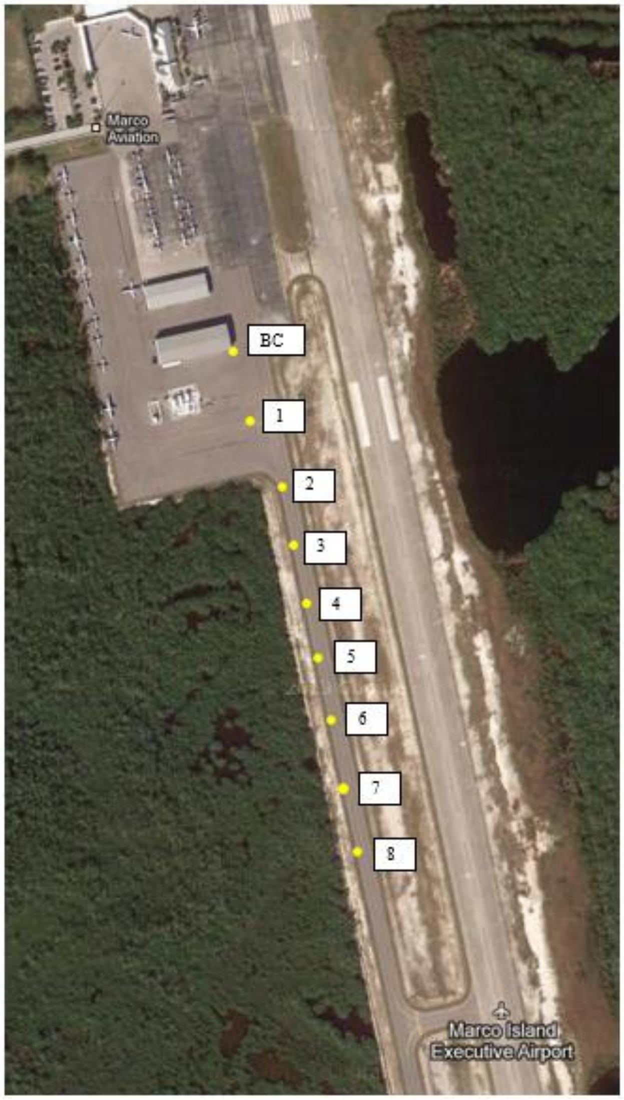

Figure 5.7. Locations of Survey Points for Macro Island Executive Airport Taxiway.

Table 5.2. Summary of Survey Elevations along Marco Island Executive Airport Taxiway.

Date	**1/31/2013**	**3/12/2013**	
Point	**EL (ft)**	**EL (ft)**	**Delta (in)**
BC	10.505	10.45	0.66
1	10.105	10.05	0.66
2	10.09	10.09	0
3	10.03	10.04	-0.12
4	10.04	10.05	-0.12
5	10.03	10.05	-0.24
6	10.04	10.05	-0.12
7	10.06	10.04	0.24
8	10.06	10.05	0.12

Table 5.3. Summary of Survey Elevations along Marco Island Executive Airport Taxiway

Date	**1/31/2013**	**3/12/2013**	**10/26/2013**	
Point	**EL (ft)**	**EL (ft)**	**EL (ft)**	**Delta (in)**
BC	10.505	10.45	10.46	-0.12
1	10.105	10.05	10.06	-0.12
2	10.09	10.09	10.09	0
3	10.03	10.04	10.05	-0.12
4	10.04	10.05	10.05	0
5	10.03	10.05	10.07	-0.24
6	10.04	10.05	10.06	-0.12
7	10.06	10.04	10.05	-0.12
8	10.06	10.05	10.05	0

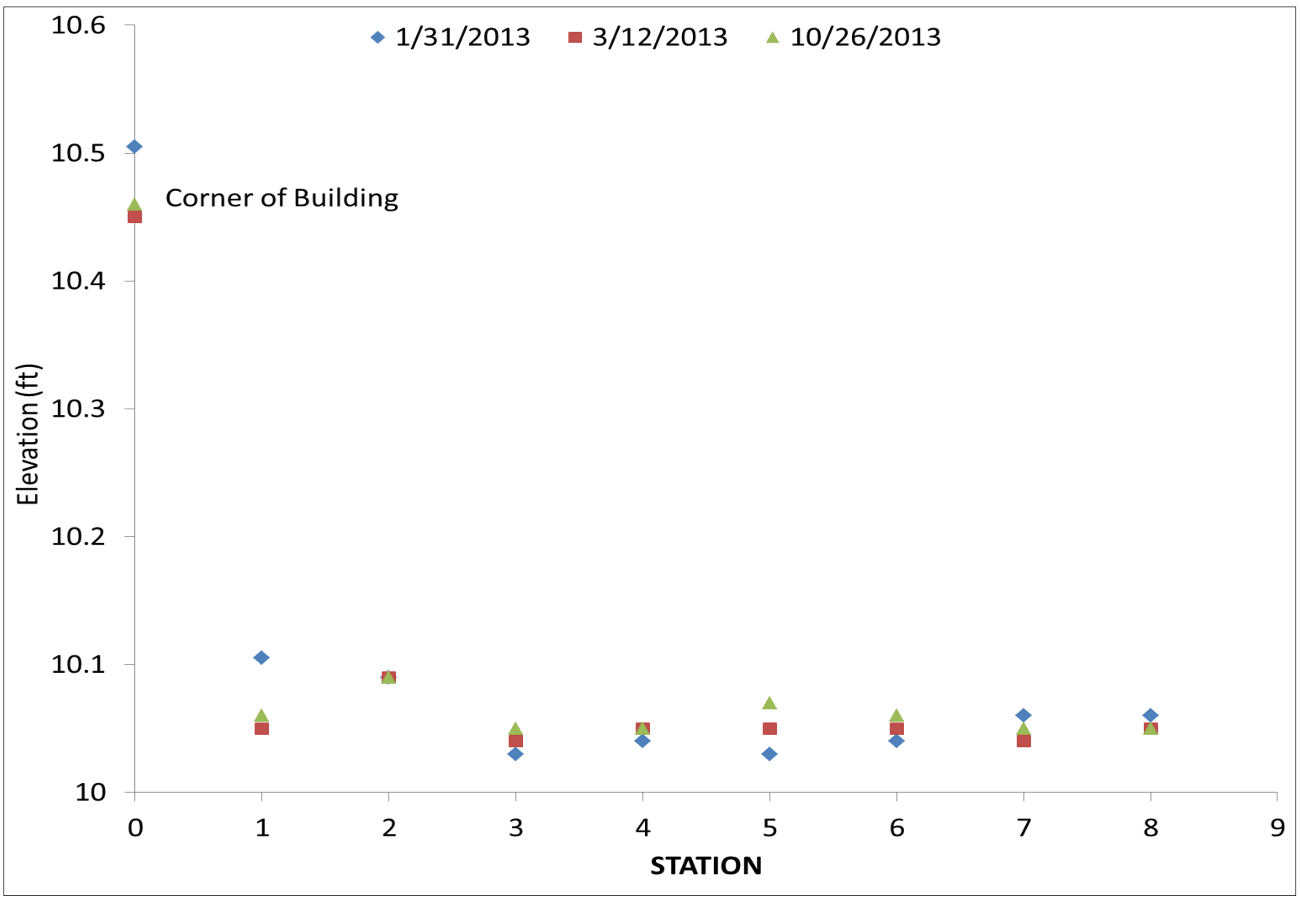

Figure 5.8. Survey Data for Marco Island Executive Airport Taxiway

In all the survey cases, no appreciable movement was detected over the three-year period of the study.

5.4 US-331 Causeway over Choctawhatchee Bay

The US-331 bridge and causeway across Choctawhatchee Bay was first built in mid-1930's where fill was pushed out into the bay to form a causeway comprising over half the entire alignment. The rest was comprised of a single bridge completed in 1940. For the ensuing 70 years, settlement and maintenance was required to combat the loose fill and soft soils over which it was placed. While the original bridge was replaced with a more modern bridge to the west leaving portions of the original bridge abandoned in place, no soil improvements to the causeway were undertaken. Recently, a widening of the entire corridor has begun which adds another parallel bridge and a comprehensive ground stabilization program involving deep and shallow soil mixing. Soils supporting the causeway fill vary including sands, silts, clays, and organic deposits. Figure 5.9 shows a concept section view of the soil treatment program which shows a 10ft thick shallow transfer platform over deep soil mixed columns to a minimum depth of 45ft.

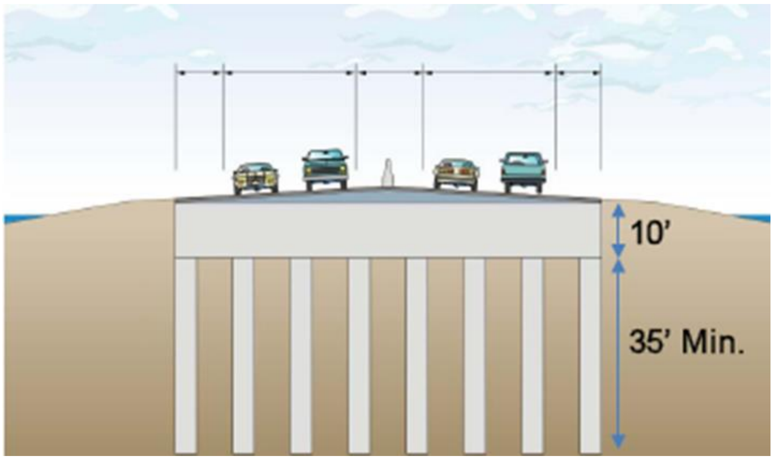

Figure 5.9. Combination of deep and shallow soil mixing used to stabilize causeway.

The overall approach to the soil mixing aspects of the project involved: (1) exploratory drilling, (2) bulk soil sampling, (3) bench scale soil mixing, (4) full scale demonstration elements constructed with varied cement contents, and (5) a surcharge program placed on cement stabilized soil mixed columns extending down to a depth of 45ft. At the time of this report, the project was largely completed and the surcharge test program was over a year old.

Typical soil strength profiles from the north and south ends of the project show consistently the roadway crust over a weak layer of soils to a depth of 40-45ft (Figures 5.10 and 5.11). Although the magnitude of additional load is minimal in most areas, treatment to depth of 45ft (elev. -40) encompasses all potentially weak soils.

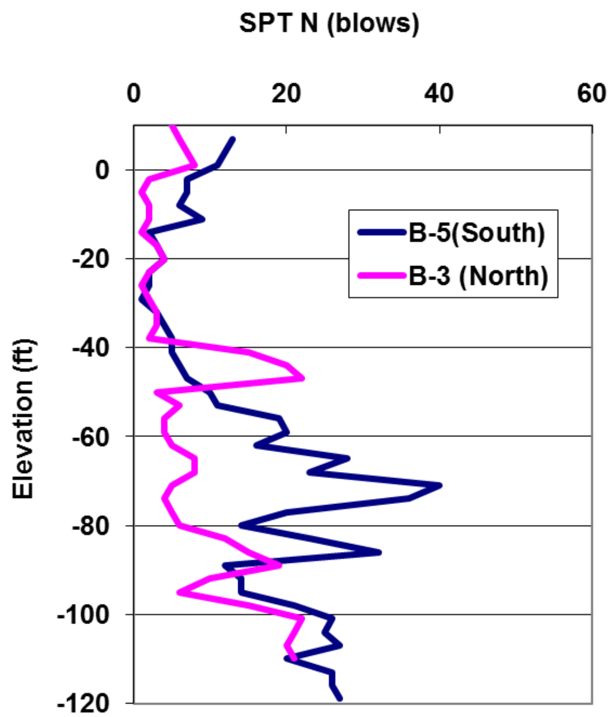

Figure 5.10. Soil strength profile from SPT blow counts at SR-331 soil mixing site.

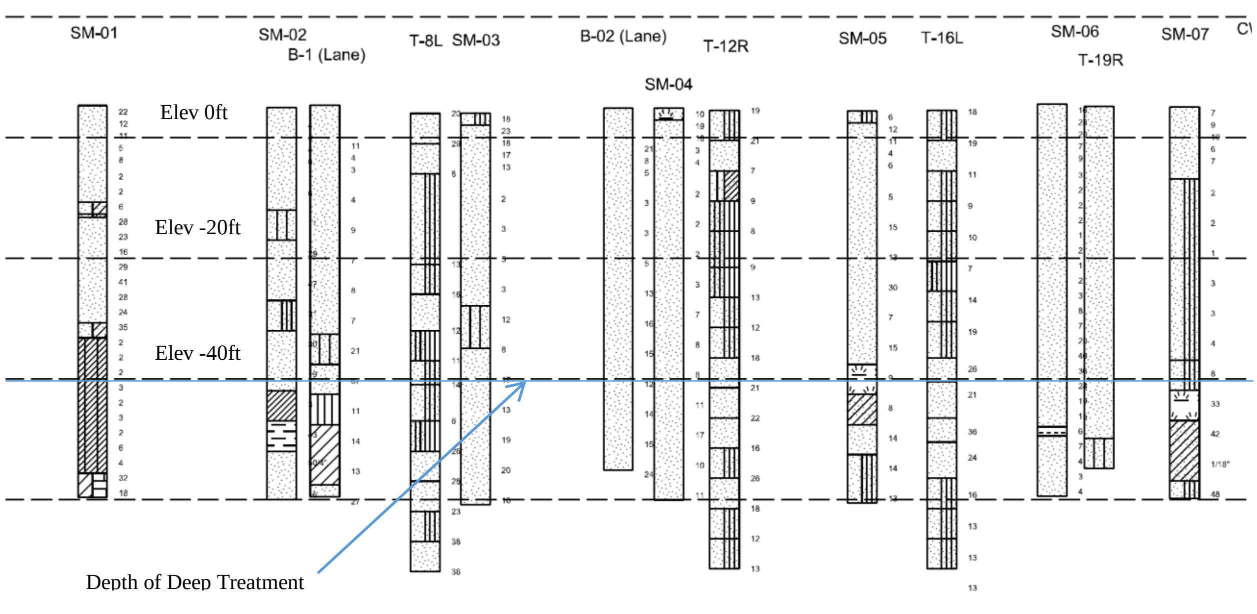

Figure 5.11. Sample of exploratory borings taken along southern portion of causeway.

Dedicated soil borings were conducted after the initial explorations to recover larger quantities of soil for bench scale tests. Bench scale tests varied the w/c ratio of the injected slurry as well as both the cement content and cement type where either 100% Portland cement was used or a 50/50 mix of Portland cement and slag. Table 5.4 shows the bench scale test matrix where CF is the cement factor in units of kg/m^3. This corresponds to 170, 340, and 425pcy for Table 5.4 CF values of 100, 200, and 250, respectively.

Table 5.4. Bench-scale test matrix for US-331 soil mixing project.

Priority	Batch ID	Cement	W/C	CF	Mixed On	Tue 09/17/13	Wed 09/18/13	Thu 09/19/13	Fri 09/20/13	Sat 09/21/13	Sun 09/22/13	Mon 09/23/13	Tue 09/24/13	Wed 09/25/13	Thu 09/26/13	Sat 10/05/13	Sun 10/06/13	Mon 10/07/13	Tue 10/08/13	Wed 10/09/13	Thu 10/10/13	Sun 12/08/13	Mon 12/09/13	Tue 12/10/13
	GROUT MIX 1	PBFC	1.25		9/10/13								14						28					
	GROUT MIX 3	OPC	1.25		9/10/13								14						28					
1	S1-B01	PBFC	1.25	100	9/10/13	7							14						28				90	
2	S1-B02	PBFC	1.25	200	9/10/13	7							14						28				90	
3	S1-B04	OPC	1.25	100	9/10/13	7							14						28				90	
4	S1-B05	OPC	1.25	200	9/10/13	7							14						28				90	
5	S2-B07	PBFC	1.25	200	9/10/13	7							14						28				90	
6	S2-B08	OPC	1.25	200	9/10/13	7							14						28				90	
7	S3-B09	PBFC	1.25	200	9/10/13	7							14						28				90	
8	S3-B11	OPC	1.25	200	9/10/13	7							14						28				90	
9	GROUT MIX 2	PBFC	1.00		9/11/13									14						28				
10	GROUT MIX 4	OPC	1.00		9/11/13									14						28				
11	S1-B03	PBFC	1.00	100	9/11/13		7							14						28				90
12	S1-B06	OPC	1.00	100	9/11/13		7							14						28				90
13	S2-B15	PBFC	1.00	200	9/11/13		7							14						28				90
14	S3-B10	PBFC	1.00	200	9/11/13		7							14						28				90
15	S3-B12	OPC	1.00	200	9/11/13		7							14						28				90
16	S4-B13	PBFC	1.25	250	9/11/13		7							14						28				90
17	S4-B14	OPC	1.25	250	9/11/13		7							14						28				90

Full scale demonstration elements were installed using the above mix ratios whereby lab results could be correlated to field performance. To ensure quality assurance measures could be properly carried out, minimum unconfined compression strength of 150psi was established such that coring could be reasonably performed and cores could be retrieved. Lower strength materials make coring impractical. Figure 5.12 shows the spoils that were left after the twin 6ft augers were finished mixing the 45ft columns. The multiple blades of the mixing paddles can just be seen on the left edge of the picture. Both the deep and shallow elements were installed with the same twin auger system.

Figure 5.12. Soil mixing spoils around twin auger soil mixed demonstration elements.

A section of the roadway where only a couple feet of planned new roadway load was selected to test the performance of the deep soil mixing effectiveness. This involved loading the treated soil with an embankment load from 19ft of fill where pore pressure and displacement transducers were installed throughout the treated soil pattern. Figures 5.13 - 5.16 show the instrumentation scheme as well as the plan and elevation views of the test section.

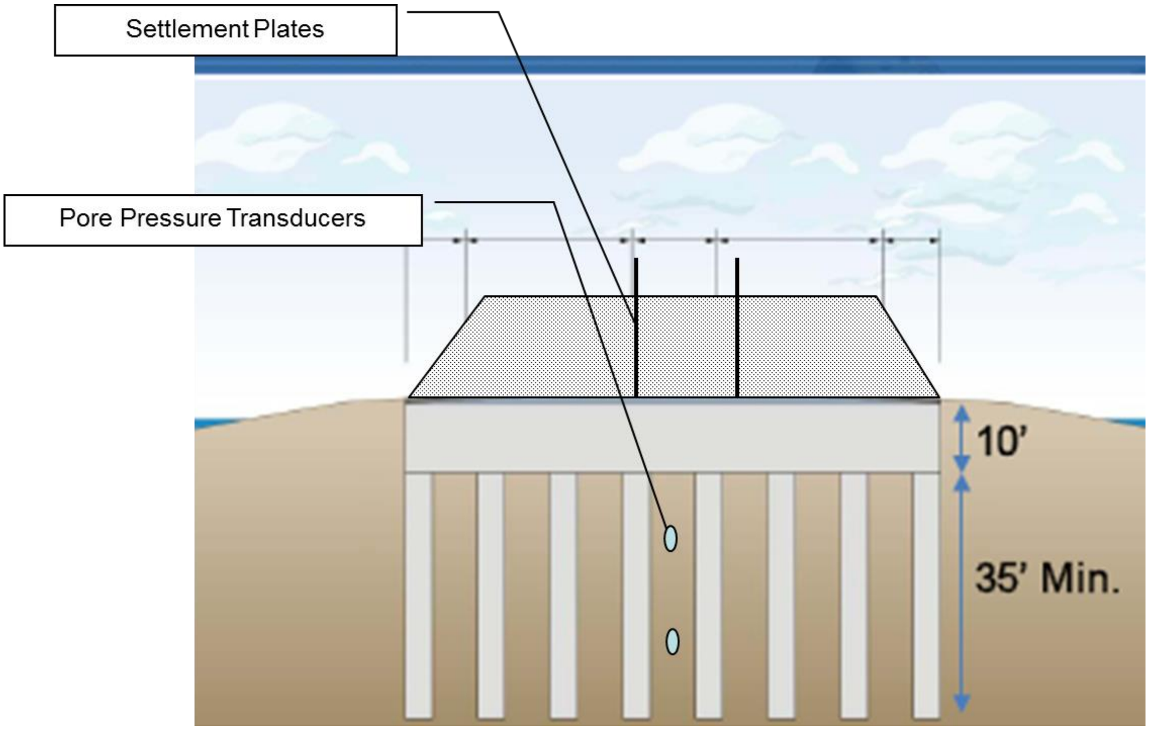

Figure 5.13. Instrumented surcharge program.

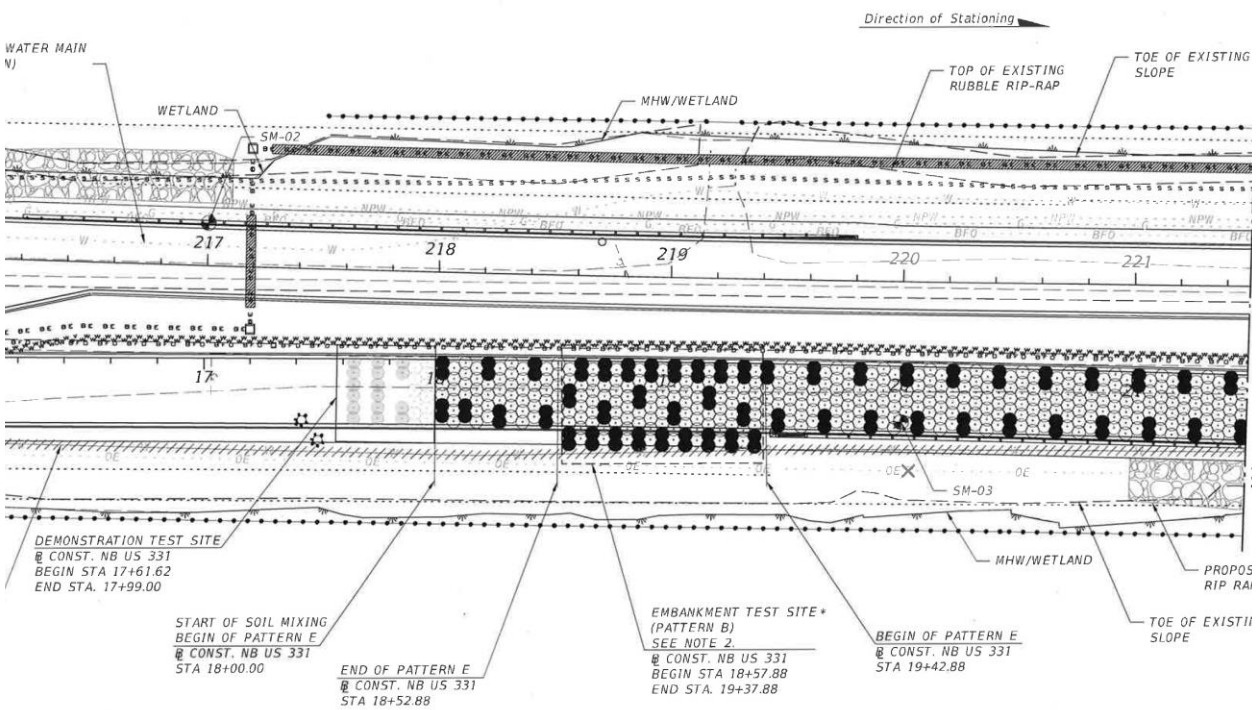

Figure 5.14. Plan view of south causeway treatment layout and test section (FGE, 2014).

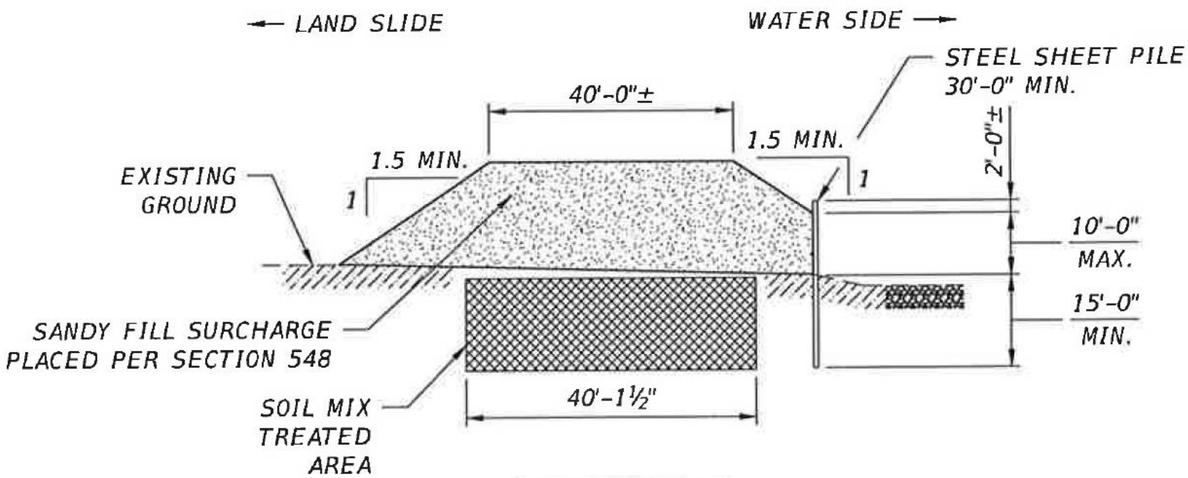

Figure 5.15. Surcharge test section showing sheet pile containment (FGE, 2014).

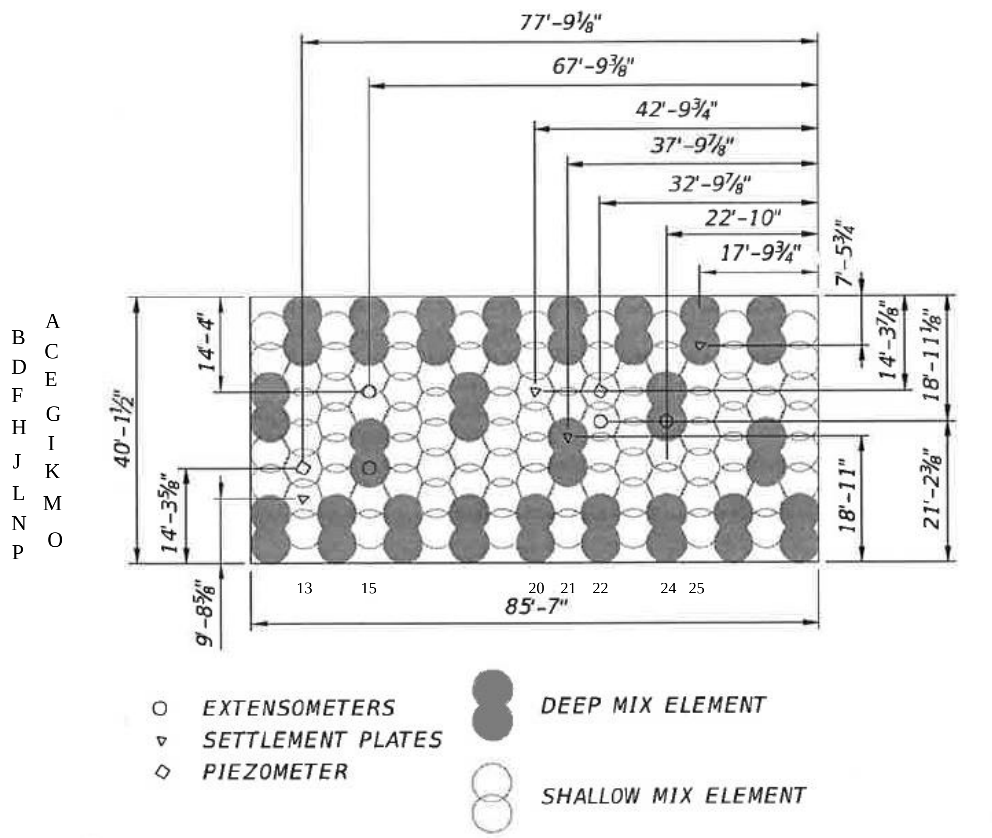

Figure 5.16. Plan view of surcharge test area (FGE, 2014).

As noted in Figure 5.14, the test area was slightly north of the project starting point (right is north). Elements were installed in pairs but each of the twin columns was denoted individually by row a column where the rows were A to P (west to east) and columns were numerical starting from the south with number 1 (Figure 5.16). Instrumentation was either installed in shallow element or deep elements. Deep elements shown with dark fill in plan views above. The instrumentation naming took on the name of the element in which it was installed (Table 5.5).

Table 5.5. Instrumentation location/naming convention (adapted from FGE, 2014).

Element	K13	M13	E/G15	K15	F20	I21	F22	H22	H24	C25
Instrument Type	Piezo.	Stlmnt. Plate	SMM Ext.	DMM Ext.	Stlmnt. Plate	Stlmnt. Plate	Piezo.	SMM Ext.	DMM Ext.	Stlmnt. Plate

As all instruments used were based on vibrating wire technology, thermistors within the unit were necessary to correct for normally experienced temperature variations and the effects on the natural frequency of the taught wire at the core of the device. For test programs in a laboratory or where only short duration tests are anticipated, the temperature is often disregarded or not even recorded. In this case, the longer duration and potential for cement hydration-induced temperature effects made it necessary to record these values. Figure 5.17 shows the temperature traces for almost a year after soil mixing; sensors were installed after at least 28 days had elapsed.

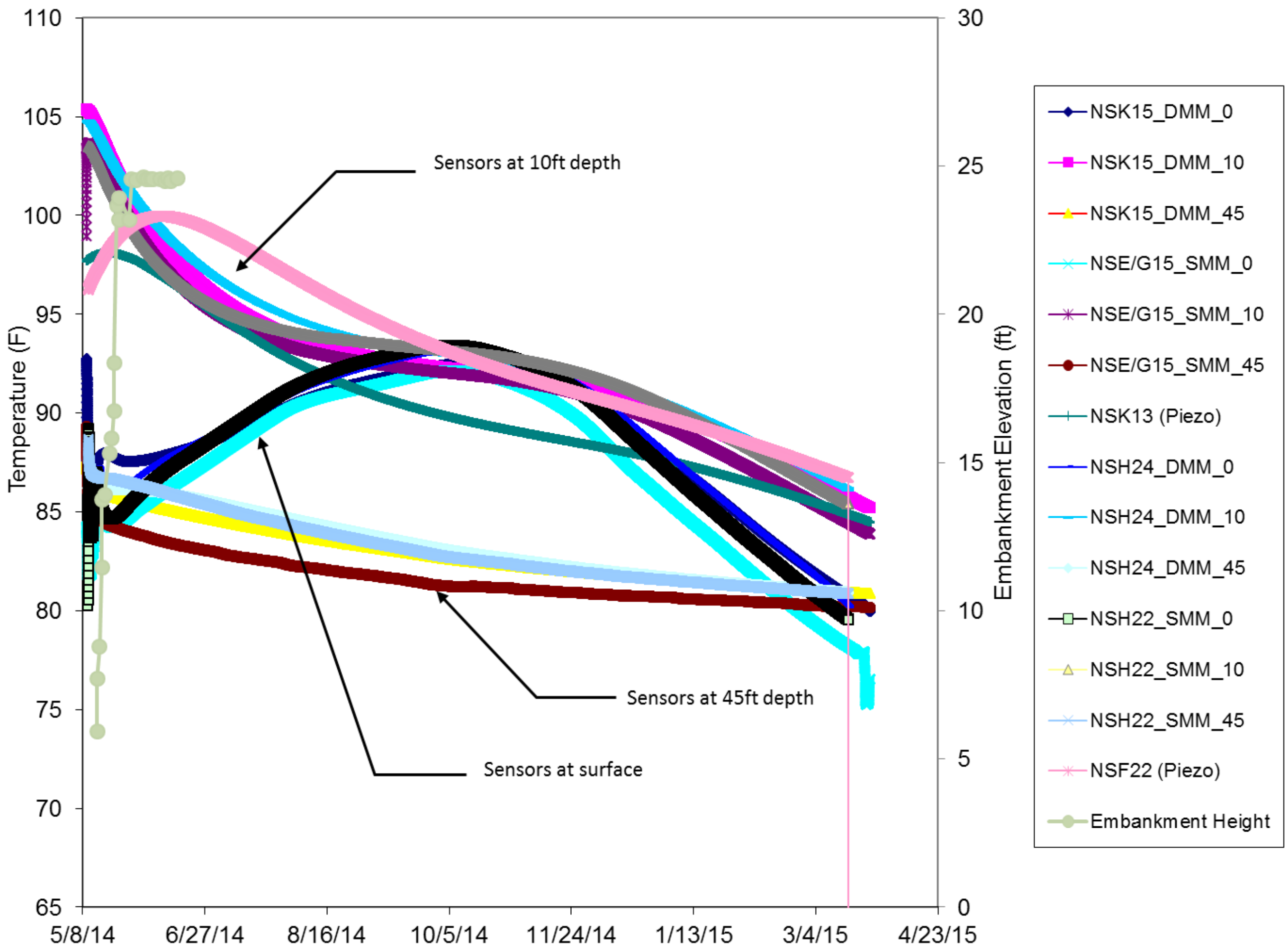

Figure 5.17. Temperature within the soil mix treatment zones (natural soil temperature is 68°F)

Due to the cement factor (10-17lbs/cu ft) used to achieve stabilization, elevated temperatures persisted for the year monitoring period installation; elevated temperatures still existed at the time the data collection system was disconnected. Average annual soil temperature in that region of the state is approximately 68F. Sensors at the surface started at a value close to air temperature and then increased due the insulating effect of the 19ft thick surcharge blanket. Surcharge was removed at the end of the monitored data (last data points shown). The sensors at 10ft depth directly beneath the shallow mass mixed region started at the highest temperature due to the concentration of cement in the upper transfer platform immediately above. Finally, the sensors at the base of the deep columns showed the coolest overall trend where only the tip of the columns influenced the local temperature and the soil beneath could diffuse more effectively than near the rest of the sensors.

Temperature data was used to correct not only the sensor frequency response but also the thermal expansion / contraction of the soil and steel rods between sensors. The compression was then computed for each of three zones beneath the ground surface: (1) the shallow mass mixed zone from 0 to 10ft, (2) the depth from the bottom of shallow platform to the bottom of deep columns, 10 to 45ft, and (3) the depth from beneath the deep columns to a datum set 15ft below the columns, 45 to 60ft. Figure 5.18 shows the fully loaded surcharge and Figure 5.19 shows each of the individual measurements along with the combined overall compression summing each of the three sensors. Surcharge was left in place for 11 months.

Figure5.18. Surcharge / embankment load fully in place on test section (approx. 19ft).

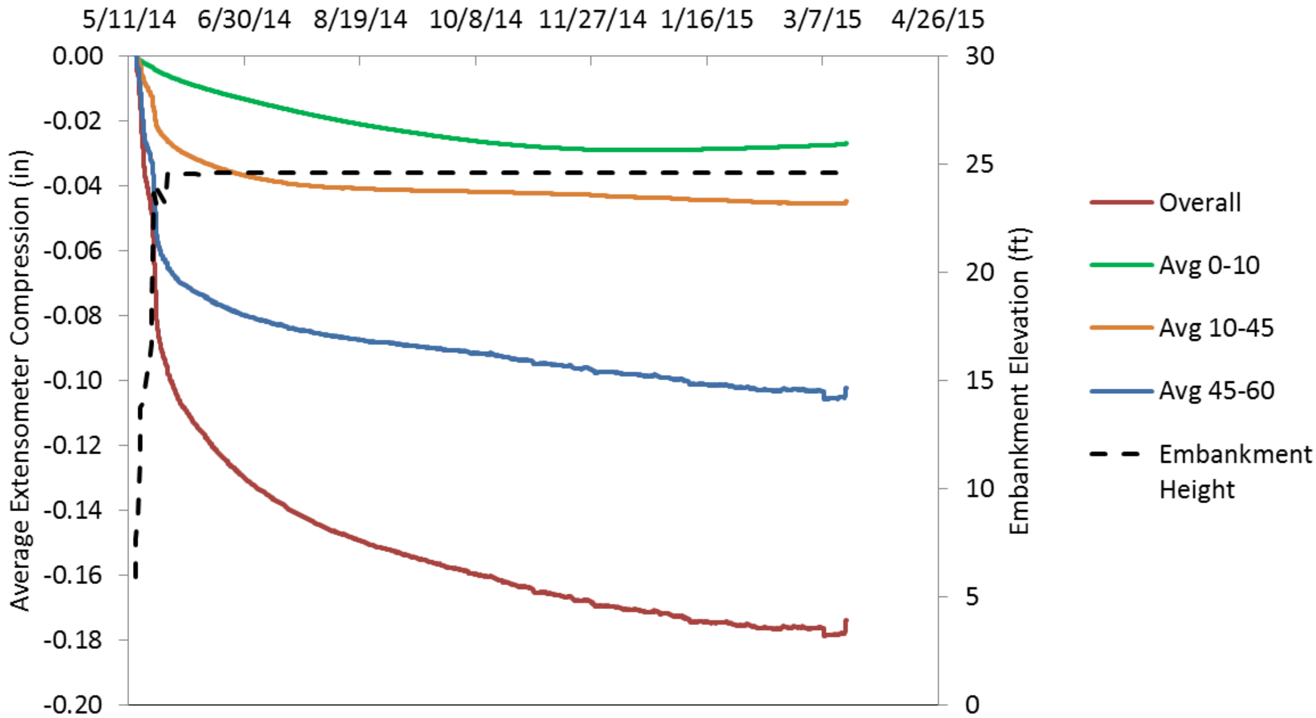

Figure 5.19. Settlement measured from 19ft surcharge loading.

While the overall settlement showed 0.04in of additional movement after completion of loading, much of this movement was more likely due to thermal cooling and contraction of the entire soil block. Assuming an average 15F drop in temperature for the entire 40ft soil mass and a thermal coefficient of expansion for cemented sand, this equates to 0.05in of contraction (settlement). This movement is negligible with regards to the intended roadway usage/purpose regardless of whether it is real movement or merely calculated movement from temperature corrections.

Plate load tests performed both directly over a deep column and between deep columns showed negligible movement of test elements outside the surcharge area. These tests were performed to demonstrate wheel loads would not affect the upper shallow platform. Figure 5.20 shows the results of the plate load tests.

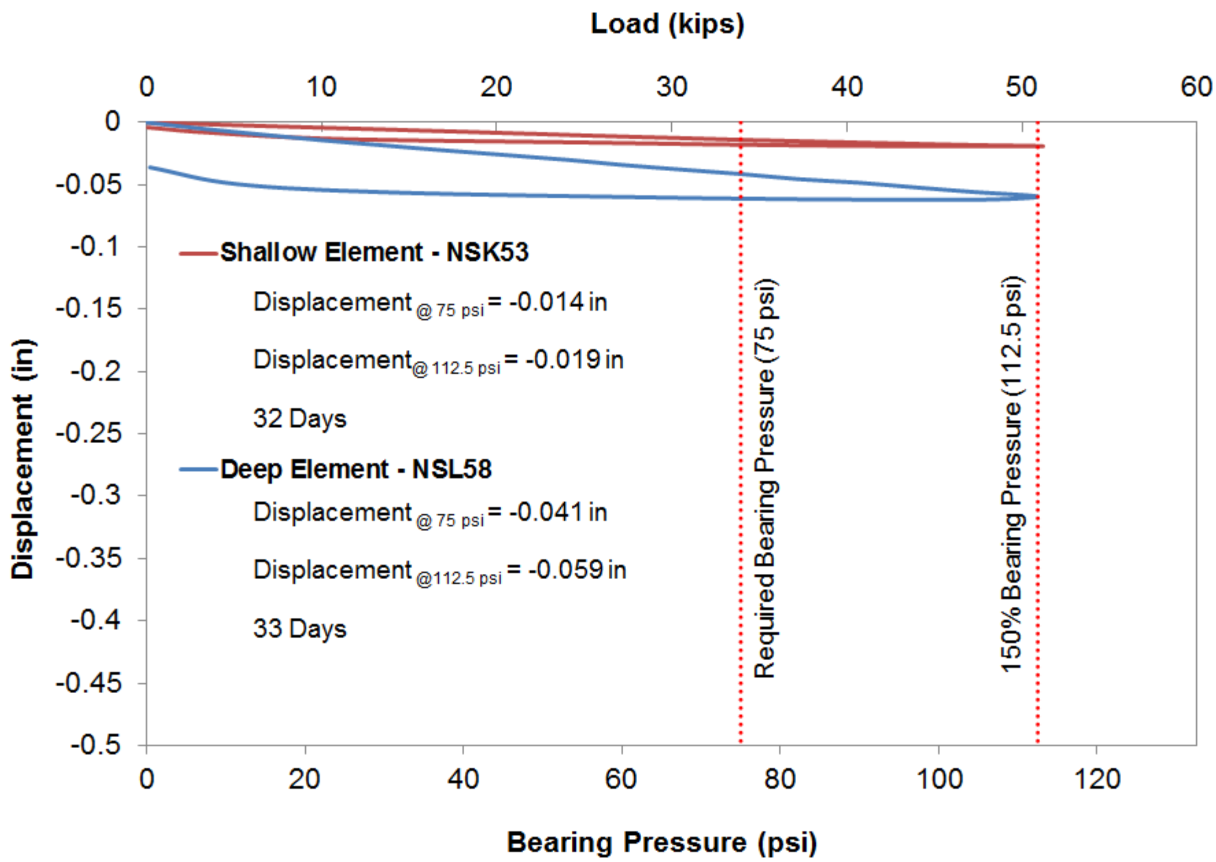

Figure 5.20. Plate load test results (FGE, 2014).

5.4.1 Quality Control/Quality Assurance

The on board computer systems of the soil mixing systems used at the US-331 site tracked the volume of cement grout installed, depth of the blades, number of blade rotations, grout pressure, inclination and forces on the auger. These systems aid in providing confidence in the as-built soil mixed elements. Commonly, coring of cured elements (discussed above) or wet grabs of the near surface mixed material are methods of obtaining test specimens. For this project, quality control and assurance protocols required a minimum amount of cement (CF > threshold), a minimum number of blade rotations (BRN > threshold), a minimum compression strength of cored and wet grab specimens, and a minimum frequency of sample testing not fall below 2% after the first 200 elements were installed (4% prior to 200 elements). Figure 5.21 shows an example field log demonstrating the installation monitoring system used by the contractor.

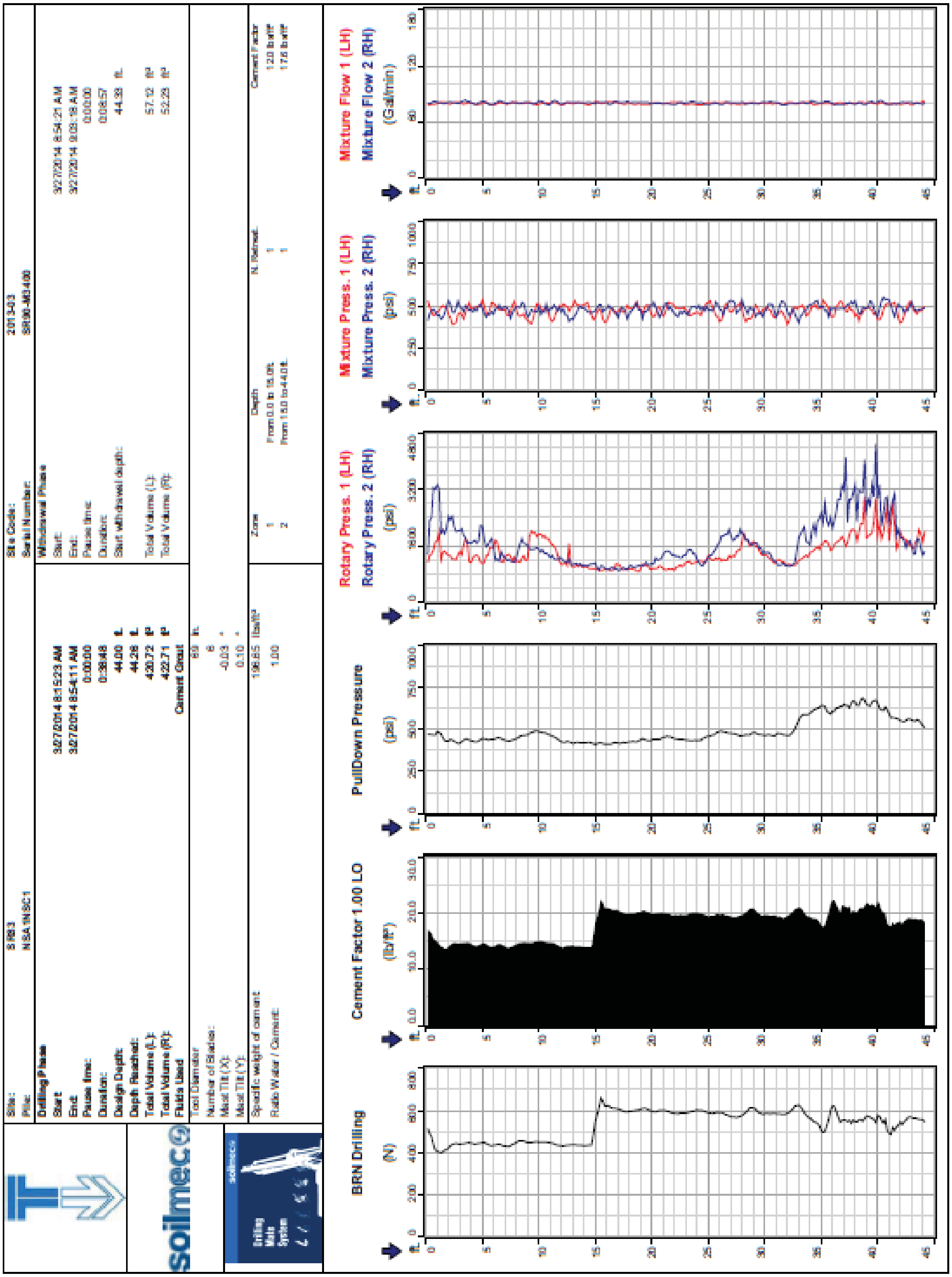

Figure 5.21. Automated measurements taken by on-board quality control system (FGE, 2014).

5.5 State Road 37 over Alafia River (Dynamic Replacement)

While not directly akin to soil mixing with cementitious binders, dynamic replacement and mixing can provide an alternative to cement based mixing soil mixing. Coincidently, a DRM program was undertaken concurrent to the study timeline. The findings of that ground improvement program are provided herein for direct comparison.

This case study dealt with the 2013-14 widening of an existing two lane rural bridge built in 1951 extending 285ft over the Alafia River on 19 – 15ft slab beams. Persistent settlement of the bridge piers required bridge replacement; narrow lanes and increased traffic demand further necessitated widening. The replacement was slated to have four lanes comprised of two side by side four span bridges 41ft wide. Due to soft clays and organics in the proposed widened alignment, dynamic replacement (DR) was selected as the ground treatment methodology.

While the main river crossing was only 20-30ft wide during the dry winter months, the flood plain of the river extended over 300ft up to and beyond the limits of the existing bridge end bents. The flood plain had a 6-10ft surficial layer of very soft clay (LL = 128; PI = 79) with *pockets of organic soil* that were speculated to be remnants of an undocumented phosphatic clay slime spill from up river. The material was too soft to support foot traffic in areas without a vegetation mat even during the dry season. Below the surficial clay was 8-10ft of grey / tan clayey sand with some silt. Limestone was encountered thereafter.

At its highest, near the end bends, the new approach embankment was planned to be 12ft above the existing grade. Since the clay deposits were too deep to easily excavate with the high groundwater water level and proximity to the river, soil stabilization was selected using DR. Left untreated, the soft clay was estimated to settle as much as 1.5ft over the period of a year; the clayey sands were estimated to take only a couple of weeks.

The DR program entailed: detailed CPT delineation of the soft clay across the site, a pilot program to set production protocols and acceptance criteria, production installation of 273 sand columns, and long term monitoring of settlement and pore pressure.

<u>Soft Clay Delineation.</u> CPT soundings were conducted on 50ft intervals along 0.25 miles of the project length and 25ft across the widened portion of the alignment. In all, 53 soundings were performed to delineate the extents of the soft clay, Figure 5.22. The design pattern and pilot test location were assigned accordingly so as to test the performance of a DR program in a representative region of the alignment.

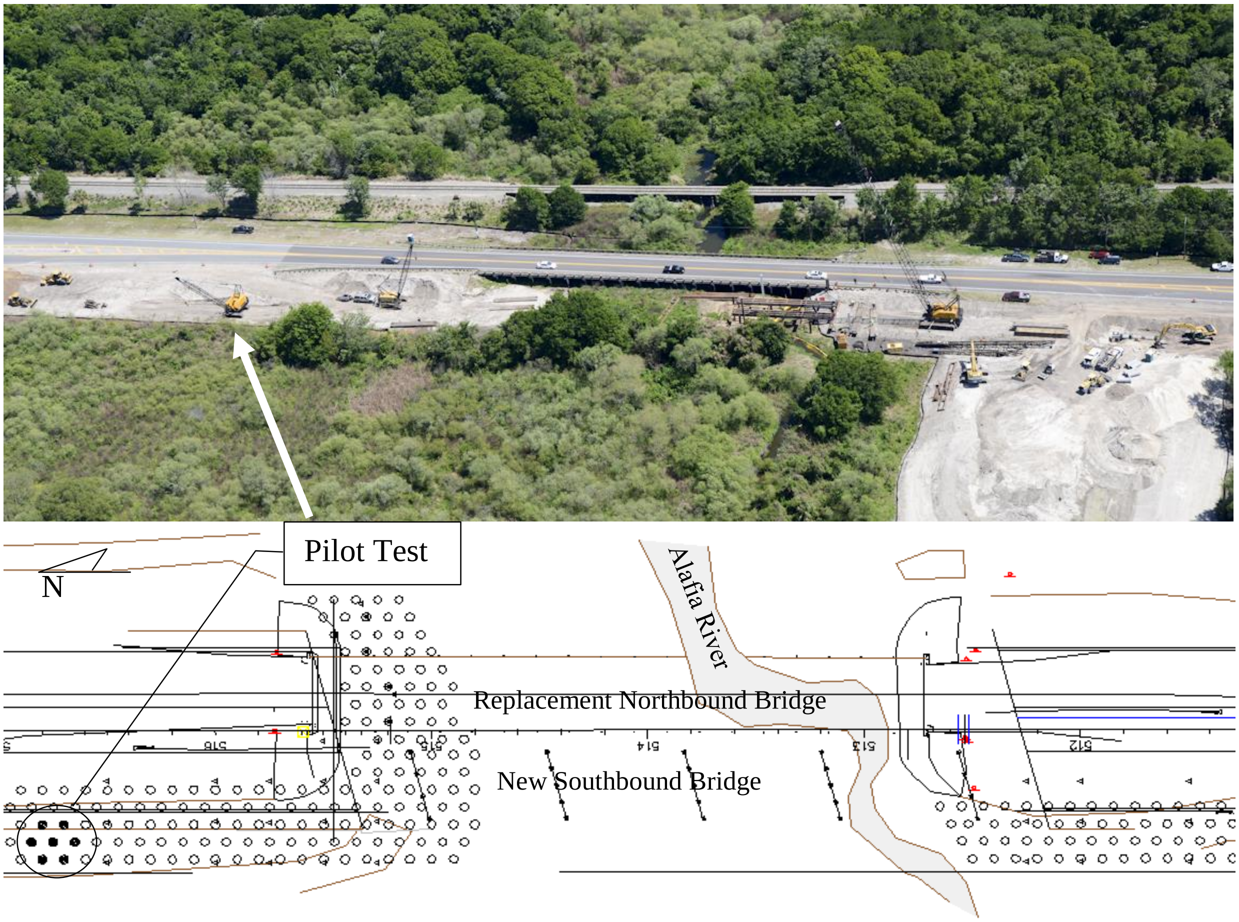

Figure 5.22. Partial aerial and plan views of overall treatment using 273 sand columns.

5.5.1 Dynamic Replacement and Mixing Pilot Program

Similar to the US-331 project, a demonstration / pilot program was conducted wherein CPT soundings were performed at each sand column location, crane drop energy efficiency was checked, the test pattern was laid out, pore pressure transducers were installed, sand columns were installed, and the resulting size (depth and diameter) of the sand columns was measured.

Cone penetration tests were performed at each of 7 column positions to set maximum energy requirements, find soil layer boundaries (for piezometers) and serve as a baseline before treatment.

Drop Energy. Test drops of the 4 ton DR pounder were conducted to determine the crane drag. While the crane operator initially opted to use a two-part line, test drops for this configuration showed only 44% of the anticipated acceleration. Subsequently, the same crane was reconfigured with a single-part line and a safety governor to control accidental drops was disabled allowing the acceleration to jump to 75%. The impact acceleration (and force) also increased from 9.5g to 27g. Figures 5.23 and 5.24 show the crane configurations and acceleration response, respectively.

Figure 5.23. Crane rigged with two-part line (left) and single-part line (right).

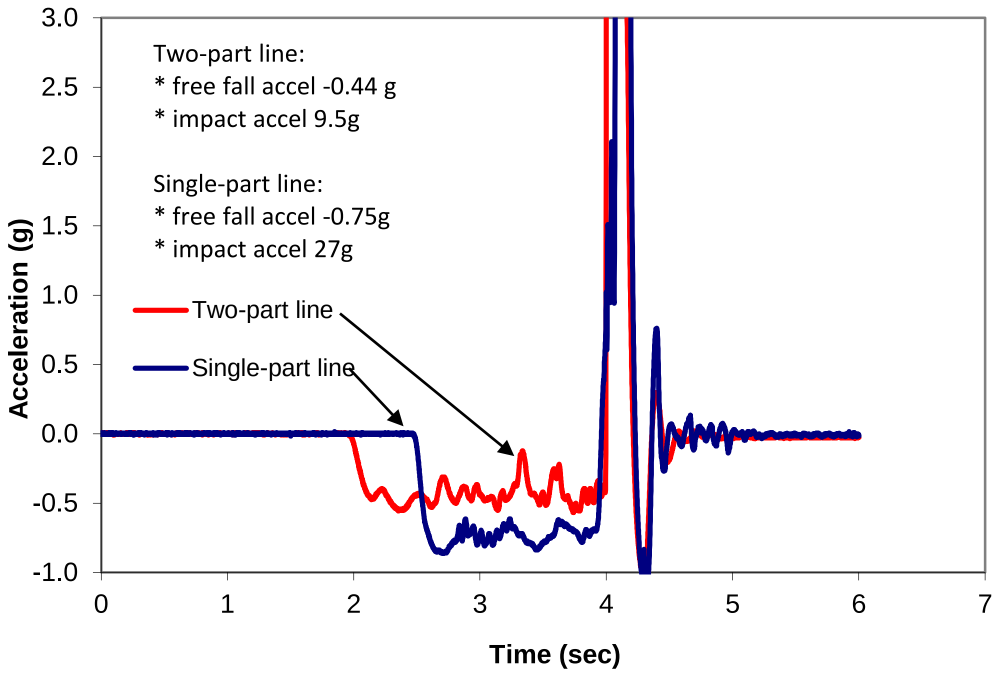

Figure 5.24. Effect of rigging on acceleration response (6m drops).

<u>Test Pattern.</u> A seven position test pattern was established based on a 3m equilateral triangular configuration and past experience that the resulting sand column would be circular with a diameter 2 times the pounder width (s/d = 2.5). Figure 5.25 shows the plan view of the pilot test layout complete with CPT and piezometer locations.

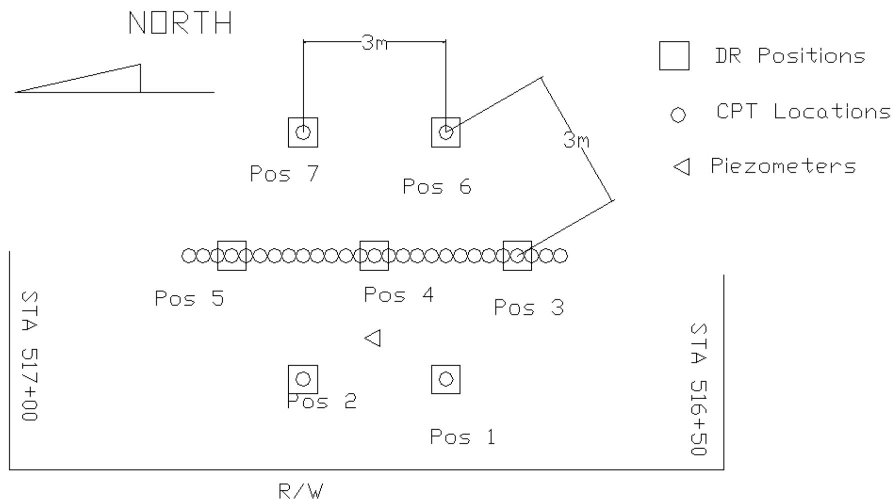

Figure 5.25. Pilot program layout (plan view).

Piezometers. Two piezometers were installed at depths of 1.7 and 3.5m roughly in the middle of the two layers (Figure 5.26) and directly between positions 1, 2, and 4. As a point of reference, friction ratios (FR) below 1% generally are associated with sand; 2 to 4% clayey material; >4% high plasticity clay or organics. Note a sand lens was detected in the bottom half of the high plasticity phosphatic clay layer.

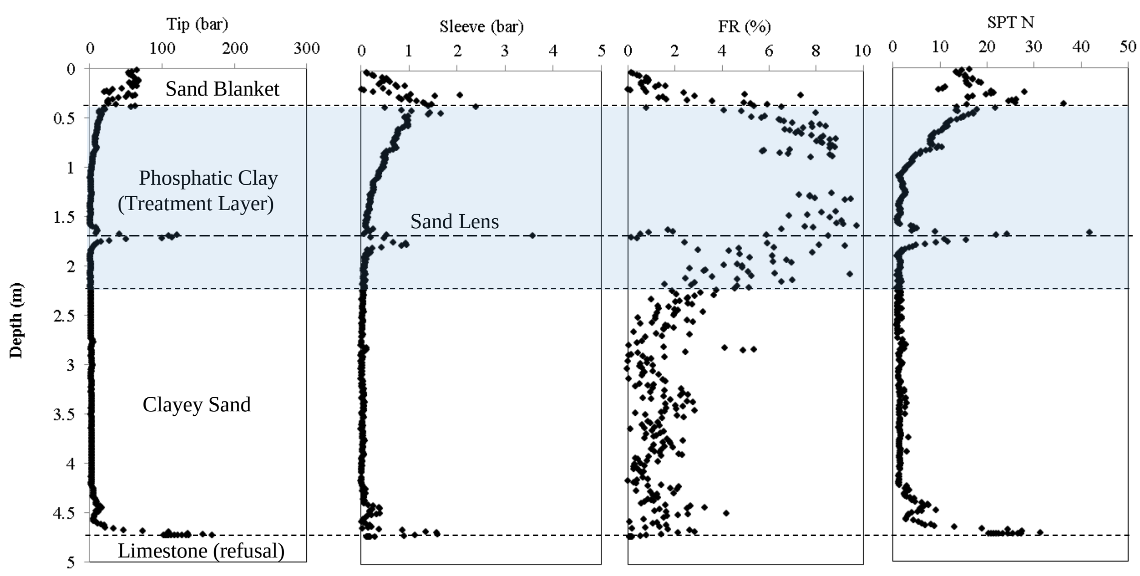

Figure 5.26. CPT soil profile used to set piezometer depths (installed to 1.7m and 3.5ft).

Direct push-in, vibrating wire pore pressure transducers were selected due to the ease of installation and the reliability of the measurement technology. A data collector was set up on-site equipped with cellular accessibility to allow for remote monitoring of the pore pressure data.

Sand Column Installation. The thickness of the sand blanket, which provides equipment support, must be balanced with the extra penetration resistance introduced by the newly placed material. The sand layer thickness was varied from 0.6 to 1m to show this effect; CPT tip stress data taken at each drop position predicted that more energy would be needed with thicker sand thicknesses based on Mullins, et al. (2000). This also confirmed the pounder would not penetrate too deeply upon the first impact (where no on-site experience had yet been developed). Predictions agreed well with that observed.

Starting with position 1, sand columns were installed with different energy protocols to assess the effectiveness of column advancement. Positions 1 - 3 had a 3ft sand blanket while positions 4 - 7 had a 2ft blanket. The net effect, less energy was needed to fully embed the pounder and advance the sand column more deeply at the latter positions. Impacts were terminated when two consecutive blows produced the same or lower embedment/crater depths.

Post Treatment Evaluation. Pore pressure measurements were continuously taken over the duration of the pilot program and subsequent construction (1 year). Figure 5.27 shows the pore pressure records over a one month period during and directly following the pilot test program.

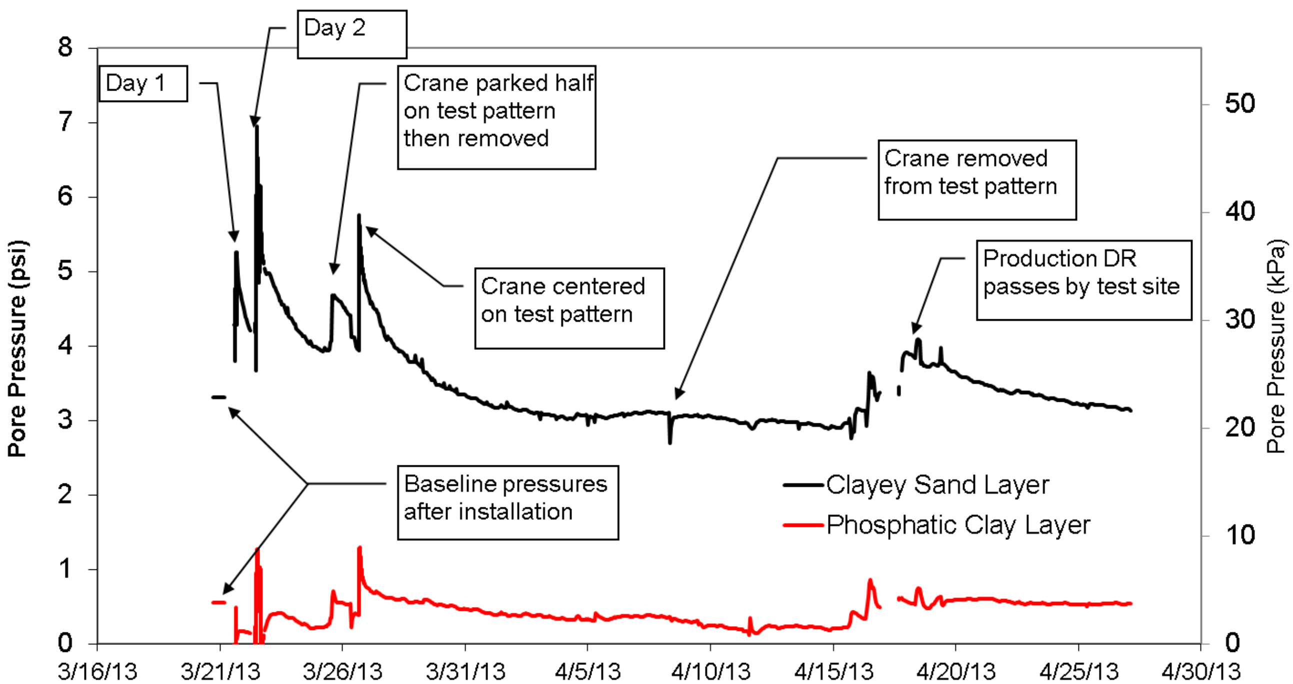

Figure 5.27. Pore pressure response over entire test program.

Pore pressure dissipation was much faster than expected in the high plasticity clay. After day 1, pore pressure levels dropped nearly to the initial baseline prior to day 2. The impacts from day 2 which involved all 7 locations showed almost full dissipation over the following weekend break. The next Monday (3/26/13) a crane was inadvertently positioned partially over the test pattern (Figure 5.22 white arrow) and the contractor quickly removed it. However, given the value of this type of data, the 70ton crane was intentionally repositioned directly over the test area to obtain further pore pressure dissipation information (Figure 5.27). Although not shown, pore pressure from embankment construction was negligible and was dominated by river level fluctuations from rainfall.

Sand column depths were determined from CPT soundings performed at each impact position (Figure 5.28). Column width was quantified from a line of 27 soundings performed on a 0.3m spacing extending from 3ft south of position 3 to 3ft north of position 5 (Figure 5.25). Figure 5.29 shows the soil profile after treatment. The thicker sand blanket over position 3 resulted in a lower overall crater depths and a shallower sand column. Moving toward position 5, the sand column was progressively deeper accounting for the reduced sand blanket and deeper soft clay deposit.

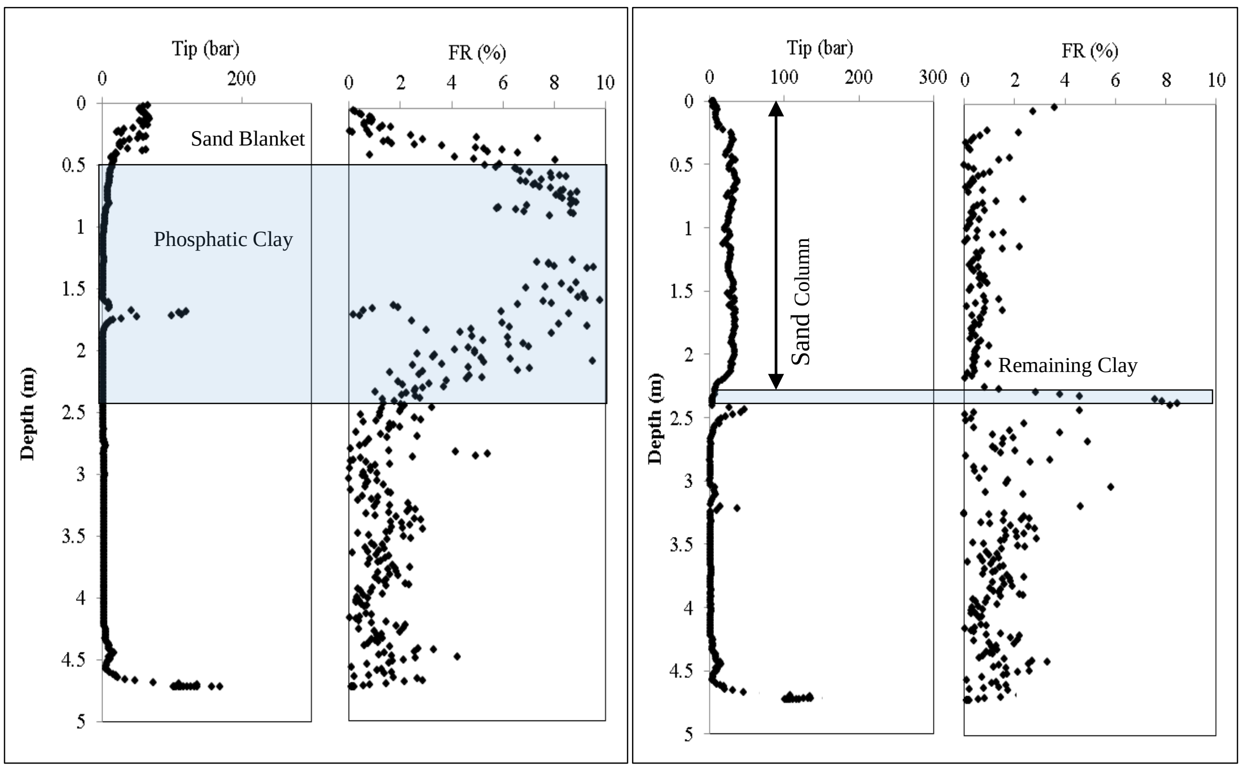

Figure 5.28. Position 4 CPT soundings before (left) and after impacts (right).

Interestingly, the position 4 sand column appeared to have burst laterally (the *M* of *DRM*) as sand layers were detected in adjacent CPT profiles within the clay. This is more common of a central column confined by surrounding columns and is more prevalent in production where there are more confined columns than peripheral.

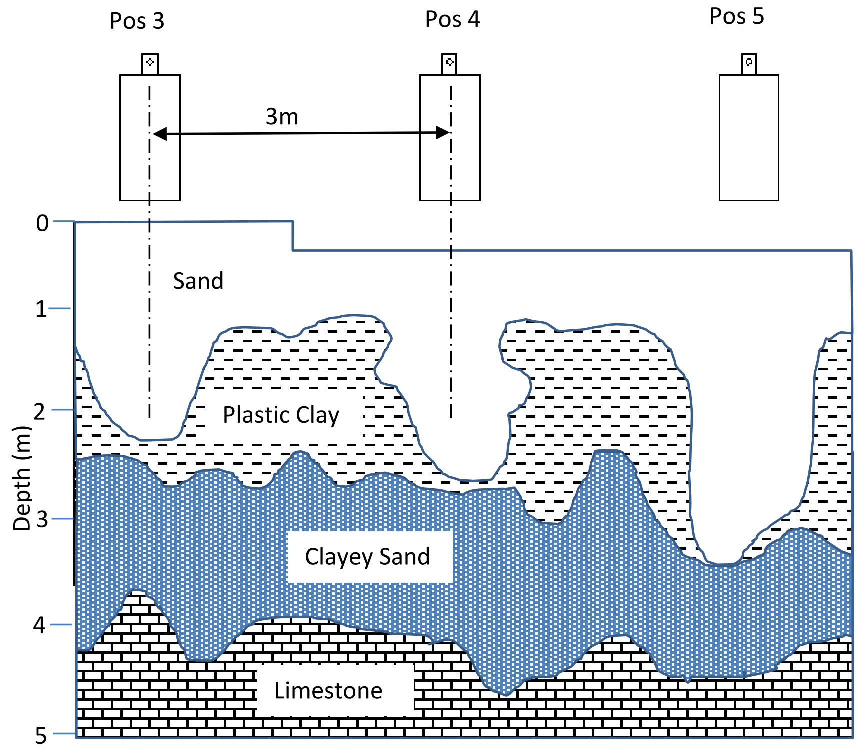

Figure 5.29. Profile view of CPT slice through Positions 3, 4, and 5 after treatment.

The sand column depth was correlated to the cumulative crater depth as a means of quality assurance (Figure 5.30). Position 1 also had CPT soundings performed between each impact which provided column depths for lower cumulative crater depth values. Intuitively, the compacted sand column will both drive deeper and/or expand laterally with continued impacts. Initially the sand columns advanced at a rate roughly half the cumulative crater depth per impact which accounted for the compaction of the loosely placed sand used to fill the crater (down to ~8ft). Afterwards, the sand columns advanced less per impact where the column then expanded more in response to the additional resistance. Figure 5.30 shows the results of both the pilot program and production verification tests.

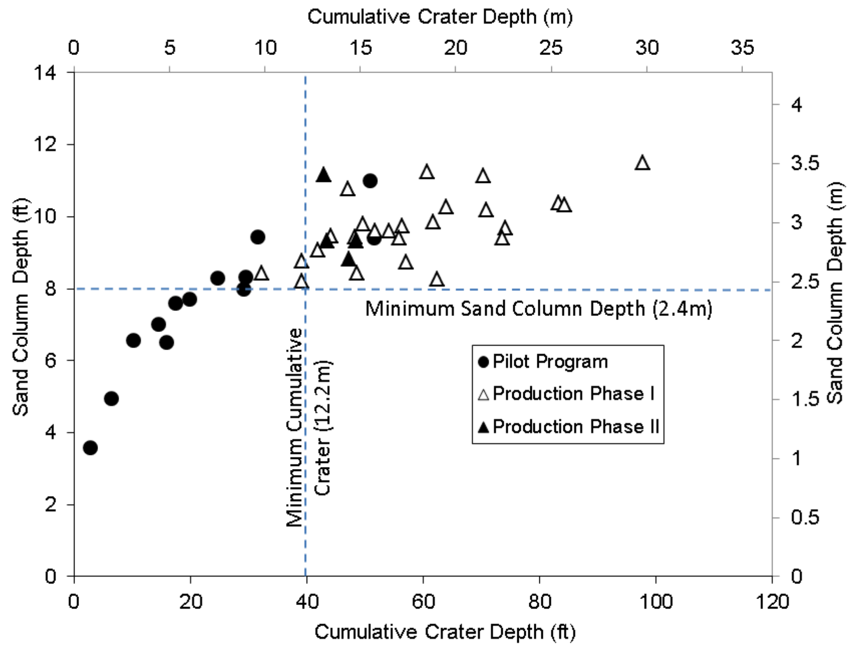

Figure 5.30. Sand column versus cumulative crater depths: pilot and production DR.

Similar to the I-4 case study (Mullins, 1996), the pilot program showed that no more than 5 to 6 drops could be applied at a given energy without reduced effectiveness (i.e. penetration/crater depths decreased). It also demonstrated the need to get more complete pounder penetration both at the low and high energy passes to more efficiently advance the sand columns. Further, it was estimated that 10 - 12 full penetrations of the 4ft tall pounder would produce a sand column 8-10ft deep. Using this information, an energy based installation procedure was established to begin production DR treatment. This entailed 5-6 low energy (20ft) drops to be conducted in the first pass; a second pass followed with 5-6 higher energy drops (40ft) to overcome the built-up penetration resistance from the compacted sand column and continue advancing the column. The sixth drop was only used if the crater depth had not decreased in the fifth blow. By first installing the surrounding low energy columns, the subsequent high energy impacts could force the columns deeper instead of spreading laterally. The sand blanket thickness was reduced to 1.5 – 2ft for all production DR.

<u>Production DR Criteria and Verification.</u> While literature often cites energy criteria (drop heights) to properly perform DR or DRM, the variations in soil strength found during production DR treatment made it necessary to refine the field installation procedures set from the pilot program. In some areas the initial 6m drop height was too much or too little energy to produce a well-defined crater for refill. In response, the field crew was instructed to vary the energy as necessary such that the pounder became as fully embedded as possible. If the pounder penetrated less than half its height, drop height was doubled; if penetration was more than 1.5 its height, the drop height was cut in half. This type of installation procedure removed issues with variability in soil strength and drop energy efficiency; displacement and not energy became the primary criterion.

As the last crater depth could be as much as 1m, a series of progressively reduced drop height impacts were used to compact the upper portions of the sand column. Therefore, after the last high energy impact (e.g. 20-40ft), two impacts at half the previous energy were imparted until the last two drops were only from a height of 5ft (typically16 total impacts: 5@20ft, 5@40ft, 2@20ft, 2@10ft, and 2@5ft). Production rates ranged from 1 to 2 minutes per impact and 20 sand columns could be installed each day.

Verification testing was performed on 10% of the 273 columns where the column depth was confirmed to meet or exceed the 8ft target depth (the depth to the bottom of soft clay). In all, twenty-nine verification CPT soundings were performed and compared to the recorded cumulative crater depth (Figure 5.30). Acceptance of each column was firstly based on achieving a cumulative crater depth of 40ft. In regions of thinner clay deposits overlying stiffer soils, the cumulative crater depth criterion was difficult to achieve, so CPT testing could also be used to verify the sand fully penetrated the undesirable materials.

Performance verification came in the form of settlement plates installed both on sand columns and between. Settlement tracked closely with the addition of embankment fill and terminated within 40days of completion with only 0.6in of additional settlement (Figure 5.31).

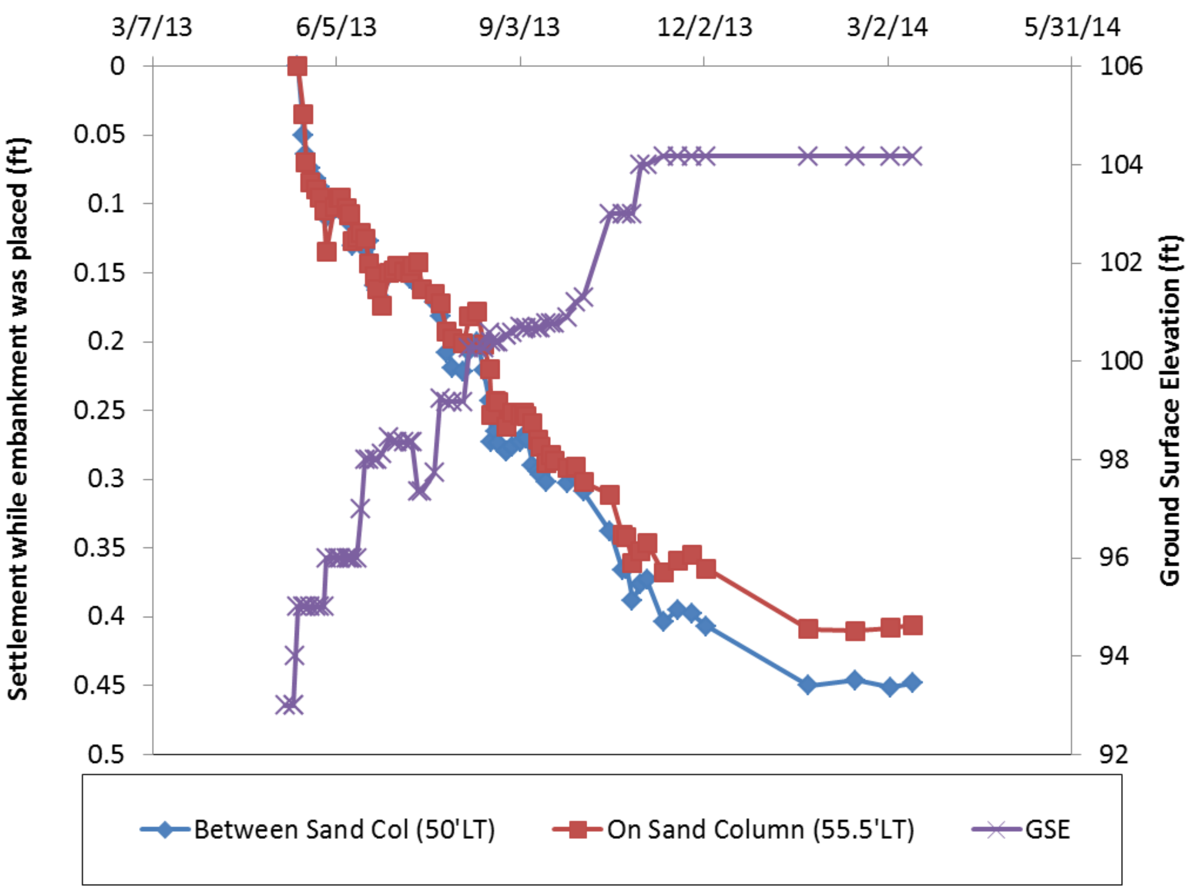

Figure 5.31. Data from settlement plates both on and between sand columns.

<u>Comparison to Cement-Based Soil Mixing.</u> The mechanisms of performance improvement from DR are based on improved drainage and increased stiffness of the upper soil layer(s). However,

when compared to the soil mixing used at US-331, the amount of settlement for the 20ft compressible soil layers was 30 times more than that observed in the 40ft deep soil mixed columns. The differences are three-fold: (1) DR only treated the upper compressible fat clay and the underlying sandy clay was permitted to settle under normal consolidation conditions, (2) the stiffness of cement stabilized columns of US-331 was higher than the compacted sand columns, and (3) cement based soil stabilization is not time / consolidation dependent making displacement from loading immediate. While DR is cost effective for shallow deposits, it cannot be used to stabilize deeper deposits that are easily accommodated by DSM methods. Even stone columns suffer from lower stiffness when compared to cement-stabilized soil mixed columns.

Chapter 6: Conclusions

Organic soils present a difficult challenge for roadway designers and construction due to the high compressibility of the soil structure, the often associated high water table, and high moisture content. For other soft or loose soils (inorganic soils), stabilization using cement or similar binders (a method called soil mixing) has proven to be an effective solution. To this end, the Federal Highway Administration has published a comprehensive design manual for these techniques. Organic soils, however, are not addressed therein to a level of confidence for design, as organic soils do not follow the trends presented for inorganic soils. This has been attributed to the high porosity, high water content, and high levels of humic acids common to organic soils.

Worldwide, the effect of organics on the strength of stabilized soils has been a recurring discussion but with vague recommendations. The FHWA manual suggests that soils with *organic contents greater than about 10 percent may produce significant interference with cementation.* It further states that *organic soils tend to require more binder than inorganic soils*. With regards to cost and due to the increased binder content the manual warns, *soils containing organics/peat are more costly to mix* where the average additional cost is on the order of $30 more per cubic yard over the average price of inorganic soils stabilization (~$77). However, discussions with soil mixing contractors revealed that the price could vary as much as $50 to $400 per cubic yard depending on numerous factors unrelated to organic content such as required strength, depth of treatment, overall size of the project, site logistics, schedule restrictions, etc. Whether or not dry or wet mixing is used has less effect.

Similarly cautionary language can be found from the Swedish Deep Stabilization Research Centre where it is simply stated that *the stabilization outcome of a binder cannot at present be definitely predicted merely by determining the organic content and humus content of the soil.* However, these recommendations provided a glimmer of insight noting that *in soils with high organic contents, such as mud and peat, the quantity of binder needs to exceed a "threshold." As long as the quantity of binder is below the threshold the soil will remain unstabilized.* This statement was supported by the findings of this study from which a proposed design approach was developed.

In the process of developing the proposed design recommendations several tasks were undertaken including: a thorough literature search, laboratory bench tests, large scale laboratory tests, field evaluation of past and on-going projects and concluded with recommendations for designing for soil mixing applications in highly organic soils.

6.1 Laboratory Bench Tests

Laboratory tests (bench tests) were performed to assess the effect of cementitious binder type, binder content, mixing method, organic content (OC), and curing time on strength gain of stabilized organic soil. This phase of the study involved over 700 samples where in all cases, specimens with organic content higher than approximately 10% required disproportionally more cement for the same strength gain when compared to inorganic or lower organic content samples.

As discussed in Chapter 2, the FHWA design manual and the Swedish Deep Stabilization Research Centre have somewhat conflicting views on the effects of slag replacement. The FHWA suggest that slag may be beneficial, while the Swedish publication recommends cement over pozzolans in organic soils with large void volumes. Depending on the type of organic soil, Chapter 3 of this report suggest that both parties may be correct. The FHWA data sets appear to be lower OC soils less than 18%. In this study, slag typically performed well in soils with organic contents lower than 20% and performed poorly in soils with organic contents above 20%, in comparison to mixes with pure cement binders. However, the point where slag was beneficial also depended on binder content and cure time. The Swedish report goes on to discuss that slag performance may be connected to the inorganic constituent (sand versus clays), with slag performing better in clays.

Figure 6.1 shows the laboratory unconfined compression strengths as a function of w/c ratio along with literature values for inorganic soils (and a few low OC cases). To be consistent with literature data, 43 points from this study are plotted; these data points all have pure cement binders and 28 day cure time. As most of the laboratory tests contained organic contents higher than 10%, most of the results do not agree with the more historically accepted trends. However, the laboratory organic specimens do follow a pattern of higher strength from lower w/c ratio mixes and vice versa.

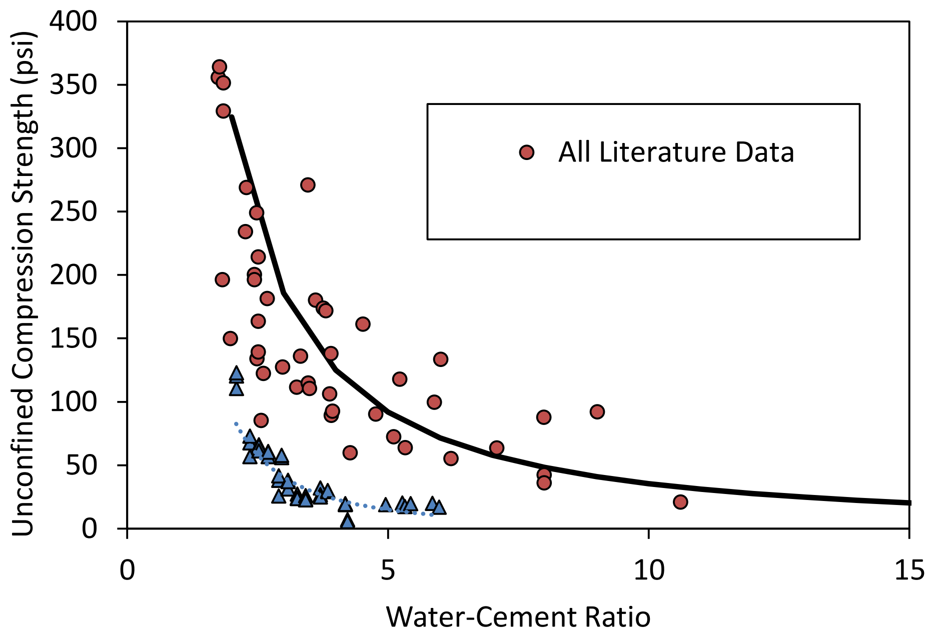

Figure 6.1. Results of laboratory unconfined compression tests along with literature values.

It should be noted that the modified FHWA design curve shown incorporated a few extra case studies discussed by the authors but not included in the curve offered by that publication. Only a subtle variation was noted between the published curve (Figure 2.38) and the modified version primarily used in this report.

6.2 Large-Scale Outdoor Laboratory Testing

Using the findings of the bench tests, a 1/10th scale test bed was built in which soil containing approximately 44% OC was placed and conditioned with rain water periodically to maintain a submerged or near submerged state. The dimensions of the bed accommodated three side-by-side tests wherein dry and wet soil mixing were performed each in one third of the bed. The remaining third of the bed was left untreated. Load tests were then performed on the three portions of the bed where the load for a simulated roadway was placed. These loads were left in place for several months and monitored for long-term movement.

Results of the simulated surcharge loading showed marked improvement for the soil mixed portions relative to no treatment (both used identical amounts of cement per system volume). The wet mix region was comprised of 20% replacement with columns of higher strength material (e.g. 20psi), and the dry mix region used an overall treatment strength (mass stabilization approach) where the required strength was closer to 4psi. Figures 6.2 and 6.3 show the load versus displacement response for the two simulated surcharges along with the untreated control. The control was not expected to ever withstand the design load (600psf) with a permissible displacement. The maximum applied surcharge loads were 800, 800, and 600psf for the wet mix, dry mix, and control, respectively; the maximum load capacity of the loading system was 800psf.

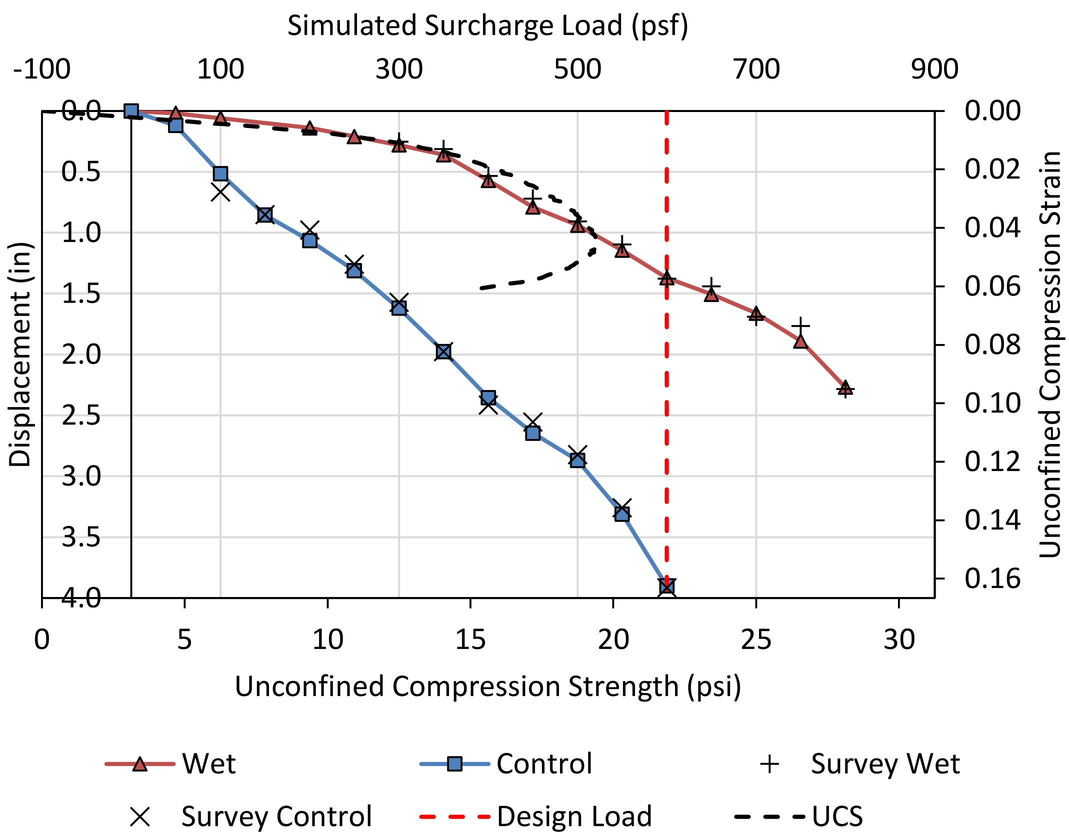

Figure 6.2. Simulated surcharge load response for the wet mix bed.

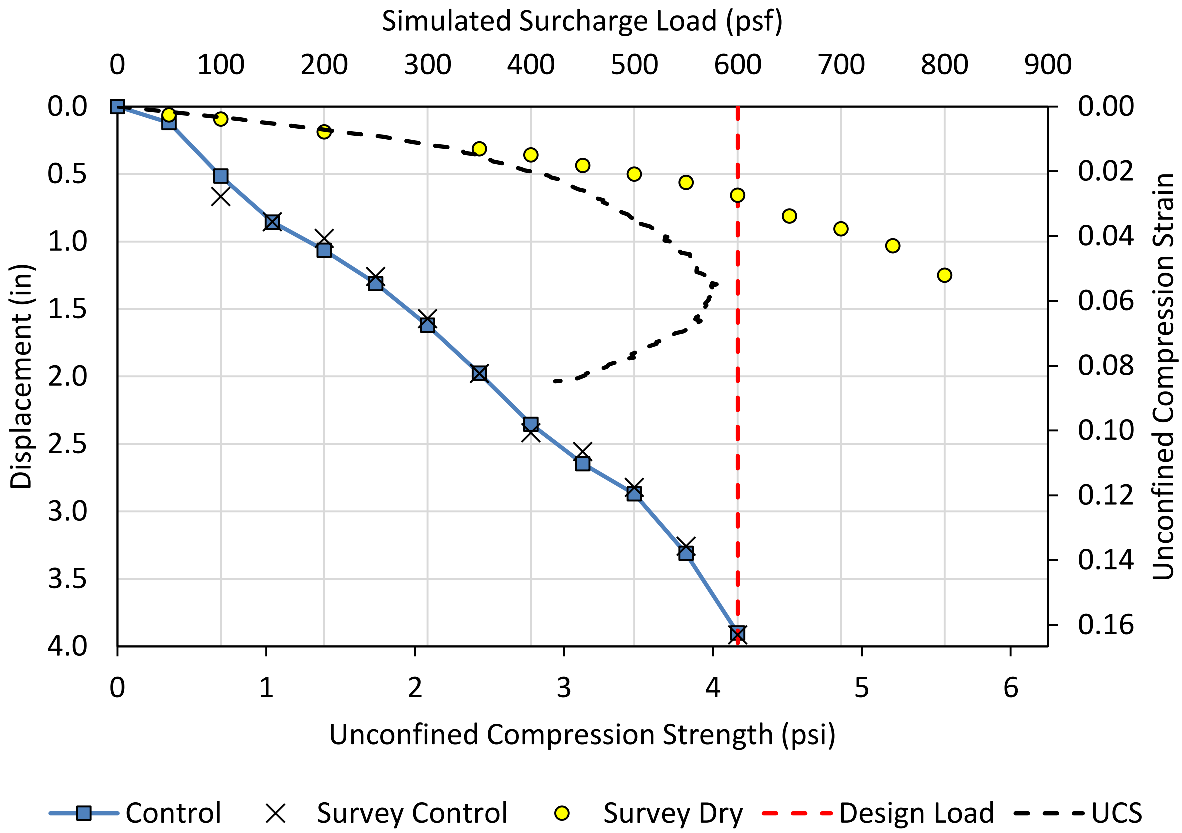

Figure 6.3. Simulated surcharge load response for the dry mix bed.

Also shown on these curves are the unconfined compression test results for laboratory samples superimposed over the two load response curves (Figures 6.2 and 6.3). Selection of the most appropriate lab test was based on the target strength of the column or mass stabilization (20 and 4psi, respectively). Further, the strain scales of the UC tests have been matched to the displacement associated with the same strain in the 24in deep soil bed. The stress-strain diagrams every for cylinder tested may be found in the Appendix.

Some variations exist, however, between the scenarios making the comparison slightly mismatched. For the wet mix, the highest cement content sample performed in the lab was 500pcy; the bed column mix was closer to 670pcy. For the dry mix, the lab specimen used 200pcy at 66% organics while the test bed used 233pcy at 40% organics. It should also be noted that the design of the soil mix in the surcharge bed was based on unconfined compression test results which are ultimate values and was not based on the non-linear stress strain response. As a result, both treatment regions showed some yielding prior to achieving the design surcharge loading.

While the stiffness of the dry and wet mixed regions were similar initially, the dry mixing continued to respond in a stiffer manner beyond the laboratory-predicted yield point. This was a side-effect of mass mixing instead of the originally planned isolated dry soil mixed columns. Whereas the wet mix surcharge plate load was solely supported by seven isolated columns, the dry mix plate load was resisted by a combination of compression of the material and distribution of stress via shear to the surrounding stabilized soil. The wet mix columns could not transfer load in the same fashion to the more peripheral columns. In essence, the wet mix load test was more representative of a continuously loaded field condition; the dry mix plate load provide a reasonable assessment of local punching stresses (wheel loads), but not necessarily global performance.

6.3 Evaluation of Full-Scale Soil Mixing Sites

Concurrent to the bench tests and the 1/10-scale load tests, field evaluation of past and on-going soil mixing programs were conducted. These showed in all cases that soil mixing has been largely successful. Both wet and dry mixing programs were reviewed. Survey data over the three year span of the study showed no new settlement. Two problematic sites were also investigated where continued subsidence of a rural road and bridge over organic and/or soft soil. The latter two sites confirmed that cement stabilized soils are not prone to time dependent deformation more typical of ground modification that uses drainage enhancing or no treatments.

Looking at the unconfined compression test results from one of the projects (Figure 6.4) showed that a wide range of strengths may result from a given soil mix approach, but where the measured strengths exceed the minimum design strength (excepting a few test columns installed before production began). The data represents all demonstration elements with high and low cement factors as well as tested values for blade rotations. The large strength variability (which is not uncommon for soil mixing) can be largely attributed to mixing thoroughness where some samples registered full strength of the injected grout and others more closely aligned with the anticipated soil mixing design strength; soil variations also contribute to the scatter. Note that where the cement factor ranged from 2.5 to 30 pcf in the mixed soil, these values are computed from the amount of grout injected at a given depth and not necessarily the amount in a given core sample. At each depth, some of the injected grout may migrate to other portions of the column and may not be uniformly spread across that cross section of the column. This can result in portions of the column that are 100% grout and others that are mixed with soil. Bench tests would not be expected to behave so erratically and contain the exact amount specified. This contributes to the difference in lab versus field performance where lab values can be 2 to 5 times higher.

While no soil mix columns or cored samples were intentionally prepared from 100% grout, these regions do exist and are evidenced by the upper bound of approximately 1275psi. Pure grout with a w/c ratio of 0.8 has a cement factor of about 47pcf and strength from laboratory tests that support the upper limit observed in the field. Figure 6.5 shows the results from compression tests independently prepared for various w/c ratios using cement with similar properties. Grout with w/c ratios above 1.0 do not maintain suspension and often result in final ratios of a lesser value with free water above the remaining tested sample. Regardless, the pure grout tested in the lab with w/c ratio of 0.8 produced a 28 day strength between 1300 and 1600psi which is in line with the upper values observed.

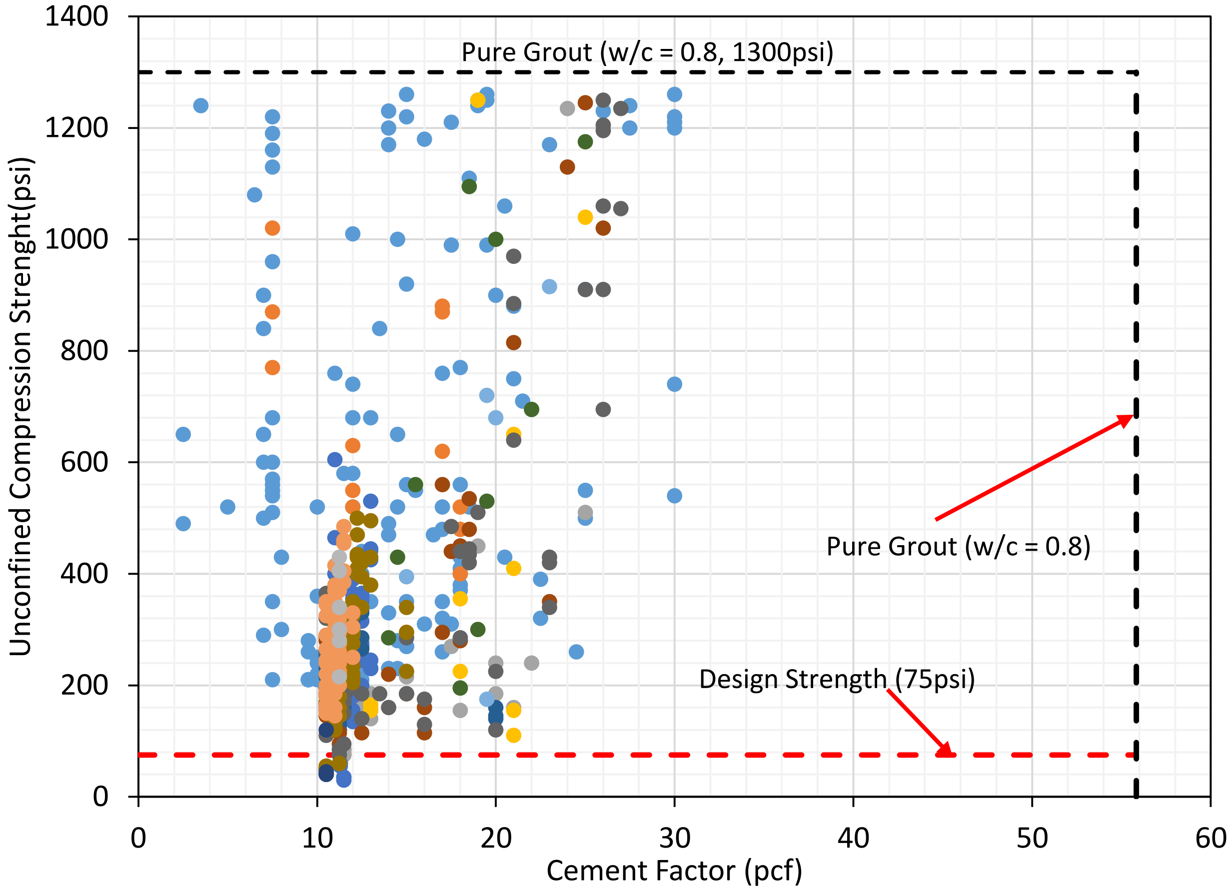

Figure 6.4. Unconfined compression tests from field collected soil mix specimens.

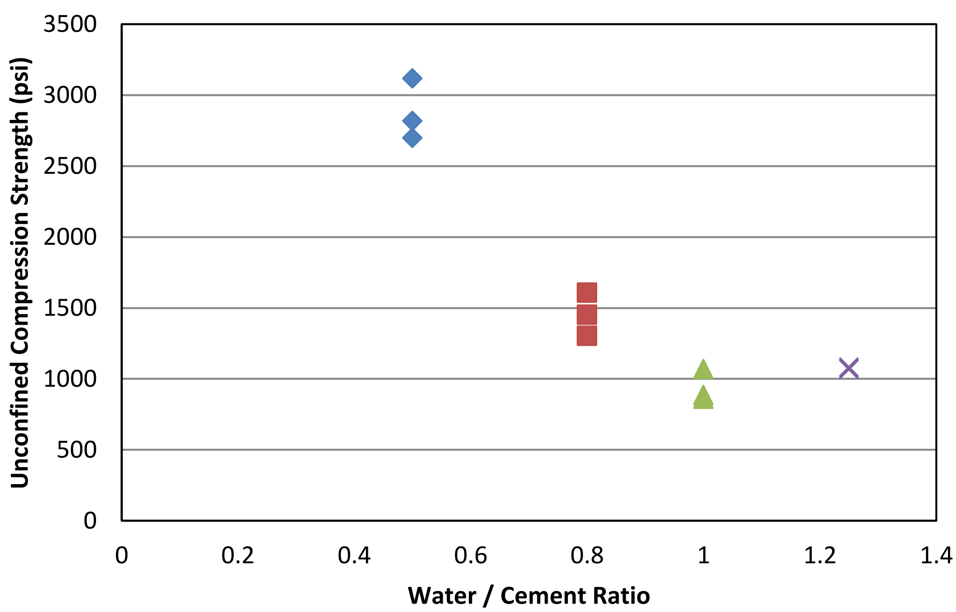

Figure 6.5. Strength of grout at various w/c ratios.

6.4 Recommendations for Designing Soil Mixed Organic Soils

Due to this study focusing on highly organic soils, the design method developed herein only uses cement as the binder as it generally performed better than slag when the organic content was greater than 20%. Using the hypothesis presented by the Swedish Deep Stabilization Research Centre, quantitative values for a cement factor threshold (***CF threshold***) were sought. The cement factor threshold was defined as necessary amount of cement below which no strength gain will be achieved. In other words, the cement factor must be satisfied before any strength gain will occur. To clarify, some meaningful definitions are provided below.

- Threshold cement: Cement that is in the system but does not contribute to strength.
- Effective cement: Cement in excess of the threshold that contributes to strength.
- Effective w/c: The w/c calculated using the effective cement.

The findings of the threshold values presented at the end of this section have the potential to improve the predictability and effectiveness of designing for soil mixing with organic soils. To calculate the cement factor threshold, two independent approaches were performed. The first approach was strictly based on laboratory data, and the second back-calculated the cement factor threshold by adjusting the cement content until the w/c ratio versus strength relationship fit the FHWA design curve (which is largely based on inorganic soils).

For laboratory data approach, the strength vs cement factor curves (Figures 3.2 and 3.13) in combination with the strength vs organic content curves (Figures 3.18) were scrutinized to determine the cement factor below which no strength was achieved (for all organic contents). This is shown below in Figure 6.6 for samples with 66% organic content. While some strength was registered at lower cement factors (i.e. usually less than 5psi), an extrapolation of the steeper linear section was used to locate the threshold value on the x-axis. In the case shown, this was around 275pcy.

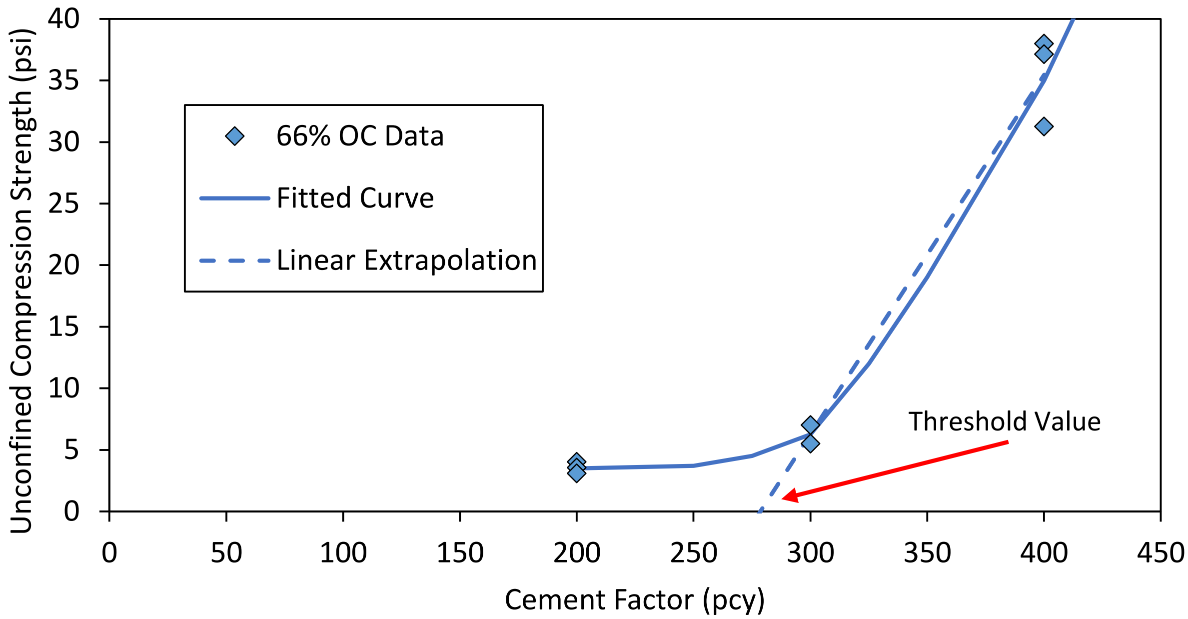

Figure 6.6 Strength vs Cement Factor for 66% OC (28 days).

This same curve (Figure 6.6) was recreated to find the cement factor threshold at other organic contents. However, as not all tests were performed at the same OC when varying CF, the strength vs OC curves were interpolated to produce strength vs CF curves for OC values ranging from 5 to 66%. Figure 6.7 plots strength vs OC for various cement factors. The dotted vertical lines show the OC values selected for this exercise. As an example, the strength vs CF curve for 30% OC is shown in Figure 6.8 alongside the 66% OC curve.

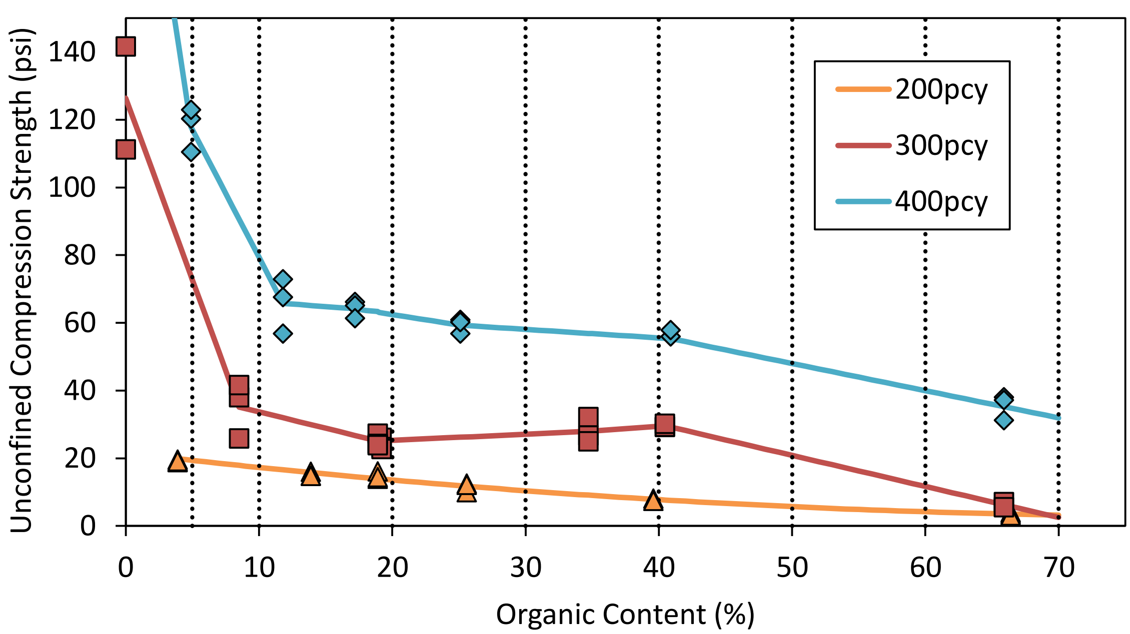

Figure 6.7 Strength vs OC for various CF (28 days).

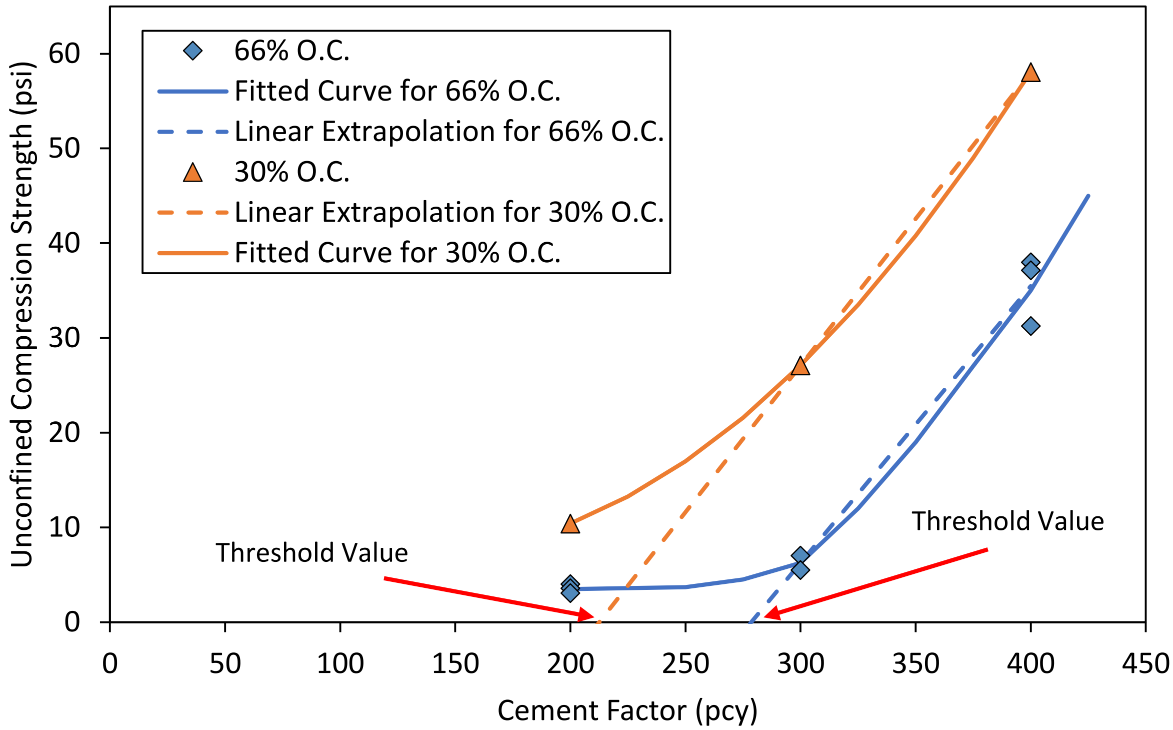

Figure 6.8 Strength vs CF for 30% and 66% OC (28 days)

Figure 6.9 shows the results for all eight OC values selected (using dashed lines) superimposed on all applicable test data (100% cement binder and 28 day cure time).

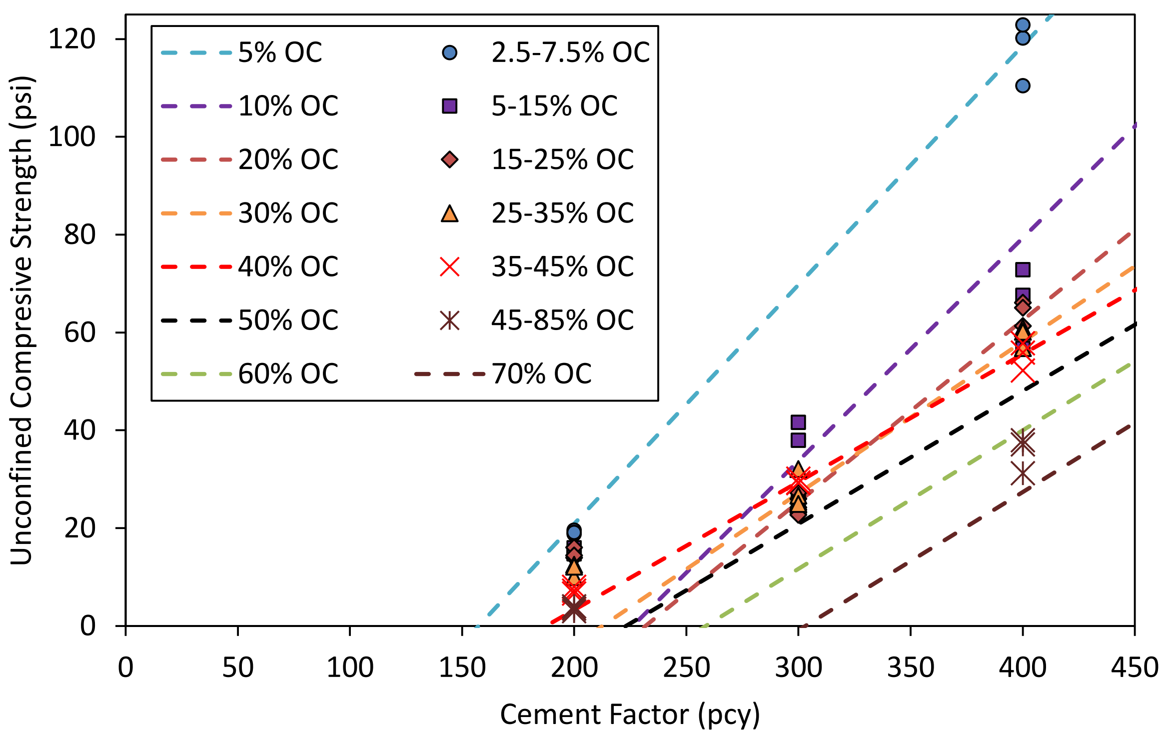

Figure 6.9 Strength vs CF for various organic contents.

By inspection, the cement factor thresholds ranged from 150 to 300pcy. Figure 6.10 plots the cement factor threshold for all OC values. The threshold varied little in the middle range of OC samples tested, however, there were notable increases and decreases in threshold at very high or low OC values. These results are not surprising considering that the strength vs OC curves show the same trend (Figure 6.7 and 3.30). For illustration, the 300pcf data series from Figure 6.7 is inverted and shown in Figure 6.10.

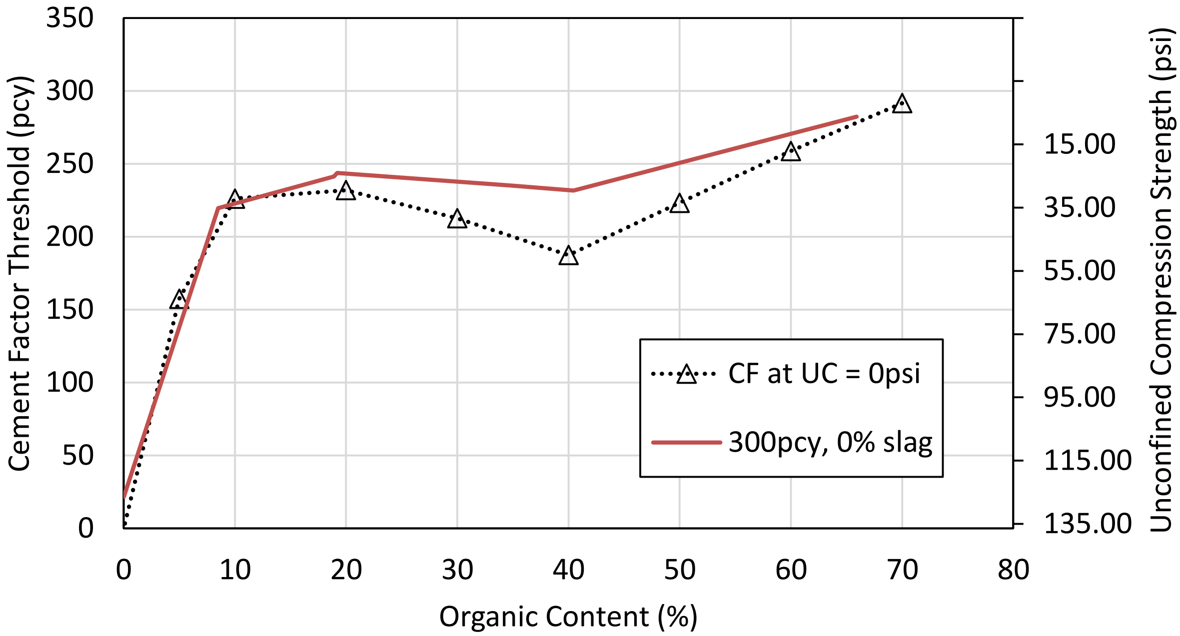

Figure 6.10 Cement Factor Threshold vs Strength Based on Extrapolation.

In order to connect this data to the FHWA design curve (Figure 6.1), a relationship between cement factor threshold and w/c was developed. Equation 6.1 uses the cement ***CF threshold*** to calculate an effective w/c.

$$w/c_{effective} = w/c_{in\ place} \frac{CF_{in\ place}}{(CF_{in\ place} - CF_{threshold})} \qquad \text{Eqn 6.1}$$

The $\boldsymbol{CF_{in\ place}}$ term represent the total cement factor (wt/vol), and that which does not contribute to meaningful soil improvement is denoted as the $\boldsymbol{CF_{threshold}}$**.** The $\boldsymbol{CF_{threshold}}$ may be thought of as “dead” cement. It takes up volume, but does not contribute to strength. Using this relationship, the laboratory data provided in Figure 6.1 was adjusted using Equation 6.1 to represent the effective w/c ratio. The cement factor threshold used in this exercise comes from linearly interpolating between the values determined in Figure 6.10. The modified lab data is shown in Figure 6.11.

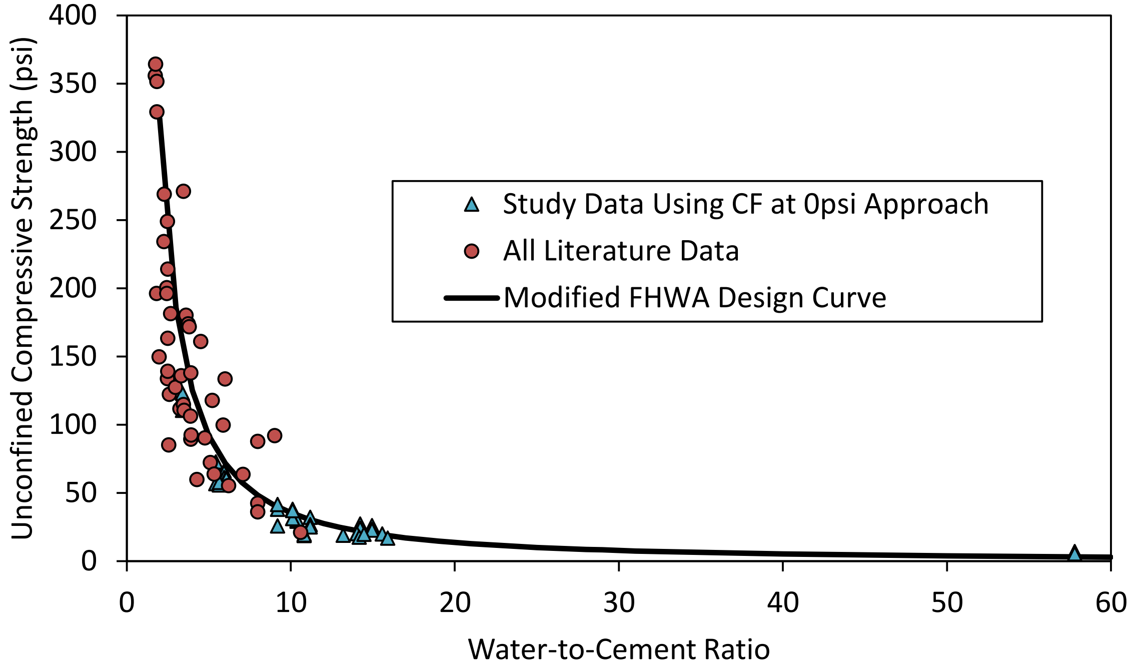

Figure 6.11 Strength versus w/c corrected for threshold using CF at 0psi.

As shown in Figure 6.11, the adjusted data fits the design curve far better than the raw data in Figure 6.1. In addition to this method, a second method for calculating the cement factor threshold was used. Cement factor threshold values were selected for each cylinder test so that the data aligned perfectly with the design curve. Figure 6.12 shows the fitted data, and Figure 6.13 shows the back-calculated cement factor threshold values used for all 43 data points.

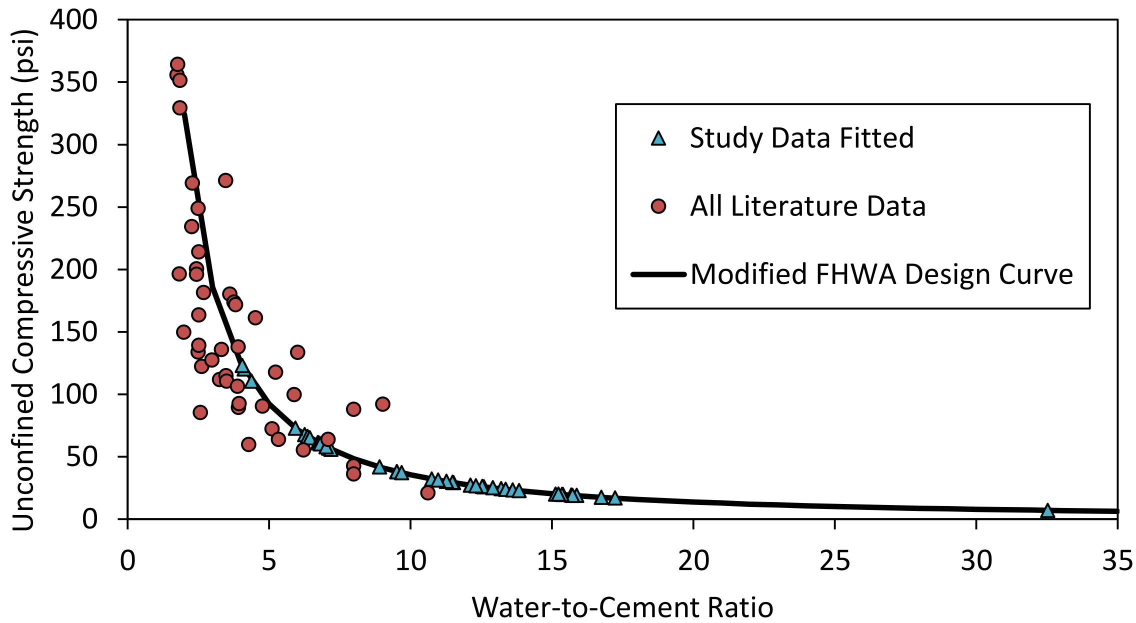

Figure 6.12 Study data fitted to modified FHWA design curve.

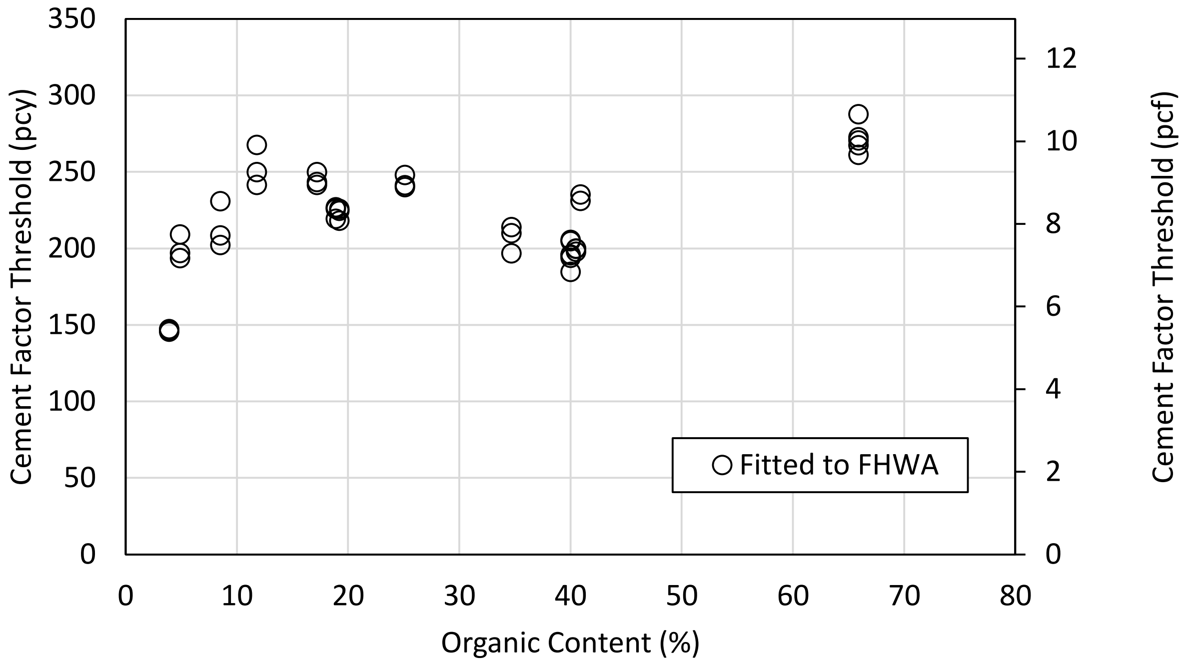

Figure 6.13 Cement factor threshold obtained by curve fitting.

When superimposing the result from both approaches, the results are strikingly similar. Figure 6.14 shows the cement factor thresholds computed for all samples versus the organic content using both methods and a recommended design value for the threshold (dashed line). Additionally, Figure 6.15 shows the laboratory data (Figure 6.1) corrected using the recommended threshold.

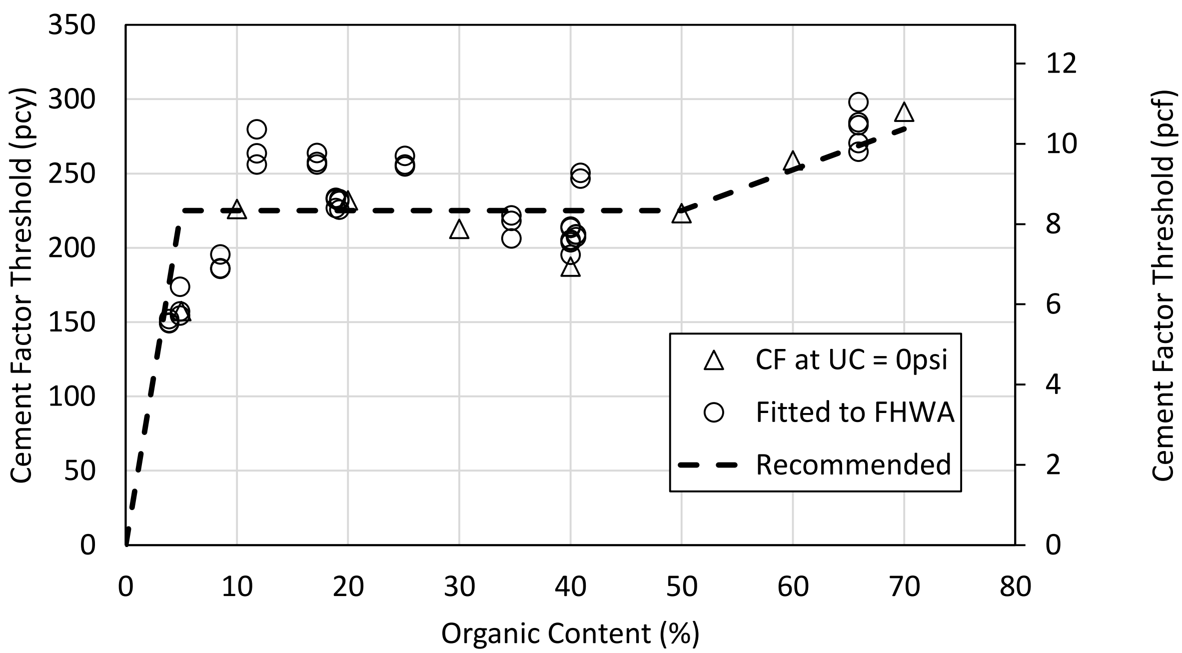

Figure 6.14. Cement factor threshold versus organic content.

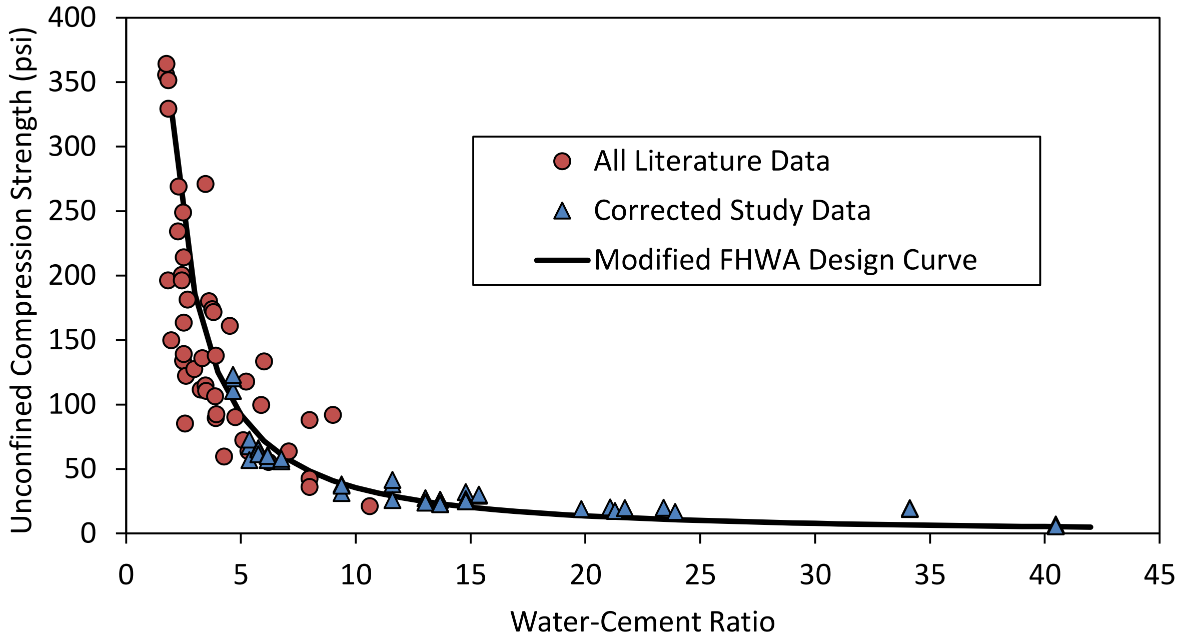

Figure 6.15. Strength versus w/c ratio corrected for cement factor threshold.

Using the threshold concept, the data collected during this study now closely correlate to previous laboratory studies that represented mostly inorganic soils. To investigate the effects of cure time, the same exercise was performed on long term data (60 days). The equation provided by FHWA for cure time was used to shift the design curve up to reflect 60 day strength instead of 28. Figure 6.16 superimposes these results over the 28 day results. The similar outcomes suggest cure time does not affect the cement factor threshold value.

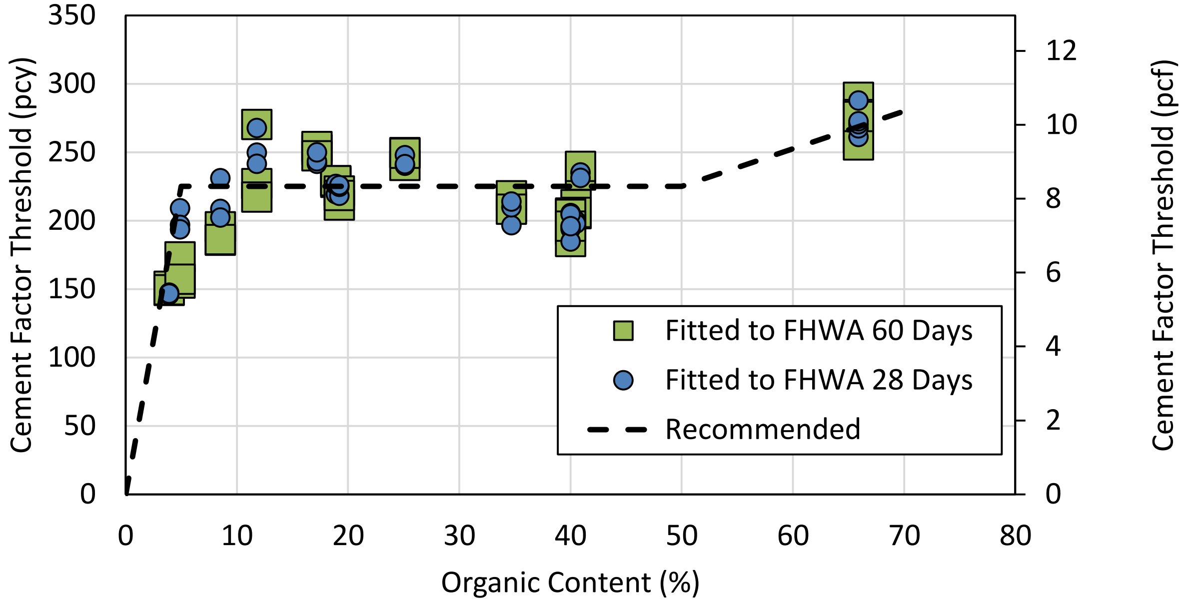

Figure 6.16 Cement factor threshold versus organic content (60 day strength).

A design process was developed with two primary components:
(1) determine the required the water-to-cement ratio to achieve the design strength using the FHWA design curve (Figure 6.13). This has a direct relationship to the total weight of water in the system and results in a weight of cement. *Note: this is the effective w/c ratio, the final in-place w/c ratio will be lower.*

(2) Account for additional cement needed to satisfy the cement factor threshold. Using this proposed design method (using Eqn 6.1 and Figure 6.15) an example design of dry mixing and wet mixing has been prepared for illustration:

Given:

- Organic soil (OC = 40%)
- Moisture Content (176%)
- Saturated unit weight (76pcf,)
- Dry Mixing

Compute the required cement factor to achieve a 50 psi required strength.

- Using FHWA design curve, 50 psi requires $(w/c)_{effective}$ ratio of 7.5.
- $1ft^3$ of the soil contains 48.4lbs of water and 27.5lbs of solids to satisfy the 176% moisture content.
- The weight of effective cement from FHWA ($wt_{effective\ cem}$) would then be 48.4lbs / 7.5 = **6.45lbs**. This is equal to an effective cement volume of **0.033ft^3**.
- Calculate the final volume after threshold cement is added using the following equation. Using Figure 6.14, the $CF_{threshold}$ for 40% OC is 225pcy (8.33pcf).

$$Final\ Volume = \frac{Volume\ of\ Soil + Volume\ of\ Effective\ Cement\ or\ Grout}{1 - \frac{CF_{threshold}}{62.4}\left(\frac{1}{3.15} + \frac{w}{c}\right)} \qquad \text{Eqn. 6.2}$$

$$Final\ Volume = \frac{1ft^3 + 0.033ft^3}{1 - \frac{\mathbf{8.33pcf}}{62.4}\left(\frac{1}{3.15} + 0\right)} = 1.079ft^3$$

- The weight of threshold cement is then calculated by:

$$wt_{threshold\ cem} = Final\ Volume * CF_{threshold} = 1.079ft^3 * 8.33pcf = \mathbf{8.99lbs}$$

- Therefore the total amount of cement to be added per cubic foot of original soil (traditional cement factor) is 6.45lbs + 8.99lbs = **15.44lbs.**
- $CF_{in\ place}$ =15.44lbs / 1.079ft^3 = **14.32pcf (or 387pcy).**
- The $(w/c)_{in\text{-}Place}$ is 48.4 / 15.44 = 3.13.
- As a check, Eqn. 6.1 may be used to calculate the effective water to cement ratio. $(w/c)_{Effective}$ = 3.13 x [387 / (387 - 225)] = 7.5.

When applying this process to wet mixing, the procedure has an iterative component. This is caused by both w/c ratio and cement factor requirements being co-dependent. In dry mixing, the w/c ratio is satisfied first and then the cement factor second. In wet mixing, the w/c ratio may be satisfied, but then becomes unsatisfied once more cement (with more water) is added. To combat this, the amount of water introduced by the threshold grout is assumed and checked. Convergence typically occurs within 2-3 iterations. To start the process, a simple assumption is that the weight of water introduced by the threshold grout is 5lbs (for a unit volume of 1ft^3).

For comparison purposes, the example from above is solved again using grout with a w/c ratio of 0.8 starting with the third step.

- Assume that weight of grout water from the threshold cement is equal to 5lbs.
- The uncorrected weight of required cement would then be (48.4lbs + 5lbs) / [7.5 – 0.8] = 7.98lbs. This cement comes with 6.38lbs of additional water, and thereby increases the total volume by 0.14ft^3.

$$\frac{7.98lbs\ of\ cement}{62.4pcf * 3.15} + \frac{6.38lbs\ of\ water}{62.4pcf} = 0.14ft^3$$

- Calculate the final volume after threshold cement is added using equation 6.2. Figure 6.14 states that the $CF_{threshold}$ for 40% OC is 225pcy (8.33pcf).

$$Final\ Volume = \frac{1ft^3 + 0.14ft^3}{1 - \frac{\mathbf{8.33pcf}}{62.4}\left(\frac{1}{3.15} + 0.8\right)} = 1.34ft^3$$

- At this point the assumption may be checked. The amount of cement in the system due to the cement factor threshold is 8.33pcy x 1.34ft^3 (final volume) = 11.19lbs. Therefore there is 11.19lbs x 0.8 = 8.96lbs of grout water from the threshold cement, so the assumption was too low. Therefore plug the new value into the original assumption. The value converges to 9.039lbs on the second iteration. The updated values are:

    - Uncorrected weight of cement = **8.57lbs**.
    - Grout water = **6.86lbs**.
    - Final volume = **1.36ft^3**.
    - Threshold cement = **11.30lbs**.

- Therefore the total amount of cement to be added per cubic foot of original soil (traditional cement factor) is 8.57lbs + 11.30lbs = **19.87lbs.**
- $CF_{in\ place}$ = 19.87lbs / 1.36ft^3 = **14.65pcf (396pcy).**
- This is about 36lbs of 0.8 w/c grout for every cubic foot of soil to be treated.
- The $(w/c)_{In\text{-}Place}$ is (48.4lbs + 9.039lbs + 6.86lbs) / 19.87lbs = 3.24.
- To check, equation 6.1 may be used to calculate the effective water to cement ratio. $(w/c)_{Effective}$ = 3.24 x [396 / (396 - 225)] = 7.5.

Using the dry mixing approach, the Marco Island Airport discussed in Section 2.3.2 should have required a cement factor of 332pcy based on the 15psi strength requirement, water content of 425%, and organic content of 58%. The proposed design methodology conservatively supports the original bench test suggested value of 275pcf which was unfortunately mistakenly interpreted to be too high due to a bent KPS rod. The subsequent reduction to 125pcf would not have crossed the cement factor threshold.

While the above examples outline a process for design, it should be understood that soil mixing in general is not a perfect science given the large variability between lab and field samples. Therefore, the values generated using the procedures above should be considered starting points for design followed by bench tests and field verification.

6.5 Summary

The findings of this study suggest that the adverse effects of organic soils can be combatted where more cement content is required to offset the acidity before the more commonly used FHWA water / cement ratio design curve can be used. While past researchers have alluded to the concept of a cement factor threshold, the study findings identified such a value below which no strength gain was achieved. This threshold was then defined as a cement factor offset above which the measured strengths matched well with other soil types. As a result, a recommended approach for designing soil mixing applications in organic soils was developed.

As this is a new development in design for organic soils, some stipulations should be placed on the proposed method:

(1) The method was developed for a given composition of organic soil which was only partially decomposed having both fibrous and amorphous attributes. A natural extension of the methodology should incorporate the assignment of a threshold on the basis of more variations in decomposition.
(2) The organic soil and range of organic contents were largely sandy organic materials with little to no clay fraction. Verification of the method should address variations in the inorganic composition of the organic soil tested. However, as the FHWA design curve is a compilation of sand and clay, this is likely to be less significant.
(3) The organic soil used in this study was not responsive to the use of slag replacement at higher OC levels; lower OC samples did response positively to slag. Other studies have shown slag to be better suited for organic soils but it is not clear at what OC. There exists the possibility that clayey organic soils or organic soils of varied decomposition may be more positively affected by slag / cement mixes than that used in this study.
(4) This study did not address slag replacement ratios other than 0, 50 and 100%. There may be more subtle improvements or applicability with other fractional components (e.g. 25/75 slag/cement mixes).
(5) Finally, in this study, Portland Type I/II cement was used due to it being the most common in soil mixing. It is conceivable that other cement types may have different results on the cement factor threshold.

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Appendix A: Lab Cylinder Information and Test Results

Table A.1. Laboratory soil mixing matrix.

Batch	CF (pcy)	Slag (%)	Organic Content (%)	Moisture Content	w/c Ratio	Mixing Date	Break Date	Days Cured	Cylinder Number	Max Strength (psi)	Average Strength (psi)
1	200	50%	66.4%	361.7%	6.88	9/23/2014			1		
									2		
									3		
									4		
									5		
									6		
									7		
									8		
									9		
2	300	50%	66.4%	361.7%	4.49	9/25/2014	10/9/2014	14	1	2.03	1.85
									2	1.79	
									3	1.73	
							10/23/2014	28	4	N/A	2.07
									5	2.3	
									6	1.84	
							11/25/2014	61	7	N/A	2.37
									8	2.28	
									9	2.46	
3	400	50%	66.4%	361.7%	3.29	9/25/2014	10/9/2014	14	1	5.75	5.9
									2	6.09	
									3	5.86	
							10/23/2014	28	4	7.04	6.1
									5	5.93	
									6	5.34	
							11/25/2014	61	7	4.36	7.25
									8	7.04	
									9	10.35	
4	200	0%	66.4%	361.7%	6.90	9/25/2014	10/9/2014	14	1	2.63	2.55
									2	2.15	
									3	2.86	
							10/23/2014	28	4	4.02	3.55
									5	3.56	
									6	3.08	
							11/25/2014	61	7	4.07	4.06
									8	N/A	
									9	4.04	
5	200	100%	66.4%	189.3%	6.10	10/7/2014			1		
									2		
									3		
									4		
									5		
									6		
									7		
									8		
									9		
6	300	100%	66.4%	211.6%	4.12	10/7/2014			1		
									2		
									3		
									4		
									5		
									6		
									7		
									8		
									9		

Table A.1 Laboratory soil mixing matrix (continued).

7	400	100%	66.4%	218.3%	3.05	10/7/2014			1		
									2		
									3		
									4		
									5		
									6		
									7		
									8		
									9		
8	300	0%	65.9%	221.1%	4.22	10/10/2014	10/24/2014	14	1	5.96	6.15
									2	6.35	
									3	6.15	
							11/7/2014	28	4	7.01	6.26
									5	5.50	
									6	N/A	
							12/10/2014	61	7	7.30	8.37
									8	8.18	
									9	9.64	
9	400	0%	65.9%	215.6%	3.08	10/10/2014	10/24/2014	14	1	37.70	27.12
									2	25.38	
									3	18.28	
							11/7/2014	28	4	37.99	35.46
									5	31.25	
									6	37.13	
							12/10/2014	61	7	34.60	38.52
									8	40.16	
									9	40.8	
10	300	50%	41.3%	135.8%	3.86	10/14/2014	10/28/2014	14	1	2.57	2.53
									2	2.46	
									3	2.57	
							11/11/2014	28	4	N/A	2.9
									5	2.97	
									6	2.82	
							12/15/2014	62	7	3.567	3.57
									8	3.567	
									9	N/A	
11	300	50%	27.8%	95.3%	3.52	10/14/2014	10/28/2014	14	1	3.62	3.57
									2	3.12	
									3	3.96	
							11/11/2014	28	4	4.06	4.26
									5	N/A	
									6	4.46	
							12/15/2014	62	7	3.8	3.8
									8	N/A	
									9	N/A	
12	300	50%	24.1%	76.2%	3.38	10/14/2014	10/28/2014	14	1	2.64	4.96
									2	4.55	
									3	7.68	
							11/11/2014	28	4	4.19	4.86
									5	5.95	
									6	4.45	
							12/15/2014	62	7	7.51	8.06
									8	8.63	
									9	8.05	
13	300	50%	11.2%	54.2%	3.00	10/14/2014	10/28/2014	14	1	24.47	24.8
									2	26.35	
									3	23.58	
							11/11/2014	28	4	42.44	40.06
									5	40.13	
									6	37.6	
							12/15/2014	62	7	63.42	59.94
									8	53.62	
									9	62.77	

Table A.1 Laboratory soil mixing matrix (continued).

14	300	50%	4.6%	36.3%	2.50	10/14/2014	10/28/2014	14	1	53.51	66.88
									2	89.24	
									3	57.89	
							11/11/2014	28	4	132.25	132.8
									5	94.45	
									6	171.7	
							12/15/2014	62	7	327.75	315.56
									8	241.4	
									9	377.52	
15	300	0%	40.5%	134.5%	3.84	10/17/2014	10/31/2014	14	1	27	25.99
									2	23.75	
									3	27.23	
							11/14/2014	28	4	29.23	29.57
									5	29.35	
									6	30.12	
							12/18/2014	62	7	31.03	29.9
									8	28.12	
									9	30.56	
16	300	0%	34.7%	105.9%	3.70	10/17/2014	10/31/2014	14	1	28.53	26.61
									2	22.48	
									3	28.83	
							11/14/2014	28	4	32.13	27.93
									5	26.66	
									6	25.00	
							12/18/2014	62	7	30.28	29.32
									8	26.61	
									9	31.07	
17	300	0%	19.2%	78.5%	3.42	10/17/2014	10/31/2014	14	1	22.06	22.62
									2	19.87	
									3	25.92	
							11/14/2014	28	4	26.03	23.99
									5	23.23	
									6	22.72	
							12/18/2014	62	7	33.06	30.19
									8	27.96	
									9	29.54	
18	300	0%	18.9%	62.9%	3.26	10/17/2014	10/31/2014	14	1	9.55	20.26
									2	21.27	
									3	19.24	
							11/14/2014	28	4	27.26	25.1
									5	24.23	
									6	23.82	
							12/18/2014	62	7	25.94	26.18
									8	26.55	
									9	26.05	
19	300	0%	8.5%	45.5%	2.90	10/17/2014	10/31/2014	14	1	33	29.85
									2	30.43	
									3	26.12	
							11/14/2014	28	4	38.02	35.17
									5	41.64	
									6	25.84	
							18-Dec	62	7	52.1	56.53
									8	58.87	
									9	58.63	
20	300	100%	43.8%	138.5%	3.80				1		
									2		
									3		
									4		
									5		
									6		
									7		
									8		
									9		

Table A.1 Laboratory soil mixing matrix (continued).

21	300	100%	40.0%	102.4%	3.56				1		
									2		
									3		
									4		
									5		
									6		
									7		
									8		
									9		
22	300	100%	30.0%	83.5%	3.47				1		
									2		
									3		
									4		
									5		
									6		
									7		
									8		
									9		
23	300	100%	20.0%	65.6%	3.29				1		
									2		
									3		
									4		
									5		
									6		
									7		
									8		
									9		
24	300	100%	6.3%	43.4%	2.76				1		
									2		
									3		
									4		
									5		
									6		
									7		
									8		
									9		
25	400	50%	41.2%	148.3%	2.88	11/5/2014	11/19/2014	14	1	15.1	14.4
									2	14.08	
									3	14.03	
							12/3/2014	28	4	13.89	14.53
									5	14.54	
									6	14.52	
							1/6/2015	62	7	N/A	16.59
									8	15.51	
									9	17.66	
26	400	50%	29.8%	108.6%	2.70	11/5/2014	11/19/2014	14	1	20.36	19
									2	18.99	
									3	17.65	
							12/3/2014	28	4	23.45	21.82
									5	21.51	
									6	20.49	
							6-Jan	62	7	23.11	25.44
									8	25.3	
									9	27.92	
27	400	50%	21.1%	82.4%	2.54	11/5/2014	11/19/2014	14	1	41.78	43.84
									2	36.56	
									3	53.19	
							12/3/2014	28	4	50.03	54.53
									5	N/A*	
									6	59.03	
							1/6/2015	62	7	85.03	82.08
									8	71.7	
									9	89.51	

Table A.1 Laboratory soil mixing matrix (continued).

28	400	50%	12.6%	60.1%	2.30	11/5/2014	11/19/2014	14	1	54.25	53.21
									2	52.06	
									3	53.33	
							12/3/2014	28	4	58.18	75.59
									5	86.05	
									6	82.53	
							1/6/2015	62	7	118.13	125.02
									8	144.59	
									9	112.35	
29	400	50%	4.2%	41.4%	1.97	11/5/2014	11/19/2014	14	1	182.95	161.89
									2	129.62	
									3	173.09	
							12/3/2014	28	4	300.95	318.12
									5	348.02	
									6	305.38	
							1/6/2015	62	7	452.65	452.23
									8	420.8	
									9	483.23	
30	400	0%	4.9%	44.4%	2.09	11/6/2014	11/20/2014	14	1	97.33	90.86
									2	91.66	
									3	83.6	
							12/4/2014	28	4	110.48	117.89
									5	120.24	
									6	122.95	
							1/7/2015	62	7	178.2	170.97
									8	159.24	
									9	175.48	
31	400	0%	11.8%	61.4%	2.35	11/6/2014	11/20/2014	14	1	62.47	56.25
									2	51.97	
									3	54.31	
							12/4/2014	28	4	67.60	65.75
									5	72.84	
									6	56.8	
							1/7/2015	62	7	93.55	85.87
									8	63	
									9	101.07	
32	400	0%	17.2%	80.3%	2.52	11/6/2014	11/20/2014	14	1	58.3	59.33
									2	60.31	
									3	59.39	
							12/4/2014	28	4	66.13	64.18
									5	65.08	
									6	61.34	
							1/7/2015	62	7	68.81	69.2
									8	67.23	
									9	71.57	
33	400	0%	25.1%	106.9%	2.70	11/6/2014	11/20/2014	14	1	57.66	58.63
									2	57.47	
									3	60.77	
							12/4/2014	28	4	60.9	59.31
									5	56.84	
									6	60.2	
							1/7/2015	62	7	69.24	65.68
									8	63.72	
									9	64.08	
34	400	0%	40.9%	156.9%	2.96	11/17/2014	12/1/2014	14	1	53.15	52.71
									2	N/A	
									3	N/A	
									4	52.27	
							12/15/2014	28	5	55.91	56.87
									6	57.82	
							1/18/2015	62	7	64.72	62.05
									8	61.4	
									9	60.04	

Table A.1 Laboratory soil mixing matrix (continued).

35	200	0%	39.6%	159.1%	6.20	11/17/2014	12/1/2014	14	1	N/A	6.95
									2	N/A	
									3	N/A	
									4	6.68	
									5	7.21	
							12/15/2014	28	6	8.03	7.71
									7	7.38	
							1/18/2015	62	8	7.84	8.03
									9	8.22	
36	200	0%	25.6%	116.5%	5.85	11/17/2014	12/1/2014	14	1	N/A	11.94
									2	12.31	
									3	11.56	
							12/15/2014	28	4	12.54	11.55
									5	9.97	
									6	12.13	
							1/18/2015	62	7	13.35	13.33
									8	13.84	
									9	12.8	
37	200	0%	18.9%	85.9%	5.44	11/17/2014	12/1/2014	14	1	13.29	14.15
									2	15.10	
									3	14.06	
							12/15/2014	28	4	16.09	14.82
									5	13.95	
									6	14.41	
							1/18/2015	62	7	16.15	16.23
									8	16.32	
									9	16.21	
38	200	0%	13.9%	64.1%	5.02	11/17/2014	12/1/2014	14	1	15.14	14.52
									2	N/A	
									3	13.9	
							12/15/2014	28	4	16	15.2
									5	14.83	
									6	14.78	
							1/18/2015	62	7	16.72	15.81
									8	14.71	
									9	16.01	
39	200	0%	3.9%	42.1%	4.18	11/17/2014	12/1/2014	14	1	17.18	16.64
									2	15.21	
									3	17.53	
							12/15/2014	28	4	18.81	19.19
									5	19.63	
									6	19.13	
							1/18/2015	62	7	18.85	19.84
									8	20.46	
									9	20.2	
40	200	50%	41.1%	161.0%	6.21	11/17/2014			1		
									2		
									3		
									4		
									5		
									6		
									7		
									8		
									9		
41	200	50%	28.7%	117.1%	5.85	11/17/2014			1		
									2		
									3		
									4		
									5		
									6		
									7		
									8		
									9		

Table A.1 Laboratory soil mixing matrix (continued).

42	200	50%	19.2%	87.4%	5.48	11/17/2014			1		
									2		
									3		
									4		
									5		
									6		
									7		
									8		
									9		
43	200	50%	11.3%	61.7%	4.89	11/17/2014	1/18/2015	62	1	N/A	4.2
									2	N/A	
									3	4.2	
									4	3.89	
									5		
									6		
									7		
									8		
									9		
44	200	50%	4.1%	44.1%	4.31	11/17/2014	12/15/2014	28	1	44.34	39.63
									2	40.62	
									3	33.93	
							1/18/2015	62	4	102.71	87.17
									5	76.66	
									6	82.13	
									7		
									8		
									9		
45	400	0%	0.0%	32.0%	1.83	11/18/2014	12/2/2014	14	1	180.4	181.06
									2	134.66	
									3	228.12	
							12/16/2014	28	4	289.61	243.64
									5	227.9	
									6	213.41	
							1/19/2015	62	7	301.58	322.28
									8	313	
									9	352.26	
46	300	0%	0.0%	32.1%	2.49	11/18/2014	12/2/2014	14	1	91.37	96.96
									2	119.82	
									3	79.68	
							12/16/2014	28	4	N/A	126.42
									5	141.6	
									6	111.23	
							1/19/2015	62	7	132.64	136.31
									8	172.01	
									9	104.28	
47	400	50%	0.0%	33.1%	1.86	11/18/2014	12/2/2014	14	1	N/A	503.18
									2	518.26	
									3	488.09	
							12/16/2014	28	4	783.75	699.98
									5	716.6	
									6	599.58	
							1/19/2015	62	7	826.93	903.41
									8	1019.32	
									9	863.98	
48	300	50%	0.0%	32.5%	2.50	11/18/2014	12/2/2014	14	1	181.84	206.51
									2	205.23	
									3	232.46	
							12/16/2014	28	4	422.72	340.28
									5	296.88	
									6	301.23	
							1/19/2015	62	7	617.39	584.24
									8	563.69	
									9	571.63	

Table A.1 Laboratory soil mixing matrix (continued).

49	300	0%	43.8%	154.3%	4.23	12/16/2014	1/7/2015	22	1	3.35	3.31
									2	N/A	
									3	3.26	
							1/13/2015	28	4	4.11	3.81
									5	3.5	
									6	N/A	
							2/16/2015	62	7	4.98	4.39
									8	N/A	
									9	3.79	
50	400	0%	43.8%	154.3%	3.17	12/16/2014	1/7/2015	22	1	7.76	7.22
									2	7.43	
									3	6.47	
							1/13/2015	28	4	7.88	6.92
									5	6.26	
									6	6.62	
							2/16/2015	62	7	6.84	6.91
									8	7.08	
									9	6.81	
51	500	0%	43.8%	154.3%	2.53	12/16/2014	1/7/2015	22	1	15.82	16.05
									2	17.38	
									3	14.96	
							1/13/2015	28	4	19.46	17.37
									5	18.06	
									6	14.6	
							2/16/2015	62	7	22.48	20.82
									8	N/A	
									9	19.15	
52	300	50%	43.8%	154.3%	4.21	12/16/2014			1		
									2		
									3		
									4		
									5		
									6		
									7		
									8		
									9		
53	400	50%	43.8%	154.3%	3.15	12/16/2014			1		
									2		
									3		
									4		
									5		
									6		
									7		
									8		
									9		
54	500	50%	43.8%	154.3%	2.51	12/16/2014			1		
									2		
									3		
									4		
									5		
									6		
									7		
									8		
									9		
55	500	0%	42.1%	154.1%	2.11	12/18/2014	1/15/2015	28	1	43.59	41.11
									2	36.16	
									3	43.59	
							2/18/2015	62	4	53.37	48.59
									5	47.62	
									6	44.78	
									7		
									8		
									9		

Table A.1 Laboratory soil mixing matrix (continued).

57	500	50%	42.1%	154.1%	2.09	12/18/2014	1/15/2015	28	1	8.42	8.61
									2	9.07	
									3	8.33	
							2/18/2015	62	4	9.25	9.18
									5	10.59	
									6	7.69	
									7		
									8		
									9		

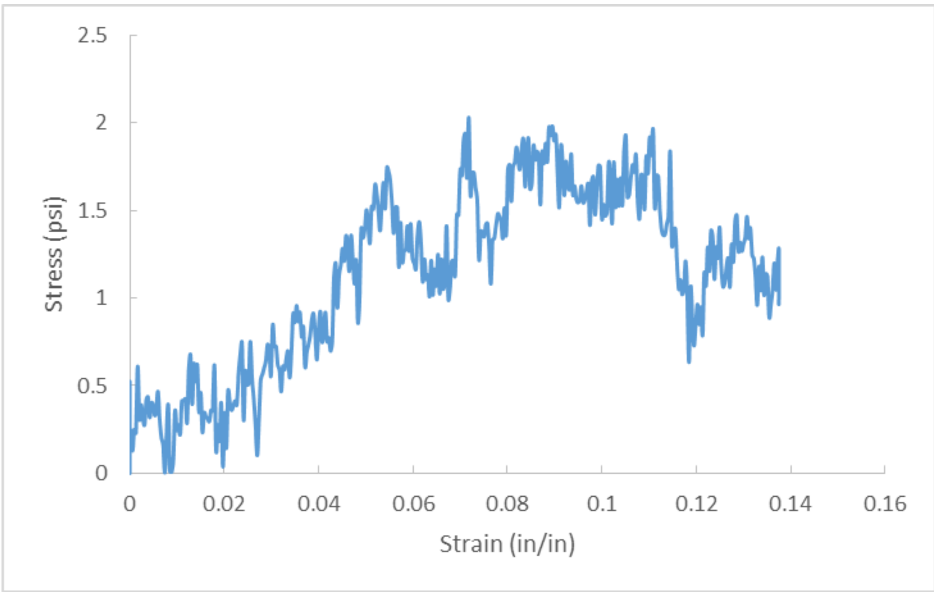

Figure A.1. D2-1, OC= 66.4%, CF= 300 pcf, T=14 days.

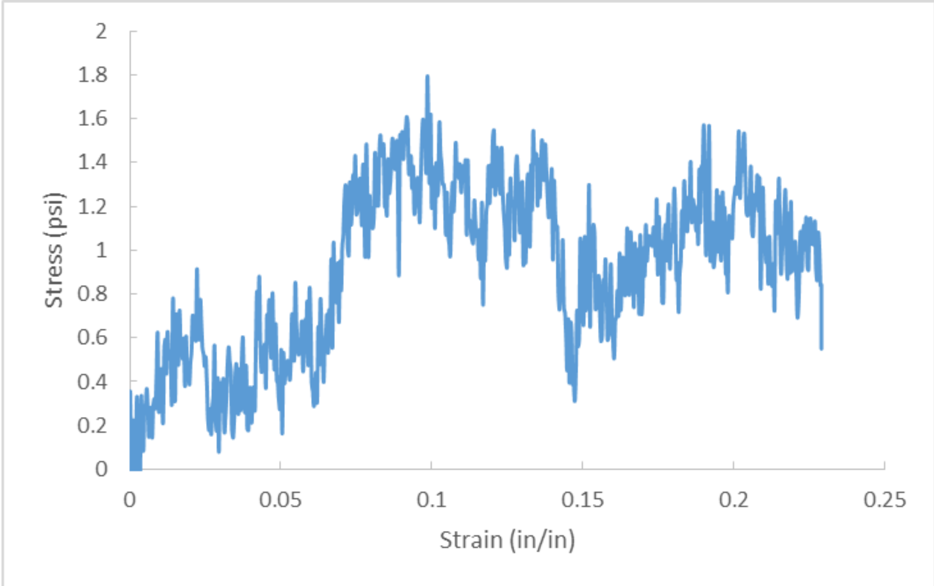

Figure A.2. D2-2, OC= 66.4%, CF= 300 pcf, T=14 days.

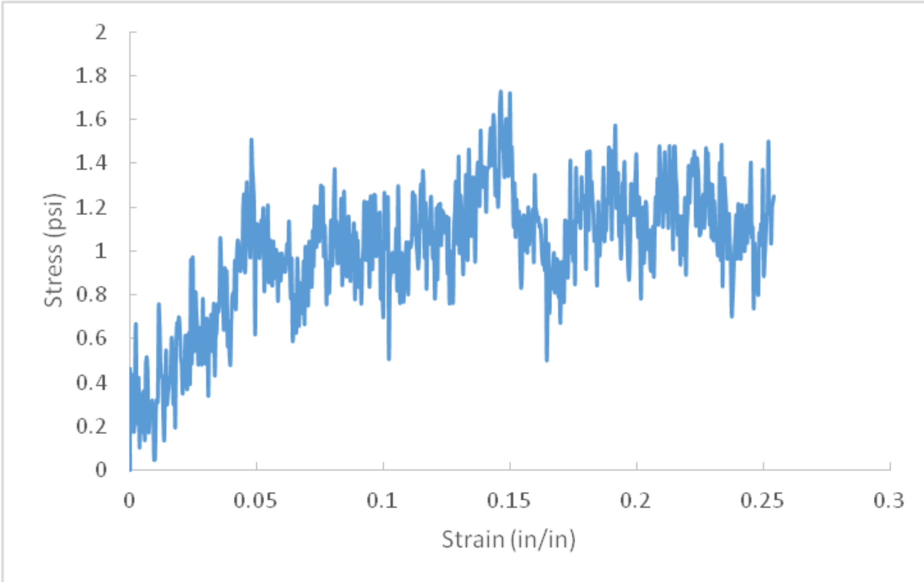

Figure A.3. D2-3, OC= 66.4%, CF= 300 pcf, T=14 days.

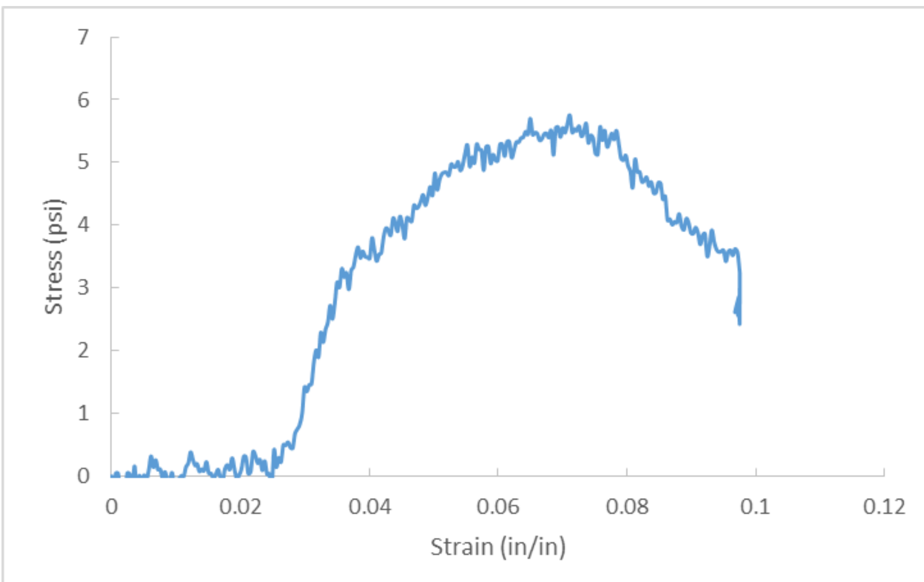

Figure A.4. D3-1, OC= 66.4%, CF= 400 pcf, T=14 Days.

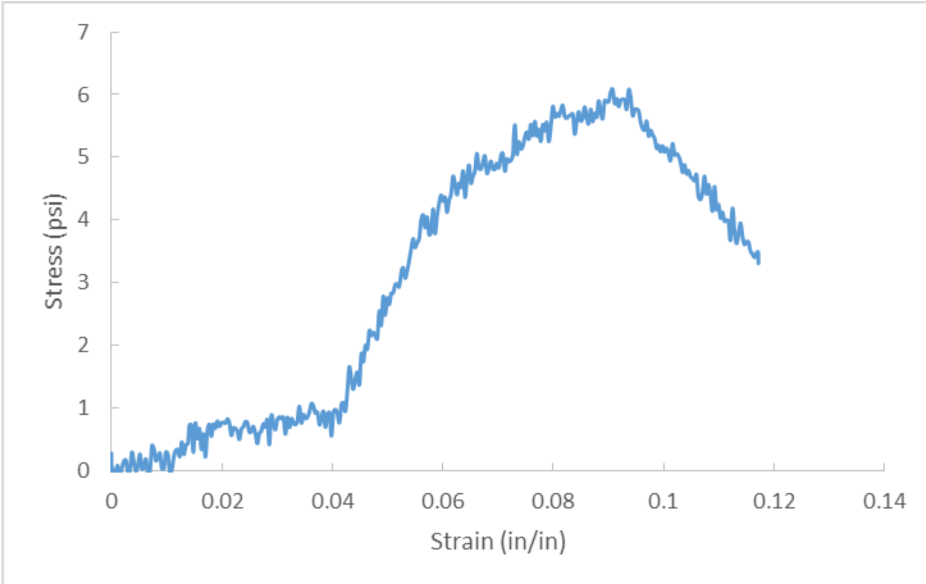

Figure A.5. D3-2, OC= 66.4%, CF= 400 pcf, T=14 Days.

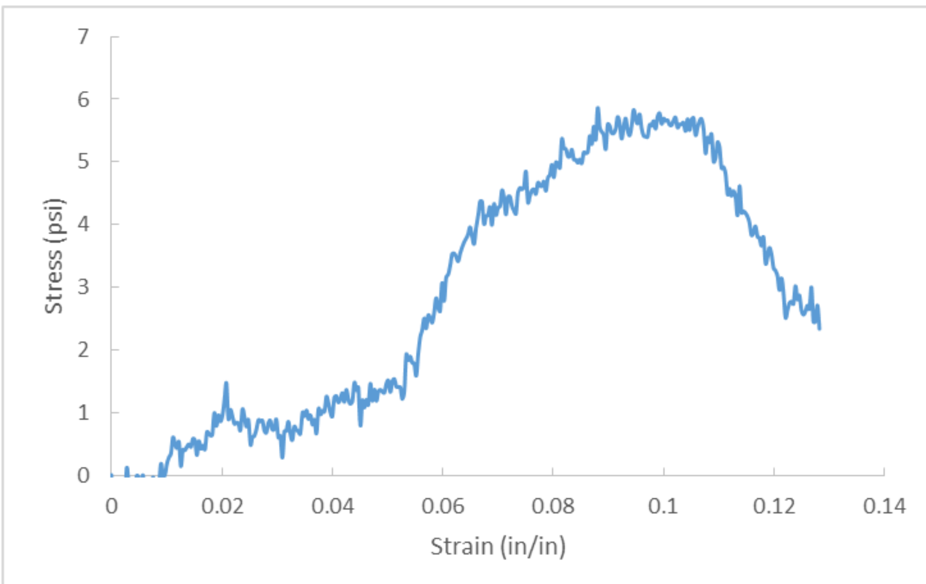

Figure A.6. D3-3, OC= 66.4%, CF= 400 pcf, T=14 Days.

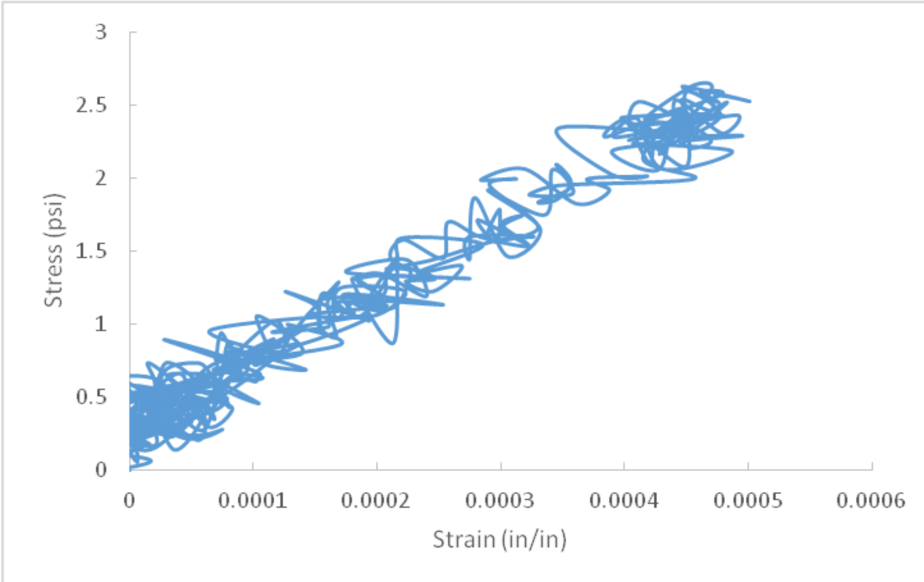

Figure A.7. D4-1, OC= 66.4%, CF= 200 pcf, T=14 Days.

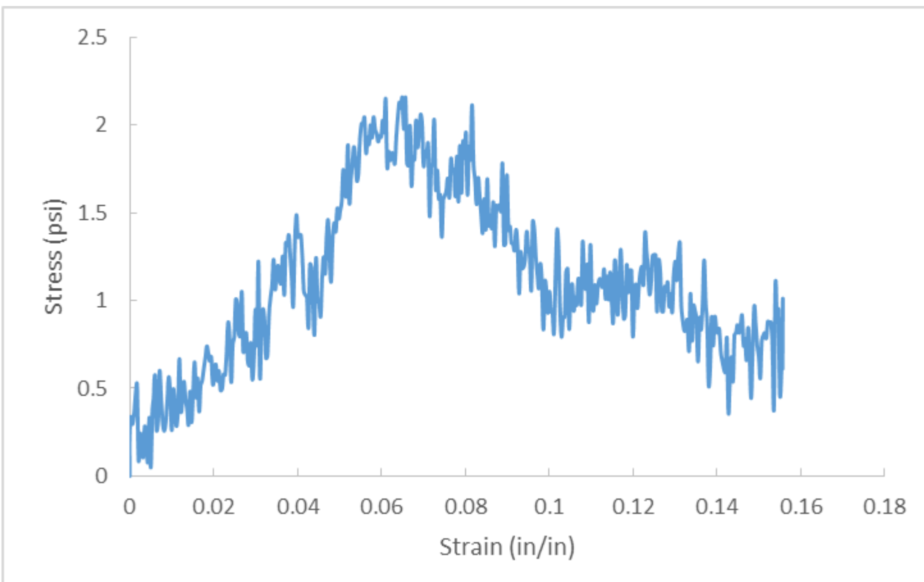

Figure A.8. D4-2, OC= 66.4%, CF= 200 pcf, T=14 Days.

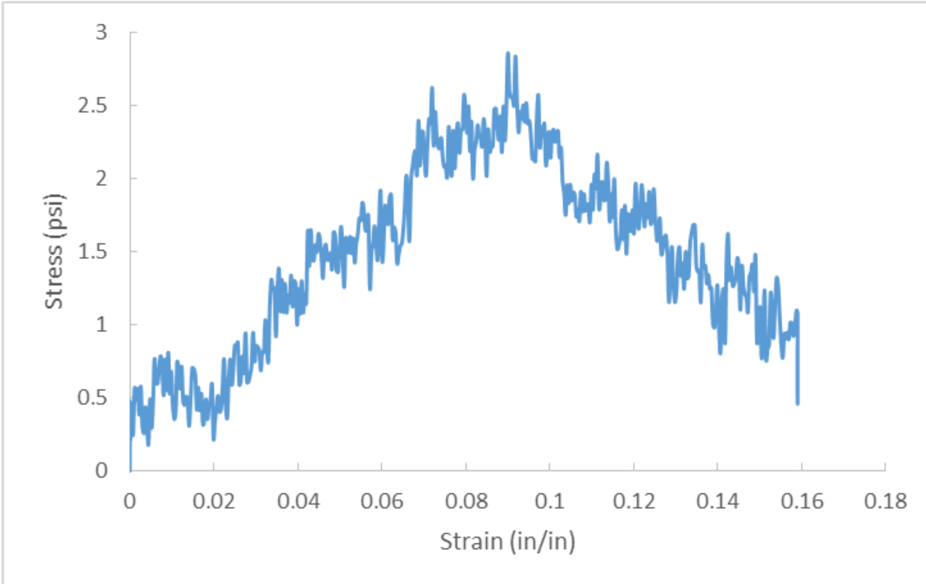

Figure A.9. D4-3, OC= 66.4%, CF= 200 pcf, T=14 Days.

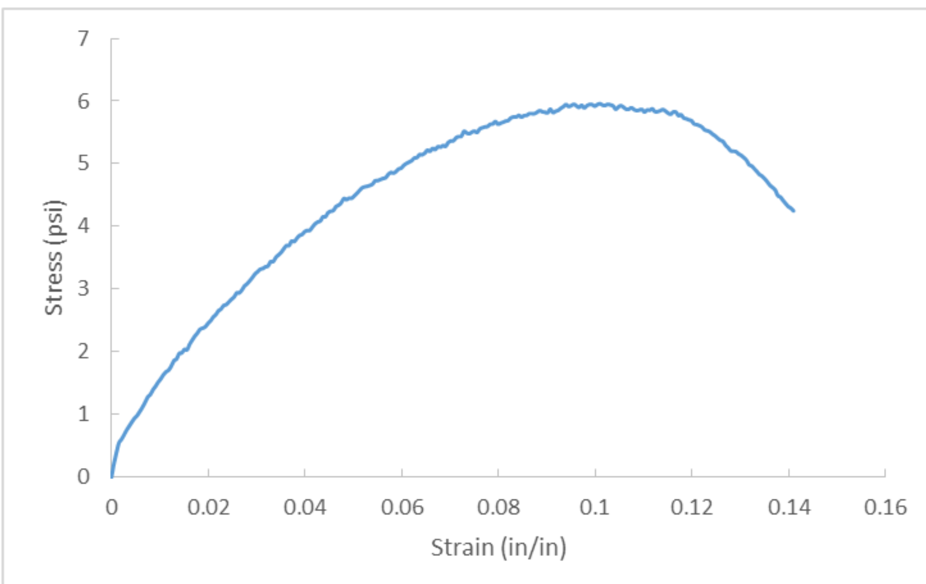

Figure A.10. D8-1, OC= 65.9%, CF= 300 pcf, T=14 Days.

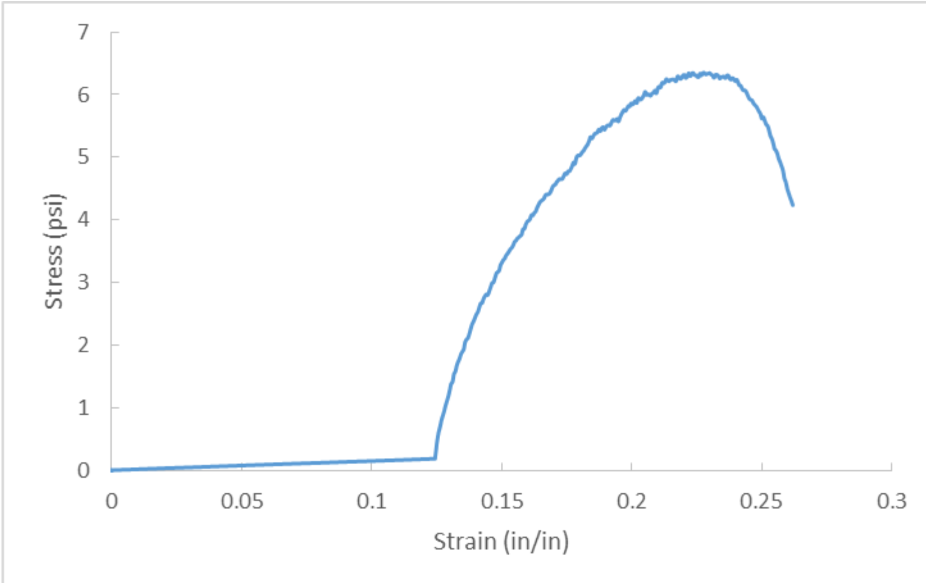

Figure A.11. D8-2, OC= 65.9%, CF= 300 pcf, T=14 Days.

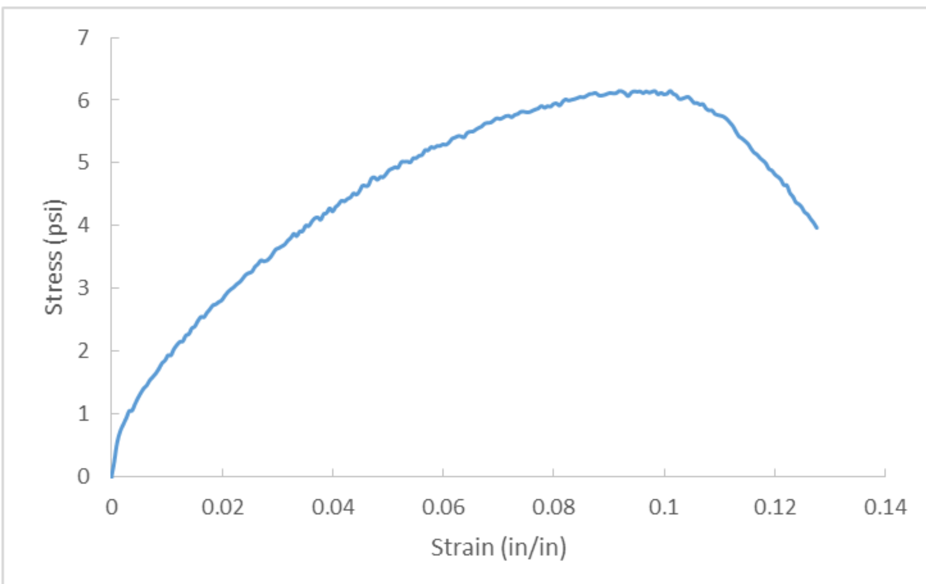

Figure A.12. D8-3, OC= 65.9%, CF= 300 pcf, T=14 Days.

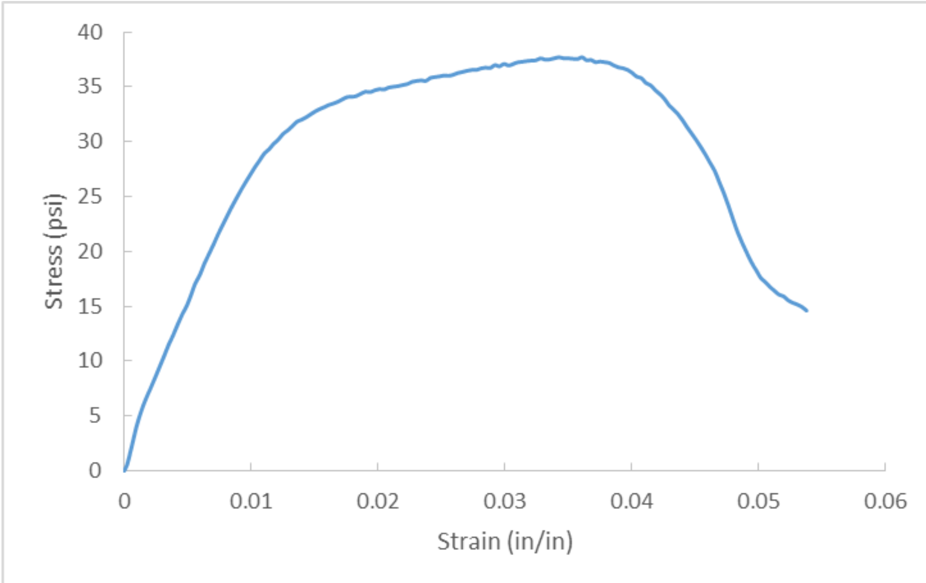

Figure A.13. D9-1, OC= 65.9%, CF= 400 pcf, T=14 Days.

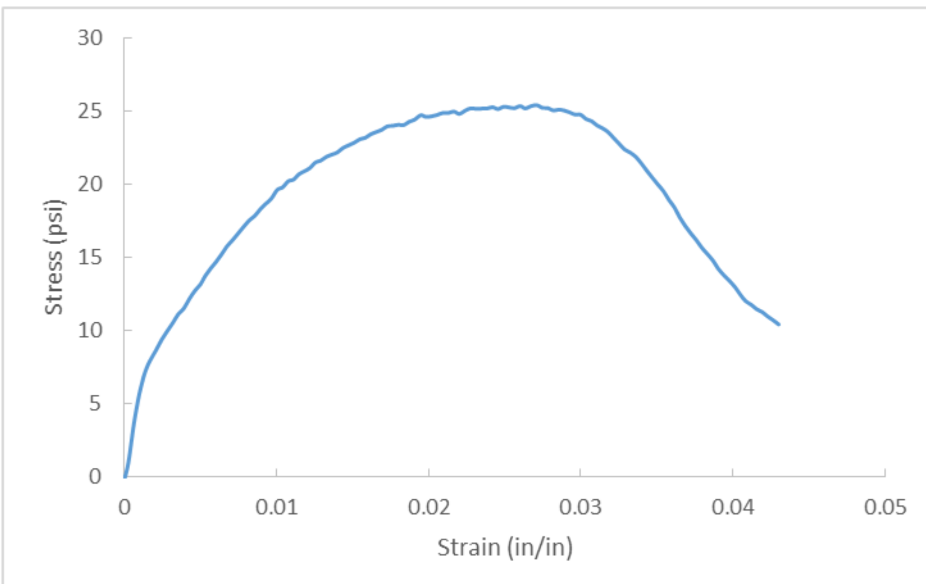

Figure A.14. D9-2, OC=65.9%, CF=400 pcf, T=14 Days.

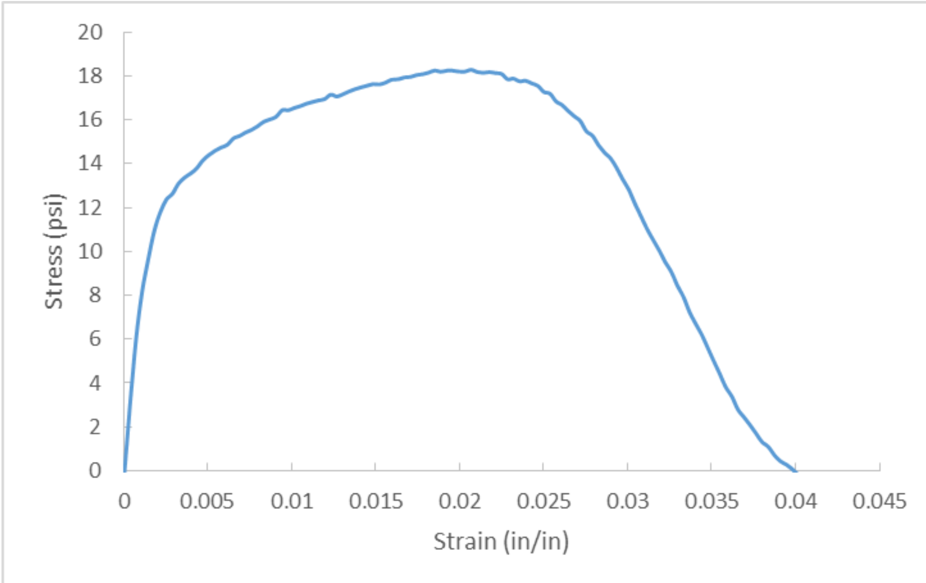

Figure A.15. D9-3, OC= 65.9%, CF= 400 pcf, T=14 Days.

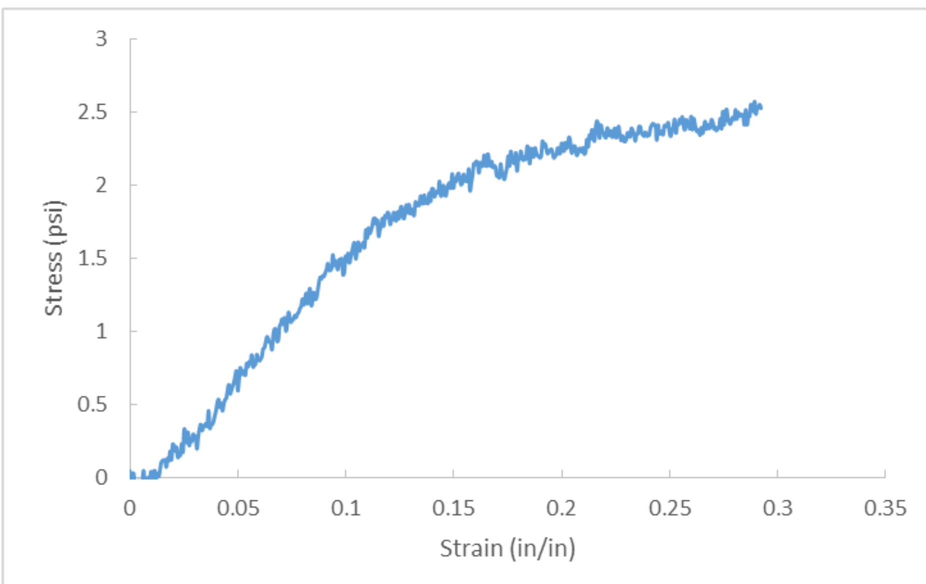

Figure A.16. D10-1, OC=41.3%, CF=300 pcf, T=14 Days.

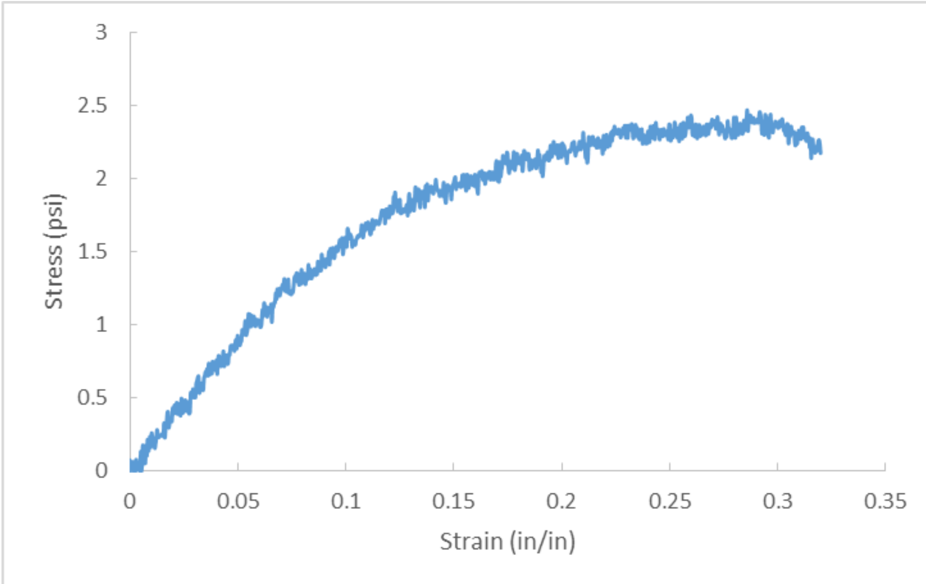

Figure A.17. D10-2, OC= 41.3%, CF= 300 pcf, T=14 Days.

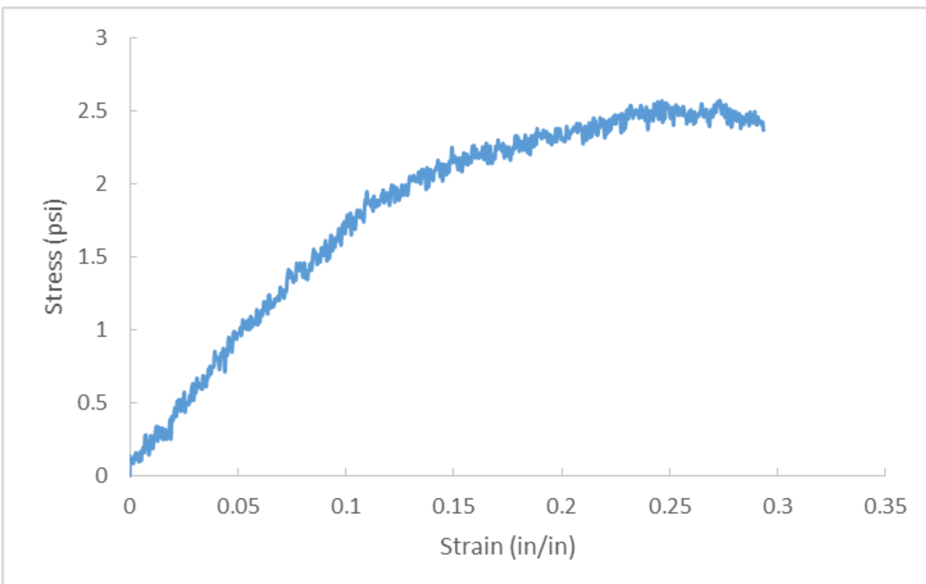

Figure A.18. D10-3, OC= 41.3%, CF= 300 pcf, T=14 Days.

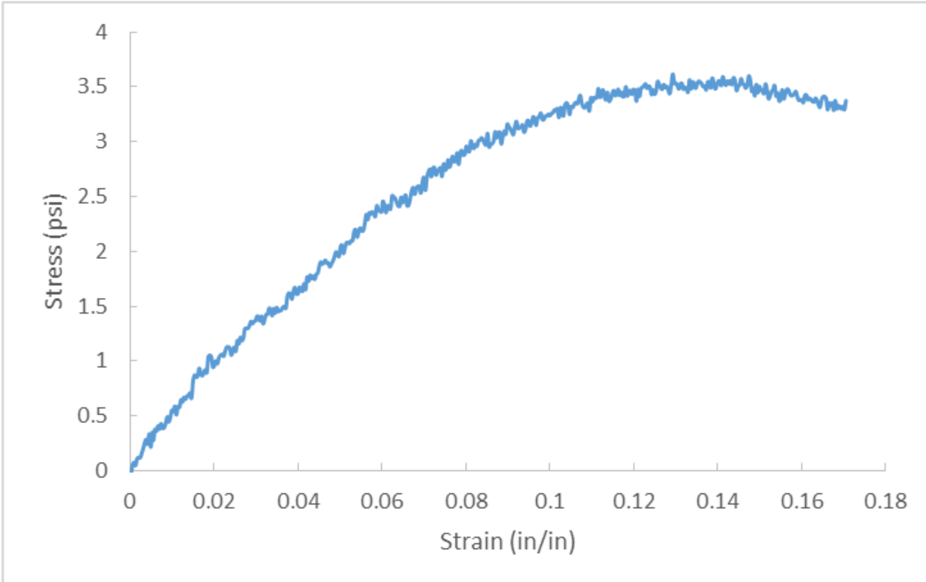

Figure A.19. D11-1, OC= 27.8%, CF= 300 pcf, T=14 Days.

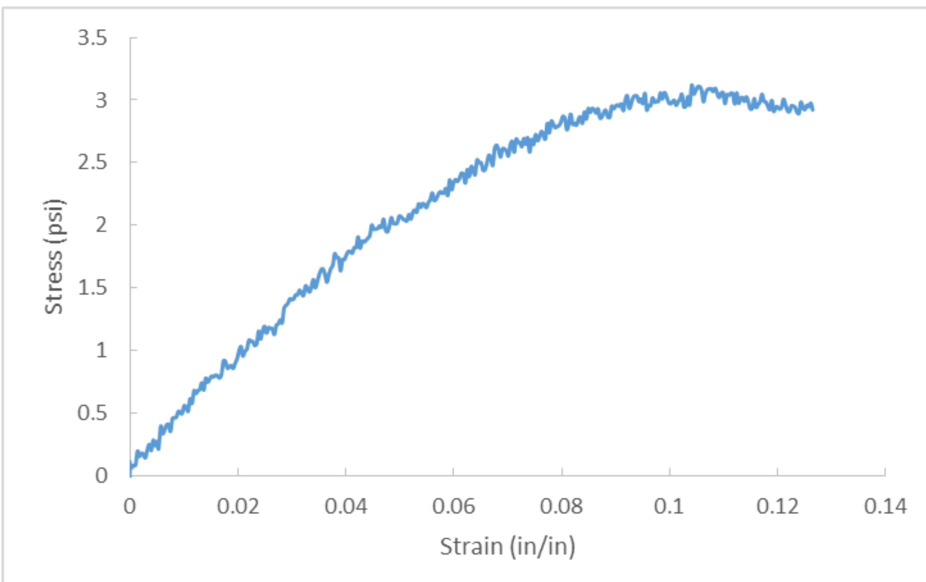

Figure A.20. D11-2, OC=27.8%, CF=300 pcf, T=14 Days.

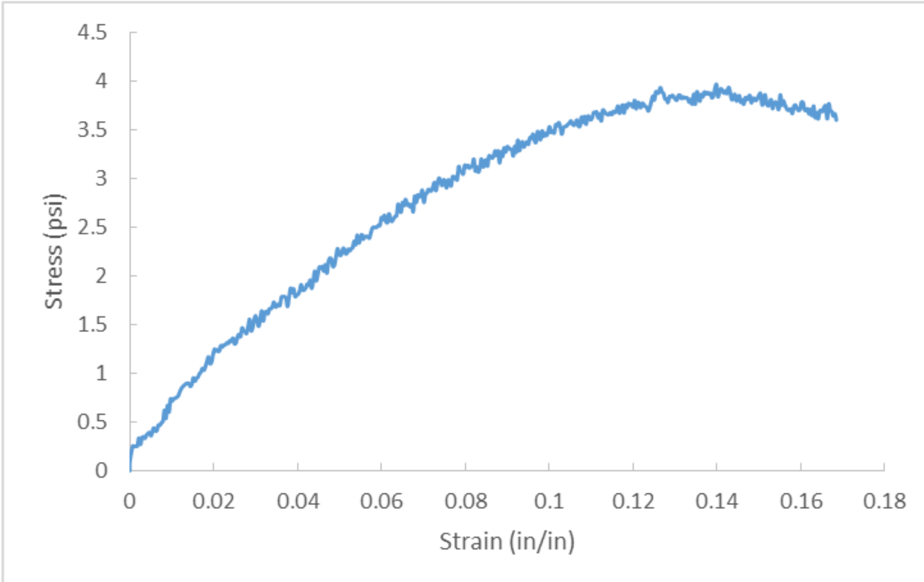

Figure A.21. D11-3, OC= 27.8%, CF= 300 pcf, T=14 Days.

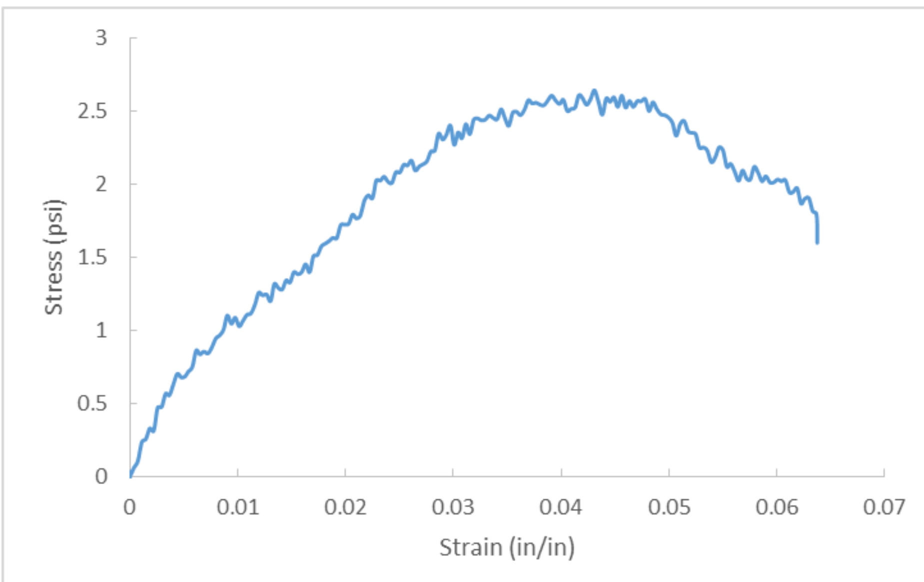

Figure A.22. D12-1, OC= 24.1%, CF= 300 pcf, T=14 Days.

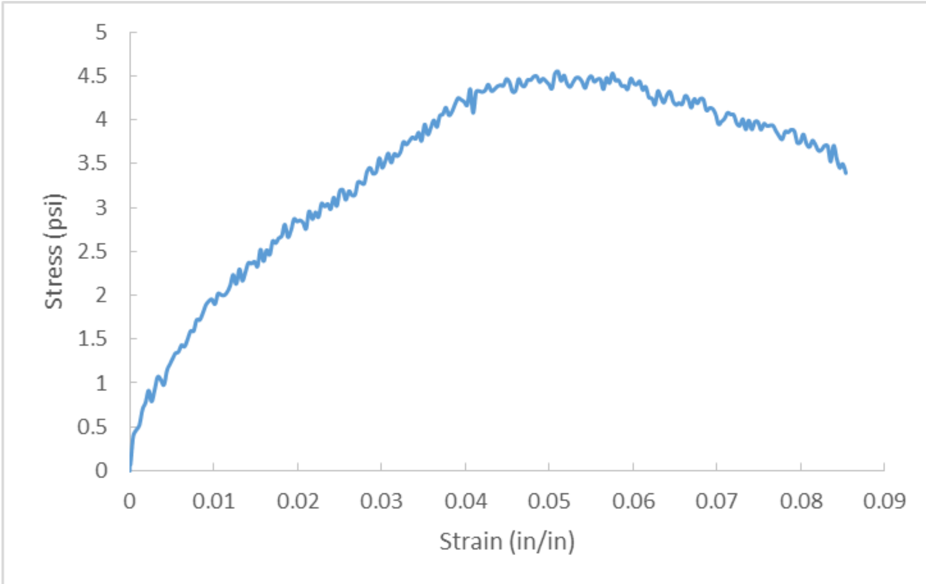

Figure A.23. D12-2, OC= 24.1%, CF= 300 pcf, T=14 Days.

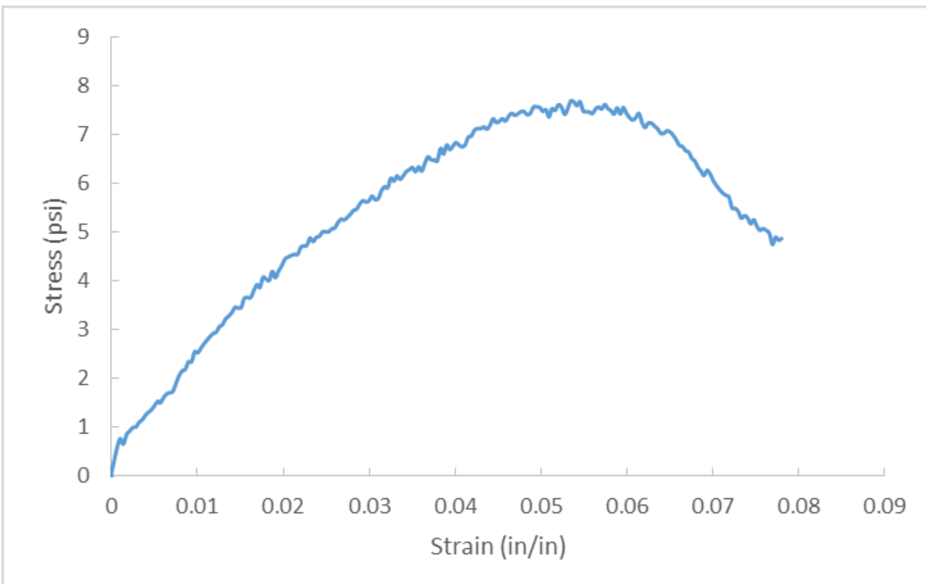

Figure A.24. D12-3, OC= 24.1%, CF= 300 pcf, T=14 Days.

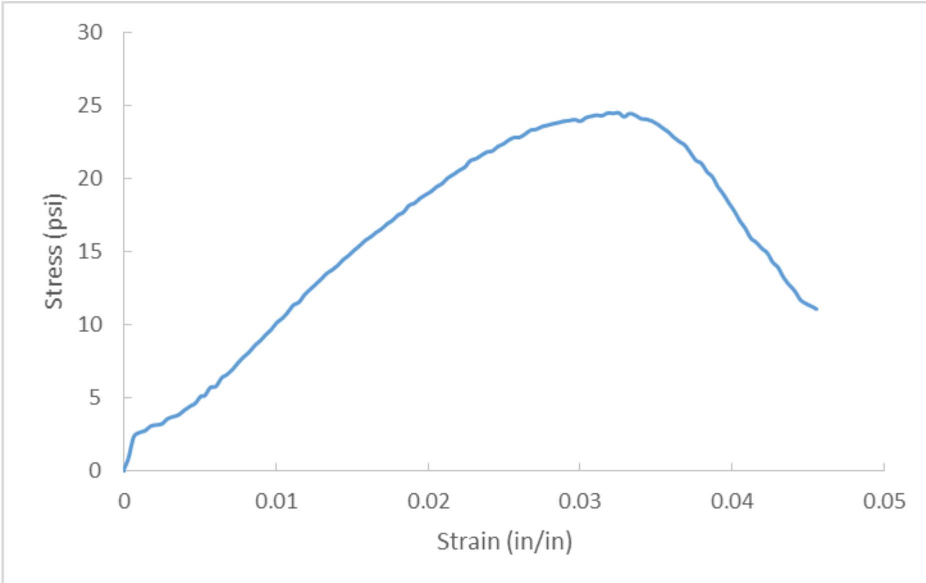

Figure A.25. D13-1, OC= 11.2%, CF= 300 pcf, T=14 Days.

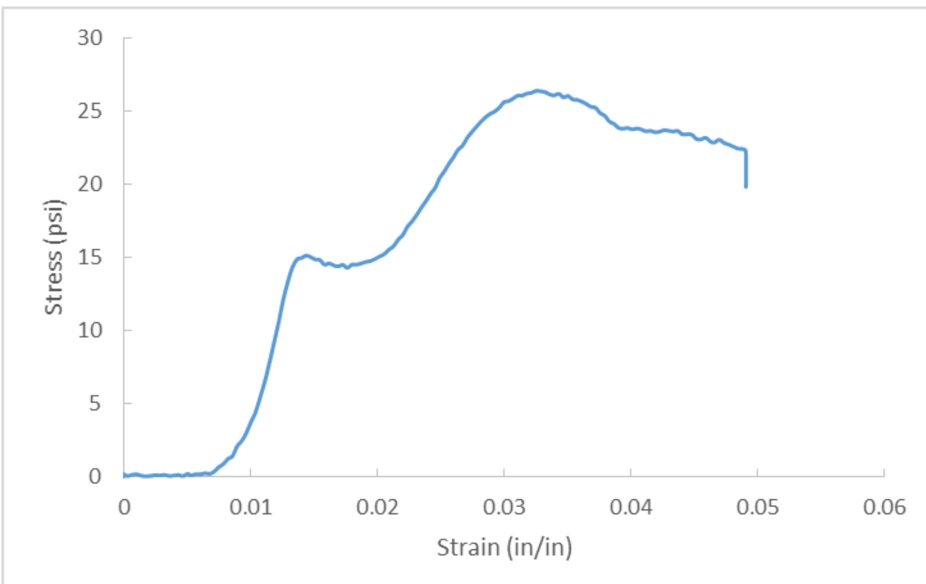

Figure A.26. D13-2, OC= 11.2%, CF= 300 pcf, T=14 Days.

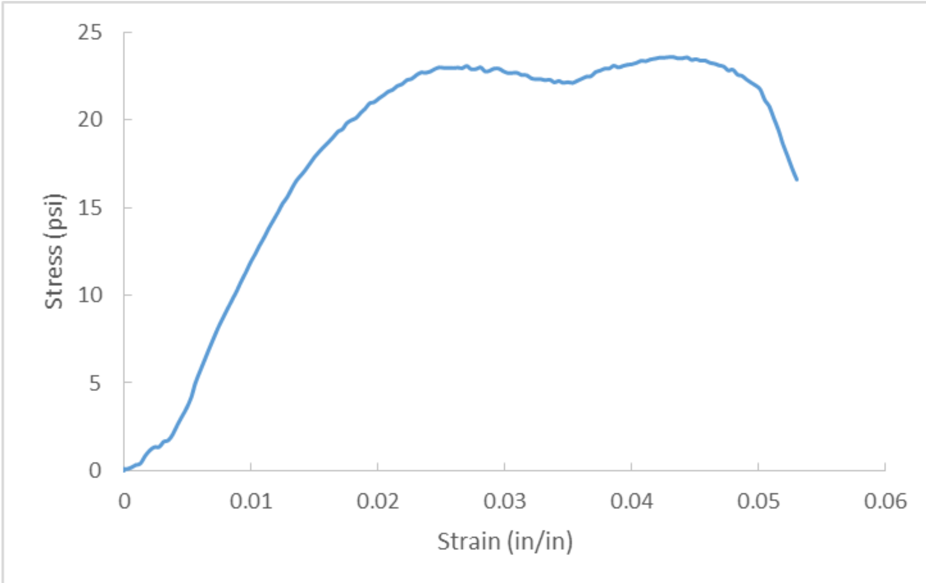

Figure A.27. D13-3, OC= 11.2%, CF= 300 pcf, T=14 Days.

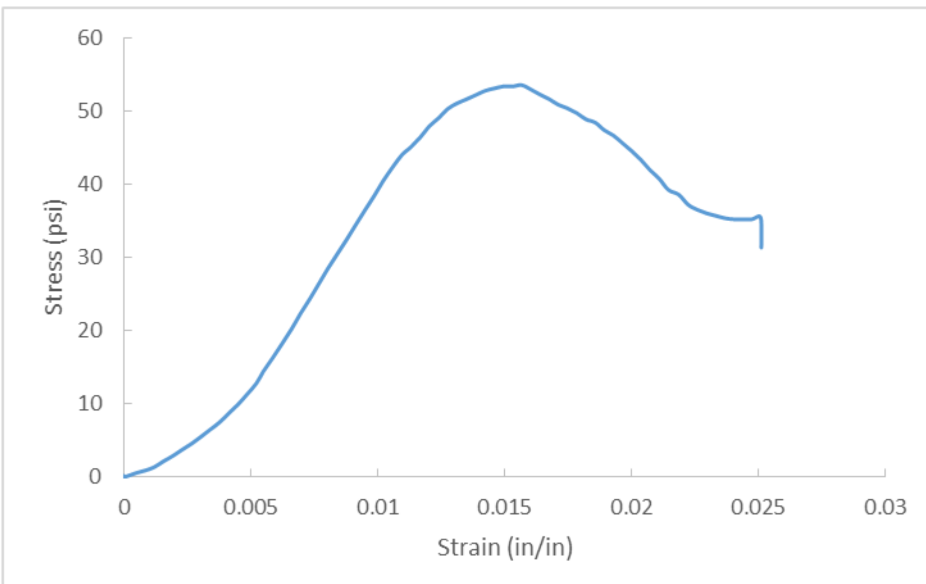

Figure A.28. D14-1, OC= 4.6%, CF= 300 pcf, T=14 Days.

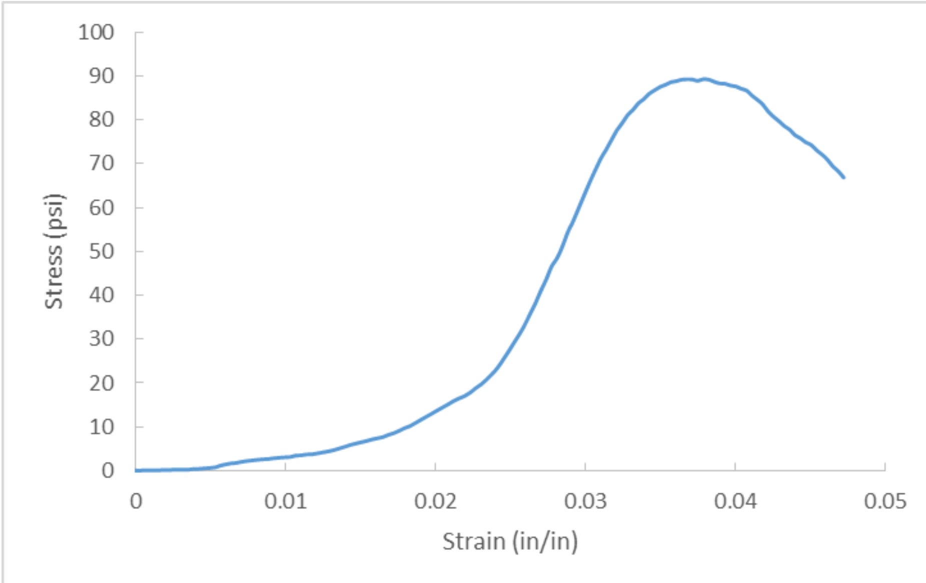

Figure A.29. D14-2, OC= 4.6%, CF= 300 pcf, T=14 Days.

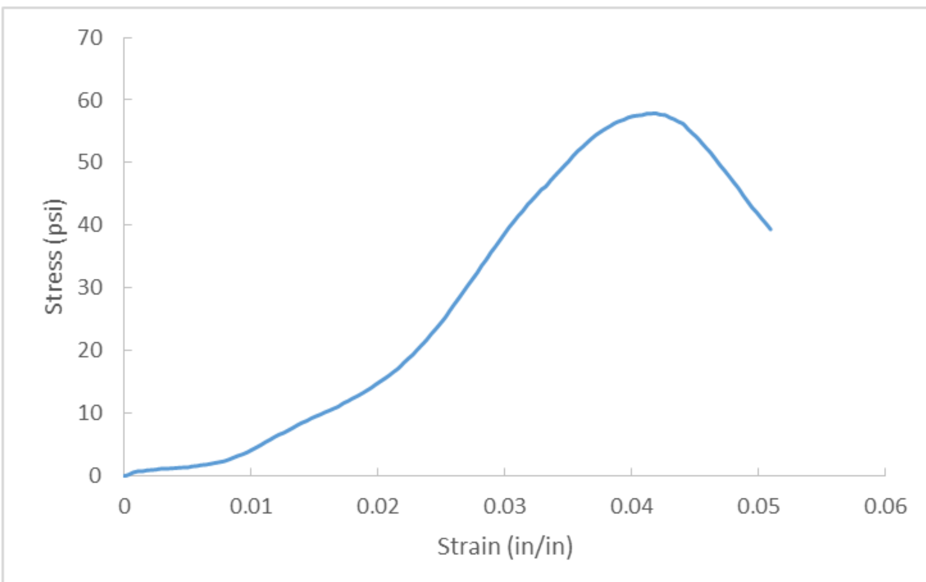

Figure A.30. D14-3, OC= 4.6%, CF= 300 pcf, T=14 Days.

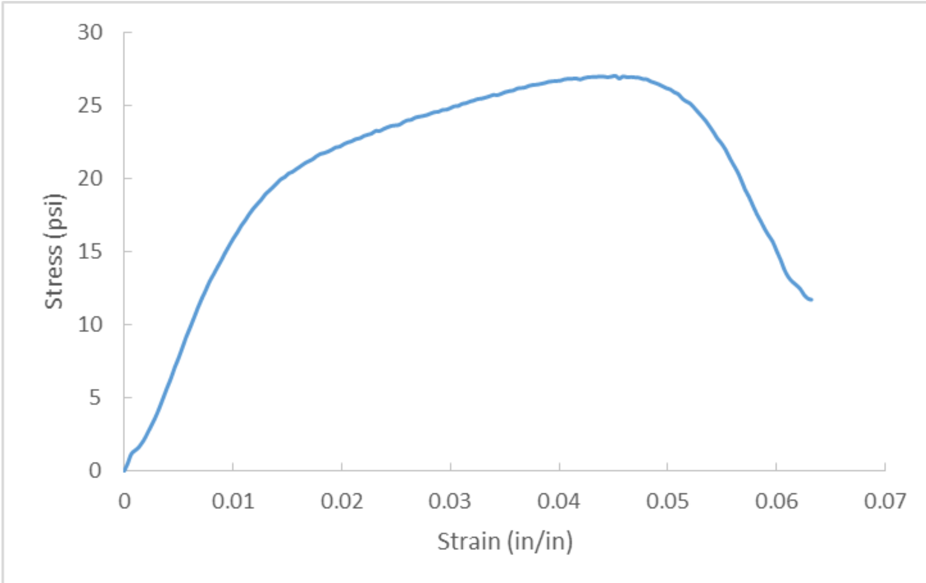

Figure A.31. D15-1, OC= 40.5%, CF= 300 pcf, T=14 Days.

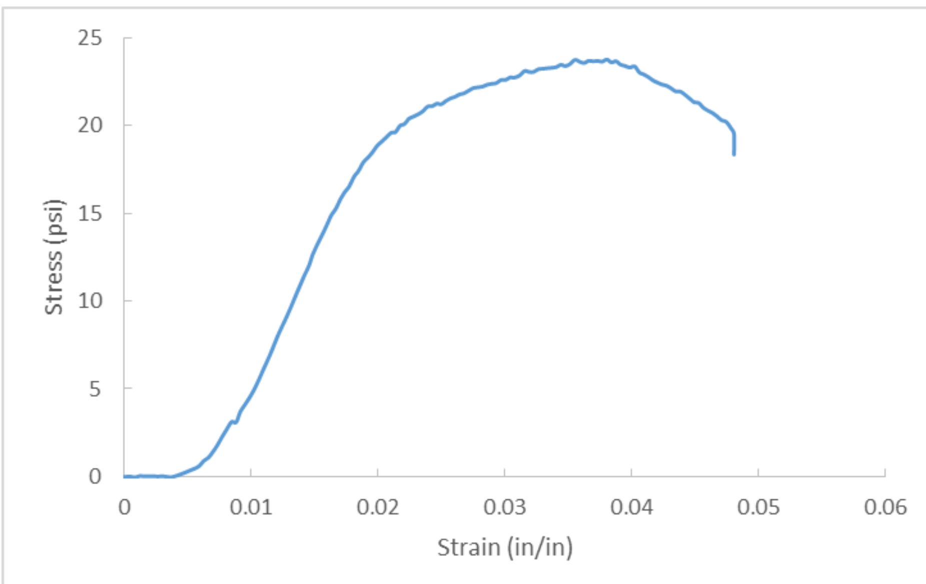

Figure A.32. D15-2, OC= 40.5%, CF= 300 pcf, T=14 Days.

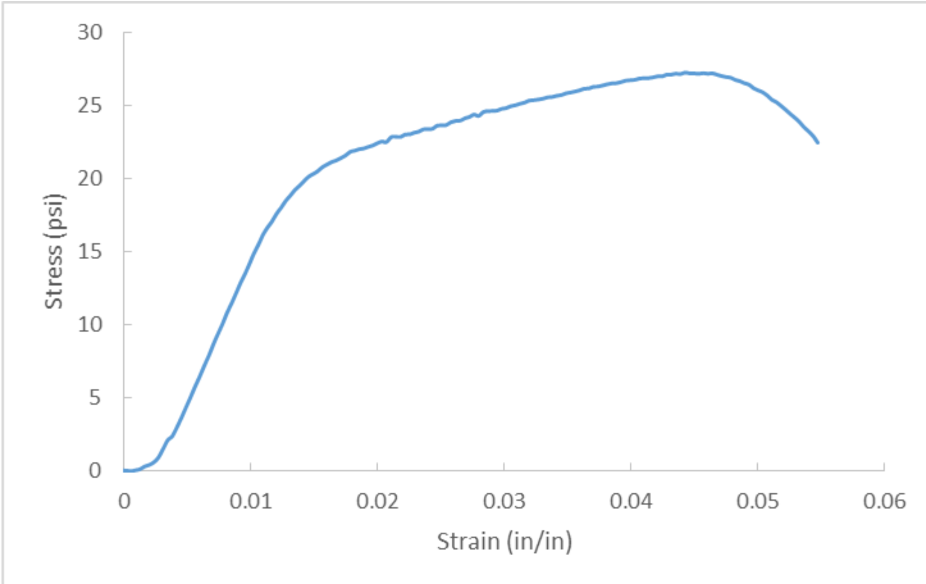

Figure A.33. D15-3, OC= 40.5%, CF= 300 pcf, T=14 Days.

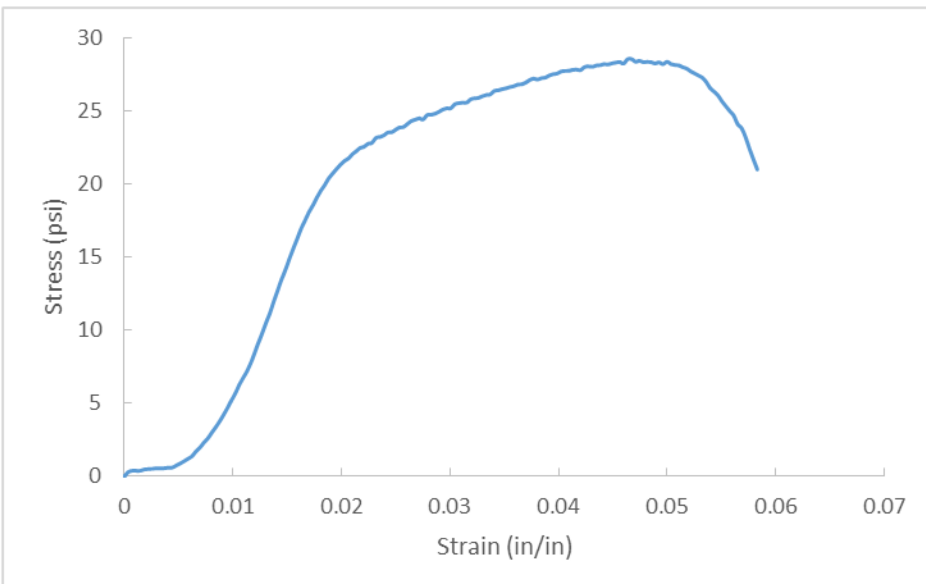

Figure A.34. D16-1, OC= 34.7%, CF= 300 pcf, T=14 Days.

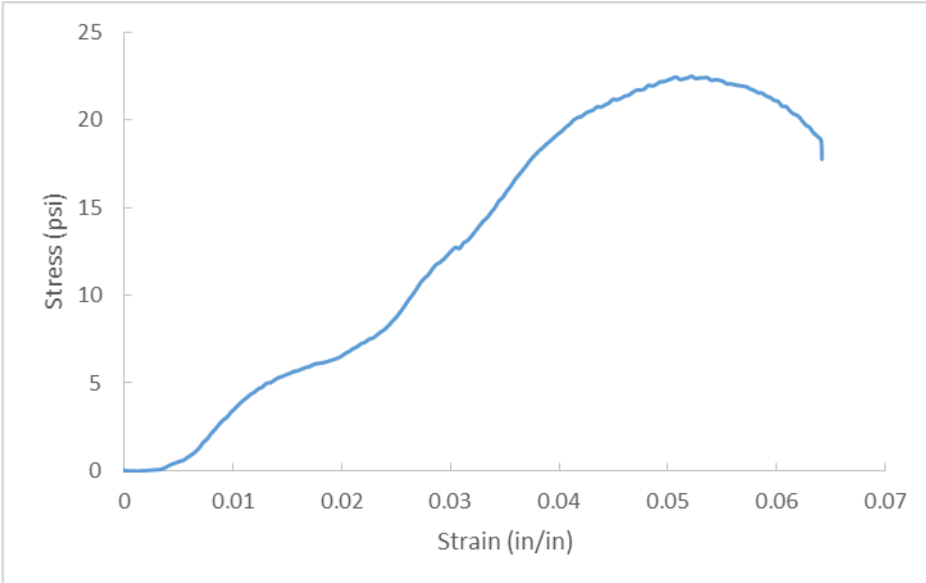

Figure A.35. D16-2, OC= 34.7%, CF= 300 pcf, T=14 Days.

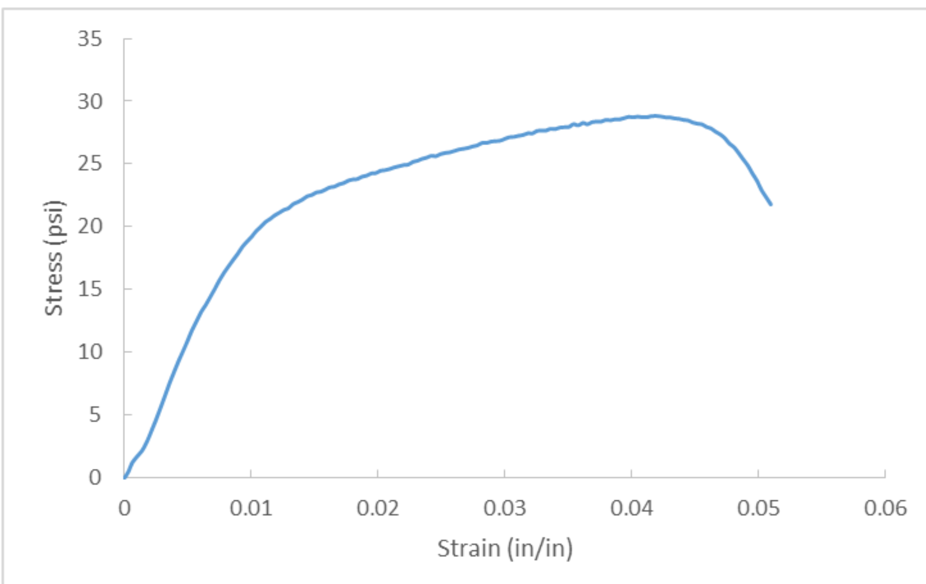

Figure A.36. D16-3, OC= 34.7%, CF= 300 pcf, T=14 Days.

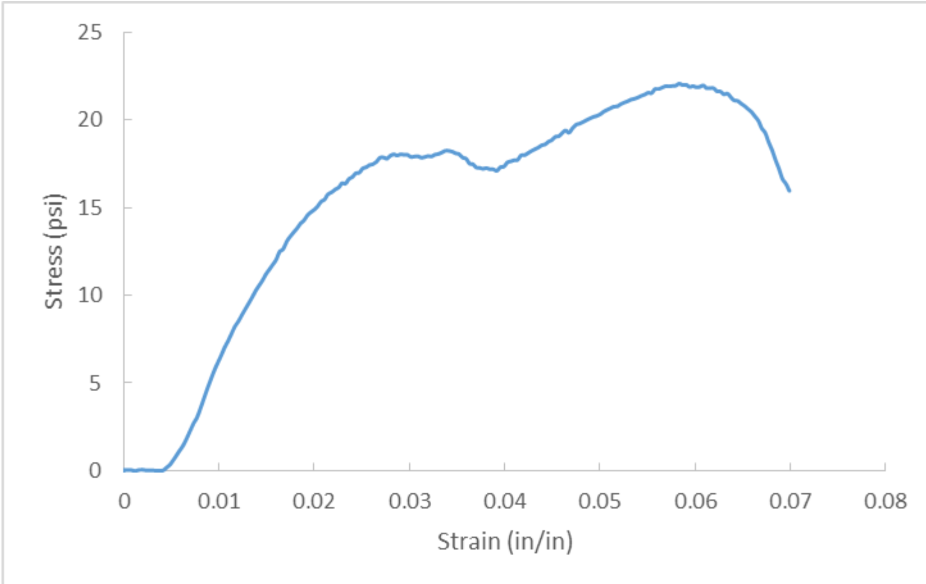

Figure A.37. D17-1, OC= 19.2%, CF= 300 pcf, T=14 Days.

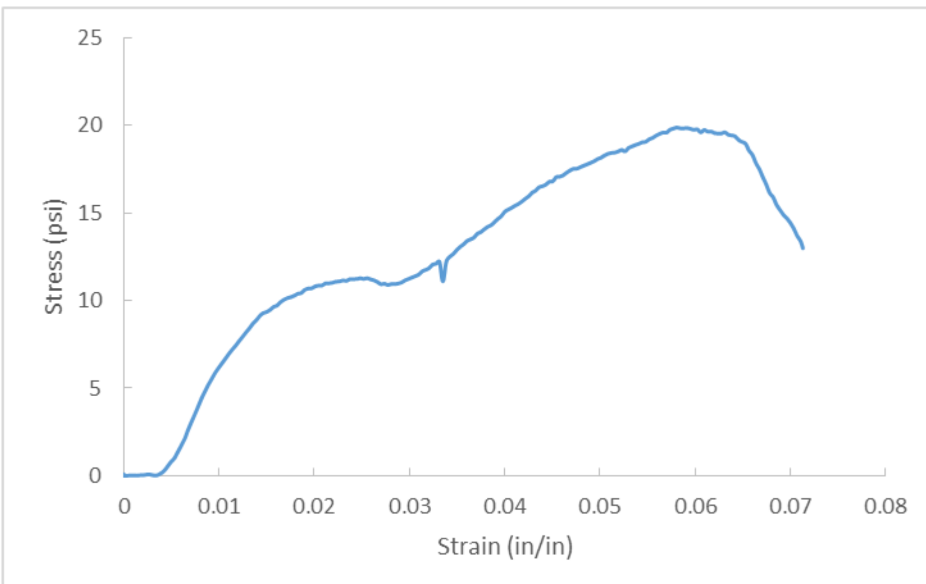

Figure A.38. D17-2, OC= 19.2%, CF= 300 pcf, T=14 Days.

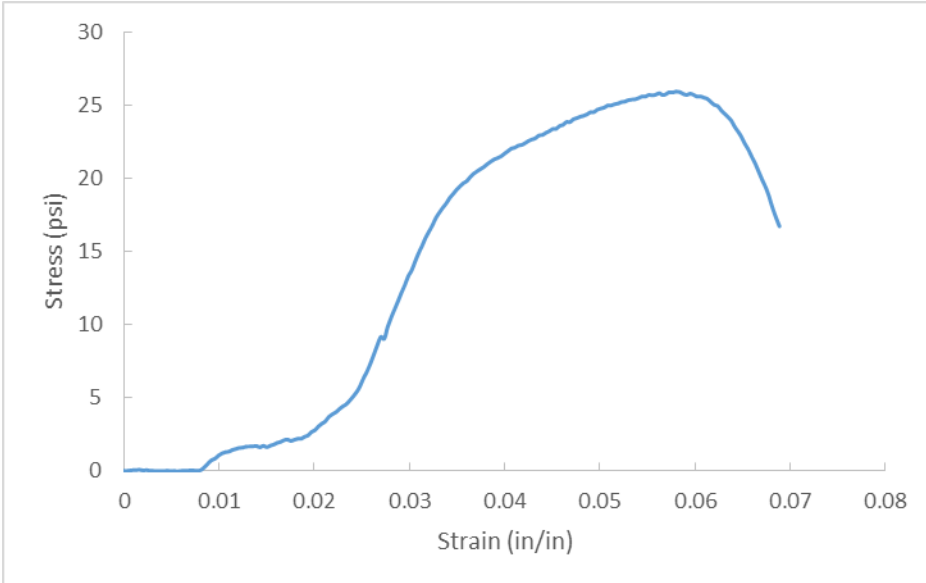

Figure A.39. D17-3, OC= 19.2%, CF= 300 pcf, T=14 Days.

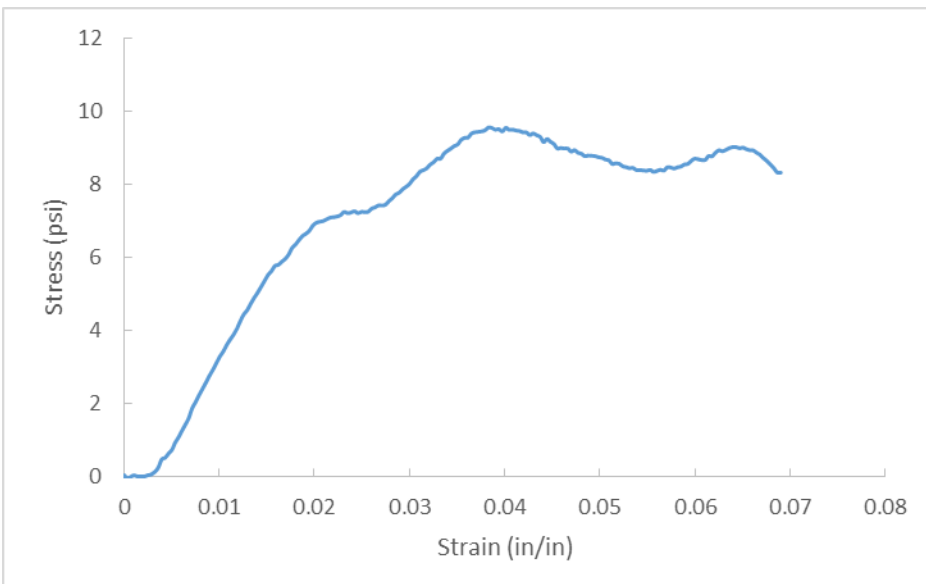

Figure A.40. D18-1, OC= 18.9%, CF= 300 pcf, T=14 Days.

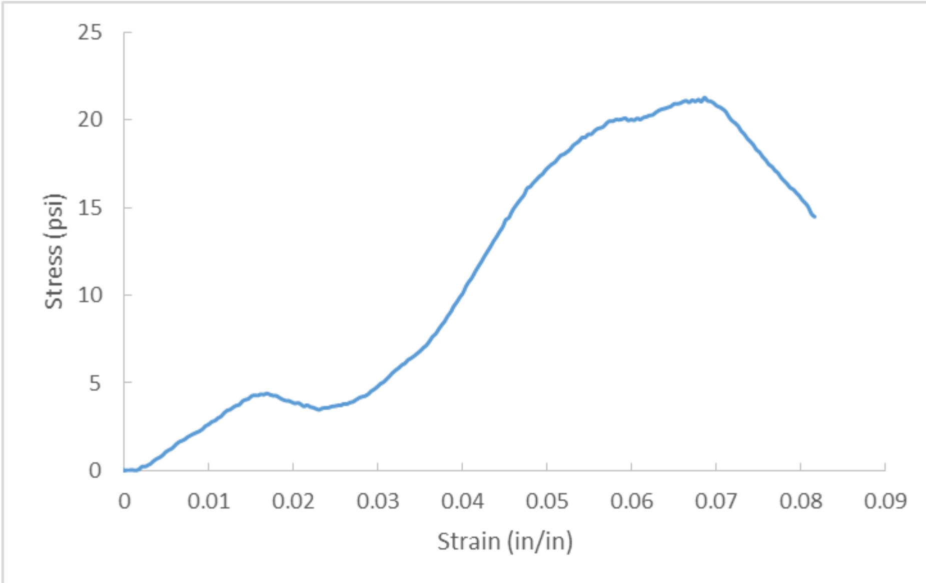

Figure A.41. D18-2, OC= 18.9%, CF= 300 pcf, T=14 Days.

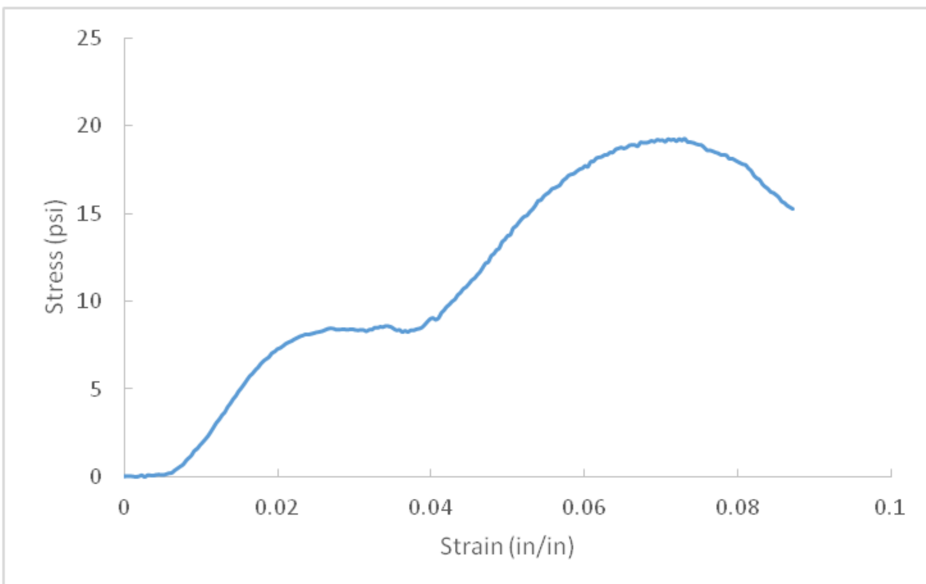

Figure A.42. D18-3, OC= 18.9%, CF= 300 pcf, T=14 Days.

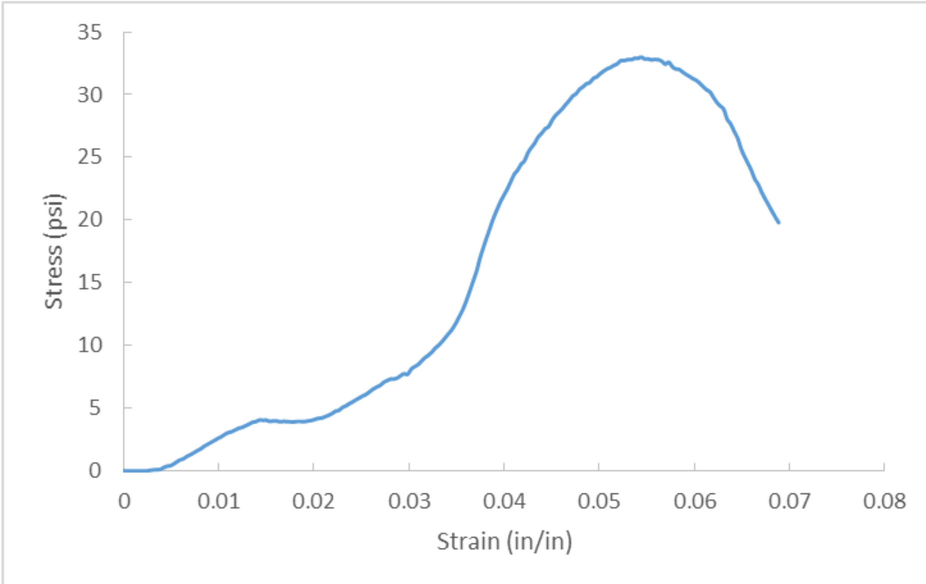

Figure A.43. D19-1, OC= 8.5%, CF= 300 pcf, T=14 Days.

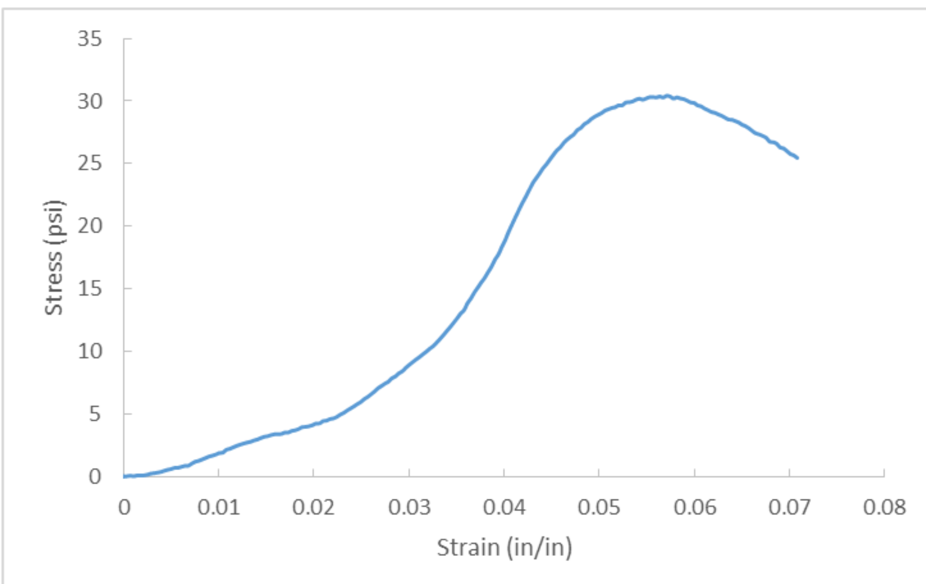

Figure A.44. D19-2, OC= 8.5%, CF= 300 pcf, T=14 Days.

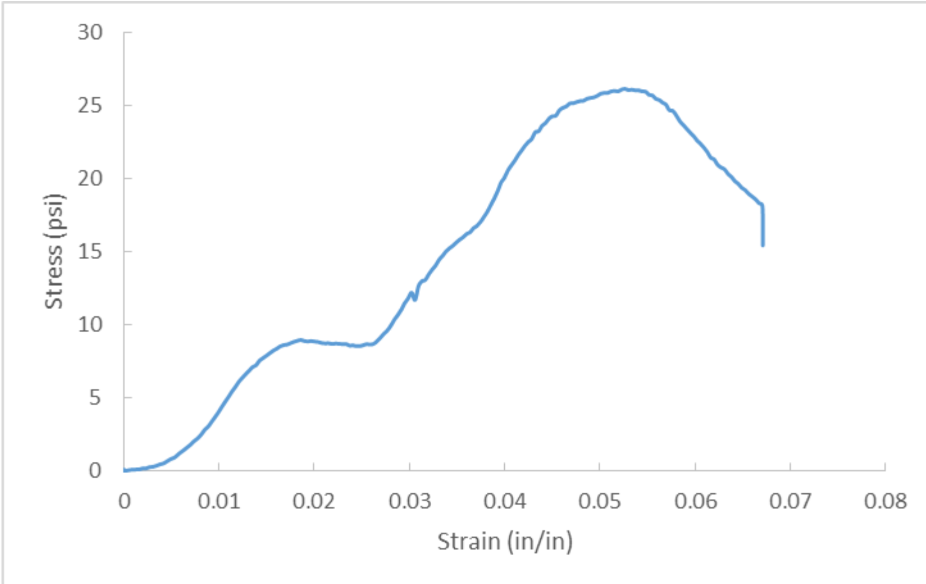

Figure A.45. D19-3, OC= 8.5%, CF= 300 pcf, T=14 Days.

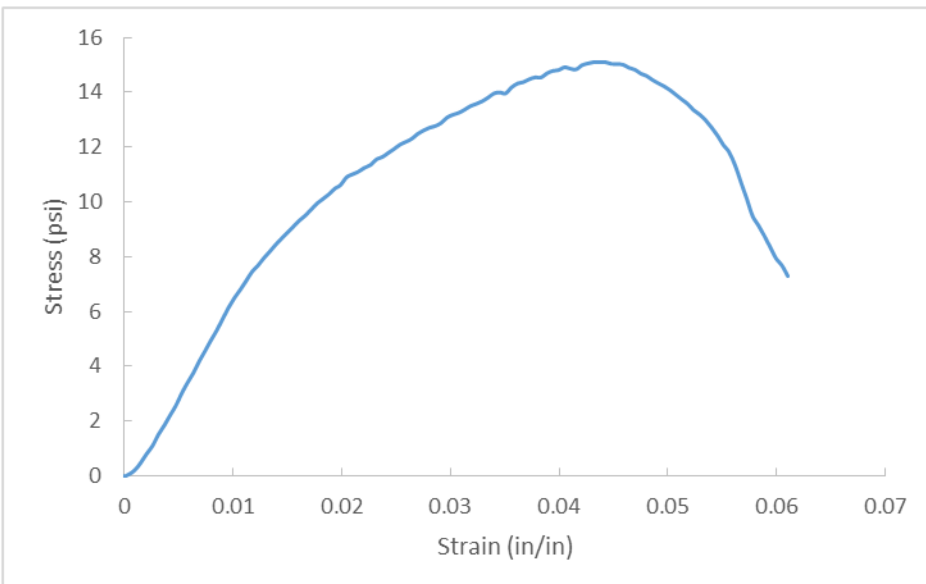

Figure A.46. D25-1, OC= 41.2%, CF= 400 pcf, T=14 Days.

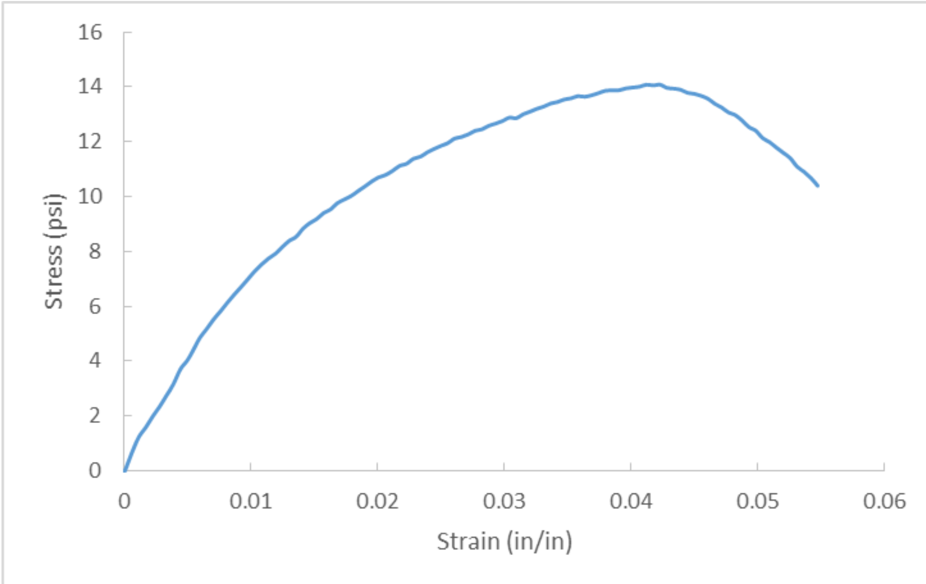

Figure A.47. D25-2, OC= 41.2%, CF= 400 pcf, T=14 Days.

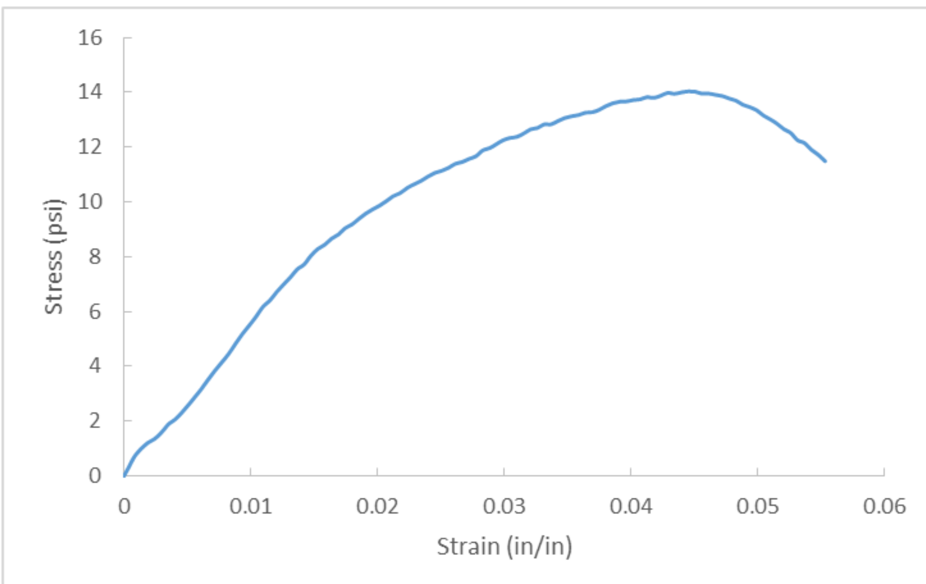

Figure A.48. D25-3, OC= 41.2%, CF= 400 pcf, T=14 Days.

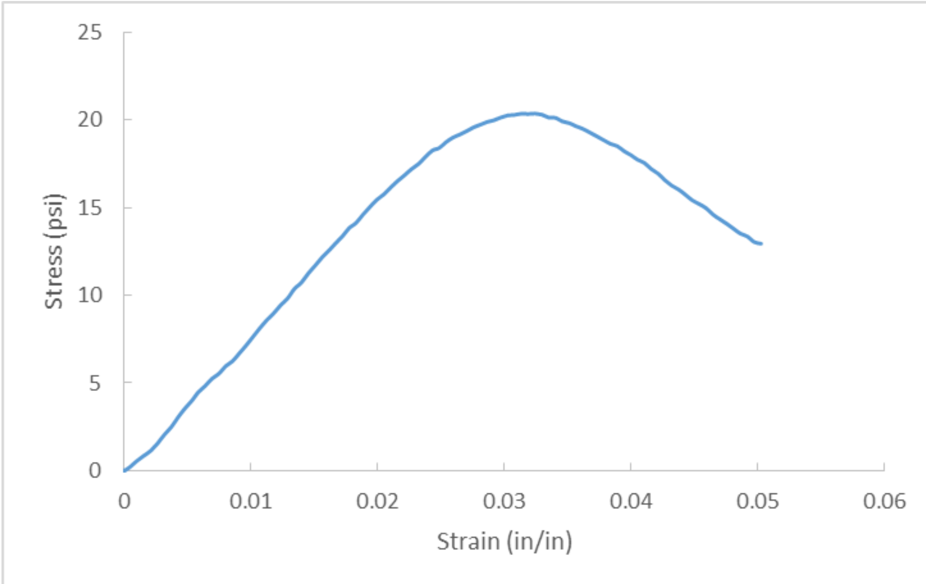

Figure A.49. D26-1, OC= 29.8%, CF= 400 pcf, T=14 Days.

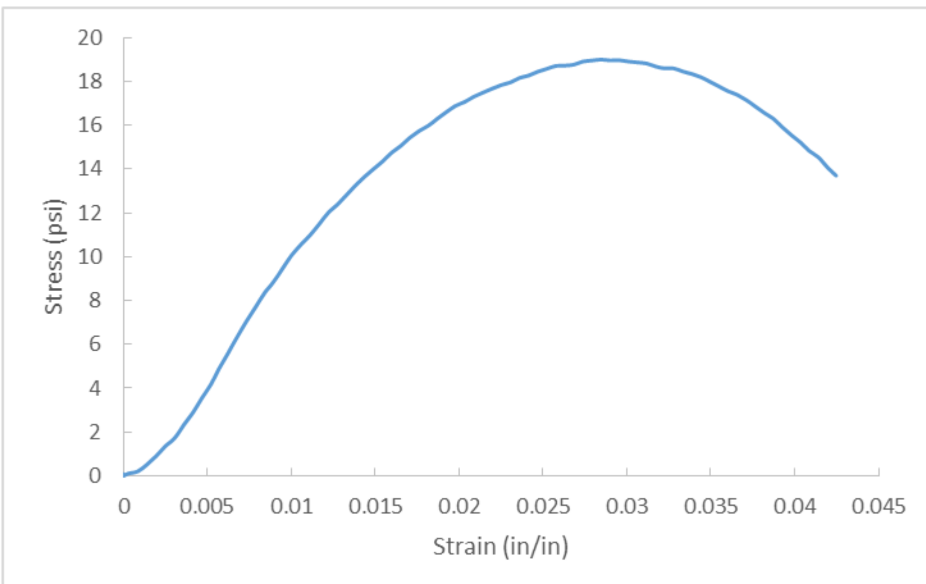

Figure A.50. D26-2, OC= 29.8%, CF= 400 pcf, T=14 Days.

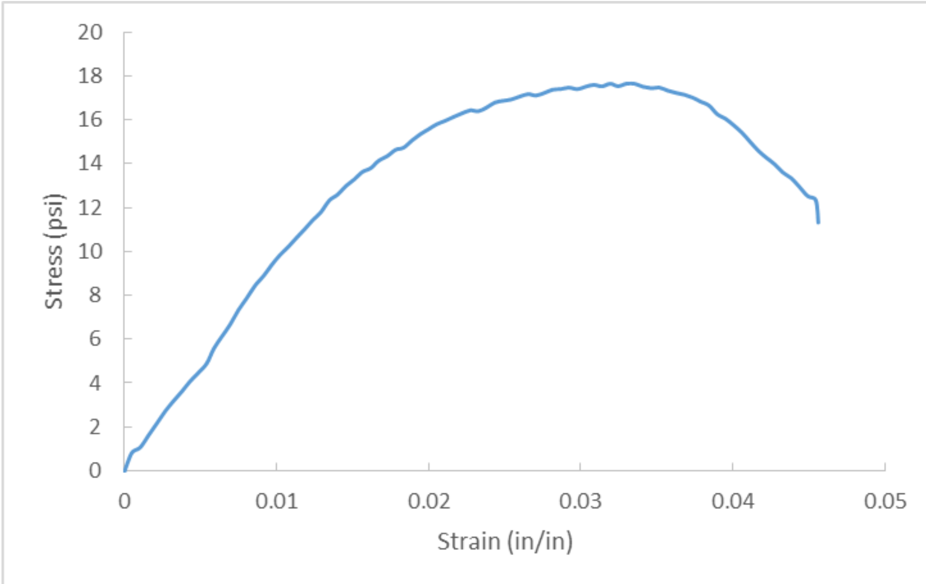

Figure A.51. D26-3, OC= 29.8%, CF= 400 pcf, T=14 Days.

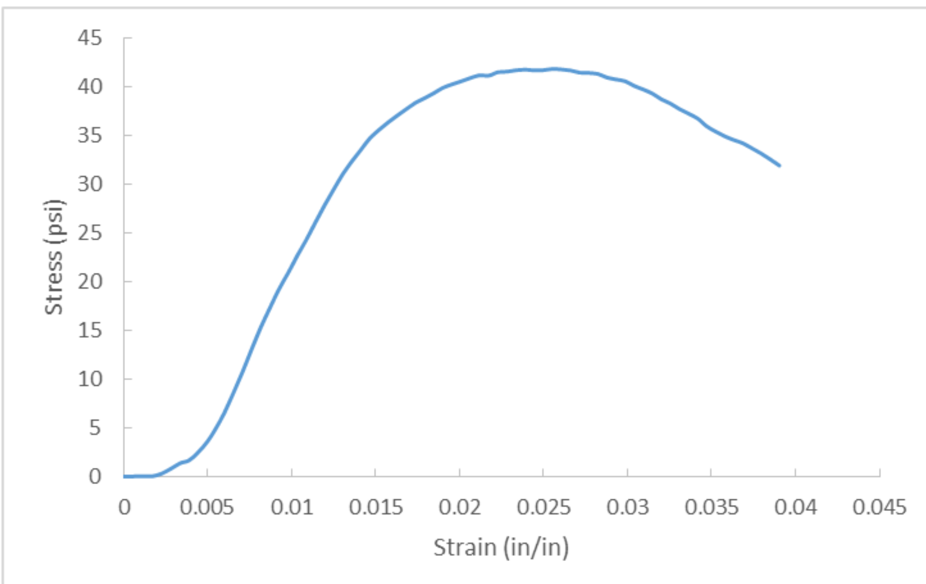

Figure A.52. D27-1, OC= 21.1%, CF= 400 pcf, T=14 Days.

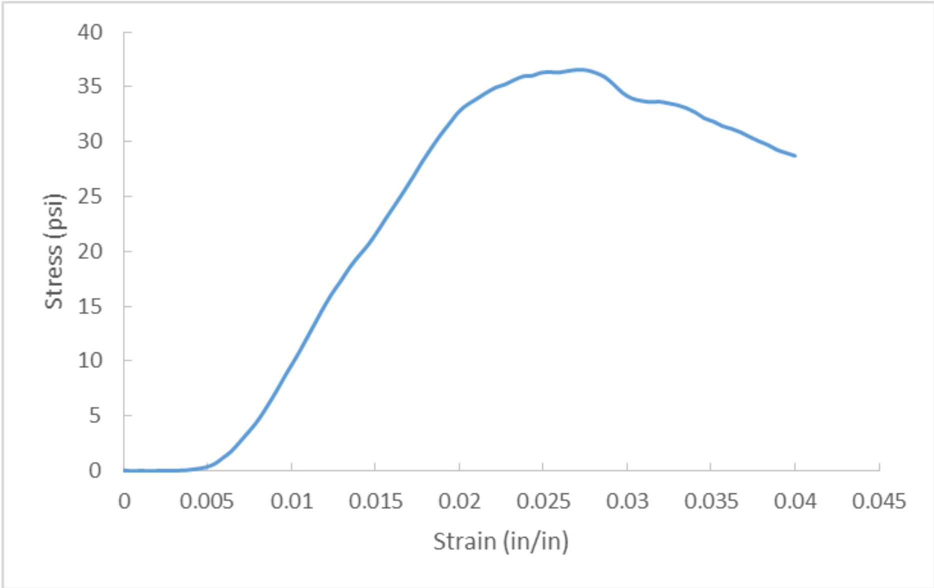

Figure A.53. D27-2, OC= 21.1%, CF= 400 pcf, T=14 Days.

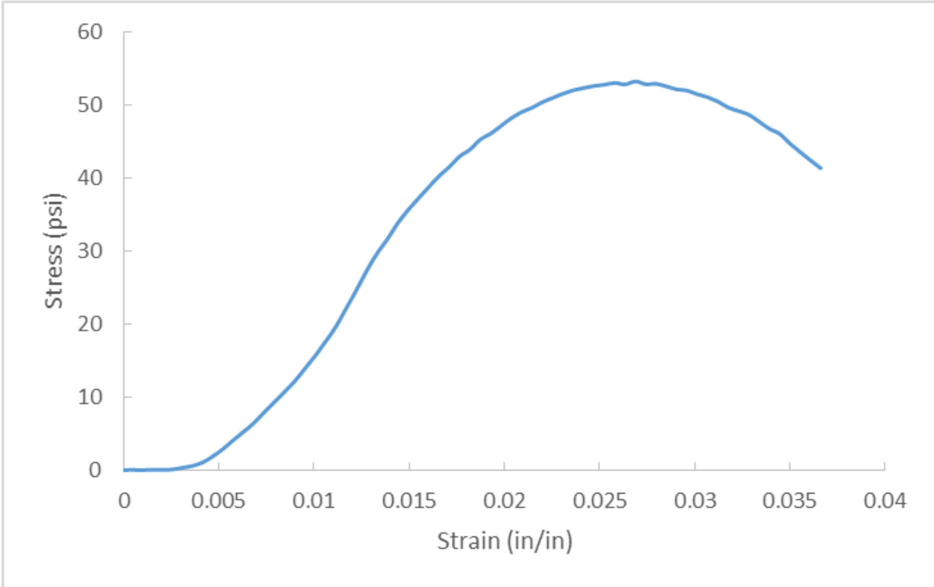

Figure A.54. D27-3, OC= 21.1%, CF= 400 pcf, T=14 Days.

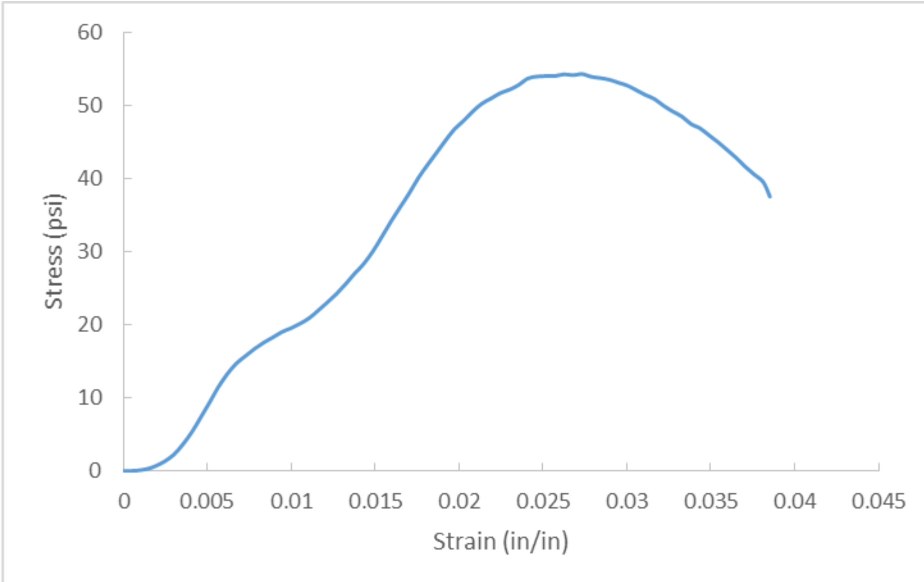

Figure A.55. D28-1, OC= 12.6%, CF= 400 pcf, T=14 Days.

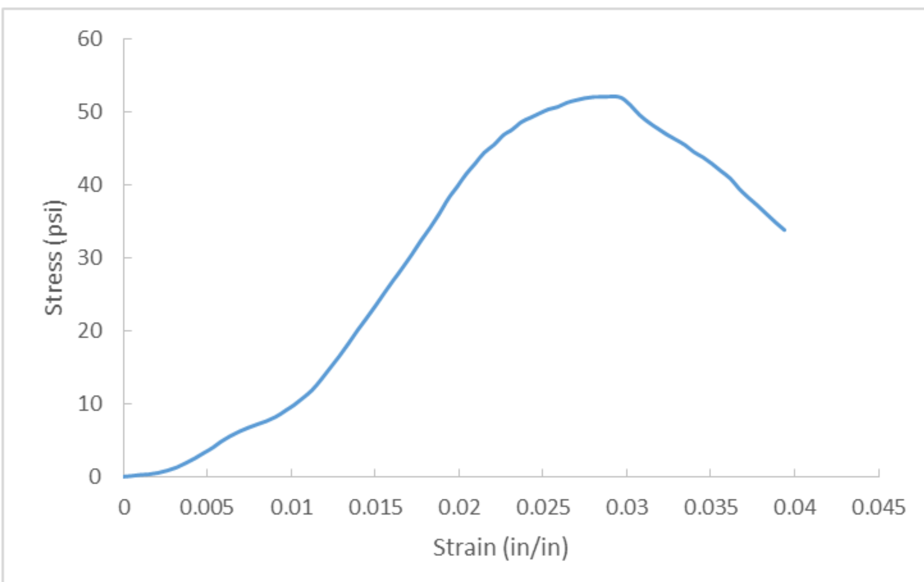

Figure A.56. D28-2, OC= 12.6%, CF= 400 pcf, T=14 Days.

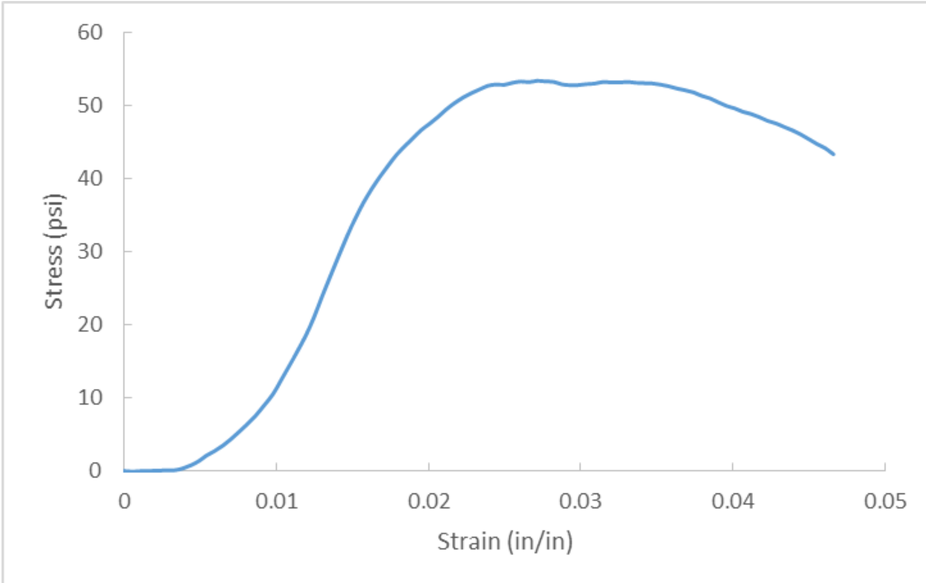

Figure A.57. D28-3, OC= 12.6%, CF= 400 pcf, T=14 Days.

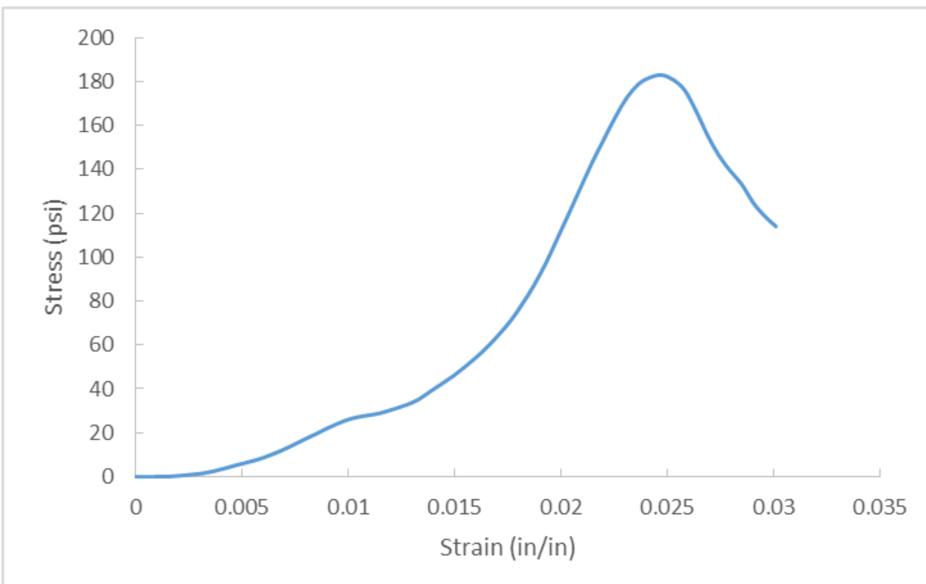

Figure A.58. D29-1, OC= 4.2%, CF= 400 pcf, T=14 Days.

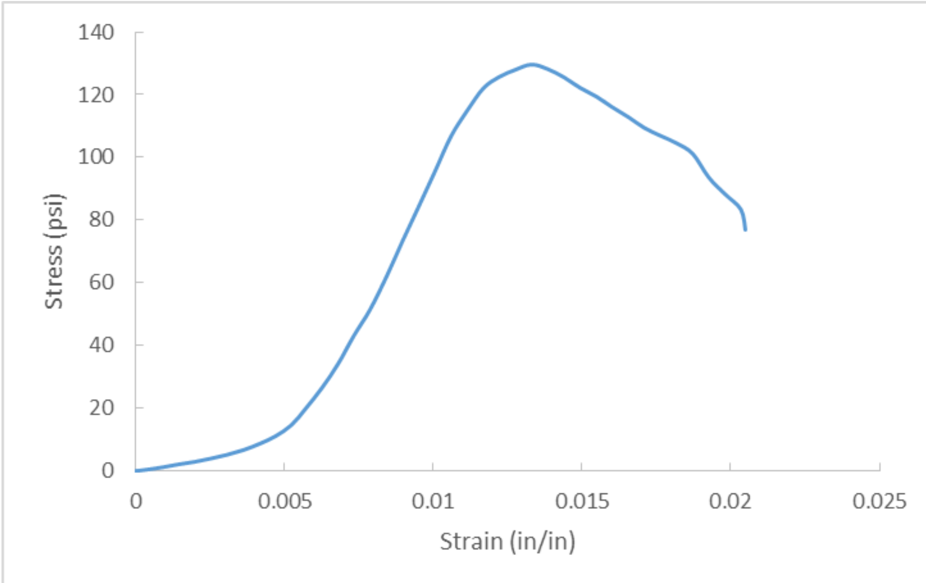

Figure A.59. D29-2, OC= 4.2%, CF= 400 pcf, T=14 Days.

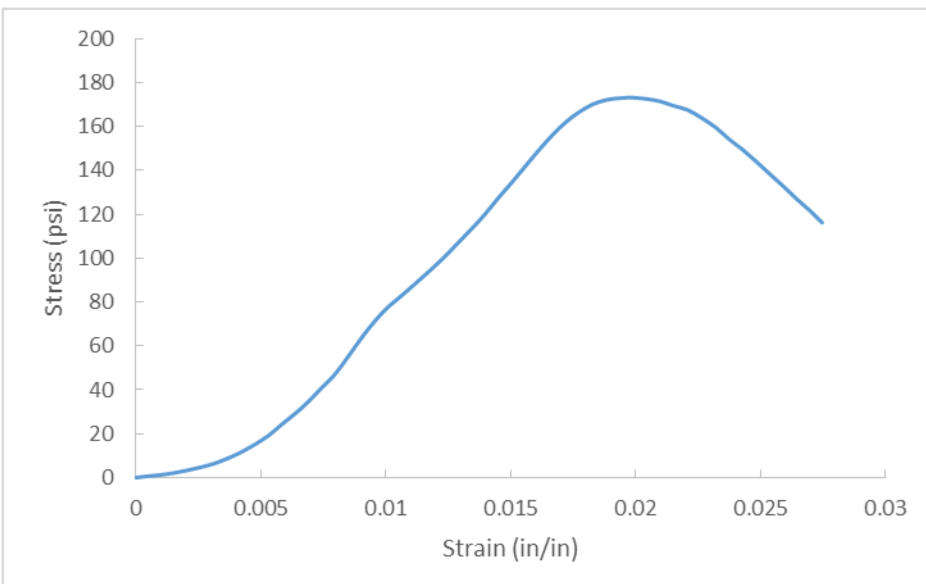

Figure A.60. D29-3, OC= 4.2%, CF= 400 pcf, T=14 Days.

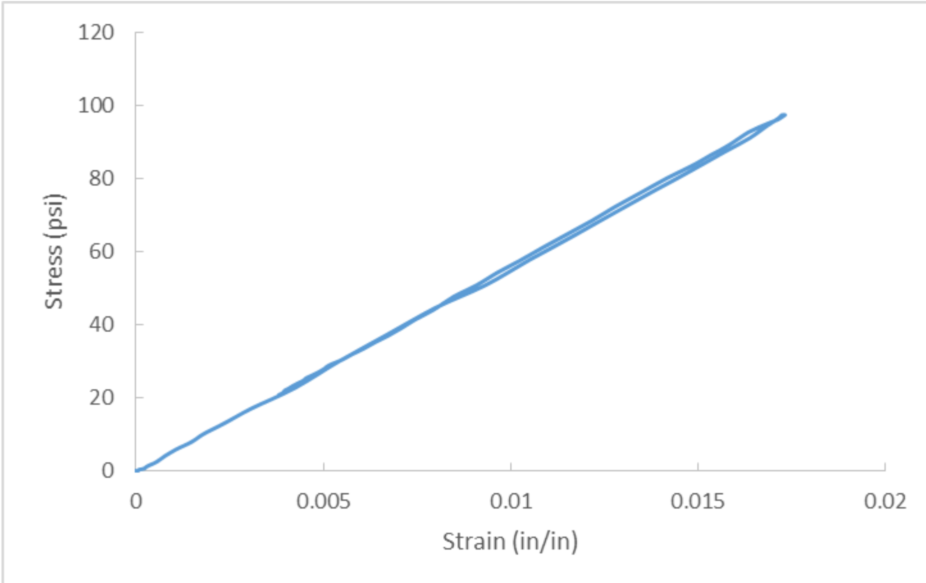

Figure A.61. D30-1, OC= 4.9%, CF= 400 pcf, T=14 Days.

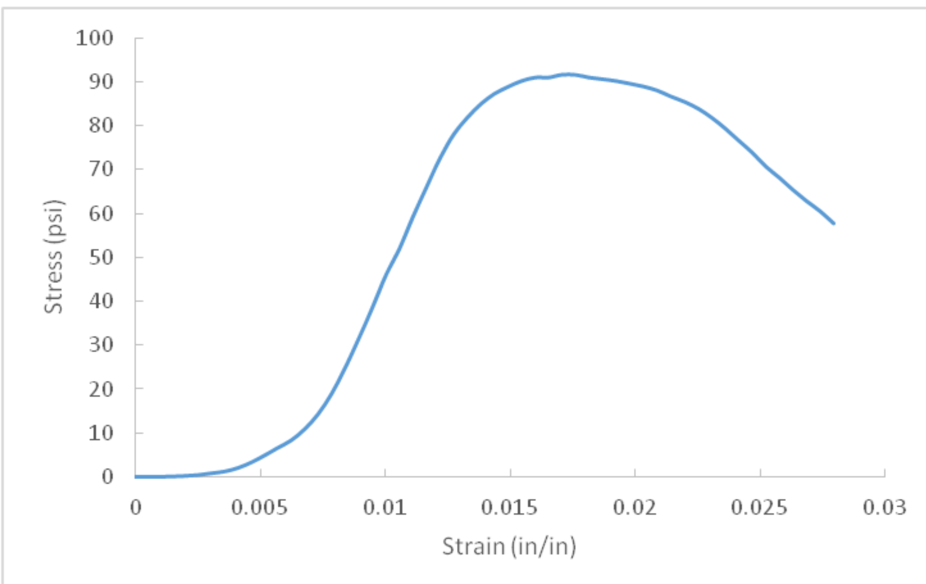

Figure A.62. D30-2, OC= 4.9%, CF= 400 pcf, T=14 Days.

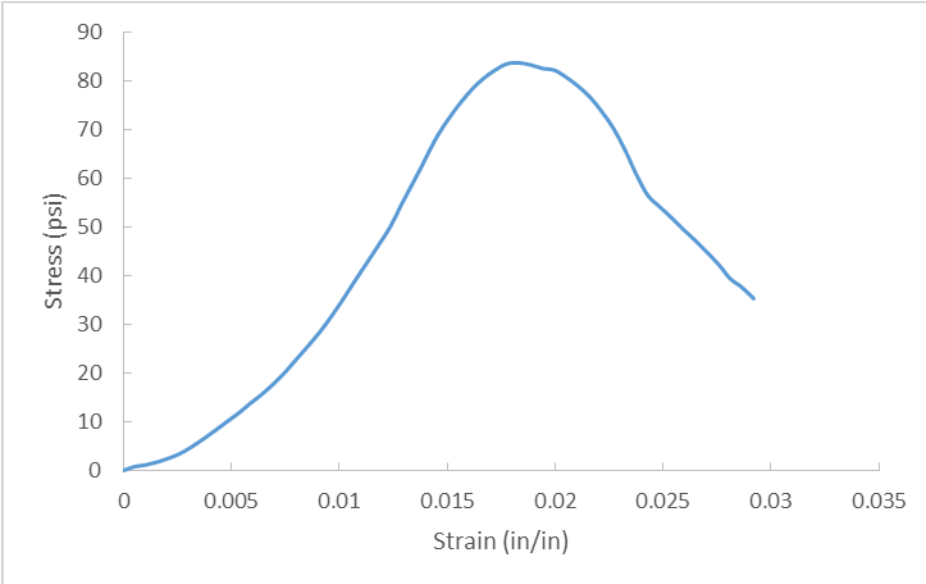

Figure A.63. D30-3, OC= 4.9%, CF= 400 pcf, T=14 Days.

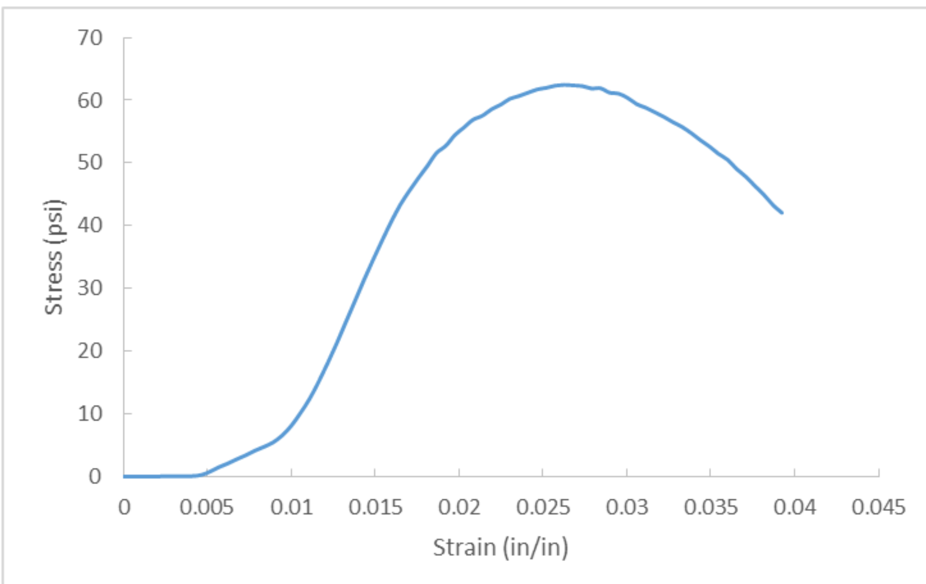

Figure A.64. D31-1, OC= 11.8%, CF= 400 pcf, T=14 Days.

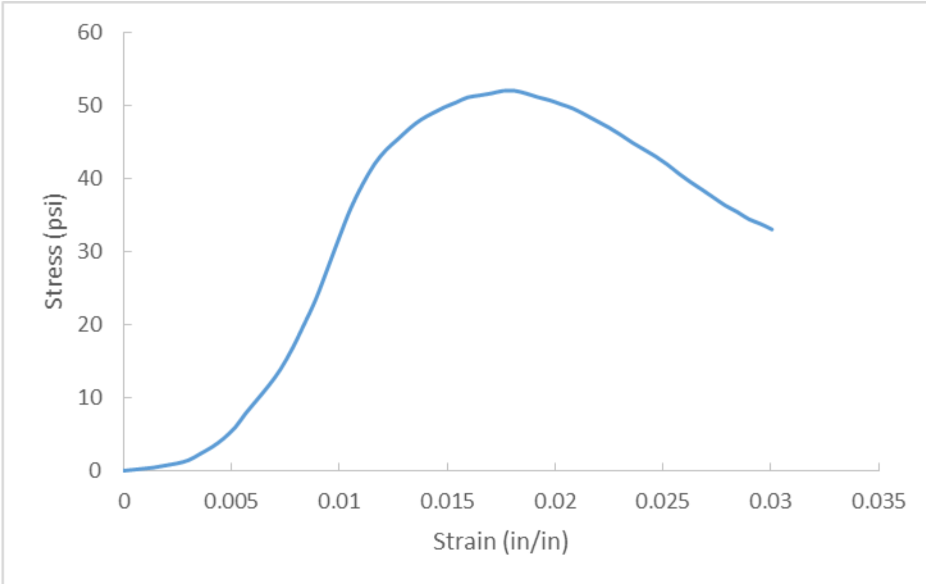

Figure A.65. D31-2, OC= 11.8%, CF= 400 pcf, T=14 Days.

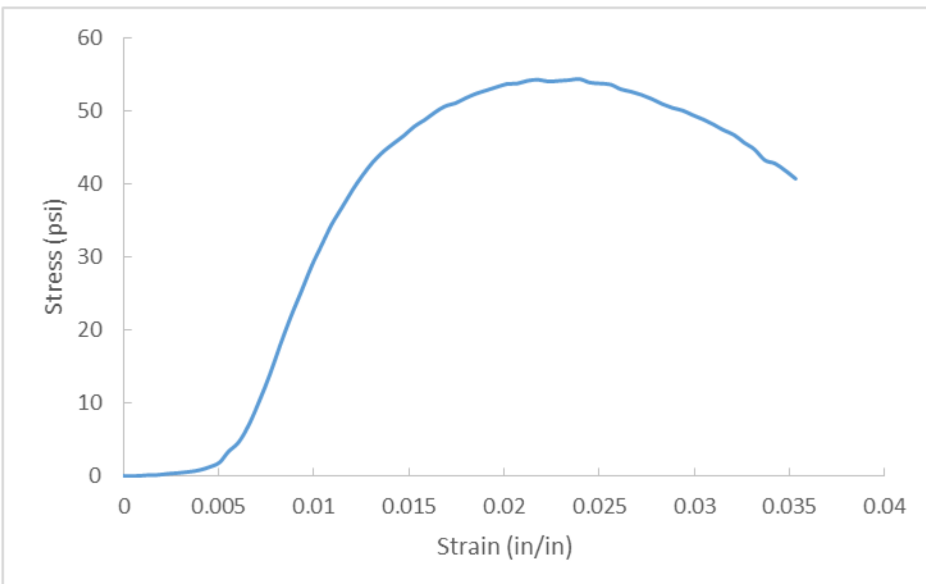

Figure A.66. D31-3, OC= 11.8%, CF= 400 pcf, T=14 Days.

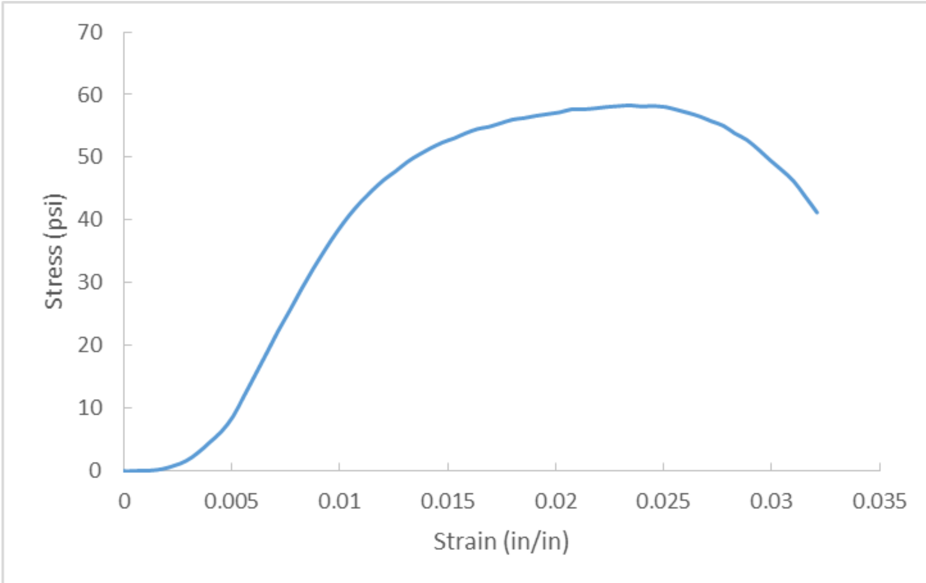

Figure A.67. D32-1, OC= 17.2%, CF= 400 pcf, T=14 Days.

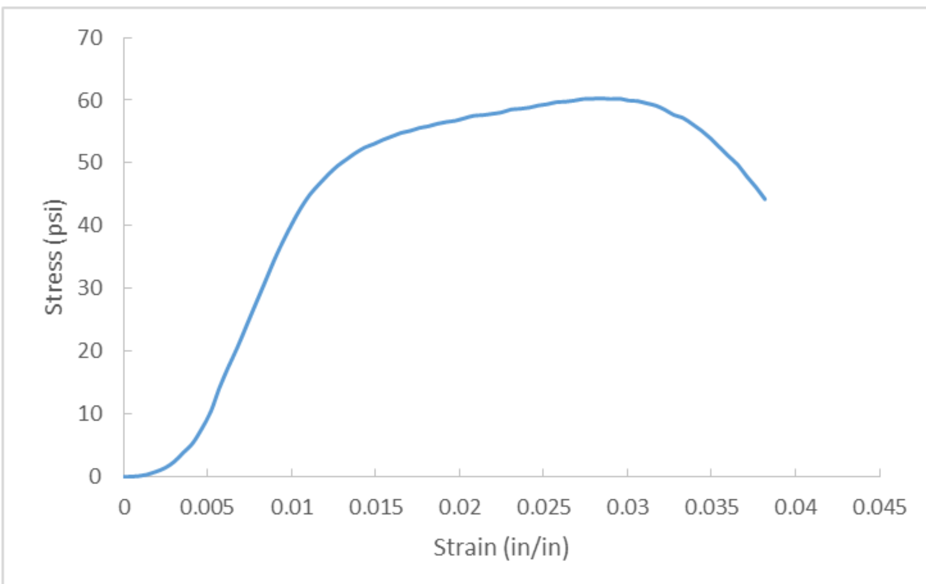

Figure A.68. D32-2, OC= 17.2%, CF= 400 pcf, T=14 Days.

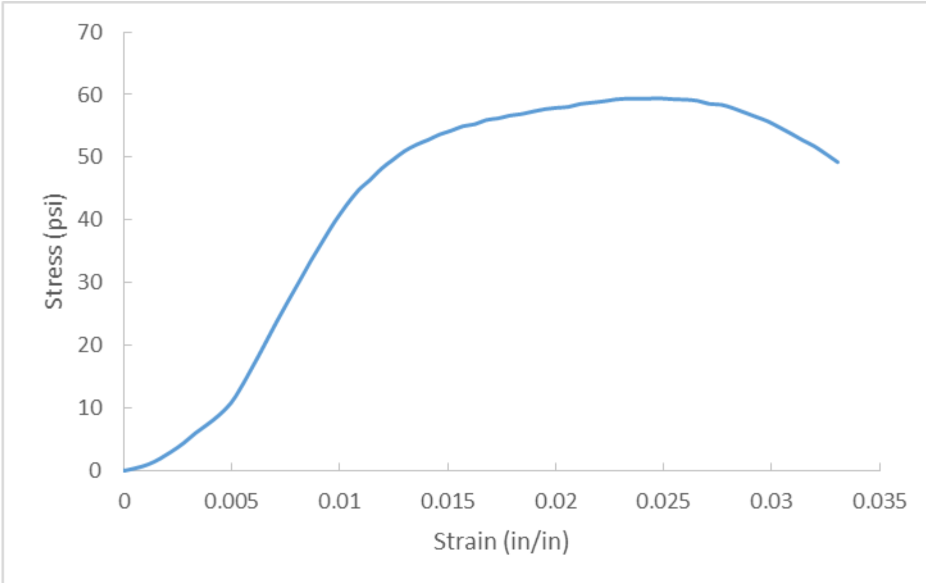

Figure A.69. D32-3, OC= 17.2%, CF= 400 pcf, T=14 Days.

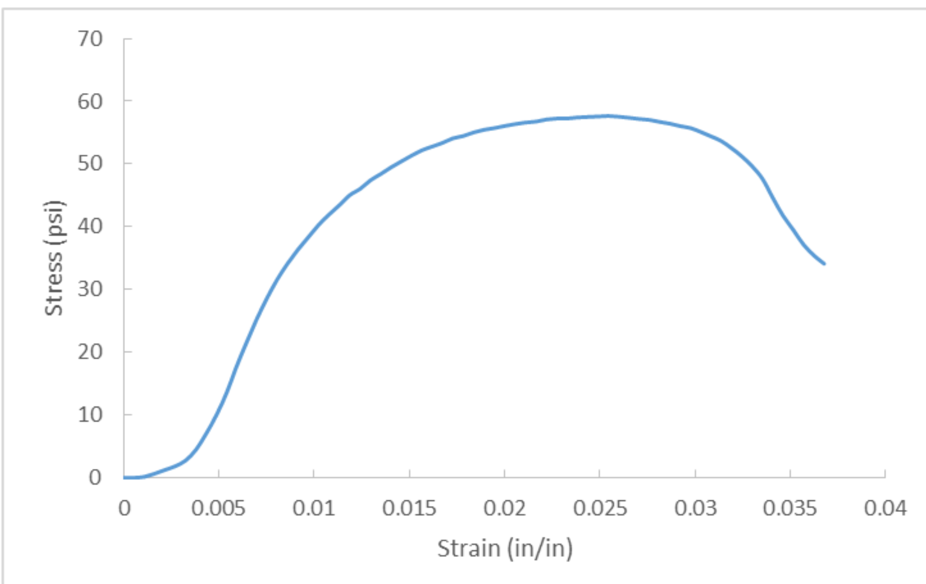

Figure A.70. D33-1, OC= 25.1%, CF= 400 pcf, T=14 Days.

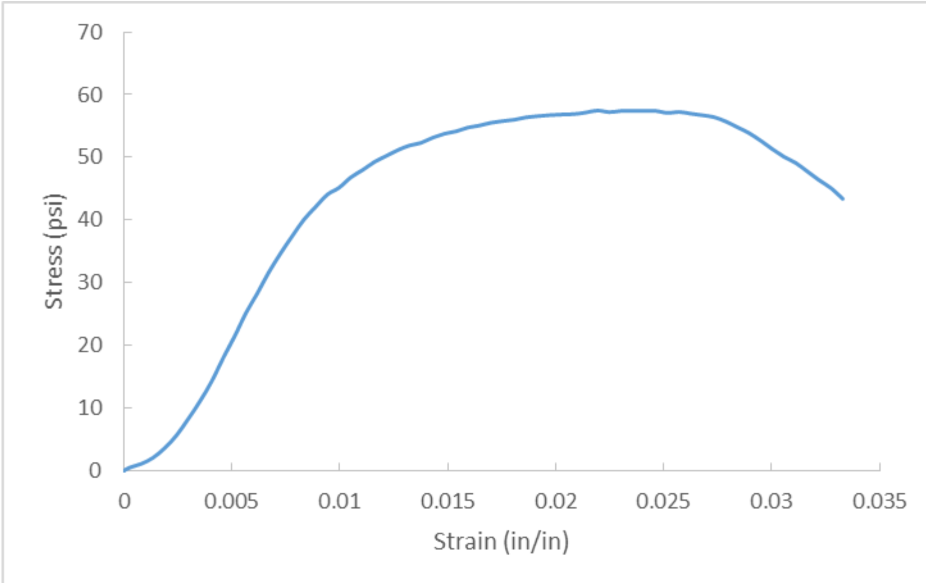

Figure A.71. D33-2, OC= 25.1%, CF= 400 pcf, T=14 Days.

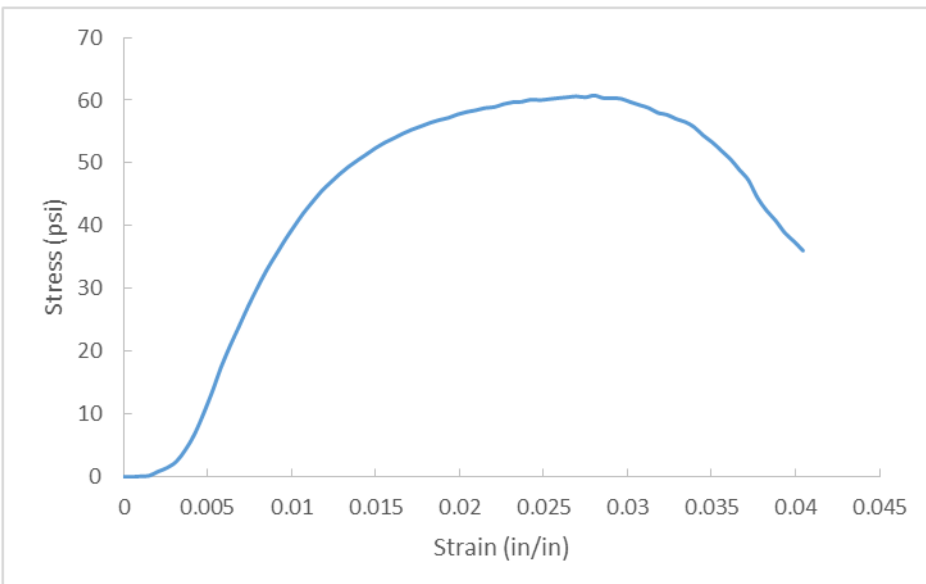

Figure A.72. D33-3, OC= 25.1%, CF= 400 pcf, T=14 Days.

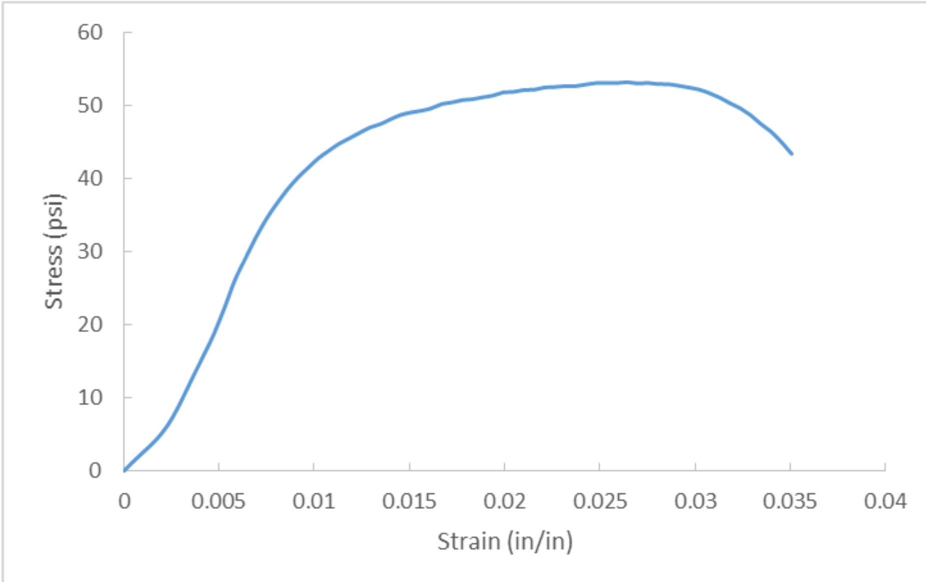

Figure A.73. D34-1, OC= 40.9%, CF= 400 pcf, T=14 Days.

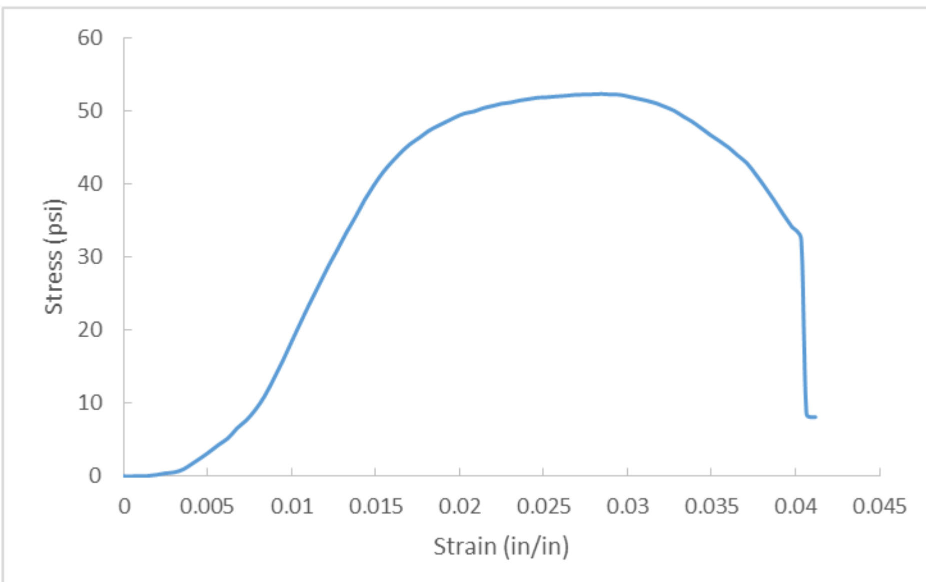

Figure A.74. D34-4, OC= 40.9%, CF= 400 pcf, T=14 Days.

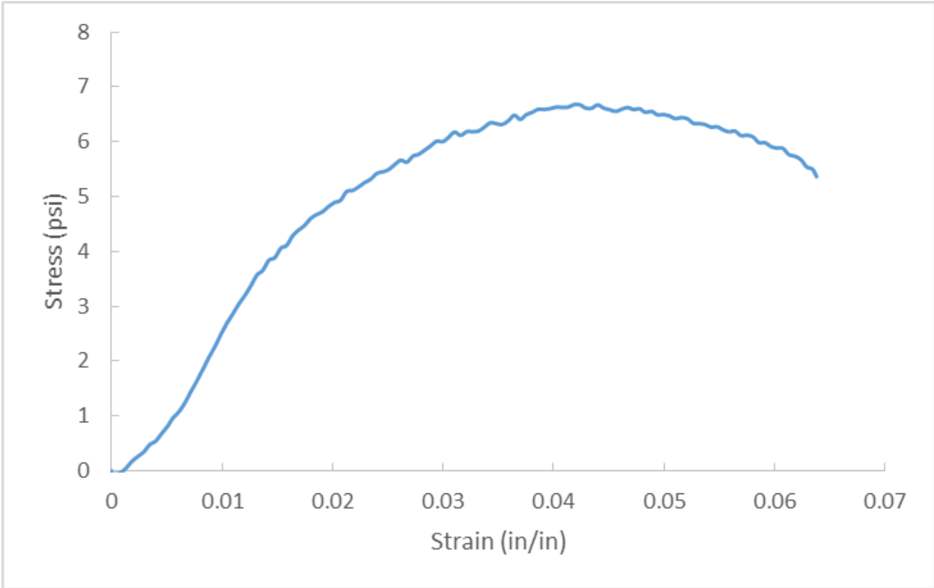

Figure A.75. D35-4, OC= 39.6%, CF= 200 pcf, T=14 Days.

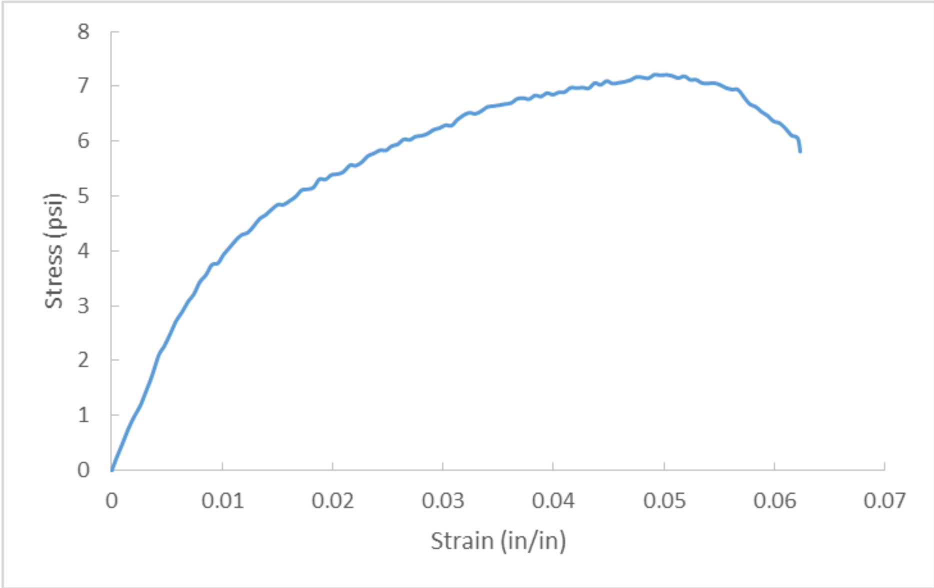

Figure A.76. D35-5, OC= 39.6%, CF= 200 pcf, T=14 Days.

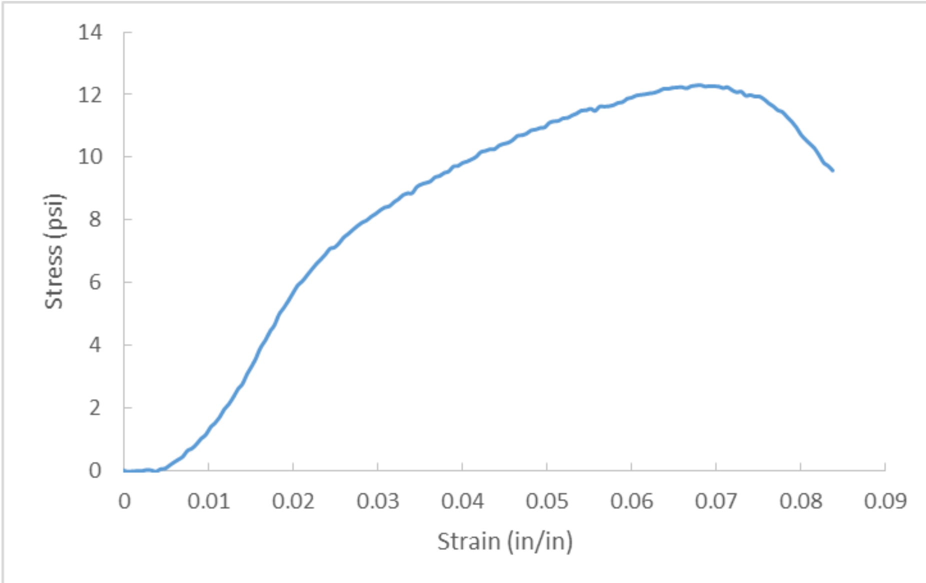

Figure A.77. D36-2, OC= 25.6%, CF= 200 pcf, T=14 Days.

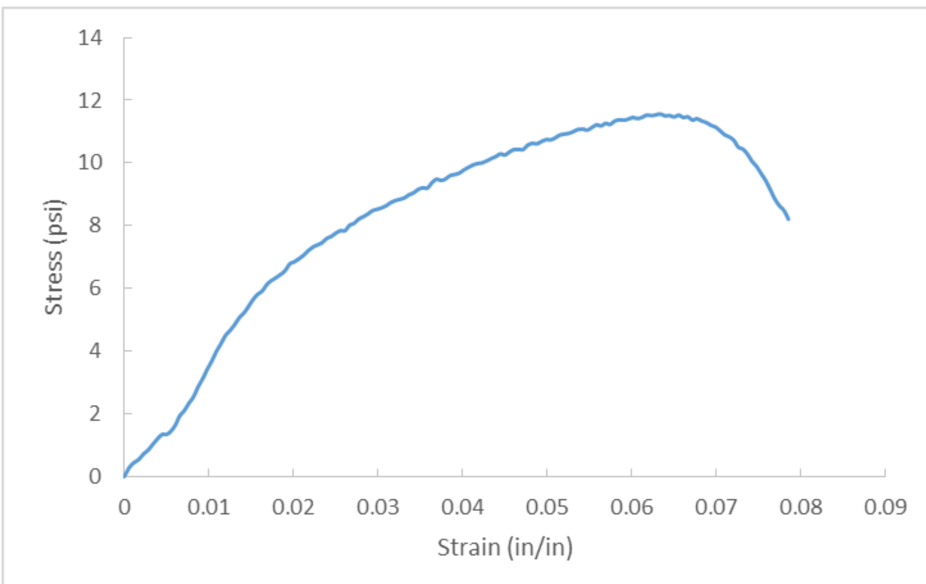

Figure A.78. D36-3, OC= 25.6%, CF= 200 pcf, T=14 Days.

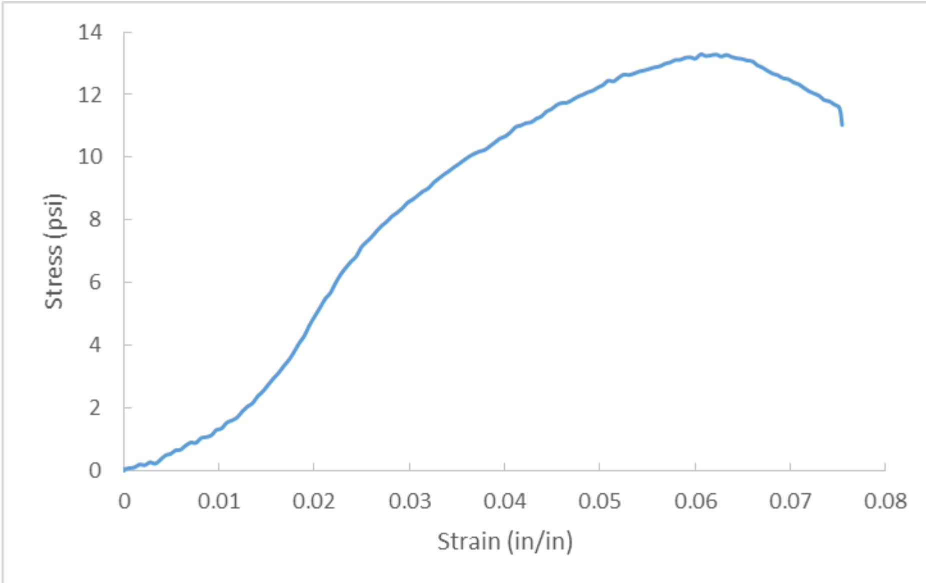

Figure A.79. D37-1, OC= 18.9%, CF= 200 pcf, T=14 Days.

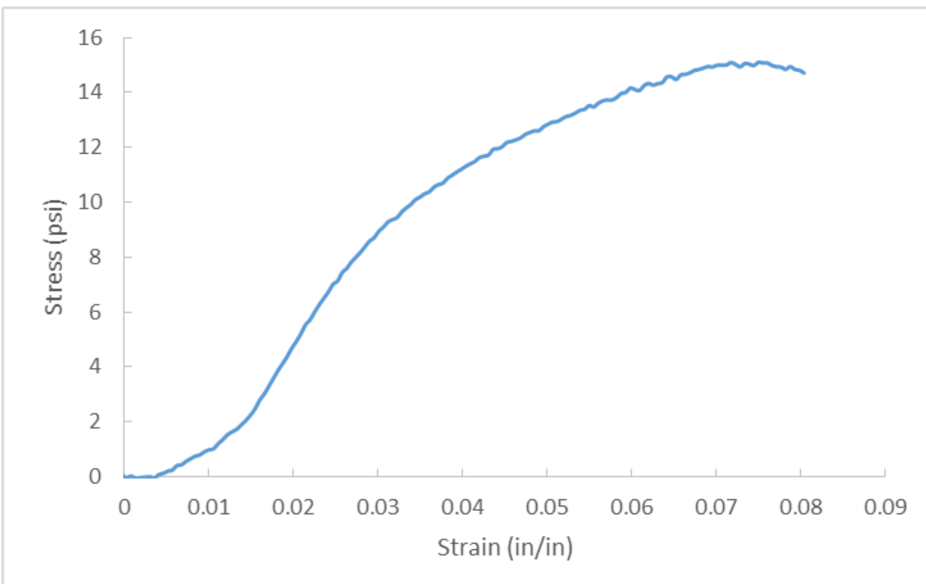

Figure A.80. D37-2, OC= 18.9%, CF= 200 pcf, T=14 Days.

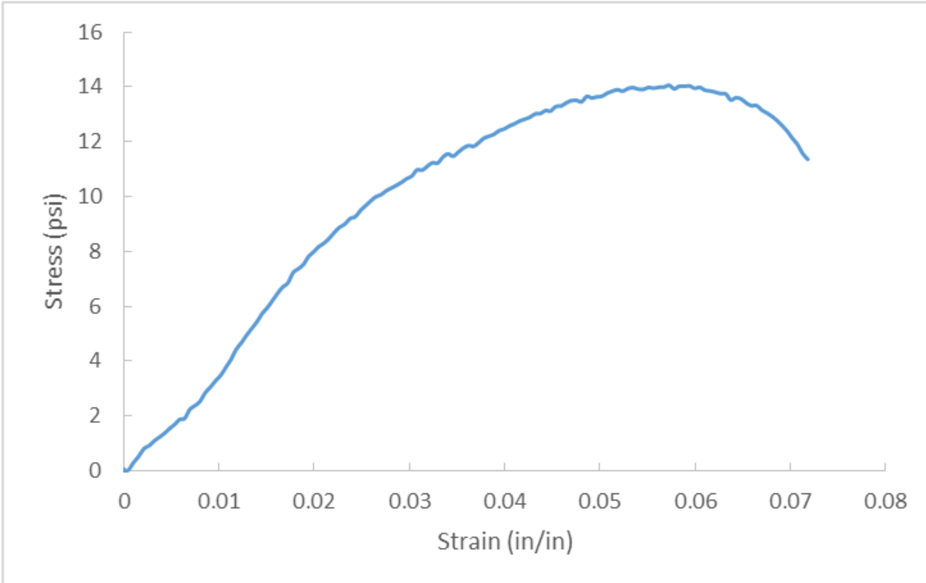

Figure A.81. D37-3, OC= 18.9%, CF= 200 pcf, T=14 Days.

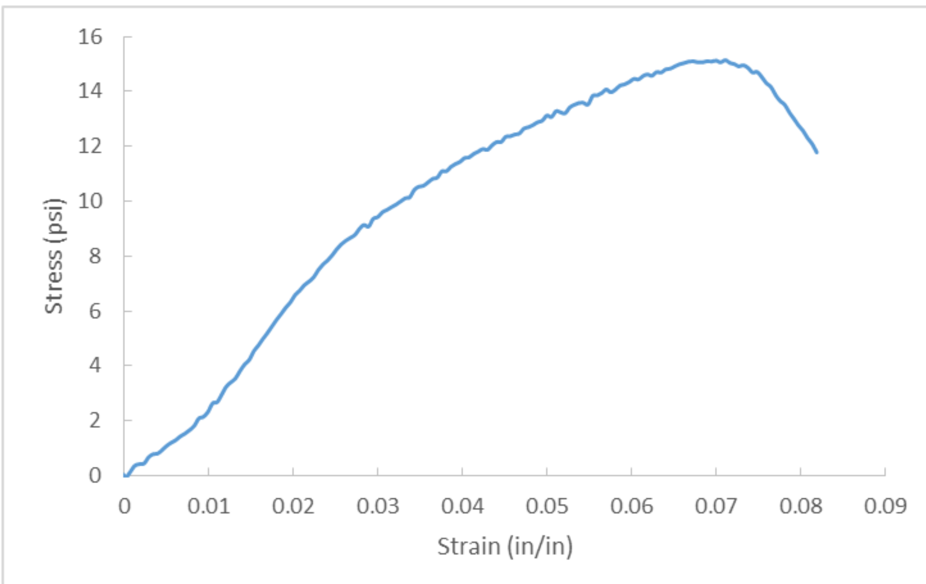

Figure A.82. D38-1, OC= 13.9%, CF= 200 pcf, T=14 Days.

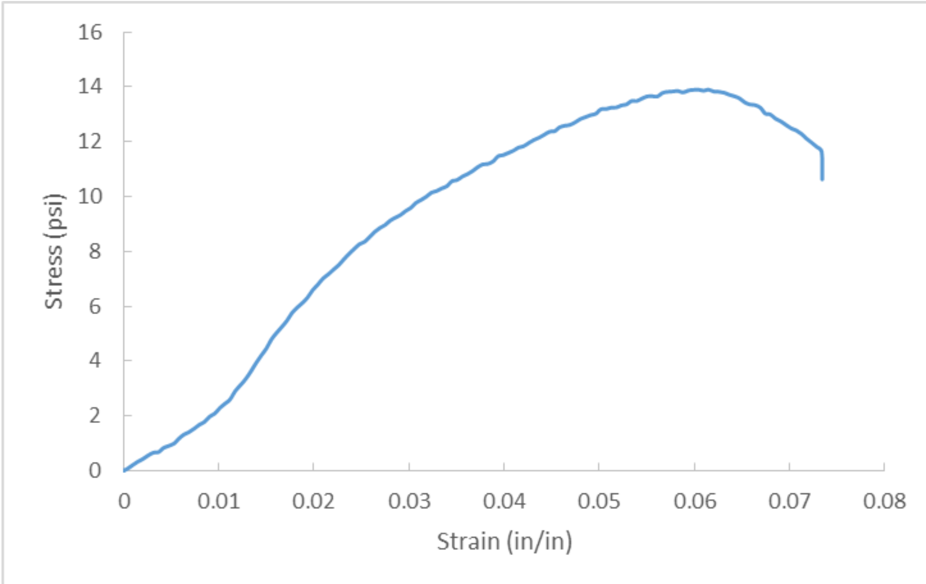

Figure A.83. D38-2, OC= 13.9%, CF= 200 pcf, T=14 Days.

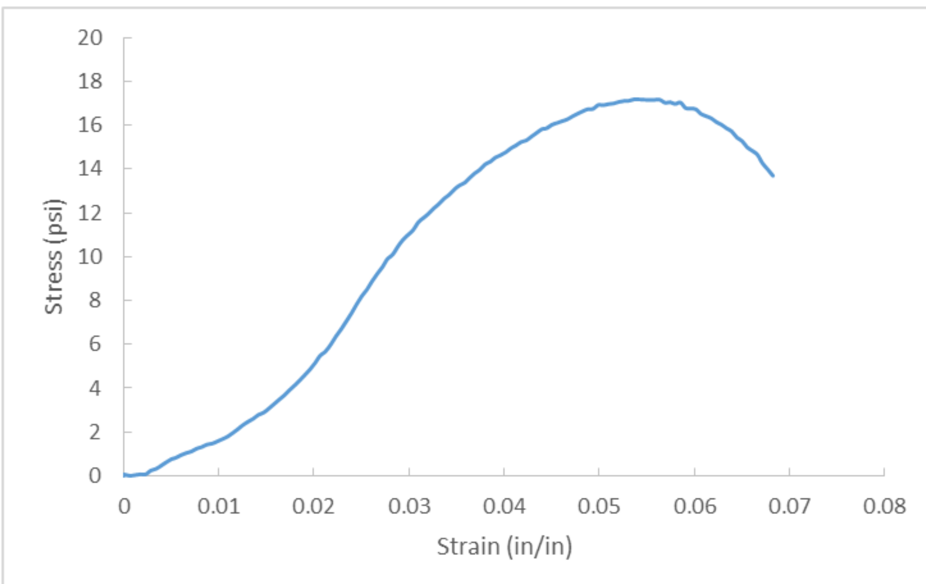

Figure A.84. D39-1, OC= 3.9%, CF= 200 pcf, T=14 Days.

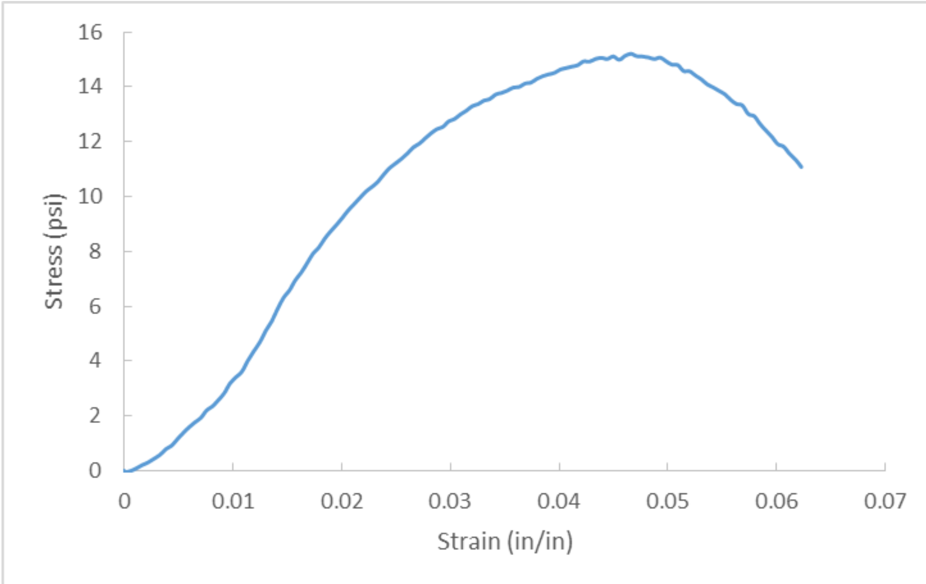

Figure A.85. D39-2, OC= 3.9%, CF= 200 pcf, T=14 Days.

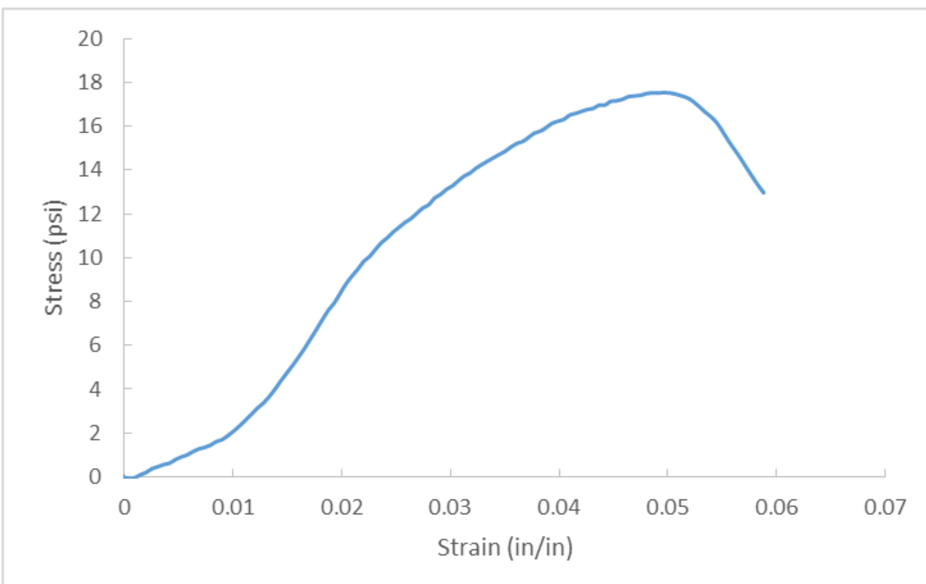

Figure A.86. D39-3, OC= 3.9%, CF= 200 pcf, T=14 Days.

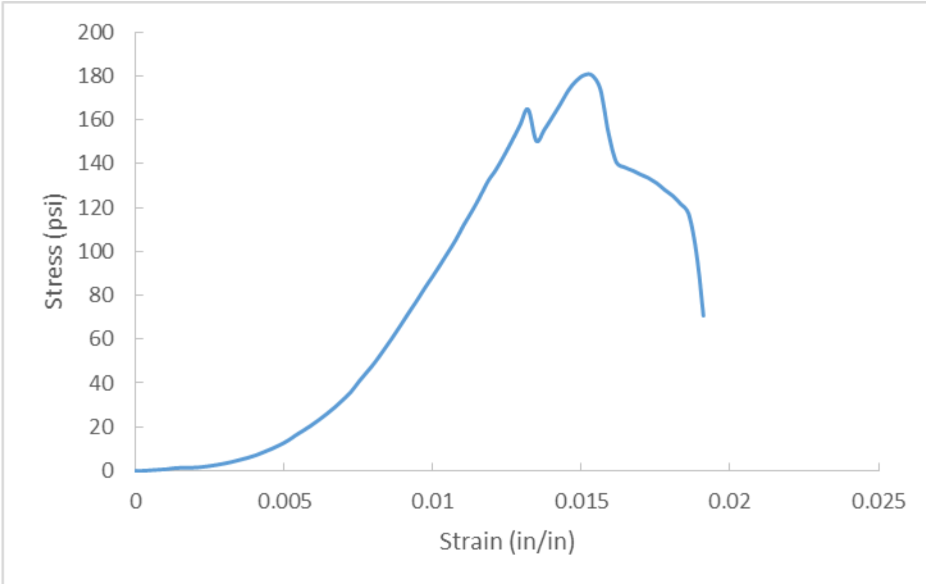

Figure A.87. D45-1, OC= 0.0%, CF= 400 pcf, T=14 Days.

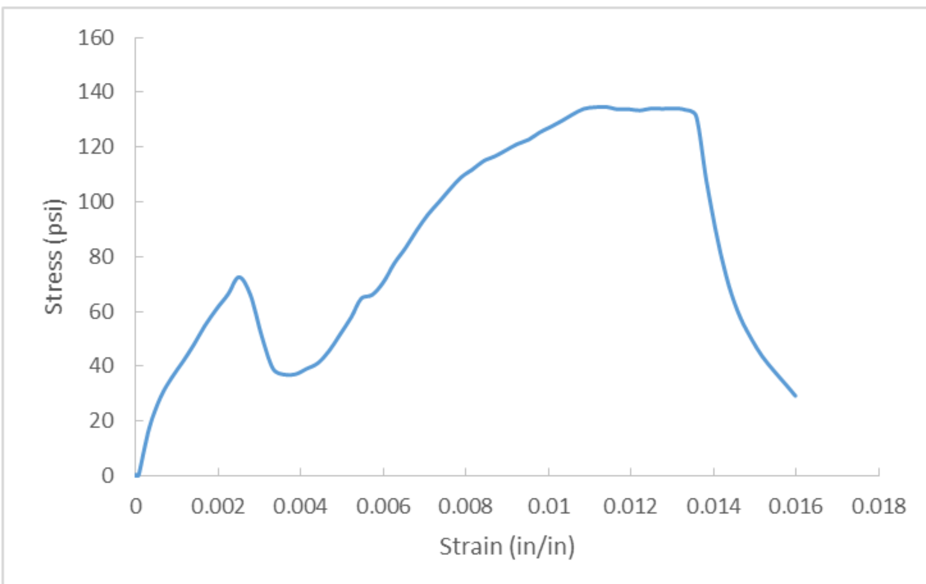

Figure A.88. D45-2, OC= 0.0%, CF= 400 pcf, T=14 Days.

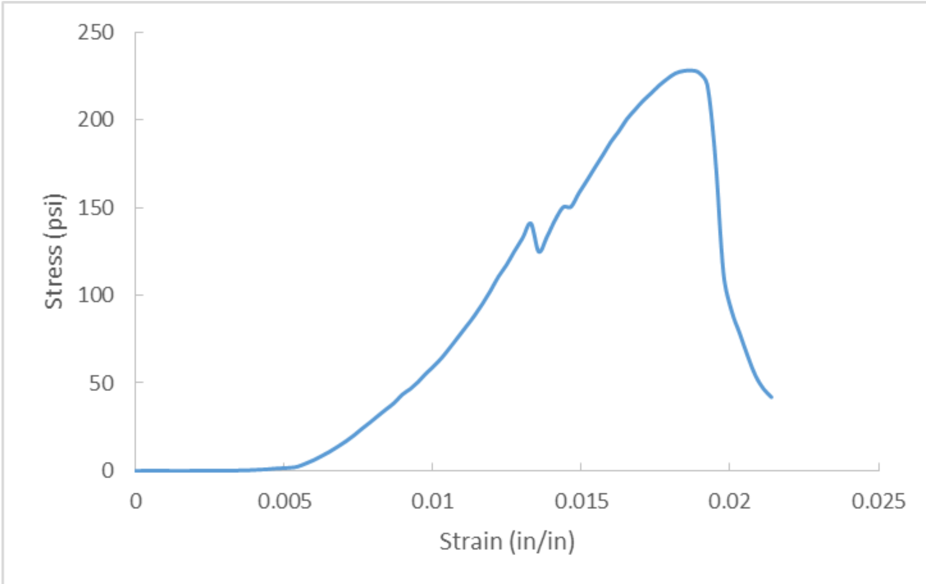

Figure A.89. D45-3, OC= 0.0%, CF= 400 pcf, T=14 Days.

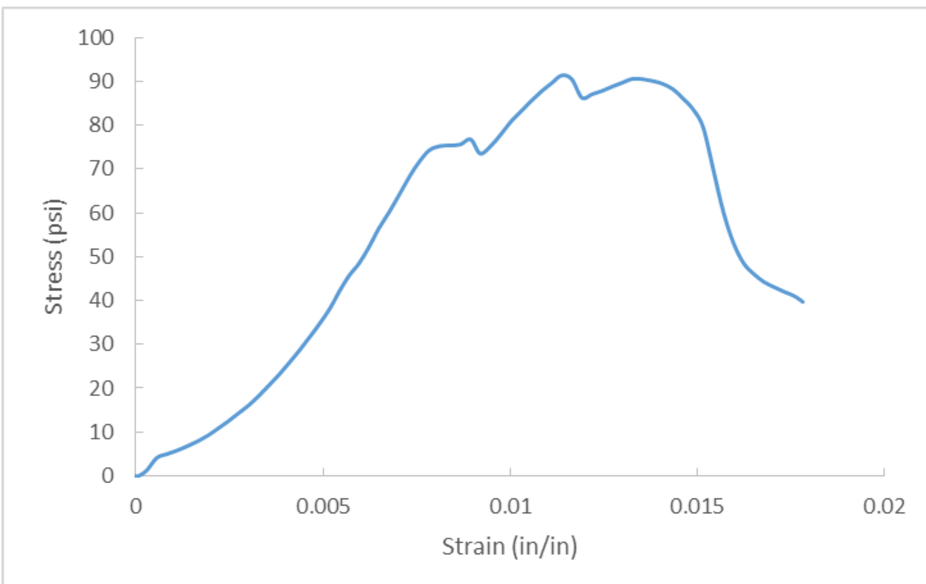

Figure A.90. D46-1, OC= 0.0%, CF= 300 pcf, T=14 Days.

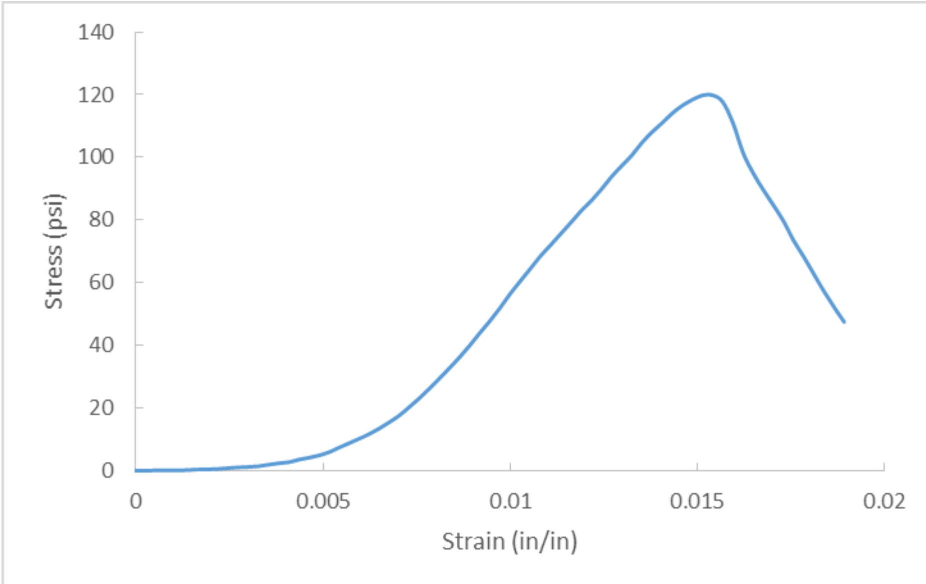

Figure A.91. D46-2, OC= 0.0%, CF= 300 pcf, T=14 Days.

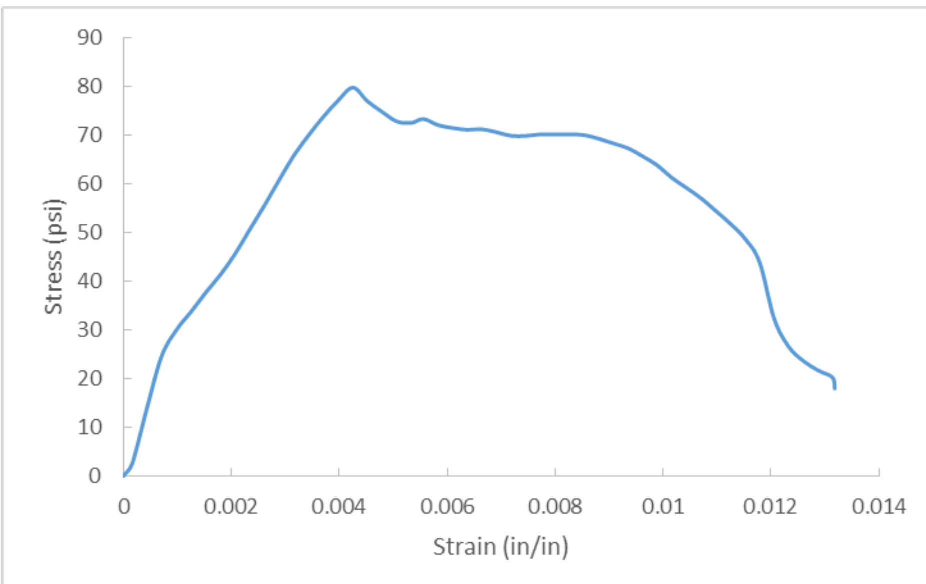

Figure A.92. D46-3, OC= 0.0%, CF= 300 pcf, T=14 Days.

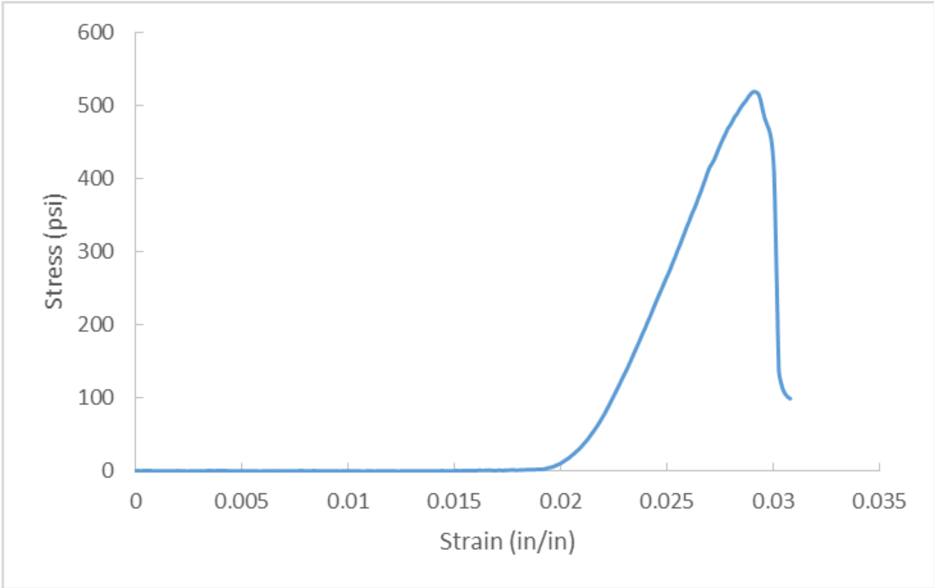

Figure A.93. D47-2, OC= 0.0%, CF= 400 pcf, T=14 Days.

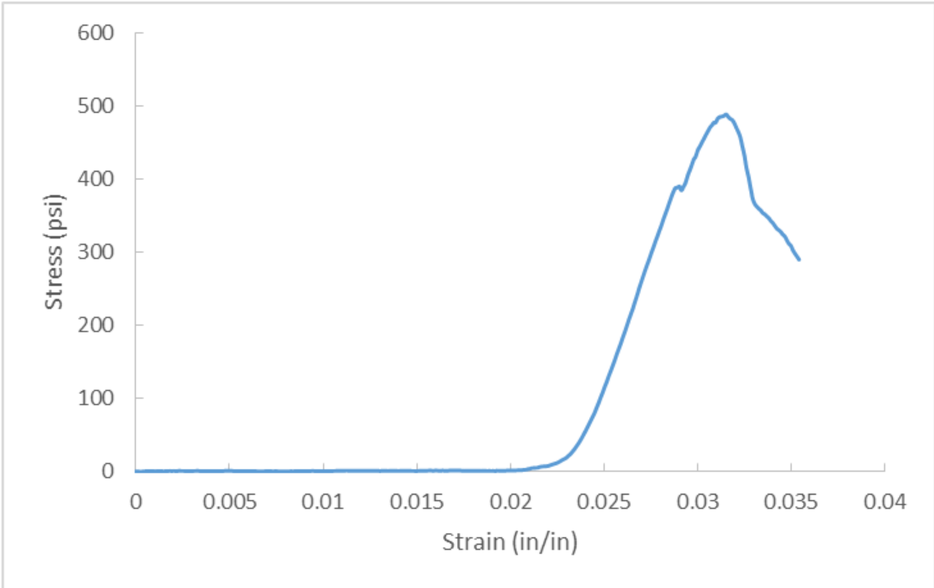

Figure A.94. D47-3, OC= 0.0%, CF= 400 pcf, T=14 Days.

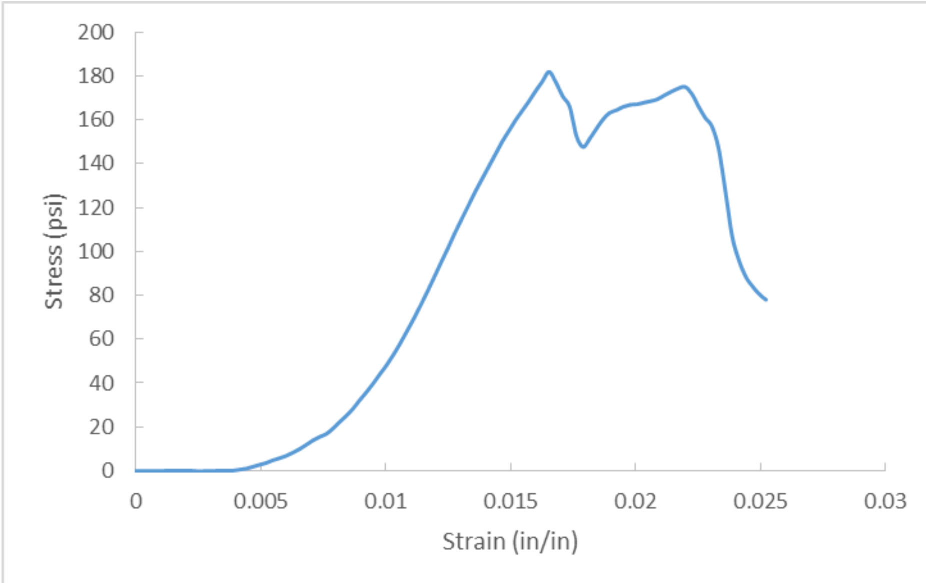

Figure A.95. D48-1, OC= 0.0%, CF= 300 pcf, T=14 Days.

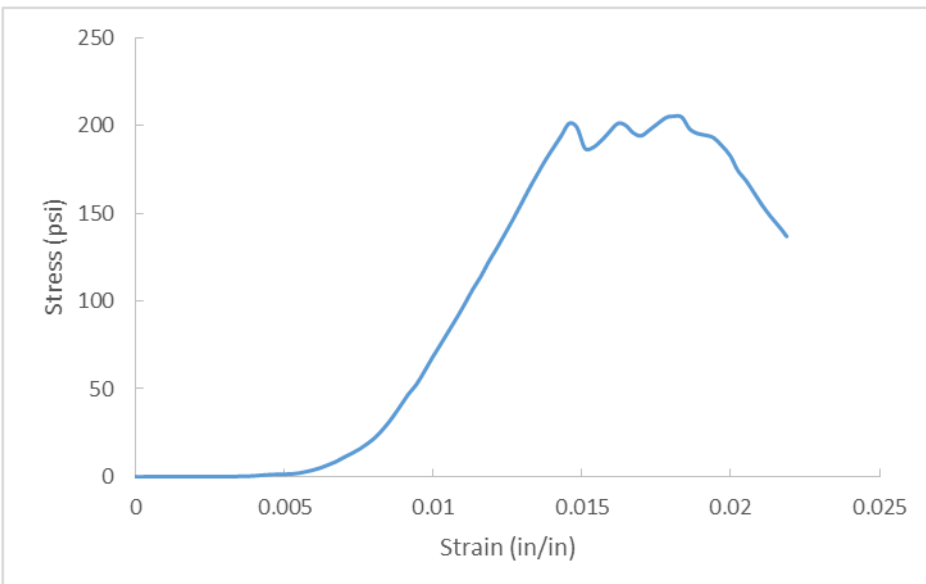

Figure A.96. D48-2, OC= 0.0%, CF= 300 pcf, T=14 Days.

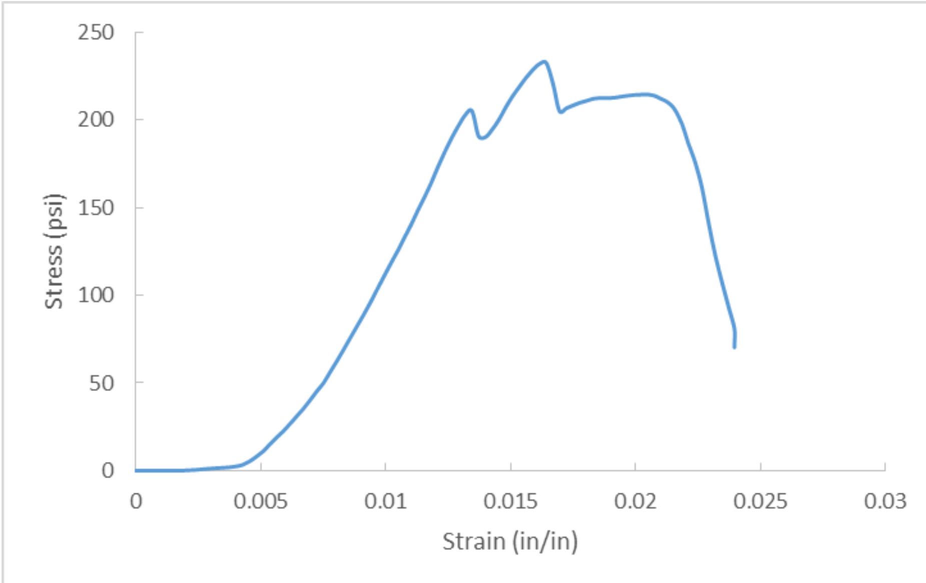

Figure A.97. D48-3, OC= 0.0%, CF= 300 pcf, T=14 Days.

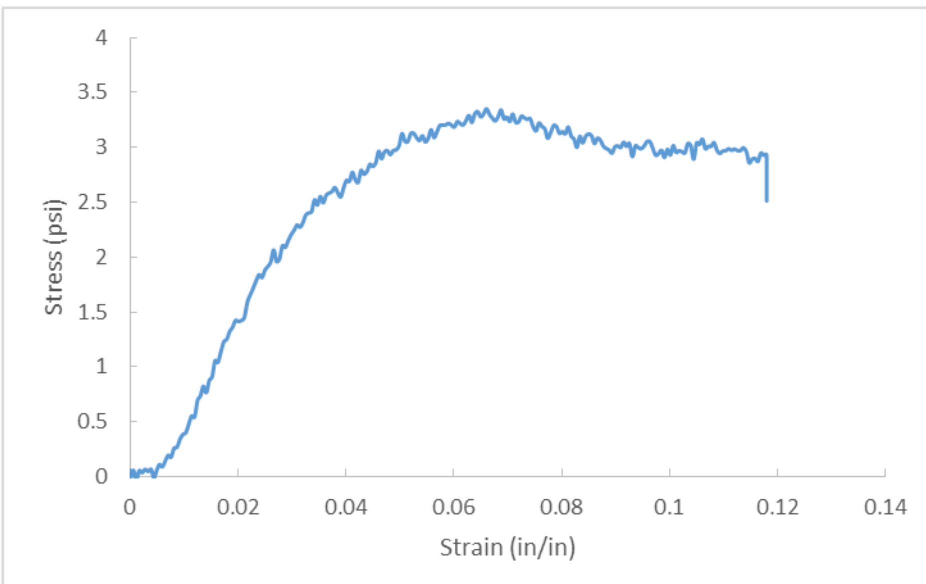

Figure A.98. W49-1, OC= 43.8%, CF= 300 pcf, T=14 Days.

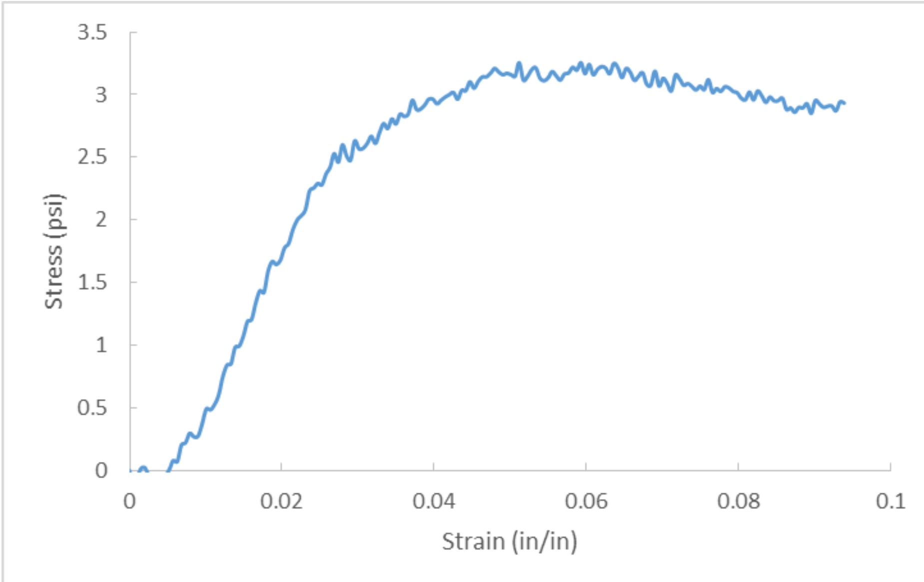

Figure A.99. W49-3, OC= 43.8%, CF= 300 pcf, T=14 Days.

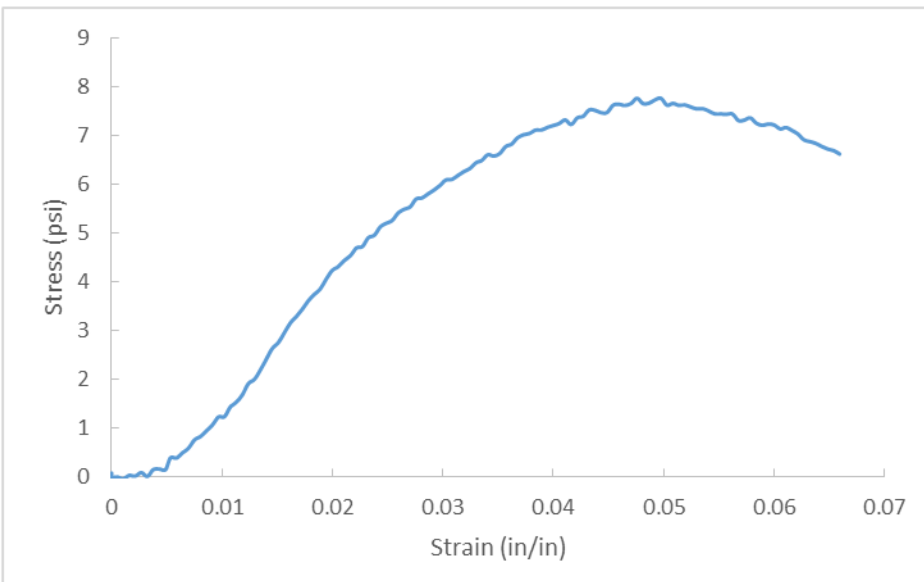

Figure A.100. W50-1, OC= 43.8%, CF= 400 pcf, T=14 Days.

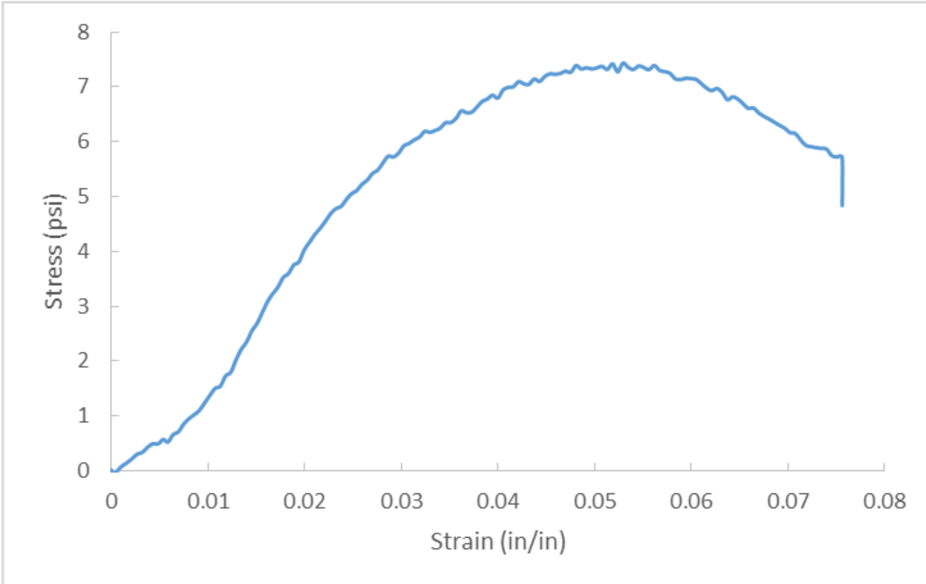

Figure A.101. W50-2, OC= 43.8%, CF= 400 pcf, T=14 Days.

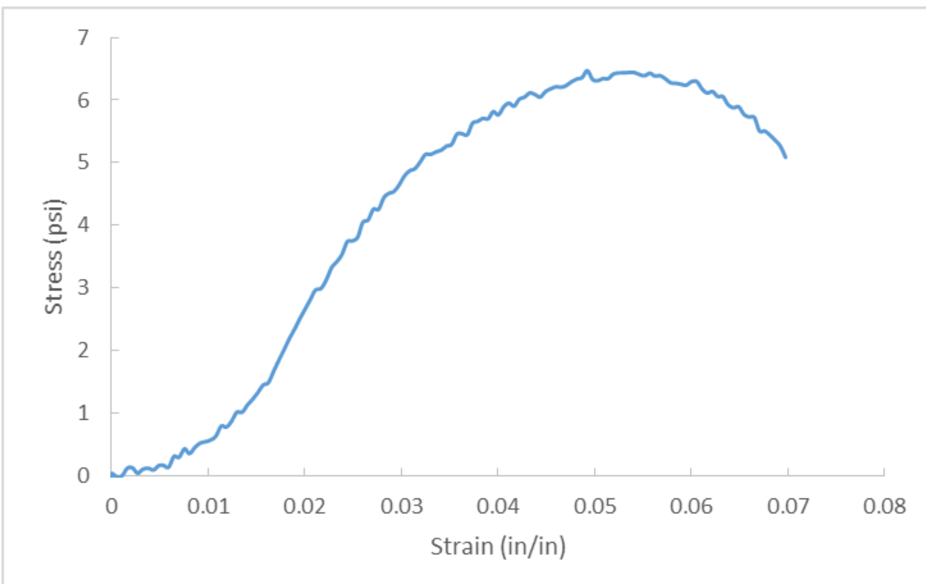

Figure A.102. W50-3, OC= 43.8%, CF= 400 pcf, T=14 Days.

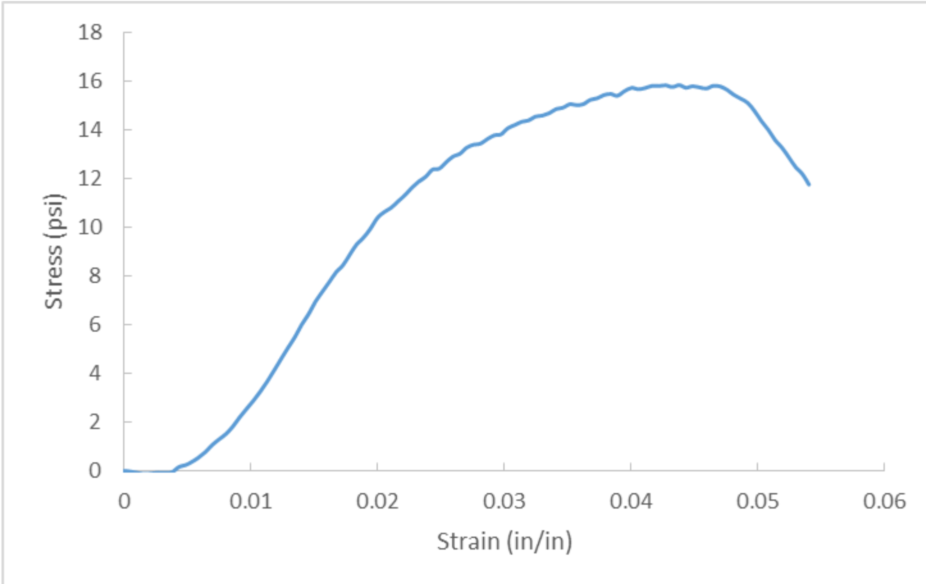

Figure A.103. W51-1, OC= 43.8%, CF= 500 pcf, T=14 Days.

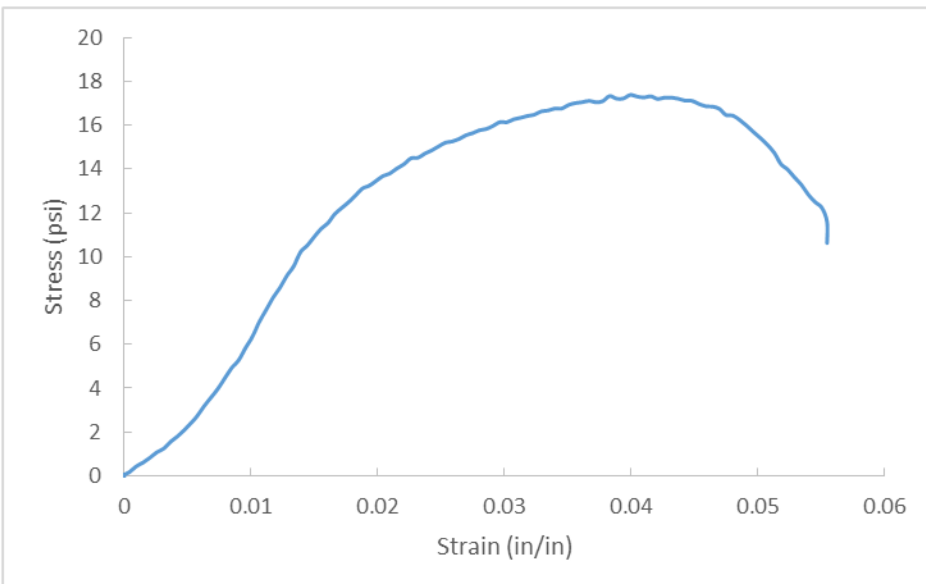

Figure A.104. W51-2, OC= 43.8%, CF= 500 pcf, T=14 Days.

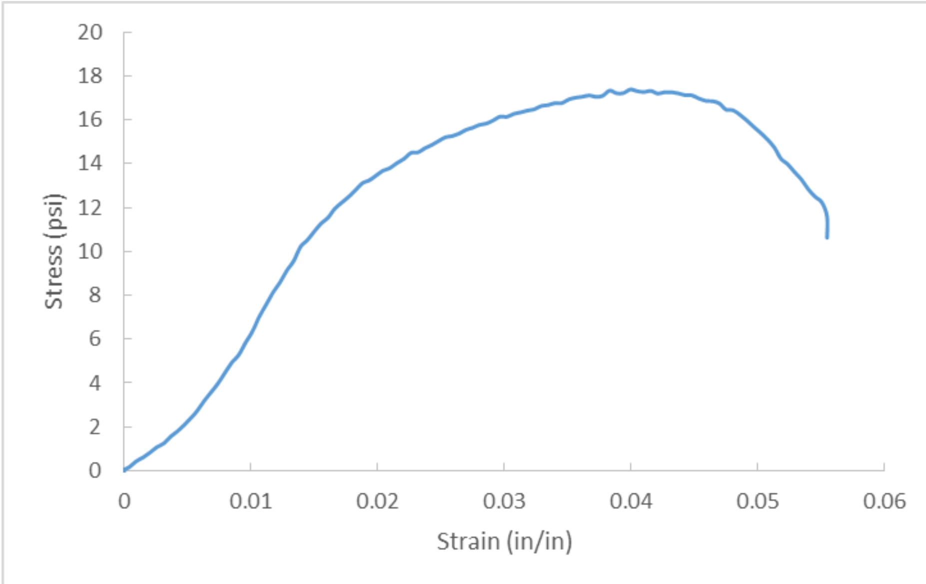

Figure A.105. W51-3, OC= 43.8%, CF= 500 pcf, T=14 Days.

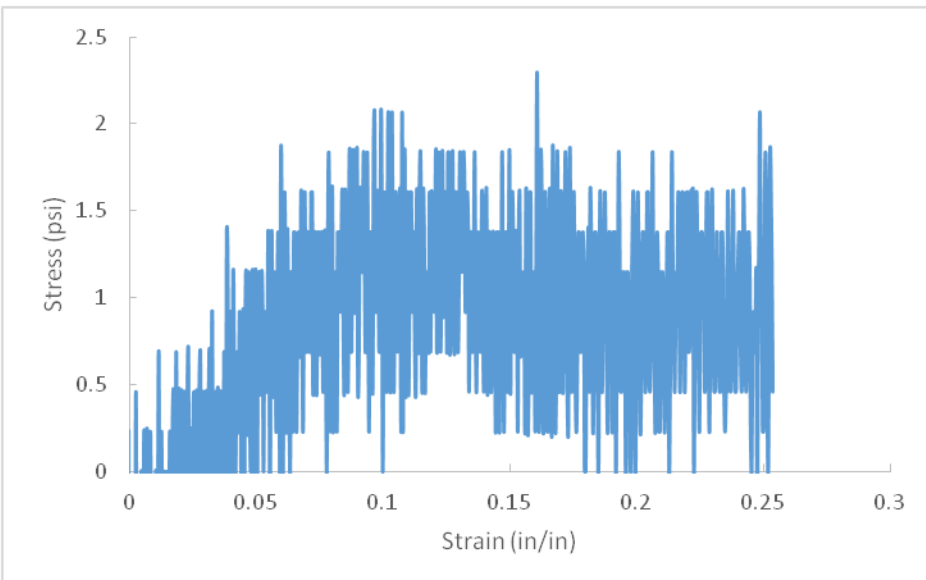

Figure A.106. D2-5, OC= 66.4%, CF= 300 pcf, T=28 Days.

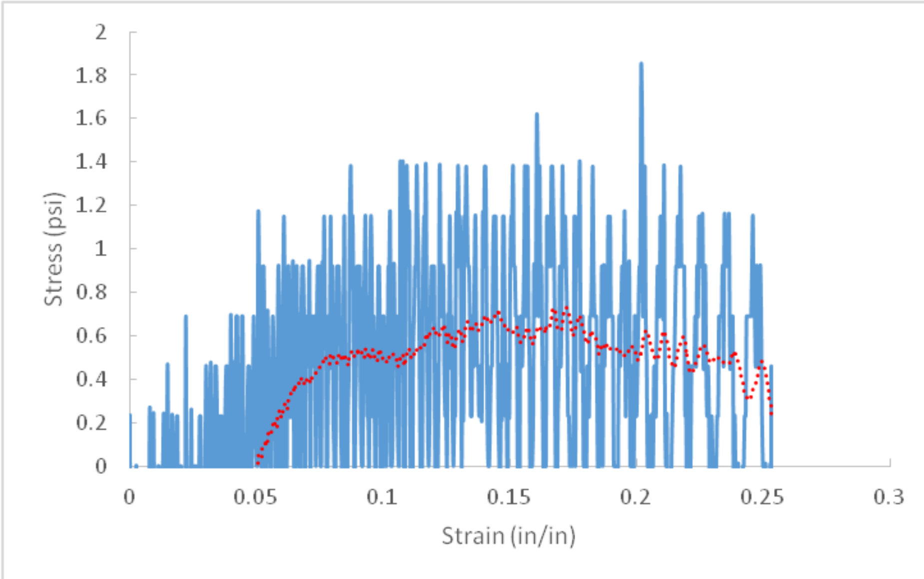

Figure A.107. D2-6, OC= 66.4%, CF= 300 pcf, T=28 Days.

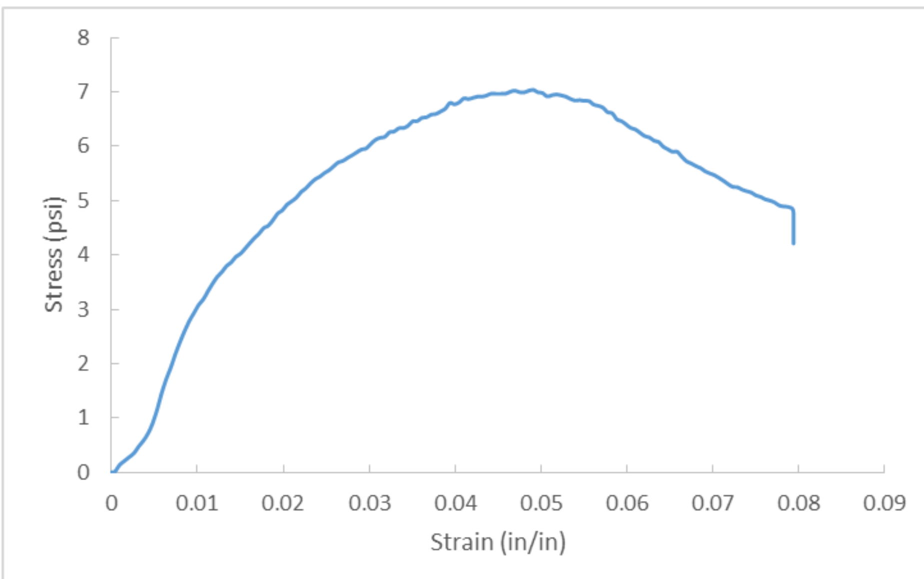

Figure A.108. D3-4, OC= 66.4%, CF= 400 pcf, T=28 Days.

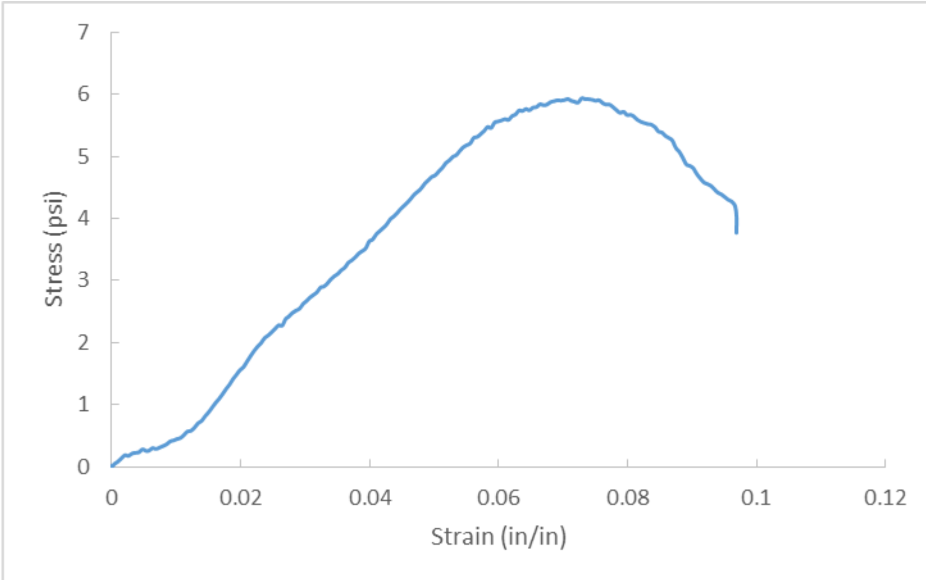

Figure A.109. D3-5, OC= 66.4%, CF= 400 pcf, T=28 Days.

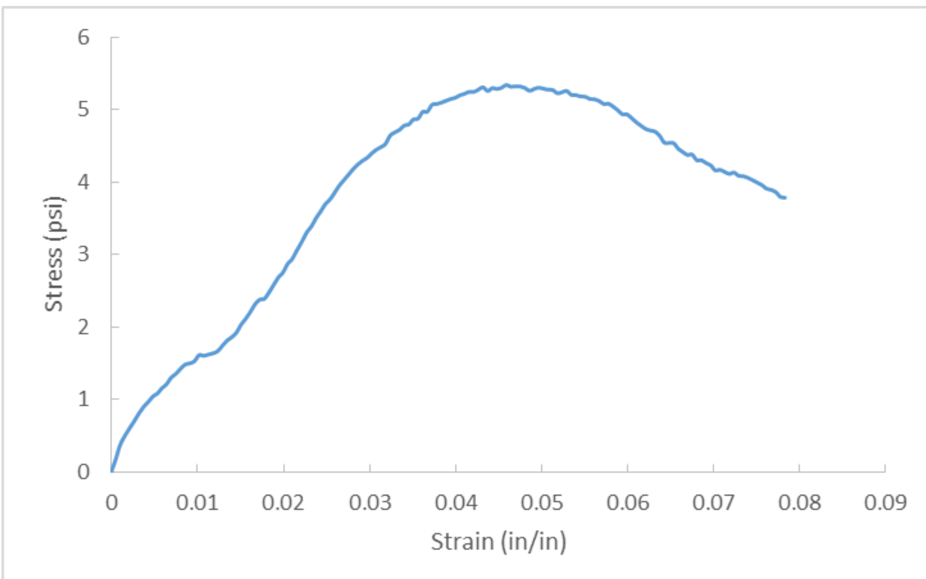

Figure A.110. D3-6, OC= 66.4%, CF= 400 pcf, T=28 Days.

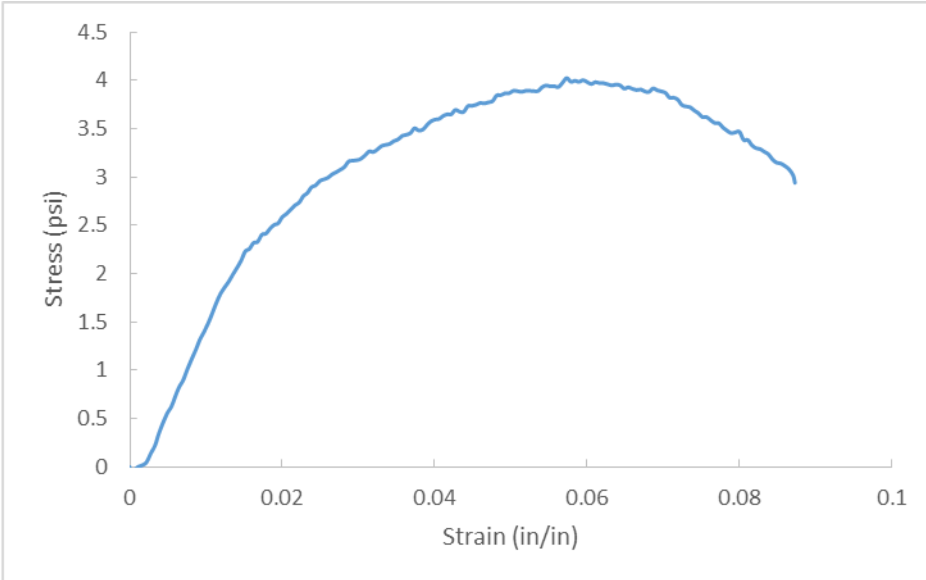

Figure A.111. D4-4, OC= 66.4%, CF= 200 pcf, T=28 Days.

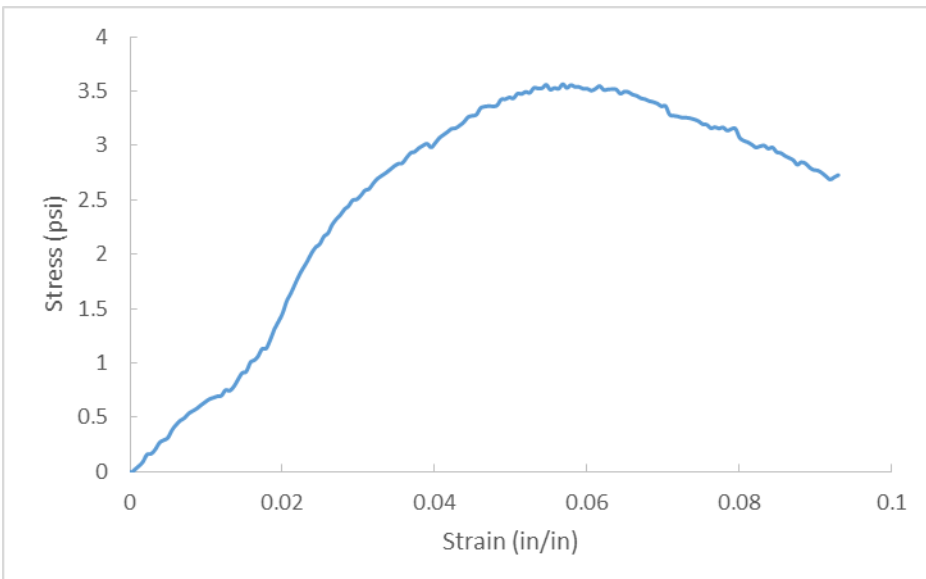

Figure A.112. D4-5, OC= 66.4%, CF= 200 pcf, T=28 Days.

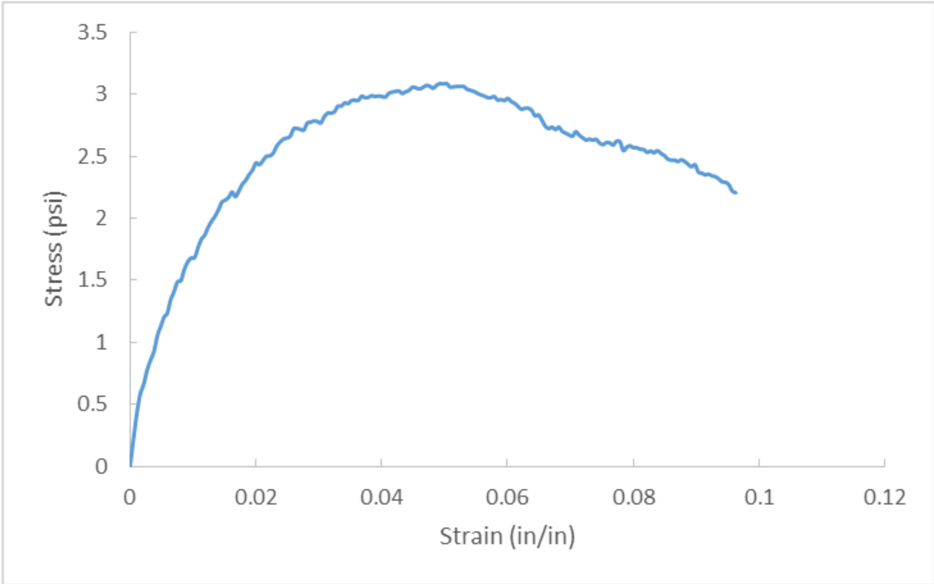

Figure A.113. D4-6, OC= 66.4%, CF= 200 pcf, T=28 Days.

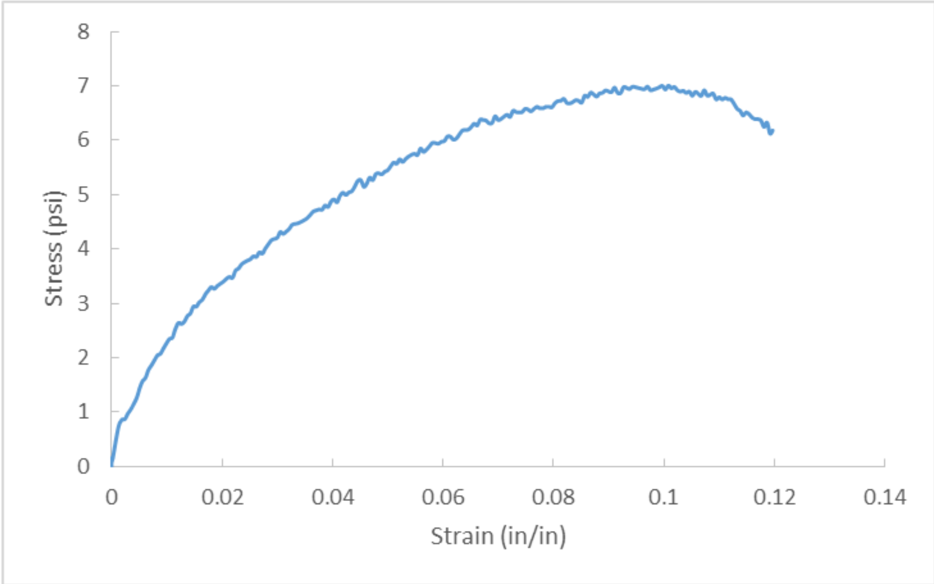

Figure A.114. D8-4, OC= 65.9%, CF= 300 pcf, T=28 Days.

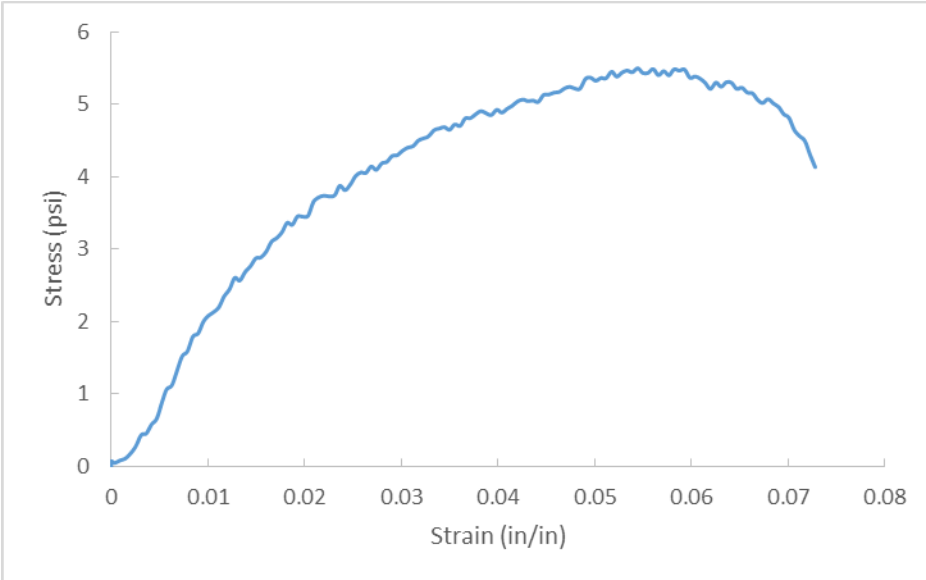

Figure A.115. D8-5, OC= 65.9%, CF= 300 pcf, T=28 Days.

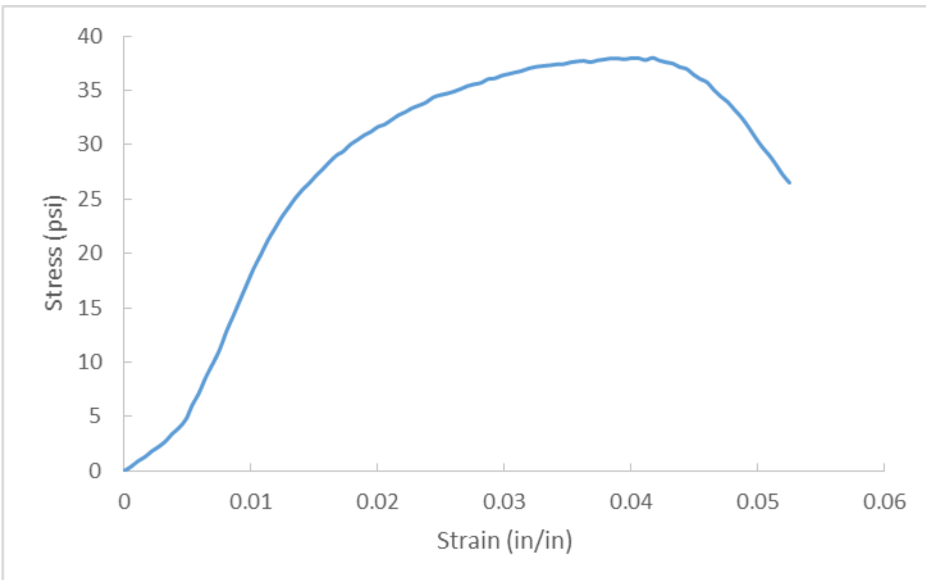

Figure A.116. D9-4, OC= 65.9%, CF= 400 pcf, T=28 Days.

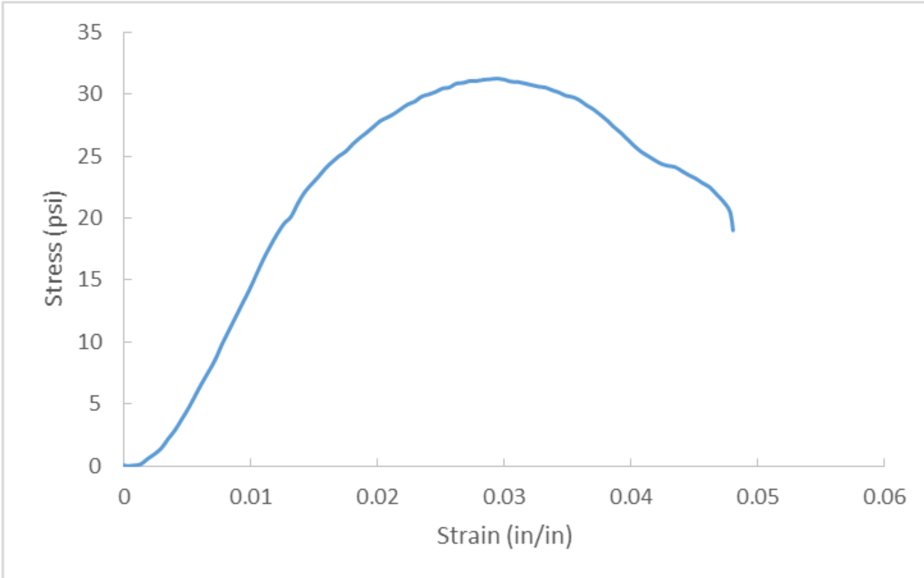

Figure A.117. D9-5, OC= 65.9%, CF= 400 pcf, T=28 Days.

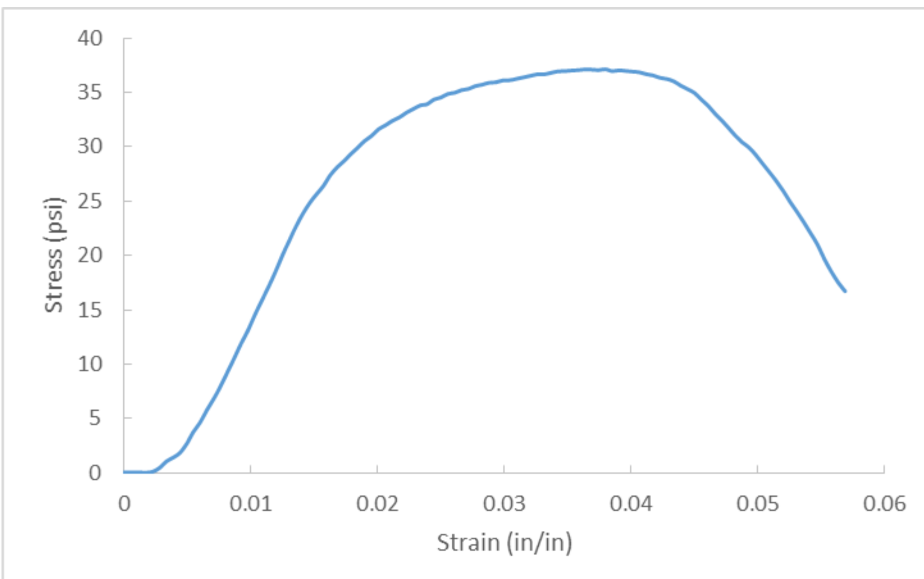

Figure A.118. D9-6, OC= 65.9%, CF= 400 pcf, T=28 Days.

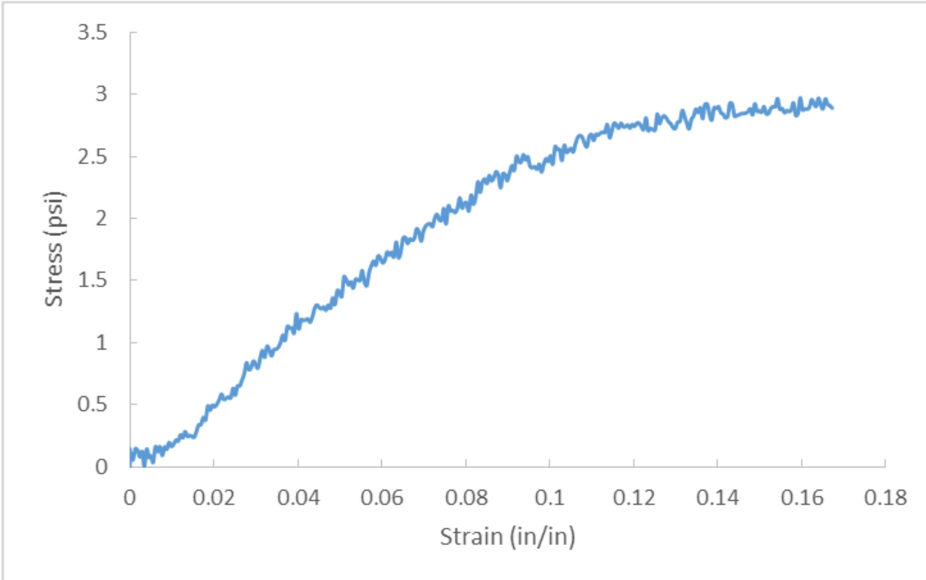

Figure A.119. D10-5, OC= 41.3%, CF= 300 pcf, T=28 Days.

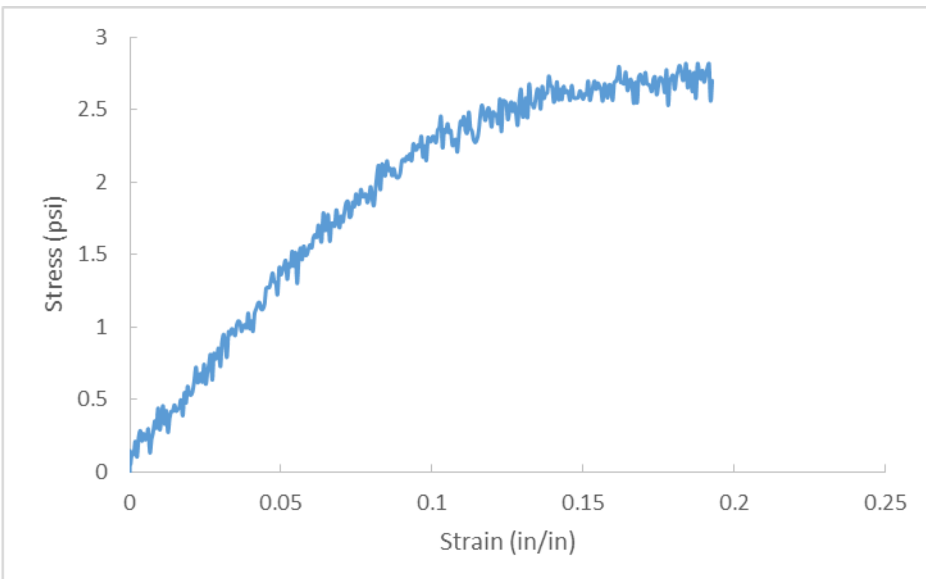

Figure A.120. D10-6, OC= 41.3%, CF= 300 pcf, T=28 Days.

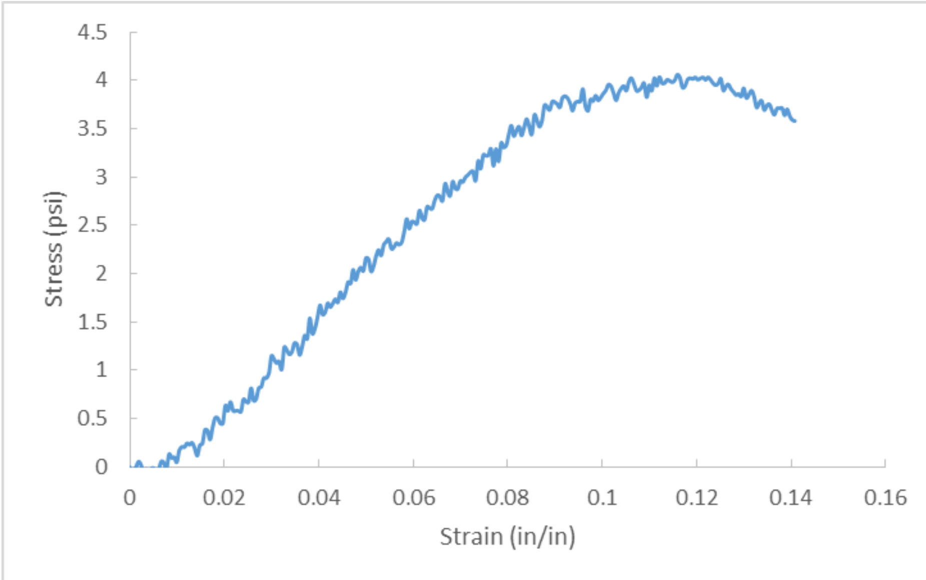

Figure A.121. D11-4, OC= 27.8%, CF= 300 pcf, T=28 Days.

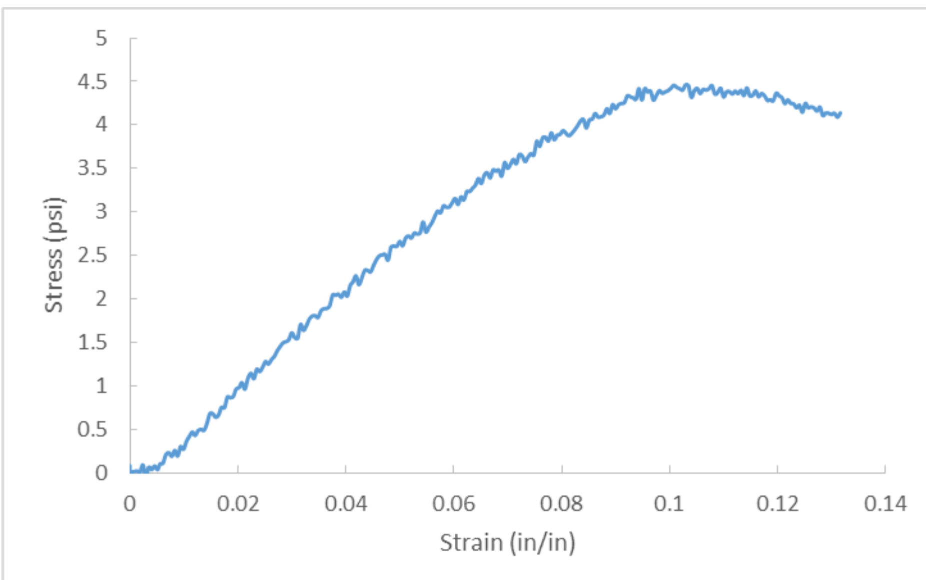

Figure A.122. D11-6, OC= 27.8%, CF= 300 pcf, T=28 Days.

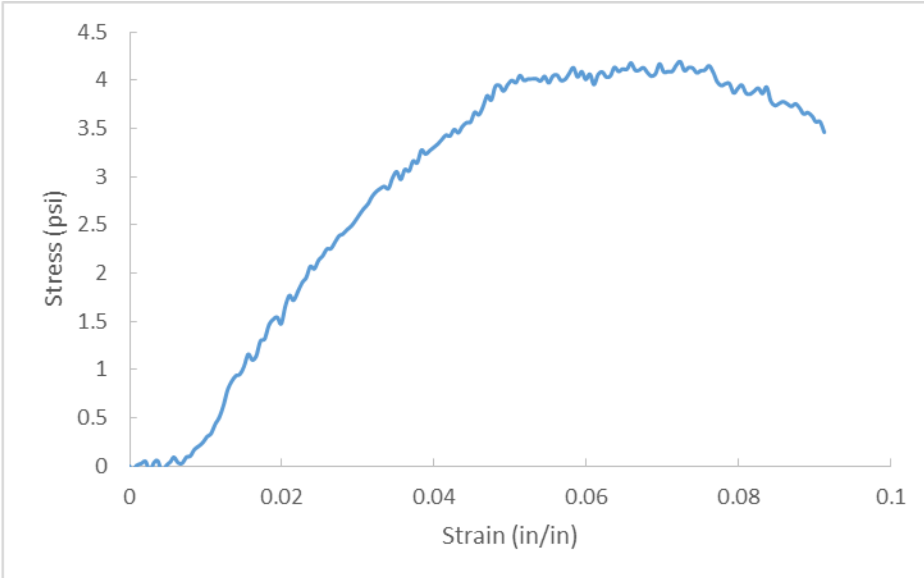

Figure A.123. D12-4, OC= 24.1%, CF= 300 pcf, T=28 Days.

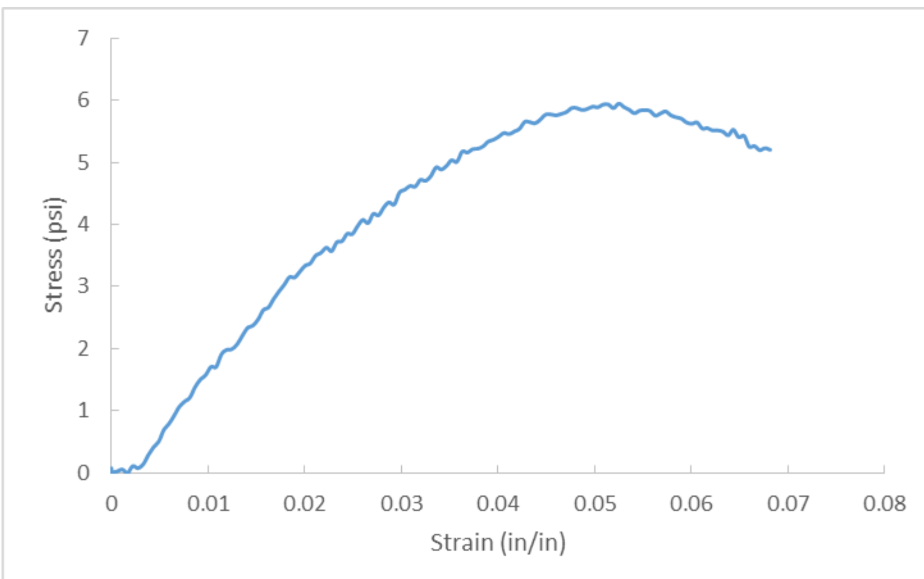

Figure A.124. D12-5, OC= 24.1%, CF= 300 pcf, T=28 Days.

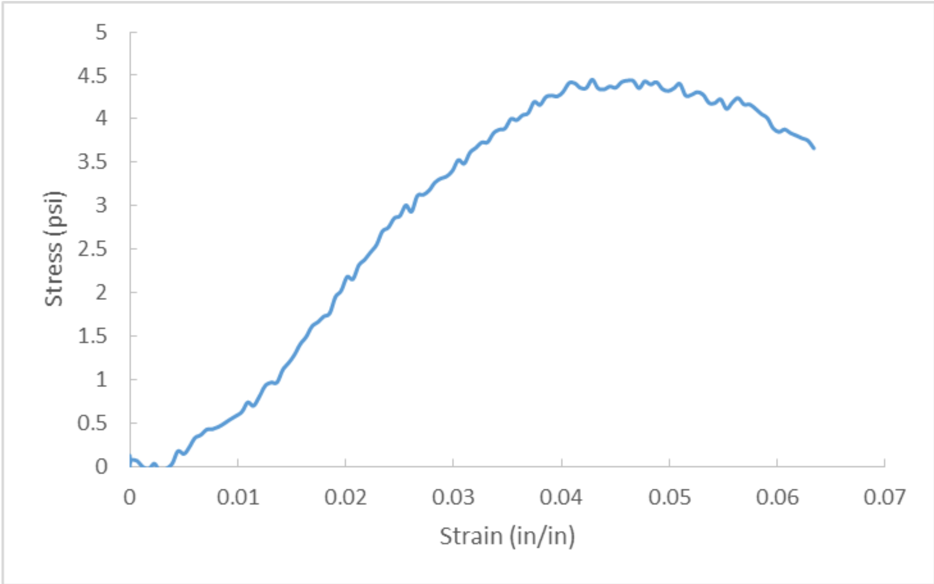

Figure A.125. D12-6, OC= 24.1%, CF= 300 pcf, T=28 Days.

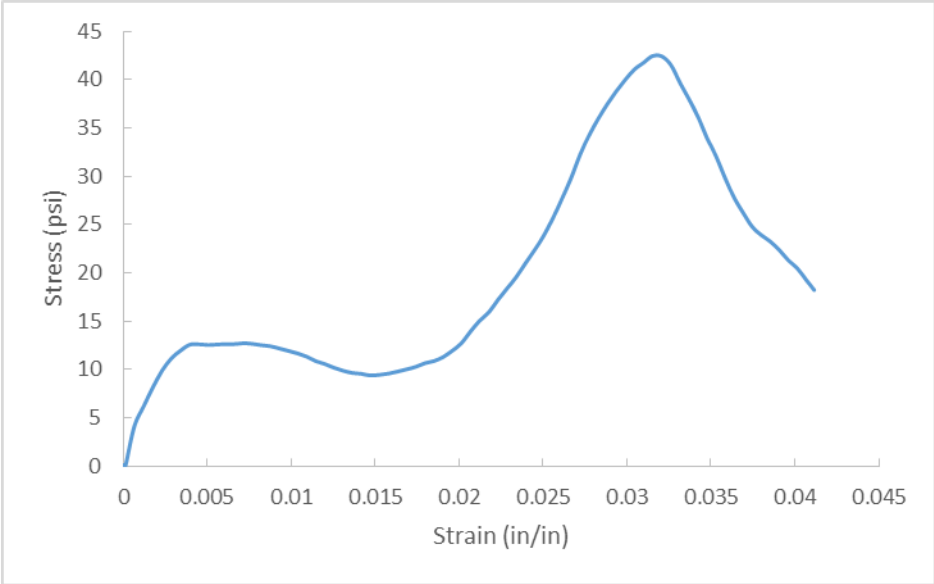

Figure A.126. D13-4, OC= 11.2%, CF= 300 pcf, T=28 Days.

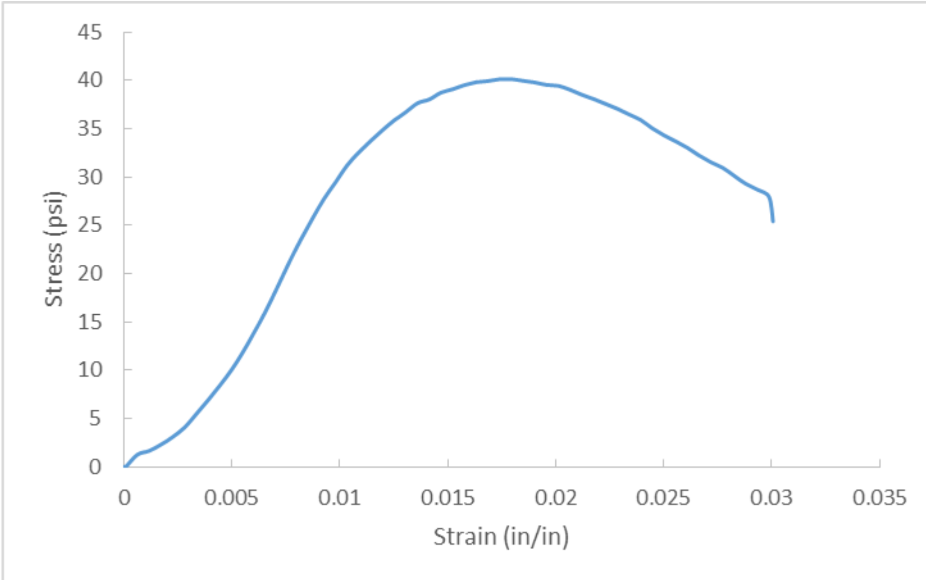

Figure A.127. D13-5, OC= 11.2%, CF= 300 pcf, T=28 Days.

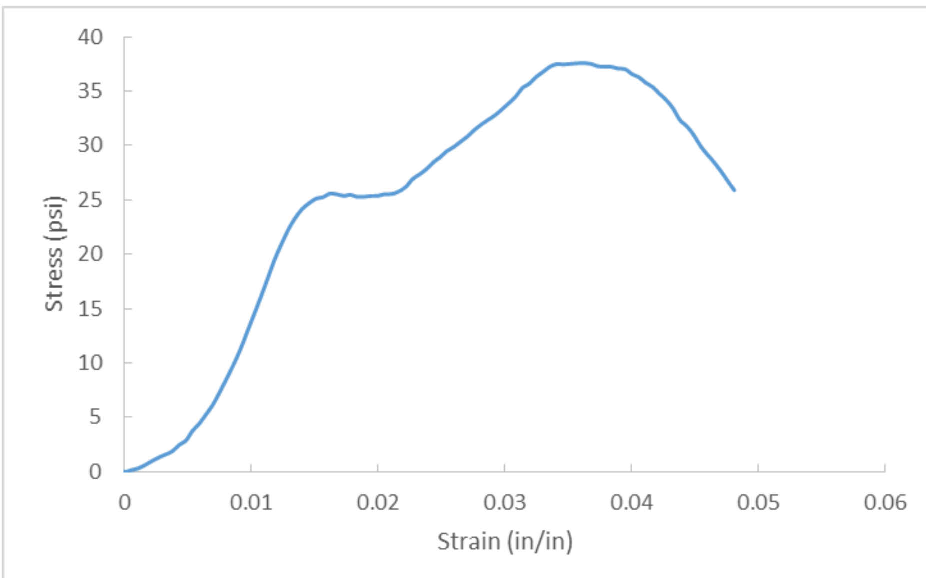

Figure A.128. D13-6, OC= 11.2%, CF= 300 pcf, T=28 Days.

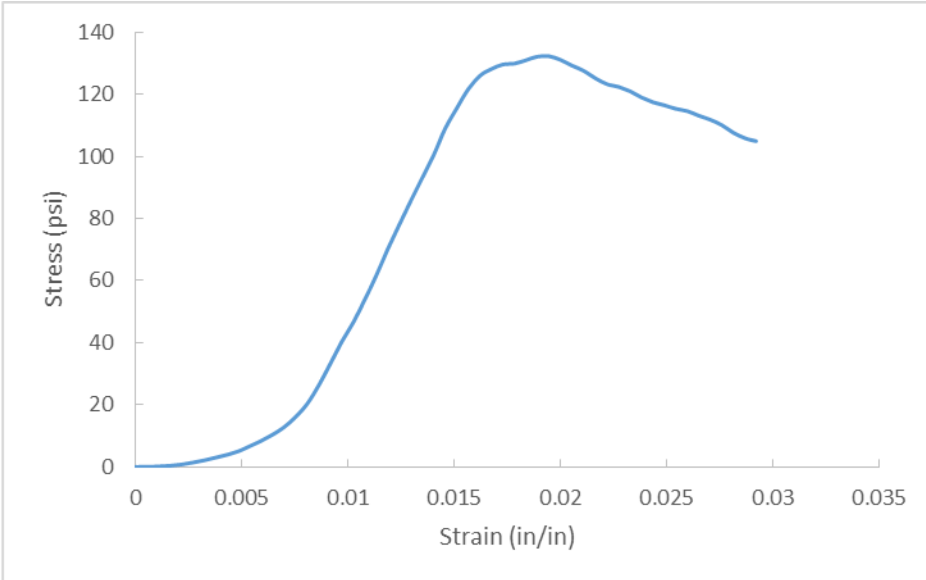

Figure A.129. D14-4, OC= 4.6%, CF= 300 pcf, T=28 Days.

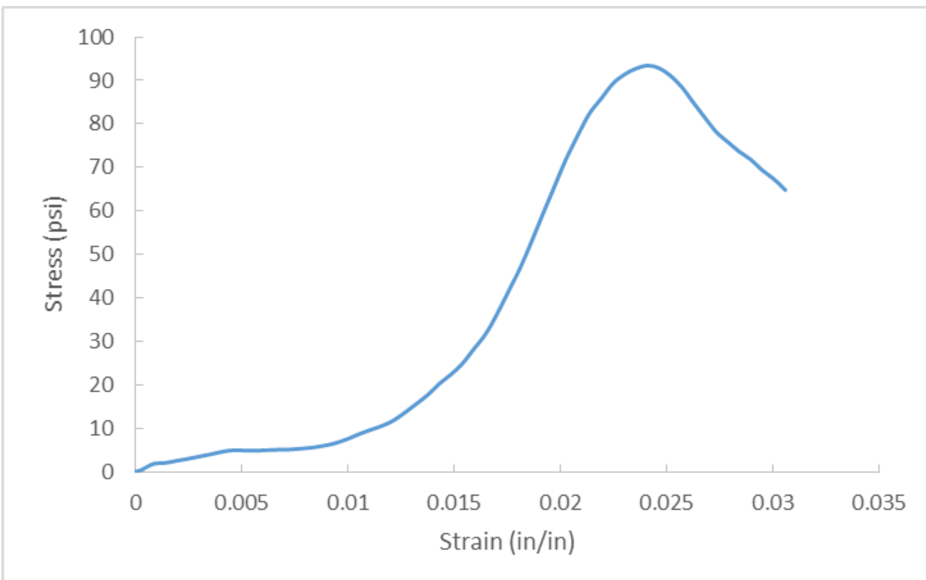

Figure A.130. D14-5, OC= 4.6%, CF= 300 pcf, T=28 Days.

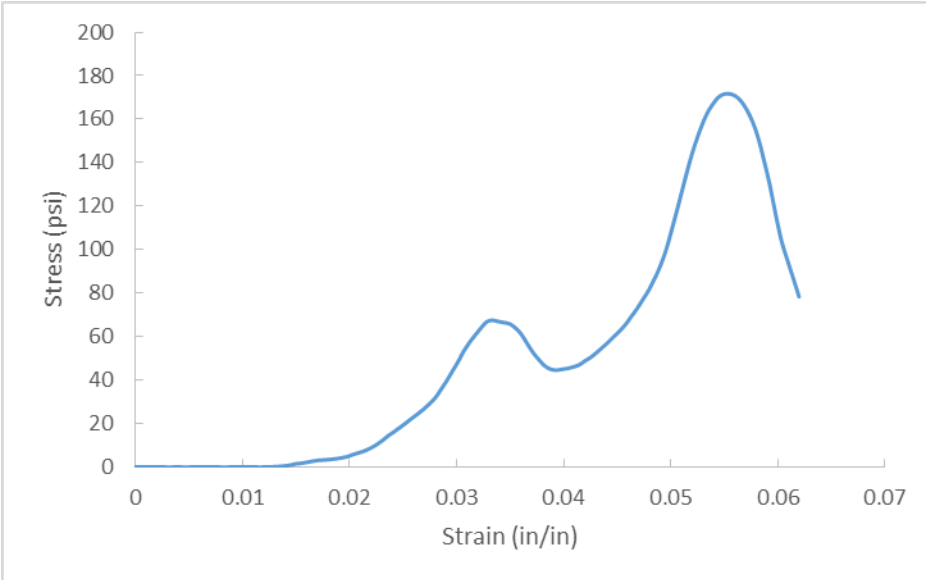

Figure A.131. D14-6, OC= 4.6%, CF= 300 pcf, T=28 Days.

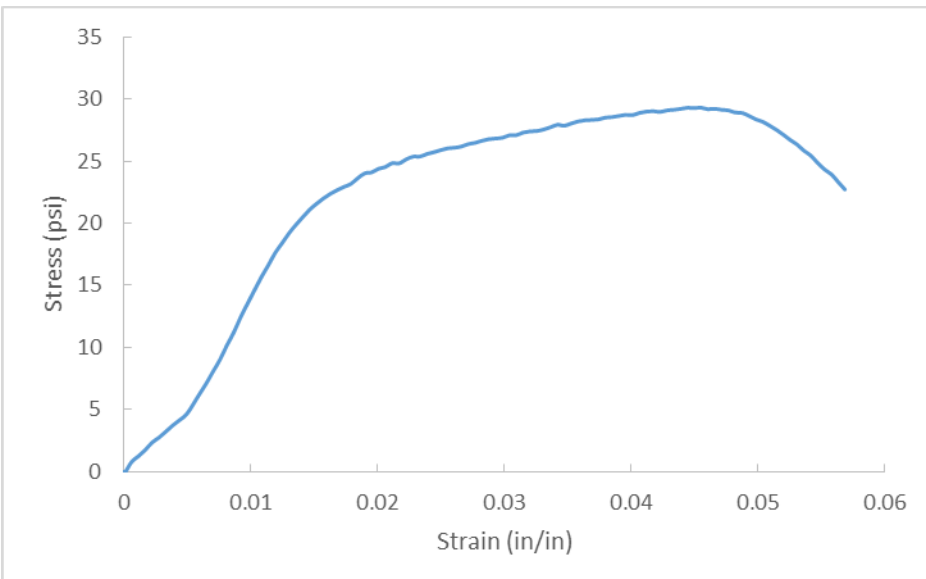

Figure A.132. D15-4, OC= 40.5%, CF= 300 pcf, T=28 Days.

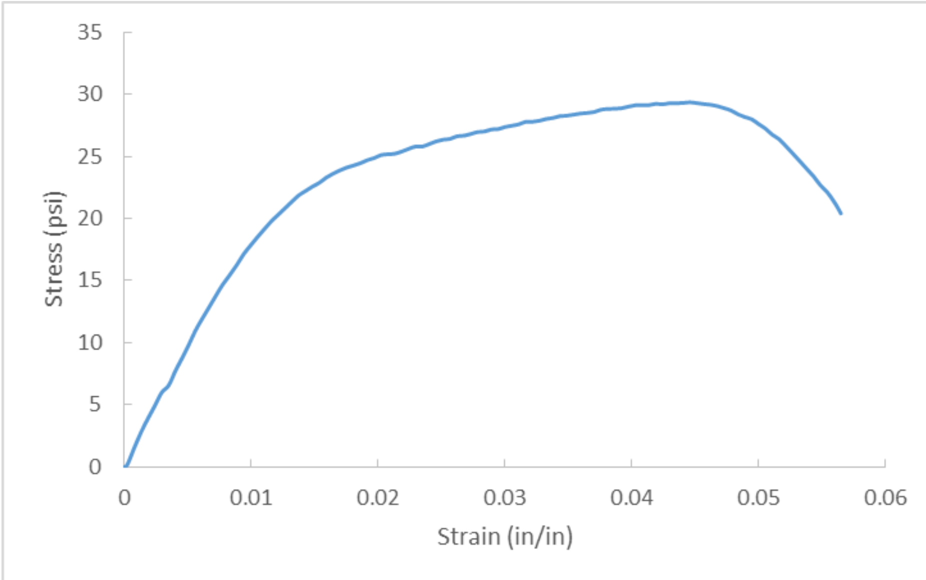

Figure A.133. D15-5, OC= 40.5%, CF= 300 pcf, T=28 Days.

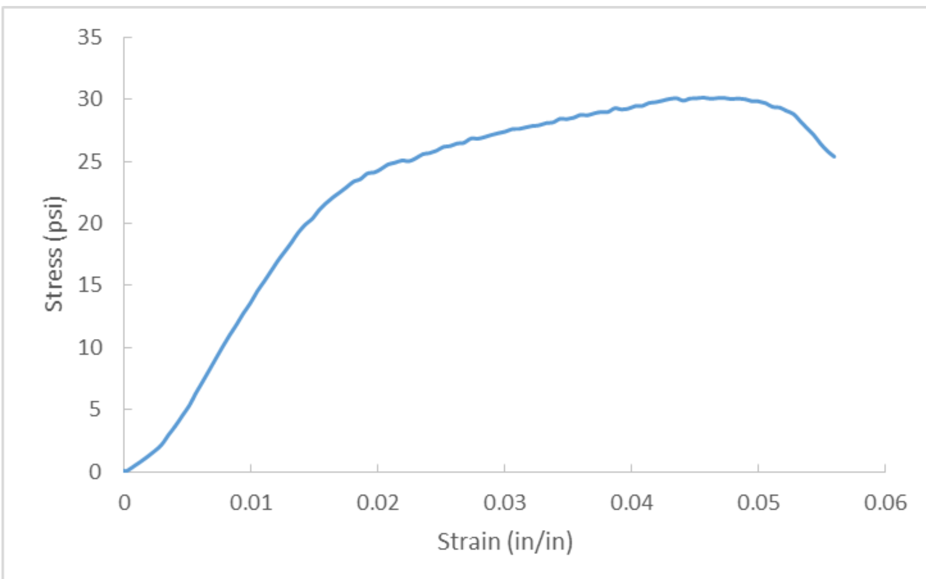

Figure A.134. D15-6, OC= 40.5%, CF= 300 pcf, T=28 Days.

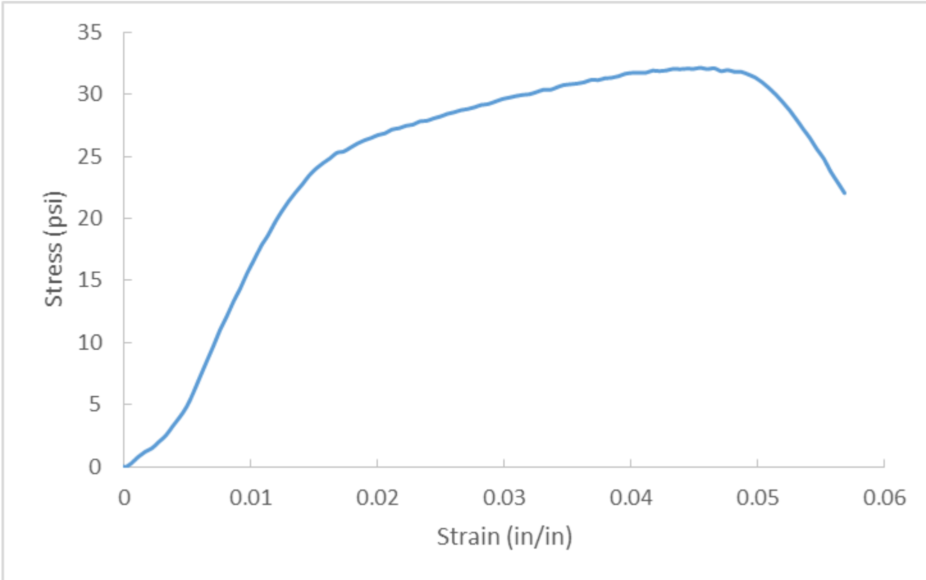

Figure A.135. D16-4, OC= 34.7%, CF= 300 pcf, T=28 Days.

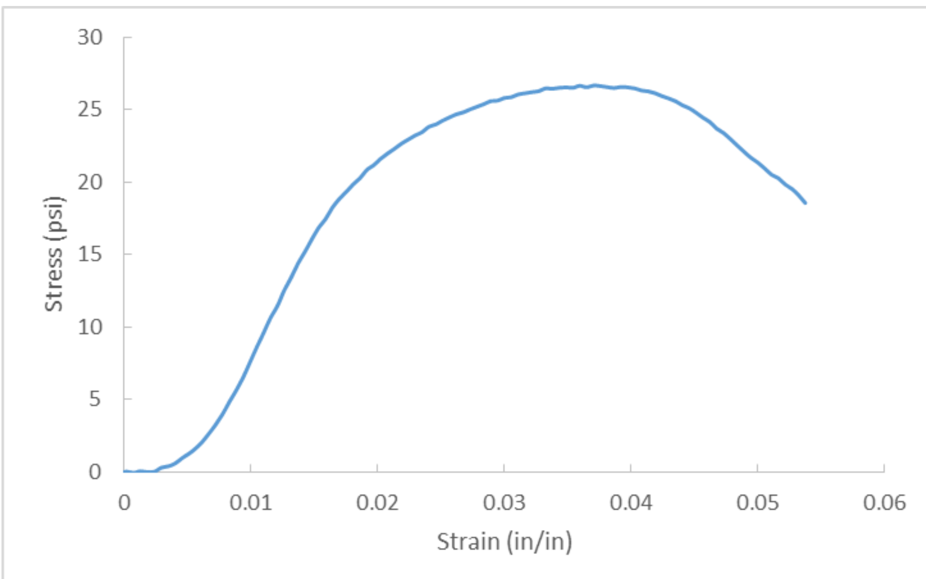

Figure A.136. D16-5, OC= 34.7%, CF= 300 pcf, T=28 Days.

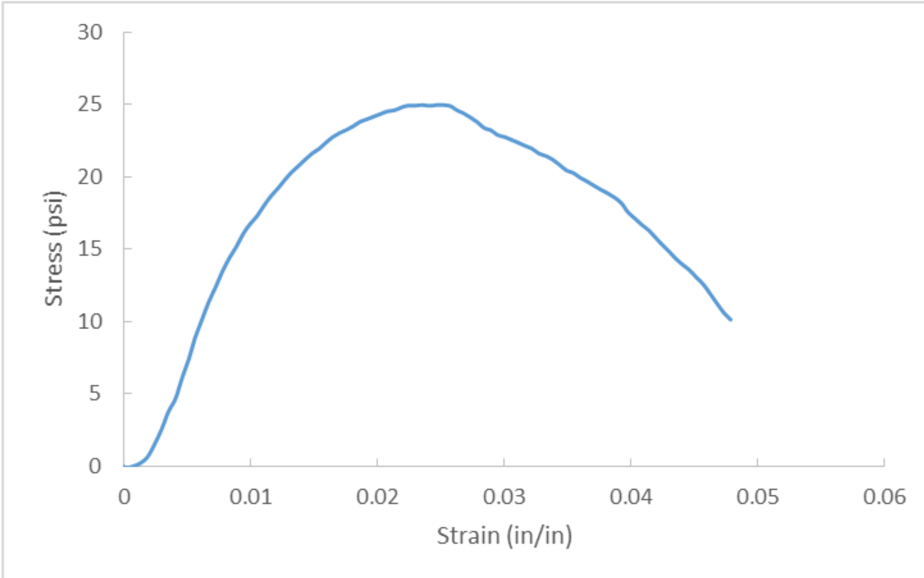

Figure A.137. D16-6, OC= 34.7%, CF= 300 pcf, T=28 Days.

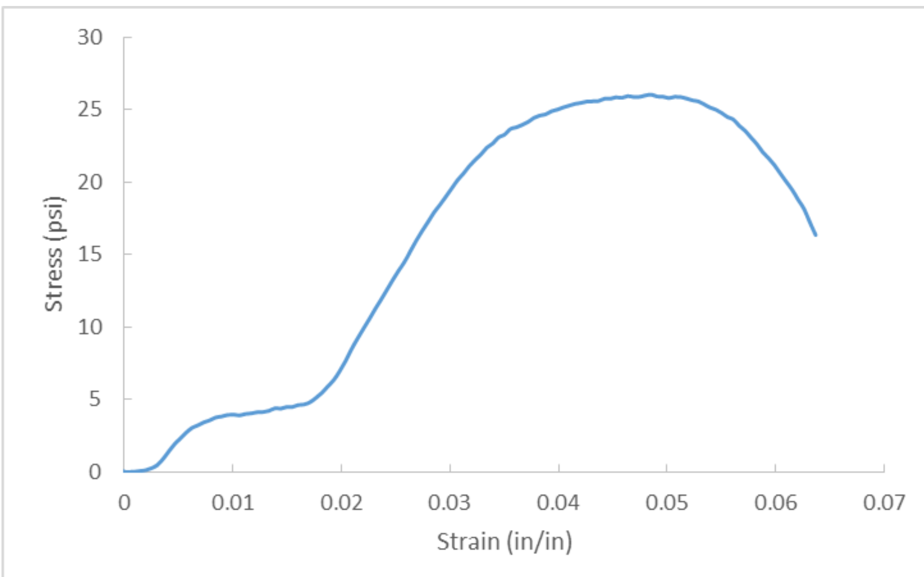

Figure A.138. D17-4, OC= 19.2%, CF= 300 pcf, T=28 Days.

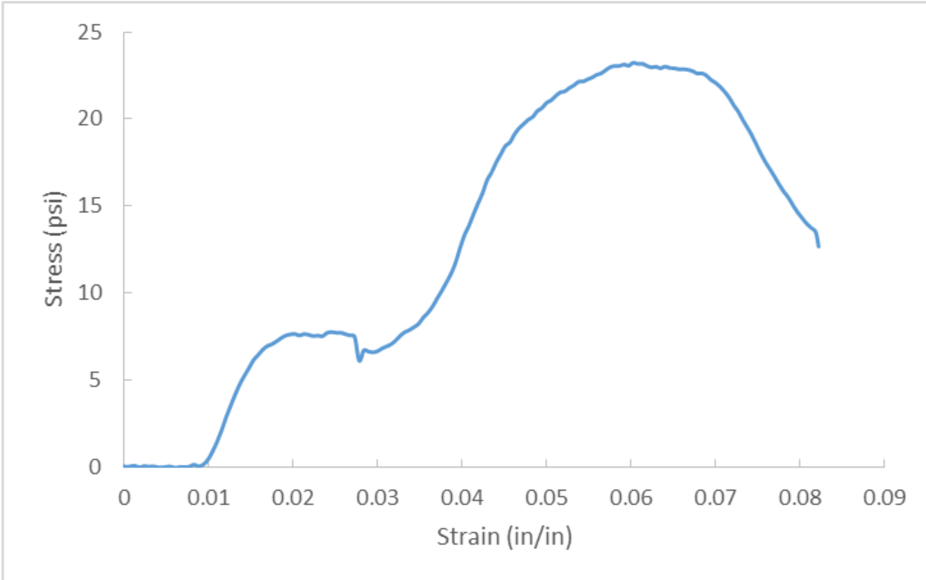

Figure A.139. D17-5, OC= 19.2%, CF= 300 pcf, T=28 Days.

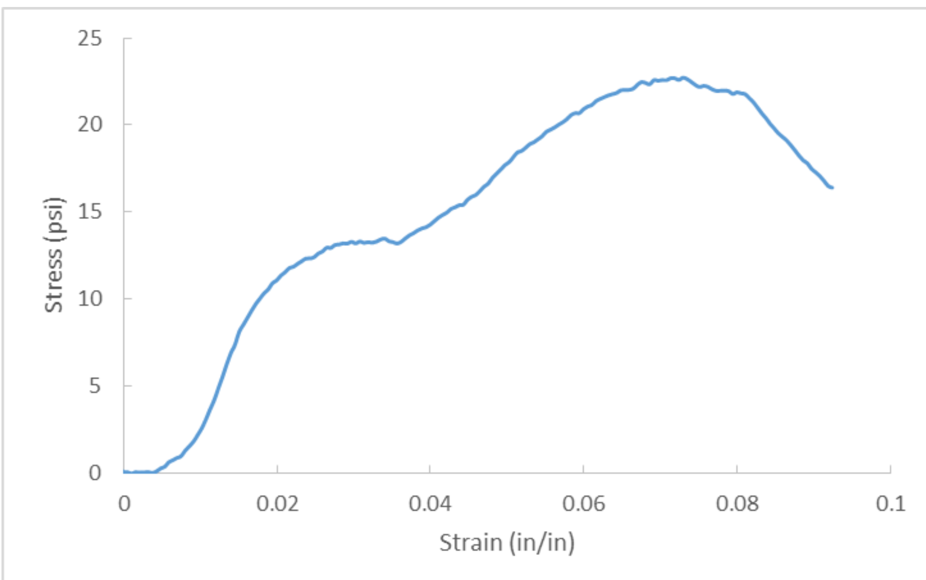

Figure A.140. D17-6, OC= 19.2%, CF= 300 pcf, T=28 Days.

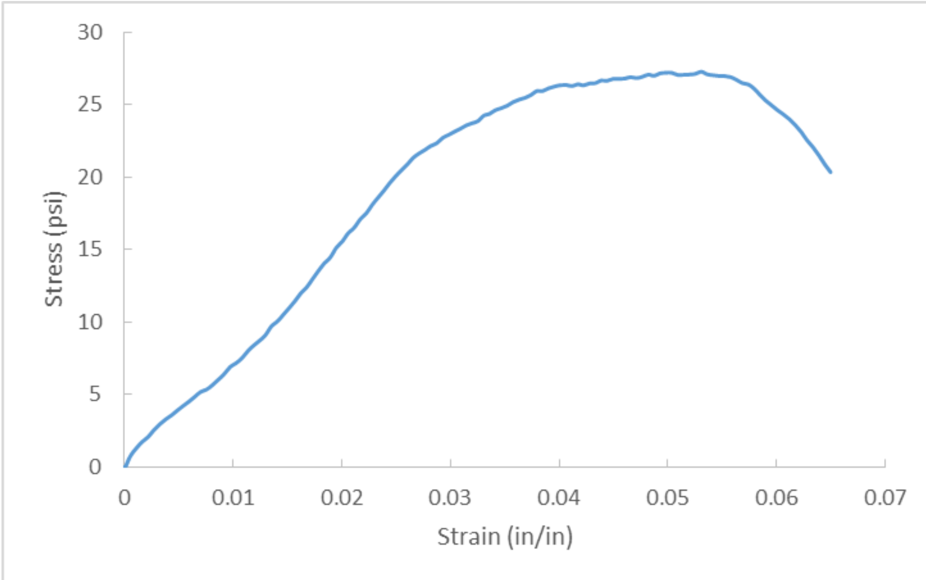

Figure A.141. D18-4, OC= 18.9%, CF= 300 pcf, T=28 Days.

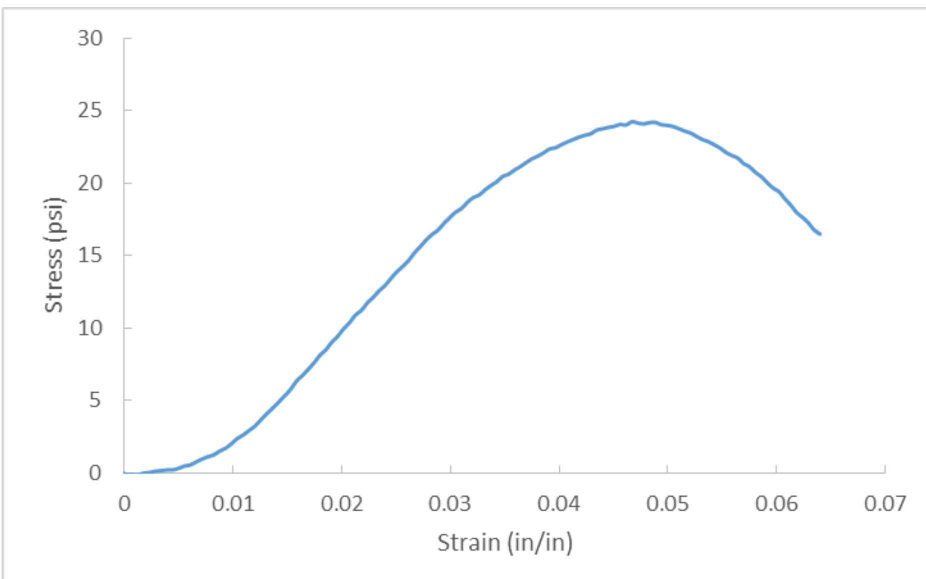

Figure A.142. D18-5, OC= 18.9%, CF= 300 pcf, T=28 Days.

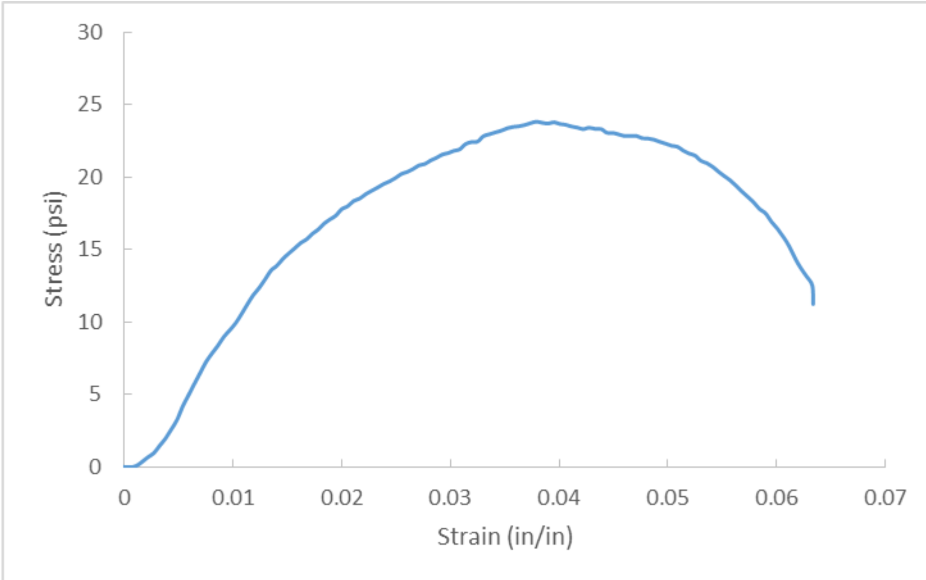

Figure A.143. D18-6, OC= 18.9%, CF= 300 pcf, T=28 Days.

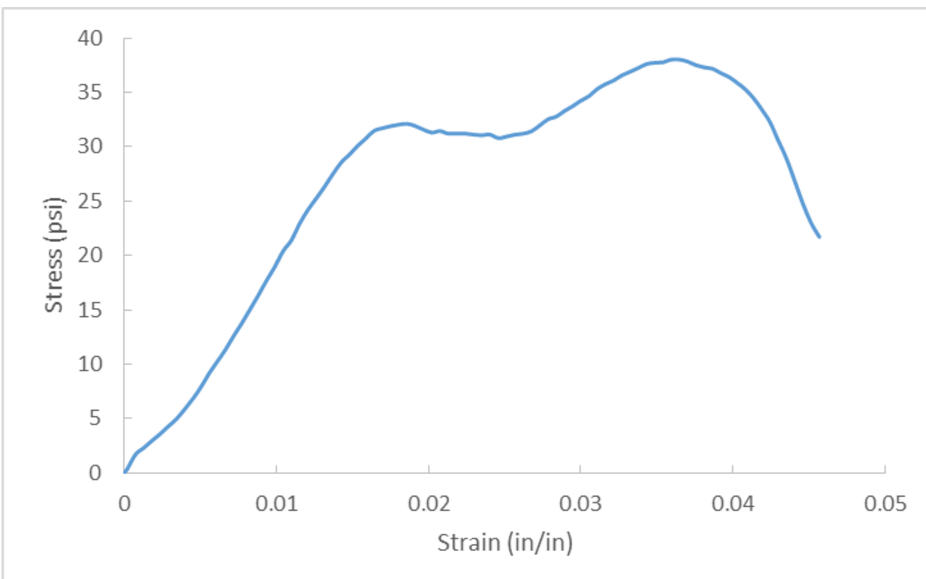

Figure A.144. D19-4, OC= 8.5%, CF= 300 pcf, T=28 Days.

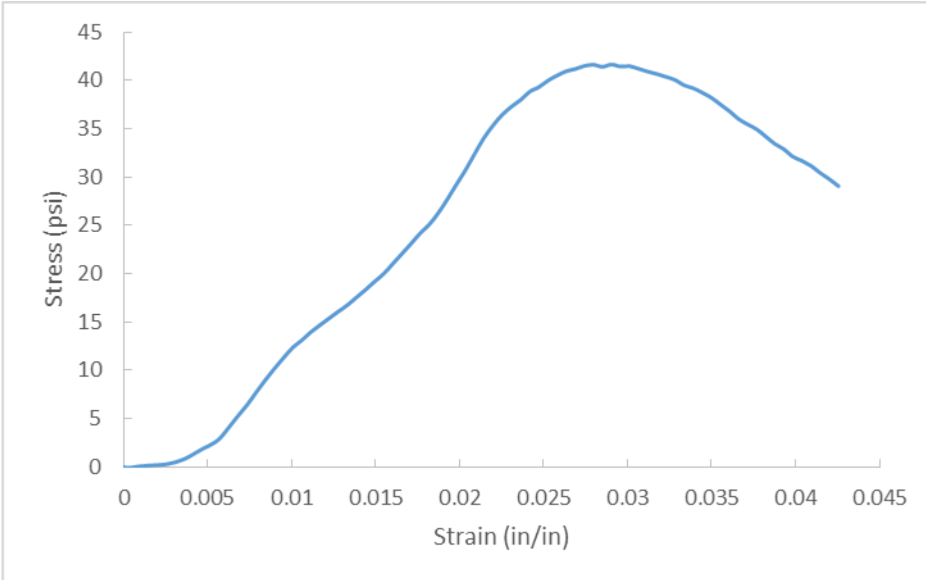

Figure A.145. D19-5, OC= 8.5%, CF= 300 pcf, T=28 Days.

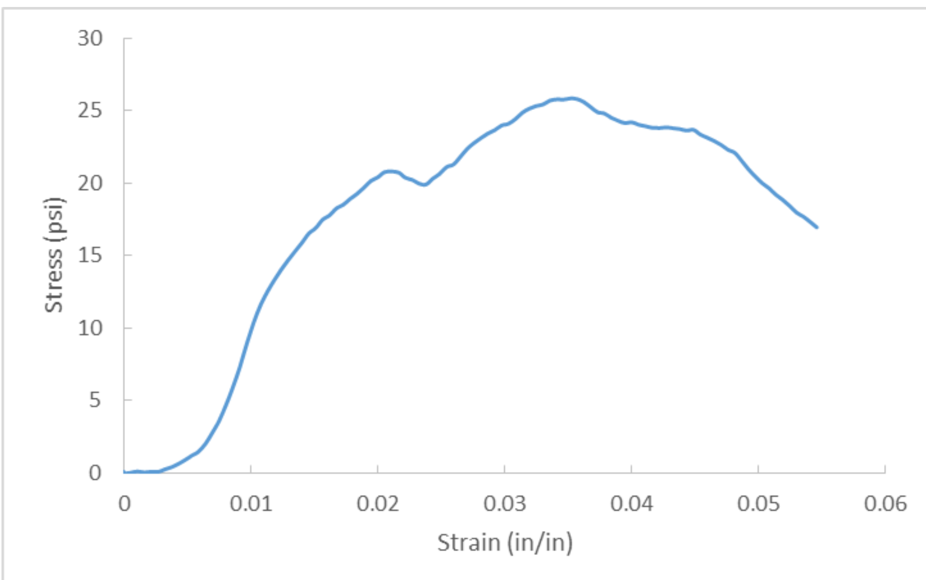

Figure A.146. D19-6, OC= 8.5%, CF= 300 pcf, T=28 Days.

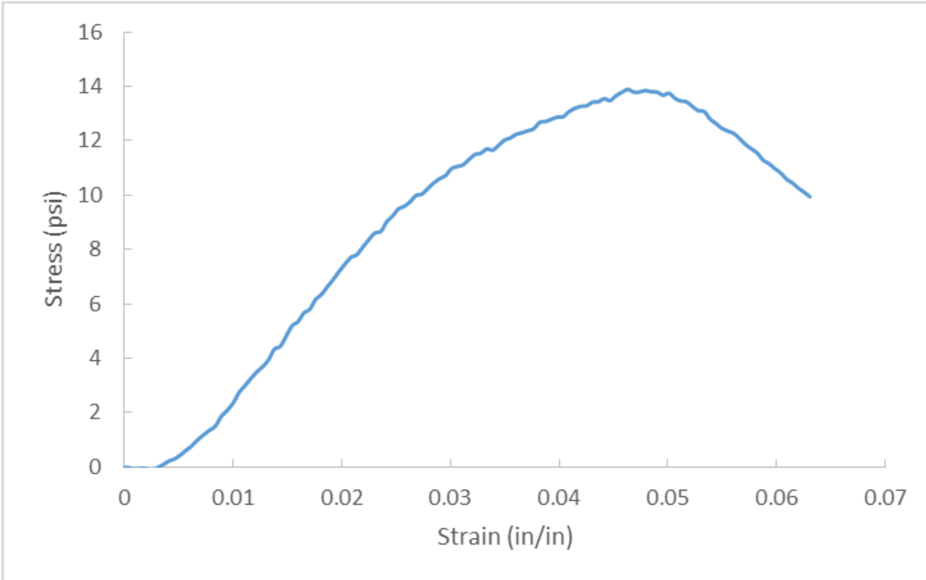

Figure A.147. D25-4, OC= 41.2%, CF= 400 pcf, T=28 Days.

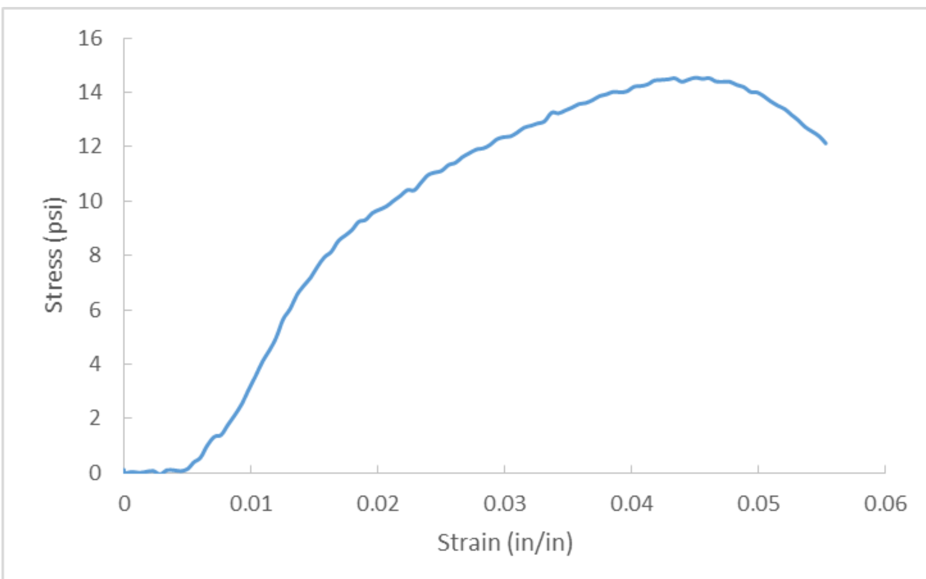

Figure A.148. D25-5, OC= 41.2%, CF= 400 pcf, T=28 Days.

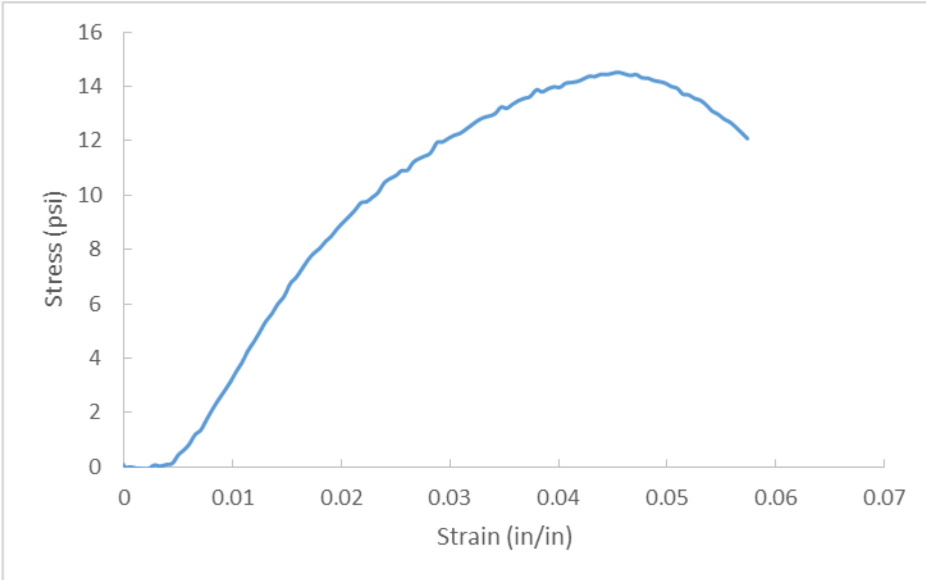

Figure A.149. D25-6, OC= 41.2%, CF= 400 pcf, T=28 Days.

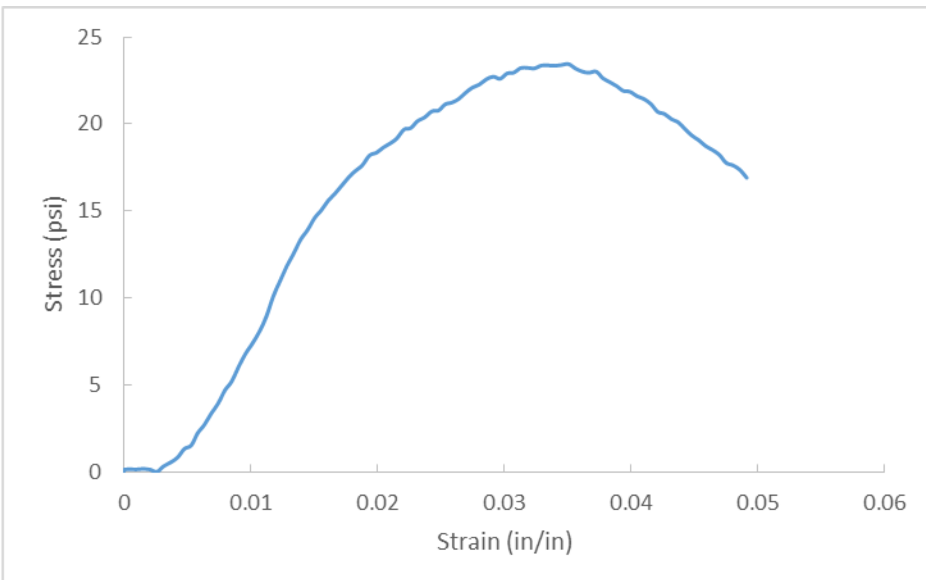

Figure A.150. D26-4, OC= 29.8%, CF= 400 pcf, T=28 Days.

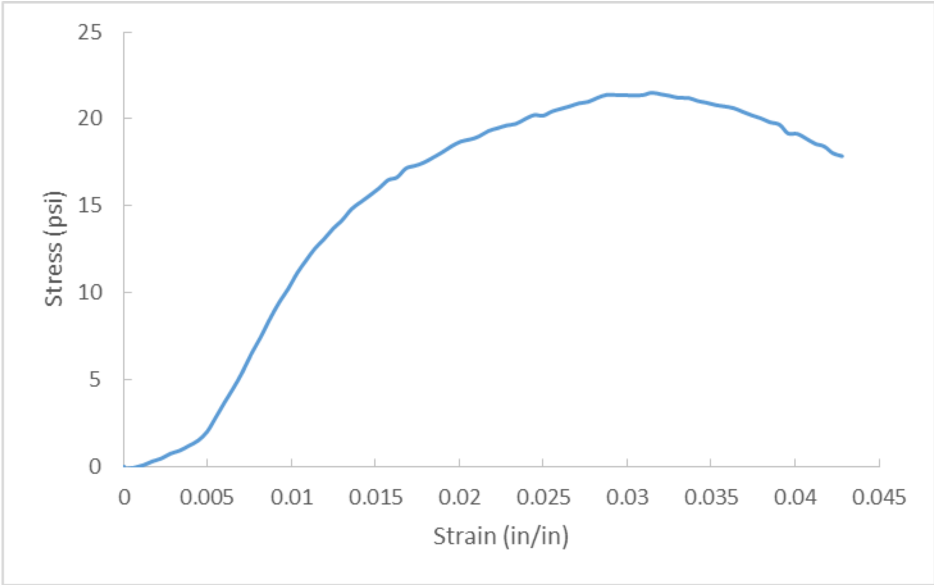

Figure A.151. D26-5, OC= 29.8%, CF= 400 pcf, T=28 Days.

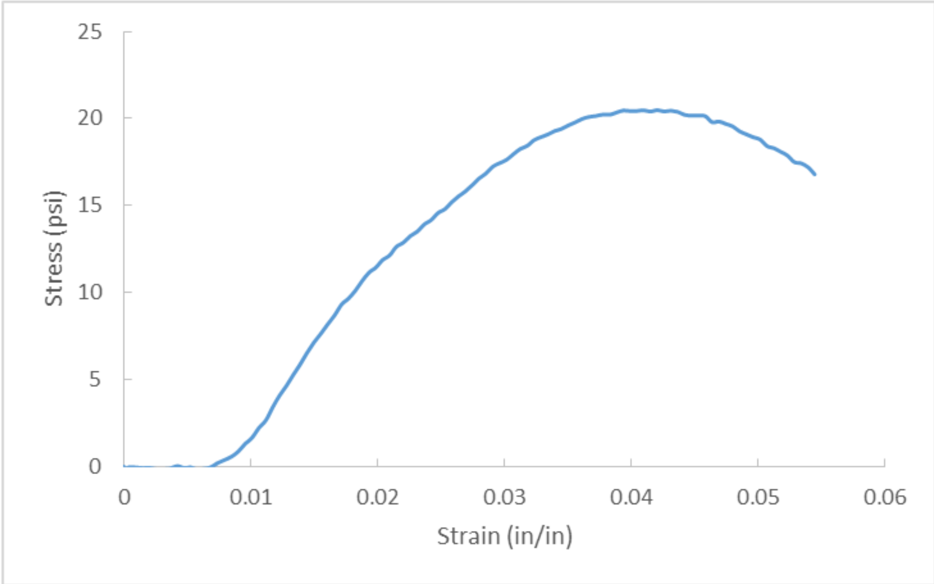

Figure A.152. D26-6, OC= 29.8%, CF= 400 pcf, T=28 Days.

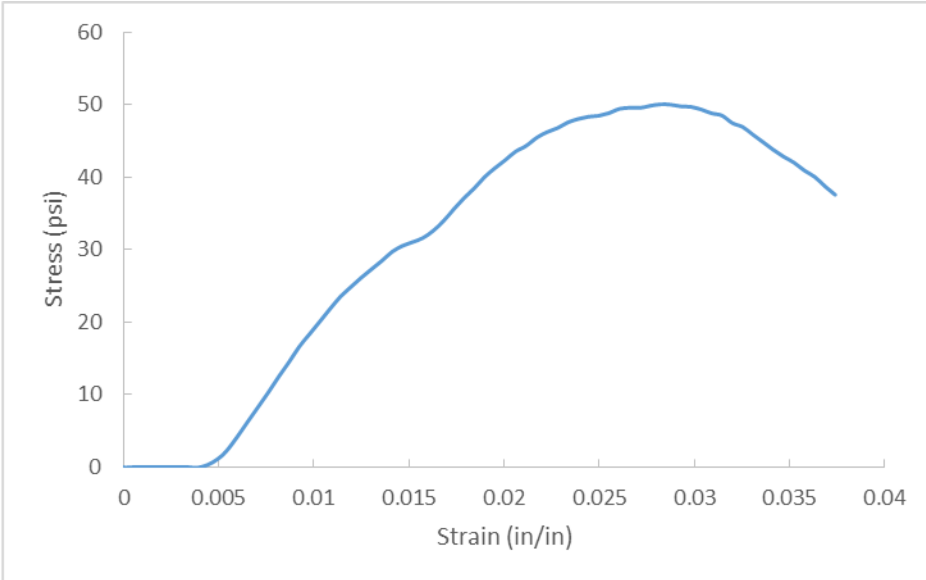

Figure A.153. D27-4, OC= 21.1%, CF= 400 pcf, T=28 Days.

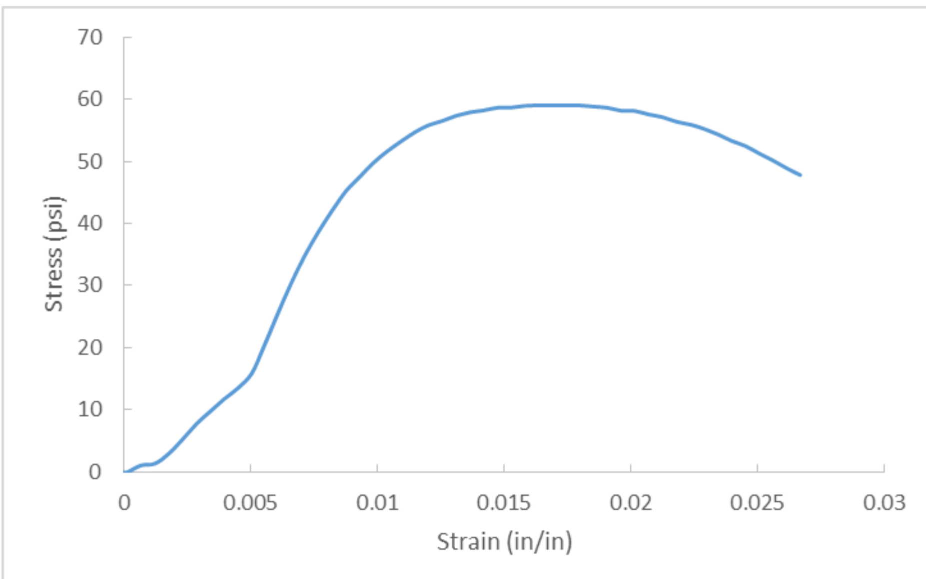

Figure A.154. D27-6, OC= 21.1%, CF= 400 pcf, T=28 Days.

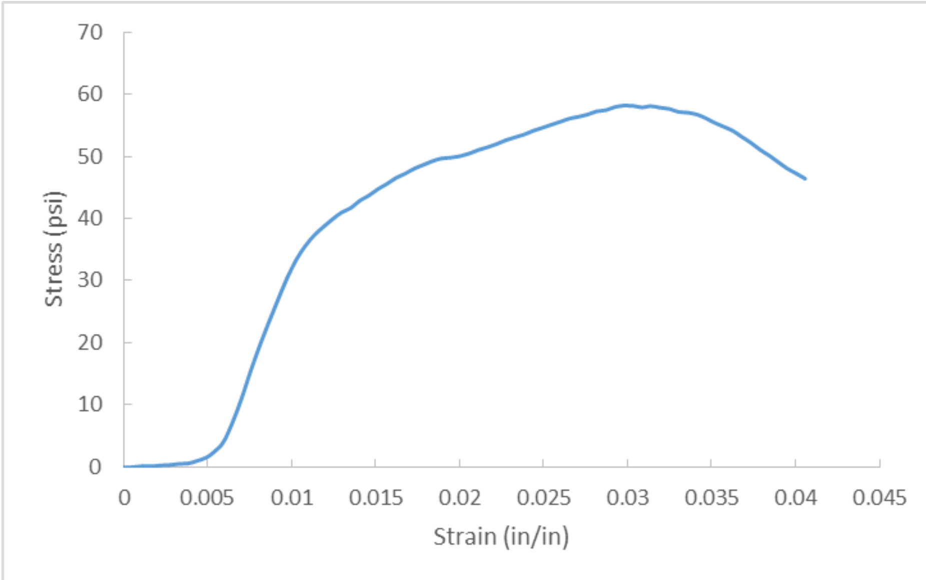

Figure A.155. D28-4, OC= 12.6%, CF= 400 pcf, T=28 Days.

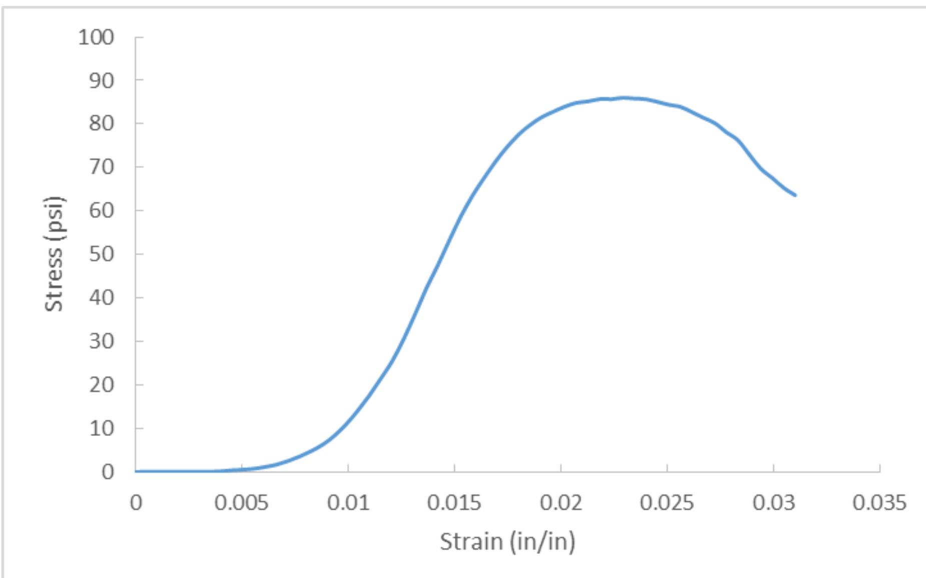

Figure A.156. D28-5, OC= 12.6%, CF= 400 pcf, T=28 Days.

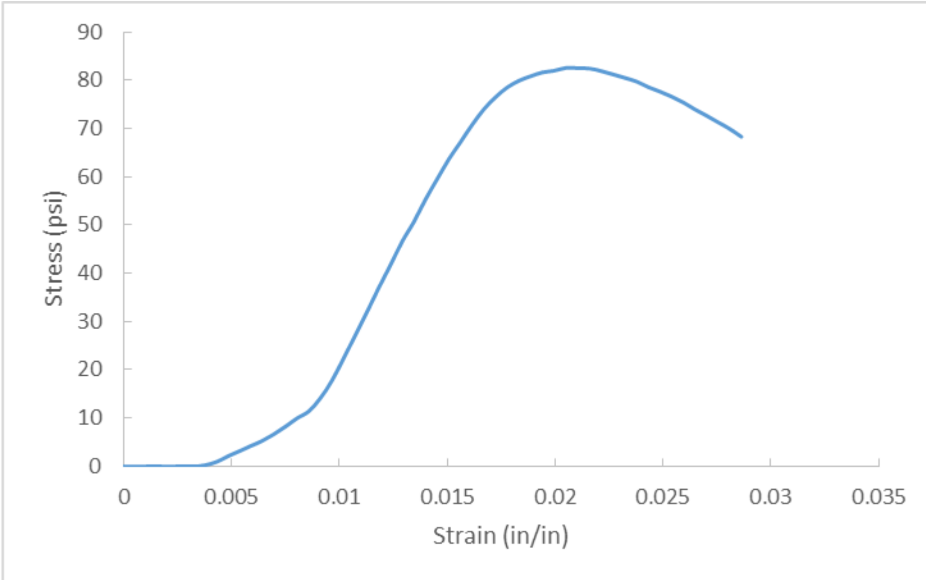

Figure A.157. D28-6, OC= 12.6%, CF= 400 pcf, T=28 Days.

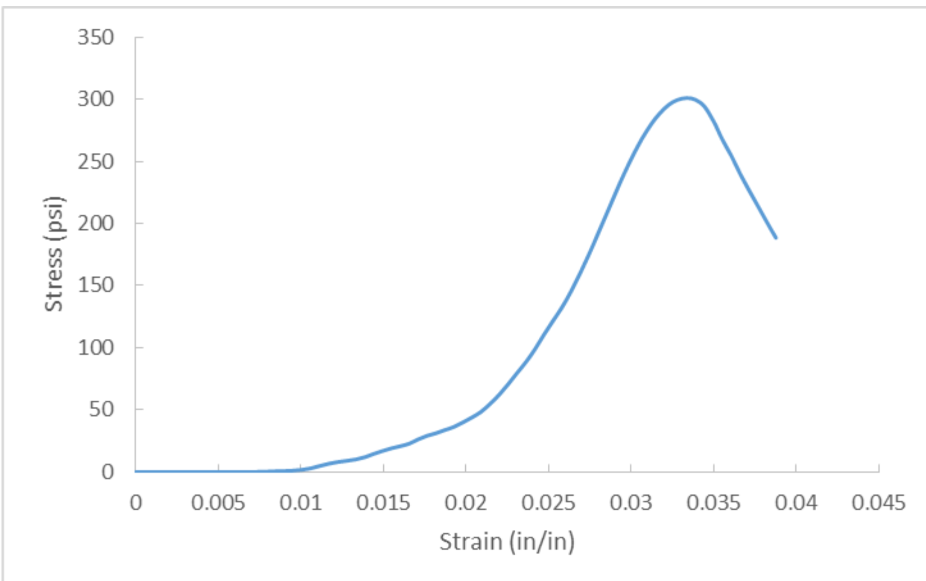

Figure A.158. D29-4, OC= 4.2%, CF= 400 pcf, T=28 Days.

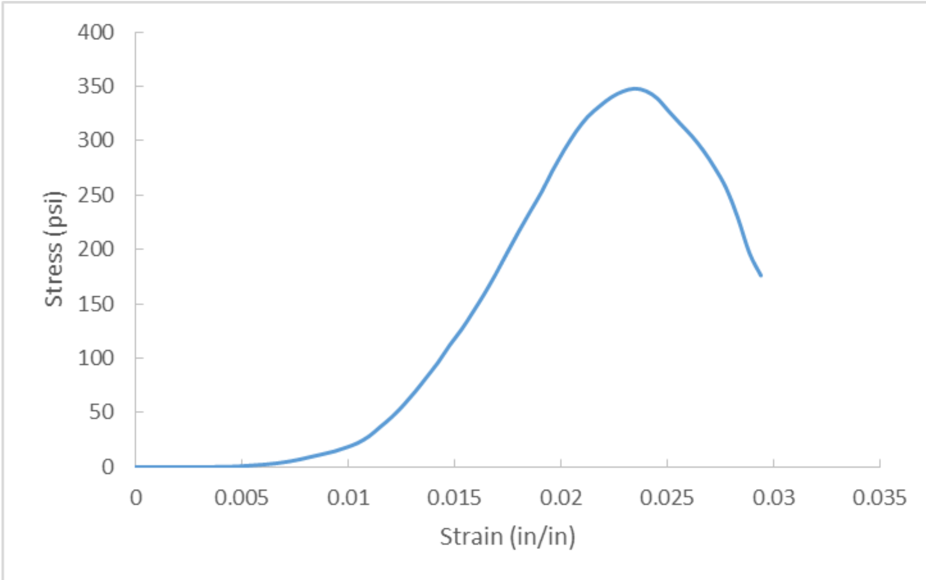

Figure A.159. D29-5, OC= 4.2%, CF= 400 pcf, T=28 Days.

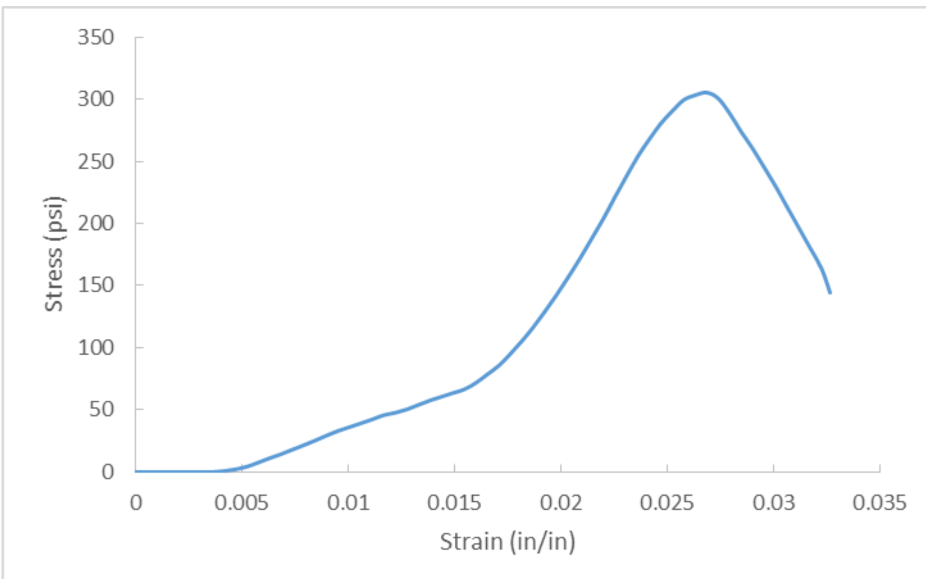

Figure A.160. D29-6, OC= 4.2%, CF= 400 pcf, T=28 Days.

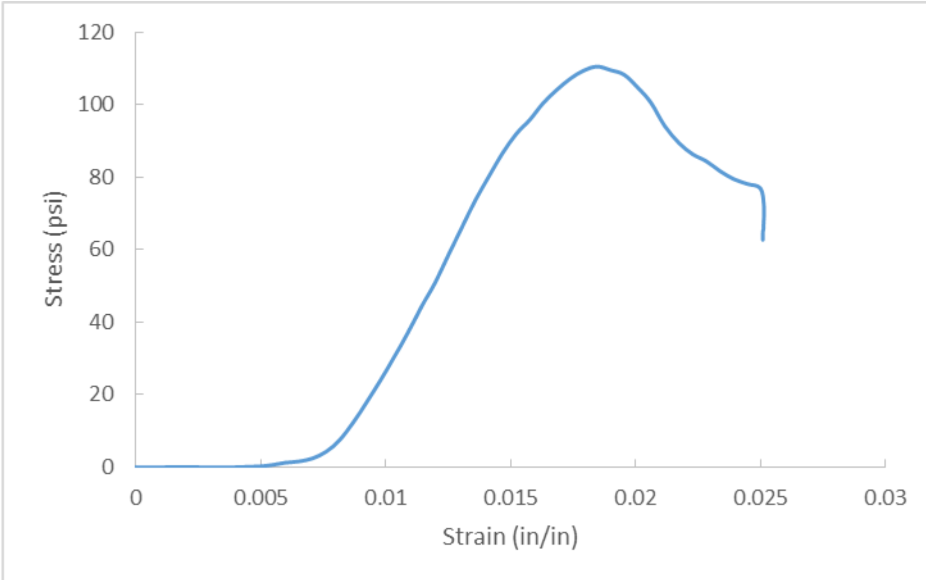

Figure A.161. D30-4, OC= 4.9%, CF= 400 pcf, T=28 Days.

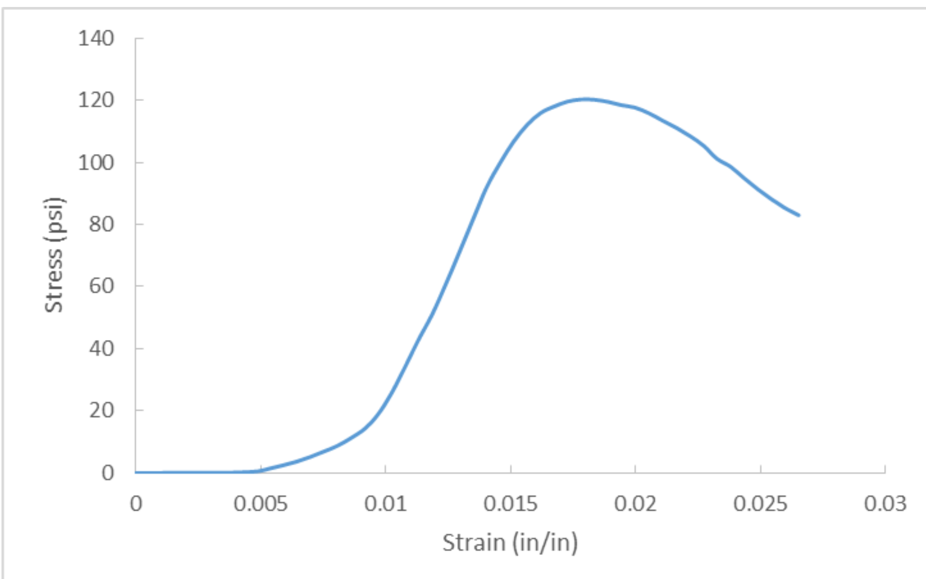

Figure A.162. D30-5, OC= 4.9%, CF= 400 pcf, T=28 Days.

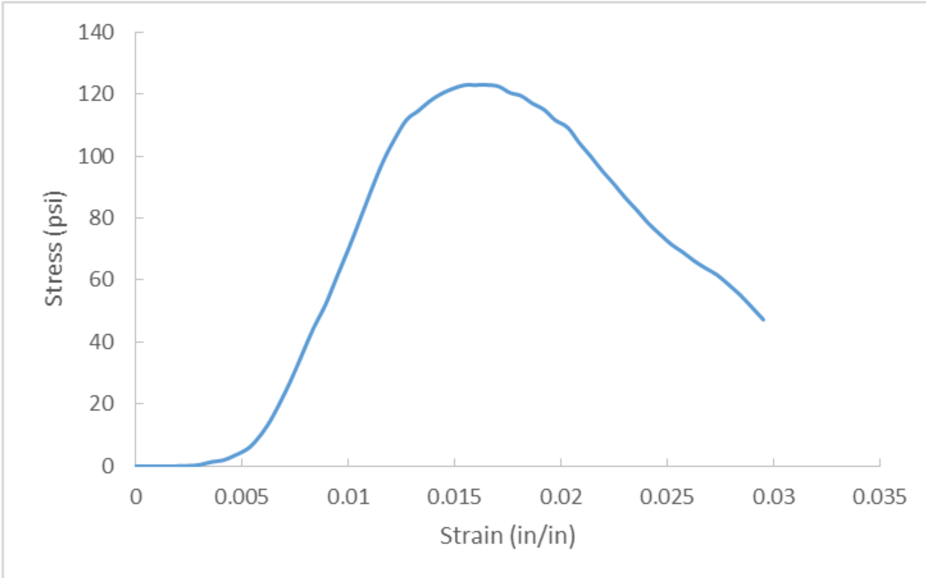

Figure A.163. D30-6, OC= 4.9%, CF= 400 pcf, T=28 Days.

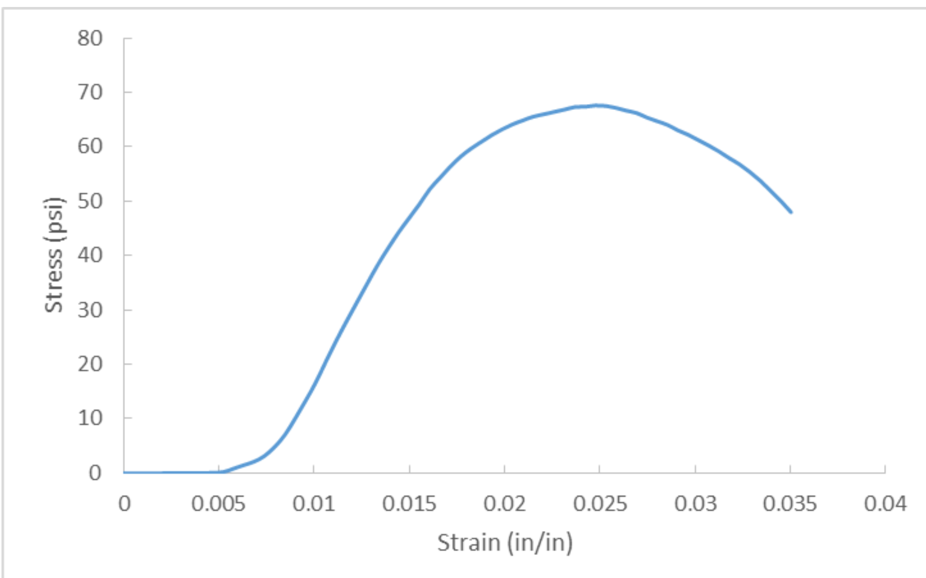

Figure A.164. D31-4, OC= 11.8%, CF= 400 pcf, T=28 Days.

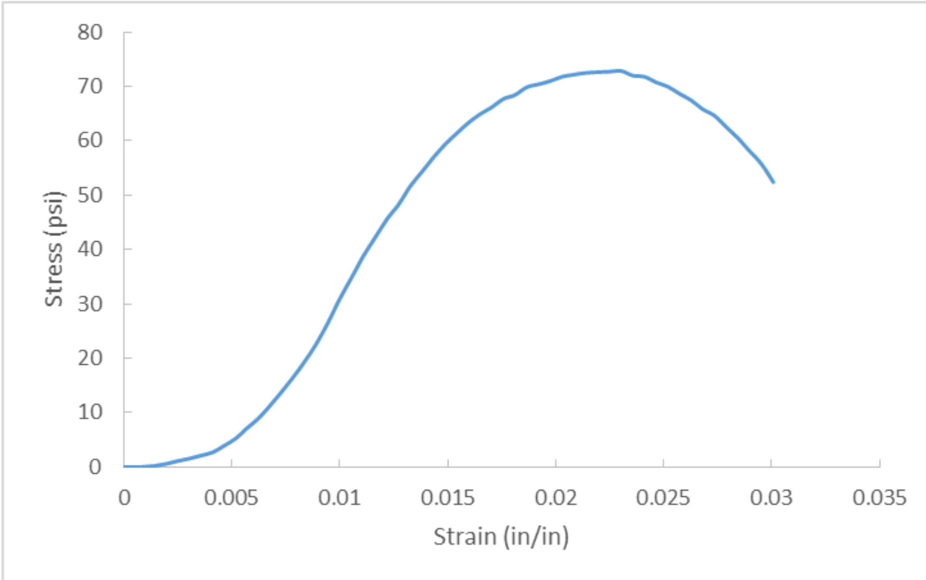

Figure A.165. D31-5, OC= 11.8%, CF= 400 pcf, T=28 Days.

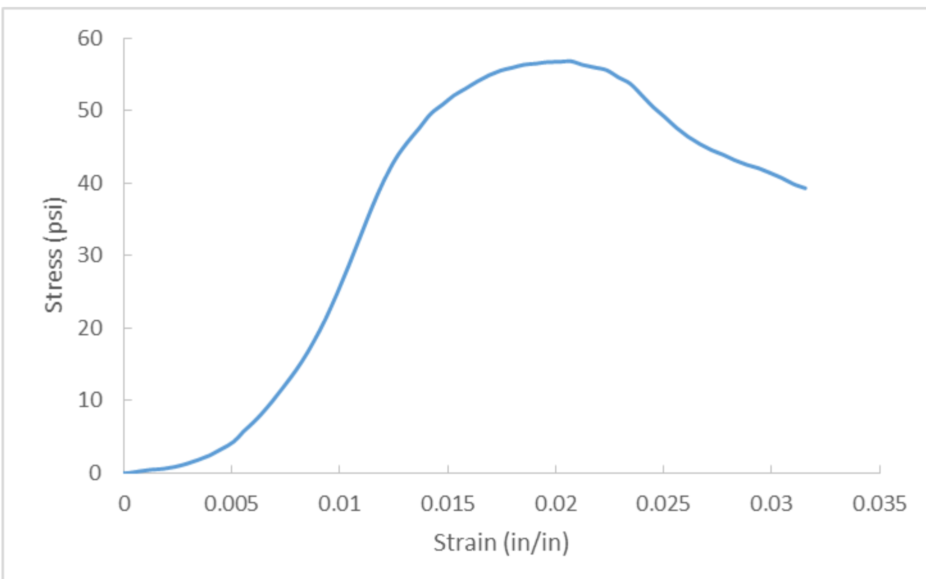

Figure A.166. D31-6, OC= 11.8%, CF= 400 pcf, T=28 Days.

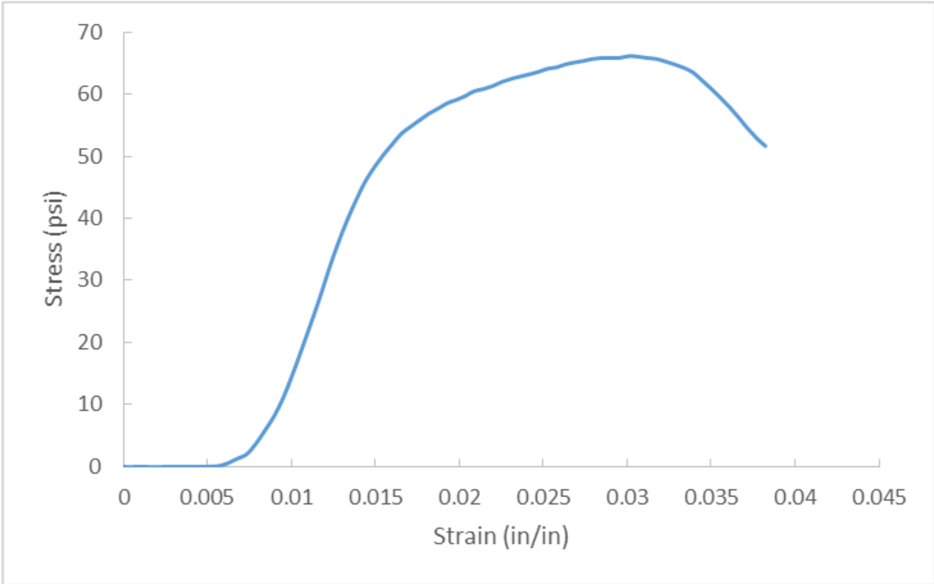

Figure A.167. D32-4, OC= 17.2%, CF= 400 pcf, T=28 Days.

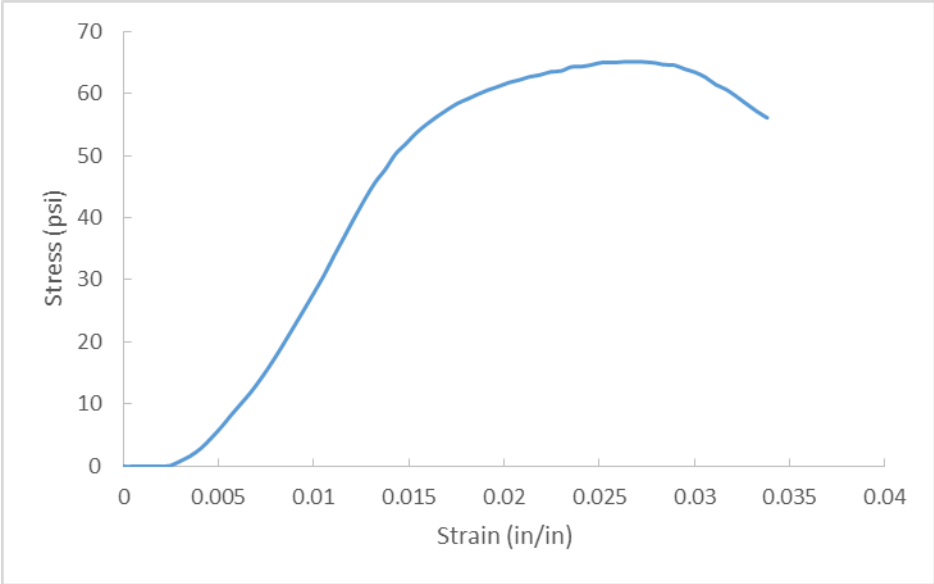

Figure A.168. D32-5, OC= 17.2%, CF= 400 pcf, T=28 Days.

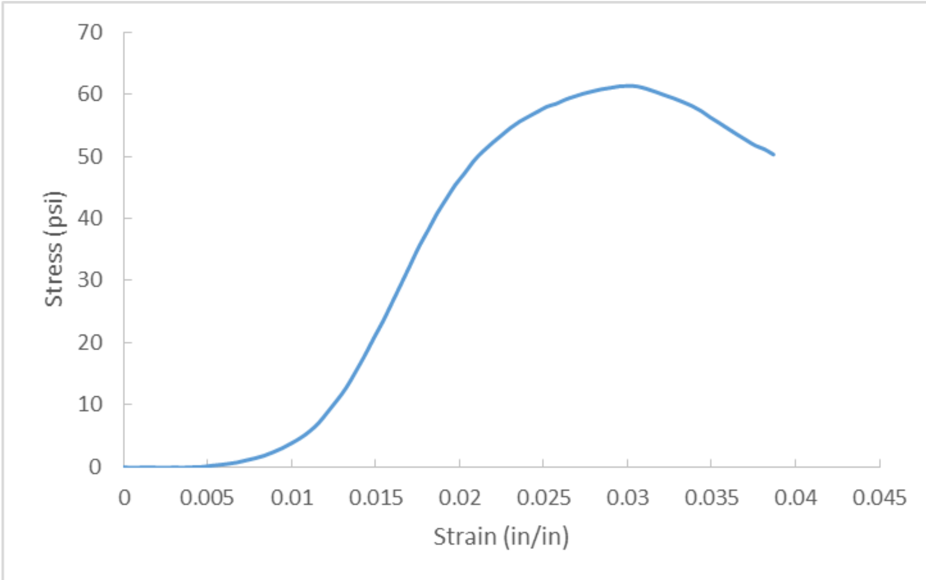

Figure A.169. D32-6, OC= 17.2%, CF= 400 pcf, T=28 Days.

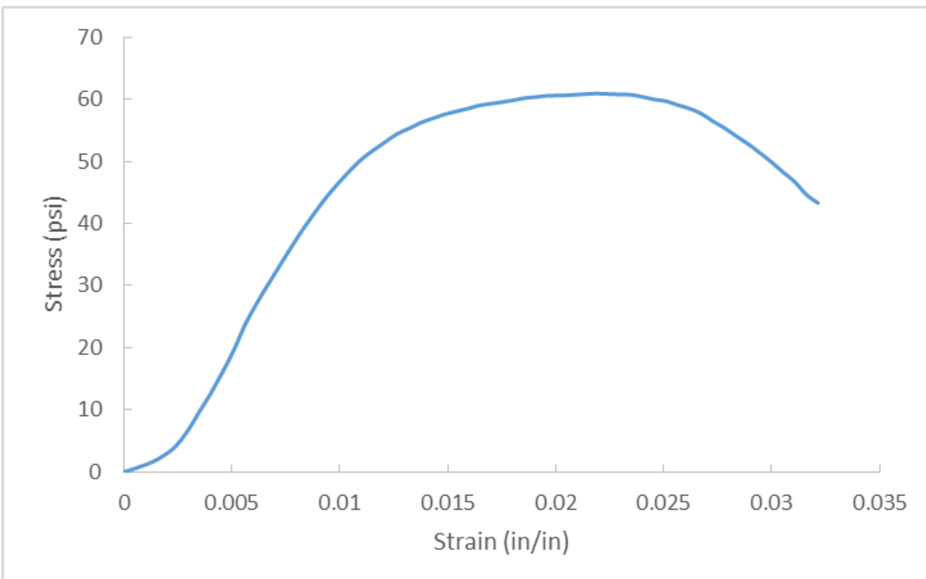

Figure A.170. D33-4, OC= 25.1%, CF= 400 pcf, T=28 Days.

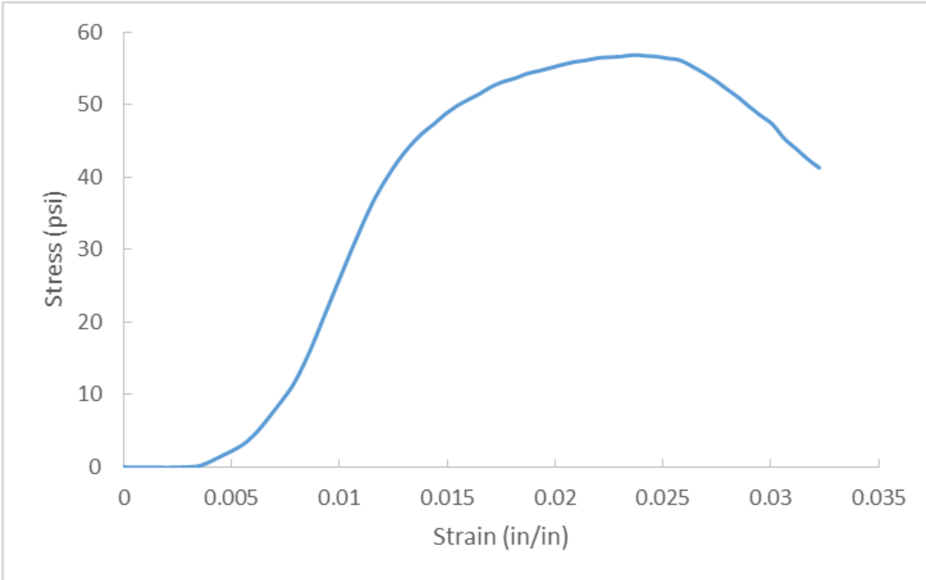

Figure A.171. D33-5, OC= 25.1%, CF= 400 pcf, T=28 Days.

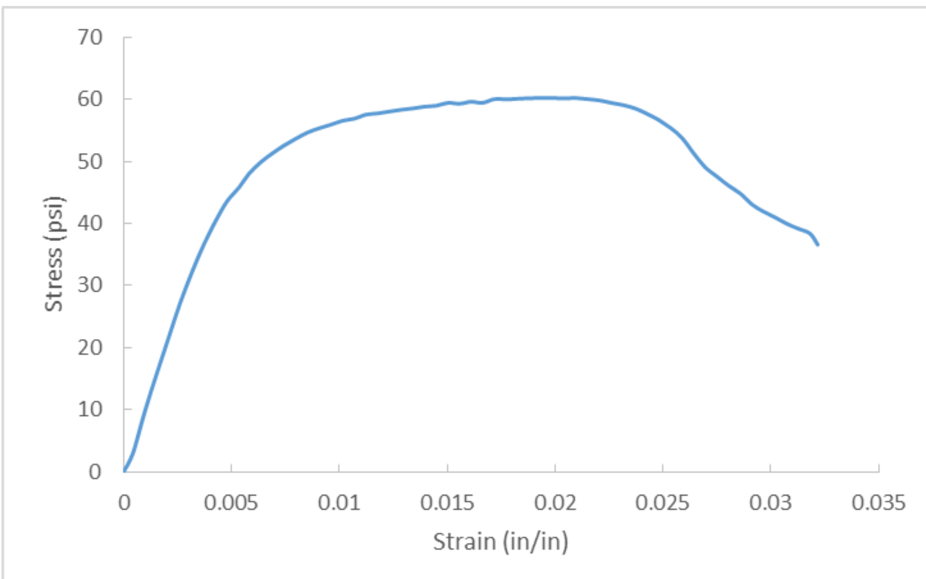

Figure A.172. D33-6, OC= 25.1%, CF= 400 pcf, T=28 Days.

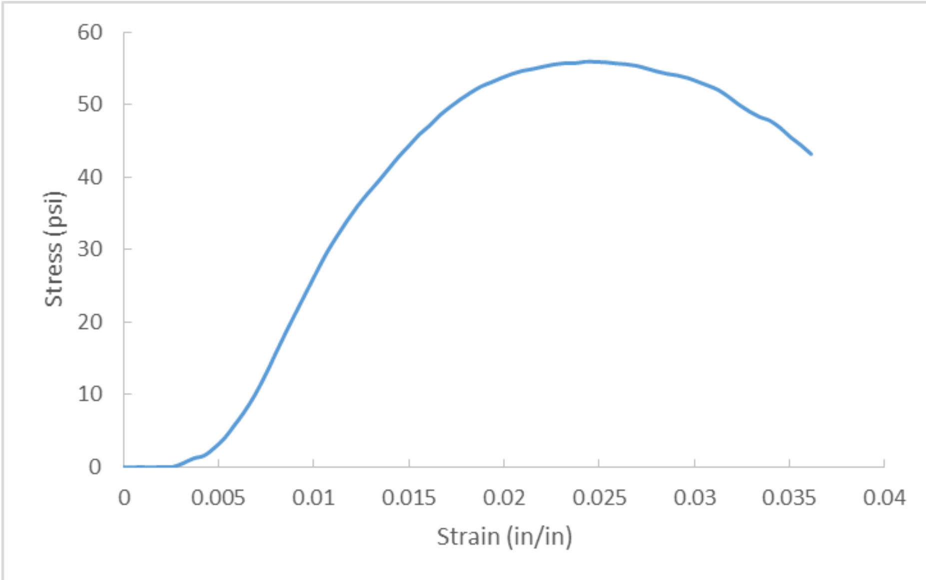

Figure A.173. D34-5, OC= 40.9%, CF= 400 pcf, T=28 Days.

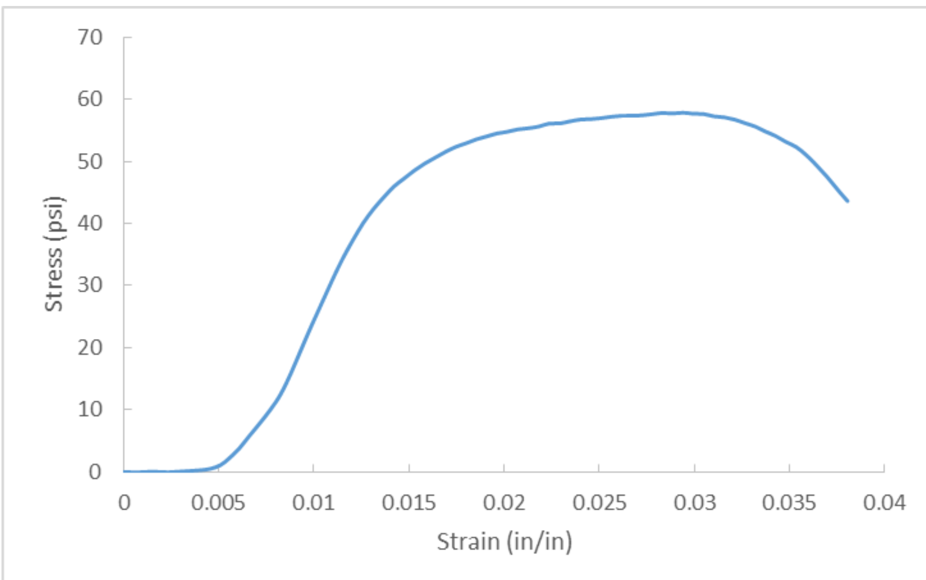

Figure A.174. D34-6, OC= 40.9%, CF= 400 pcf, T=28 Days.

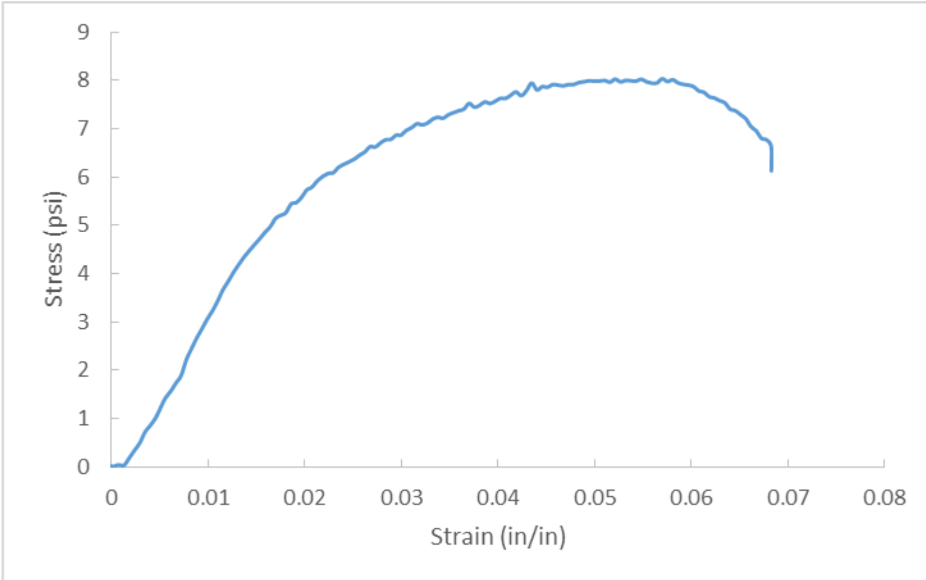

Figure A.175. D35-6, OC= 39.6%, CF= 200 pcf, T=28 Days.

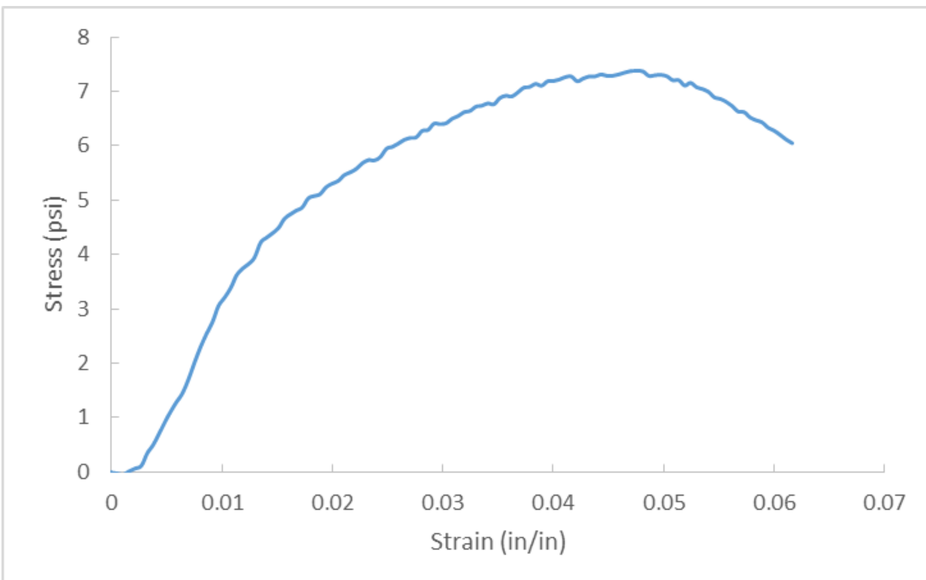

Figure A.176. D35-7, OC= 39.6%, CF= 200 pcf, T=28 Days.

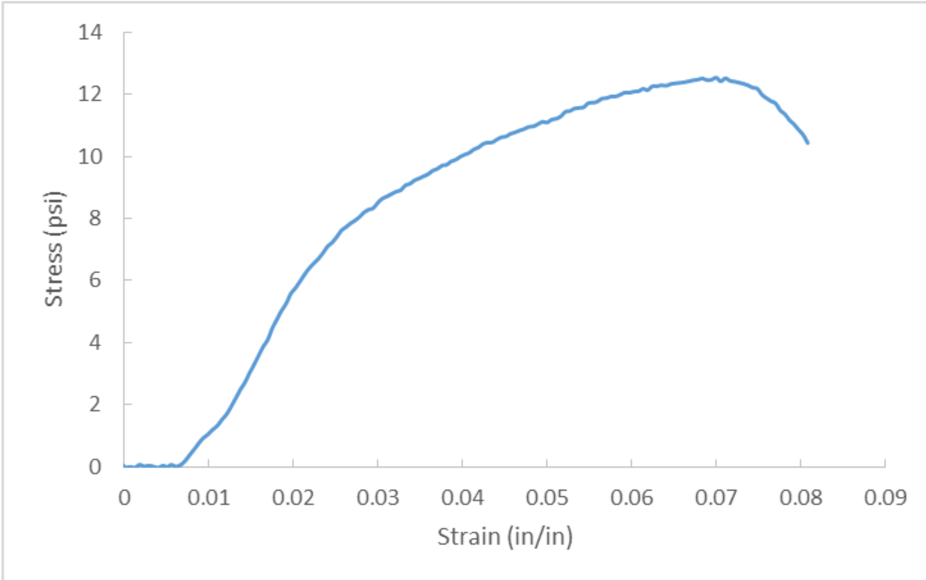

Figure A.177. D36.4, OC= 25.6%, CF= 200 pcf, T=28 Days.

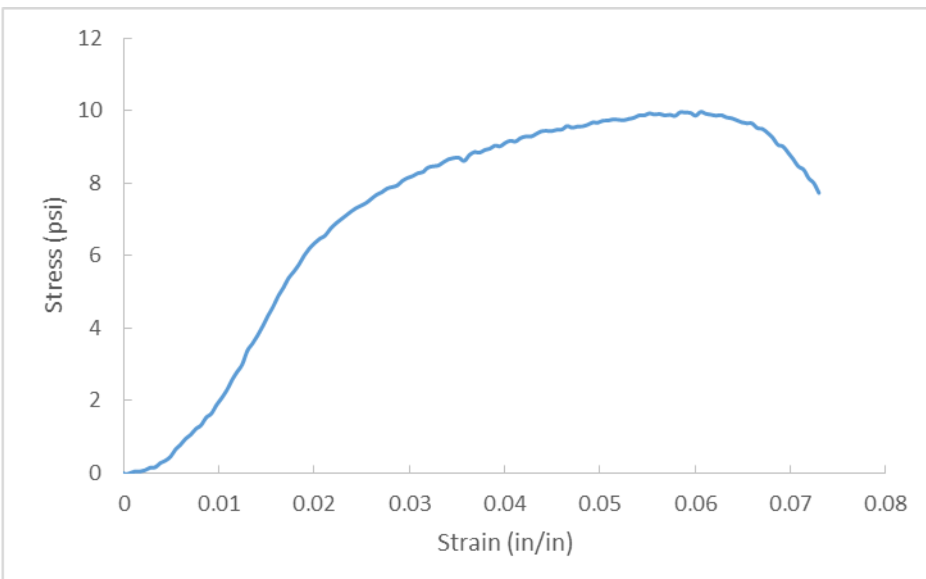

Figure A.178. D36.5, OC= 25.6%, CF= 200 pcf, T=28 Days.

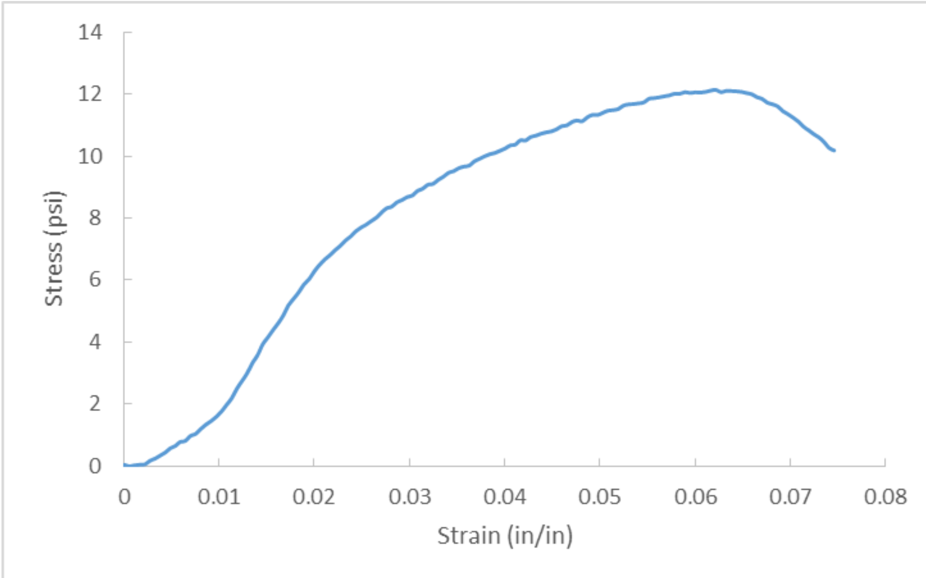

Figure A.179. D36.6, OC= 25.6%, CF= 200 pcf, T=28 Days.

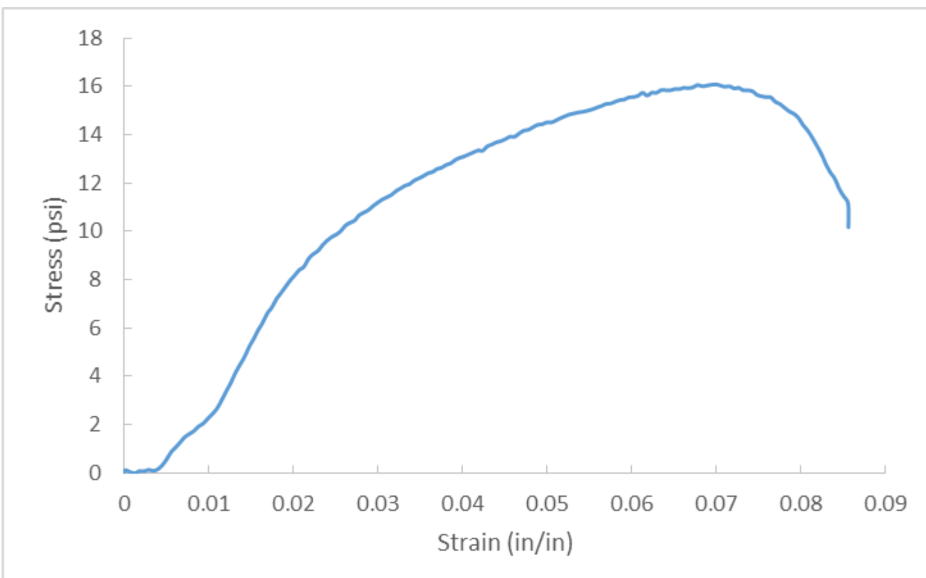

Figure A.180. D37-4, OC= 18.9%, CF= 200 pcf, T=28 Days.

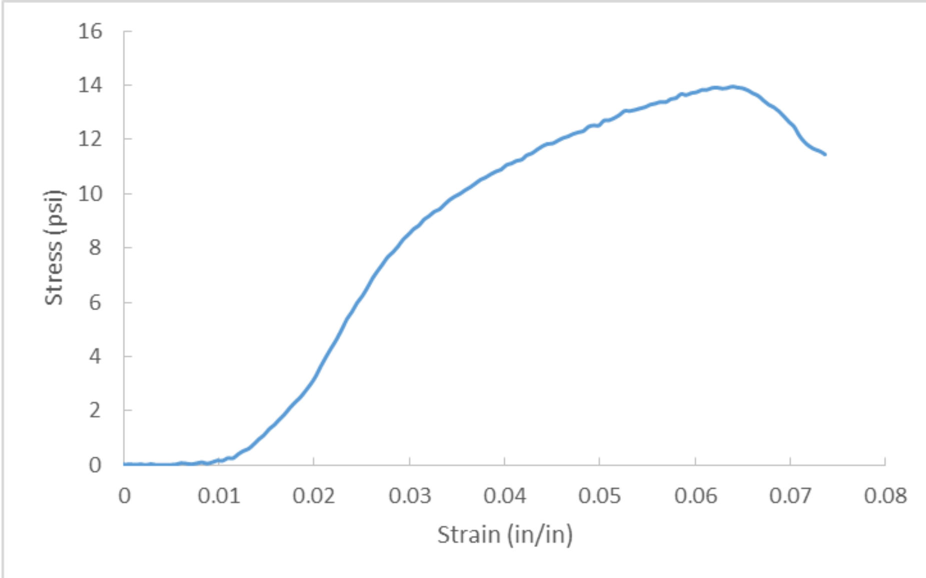

Figure A.181. D37-5, OC= 18.9%, CF= 200 pcf, T=28 Days.

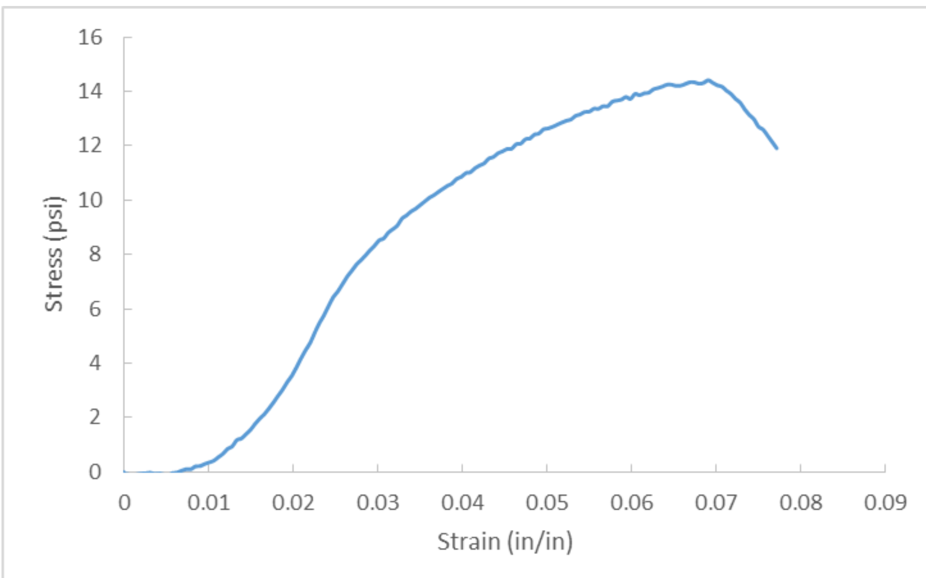

Figure A.182. D37-6, OC= 18.9%, CF= 200 pcf, T=28 Days.

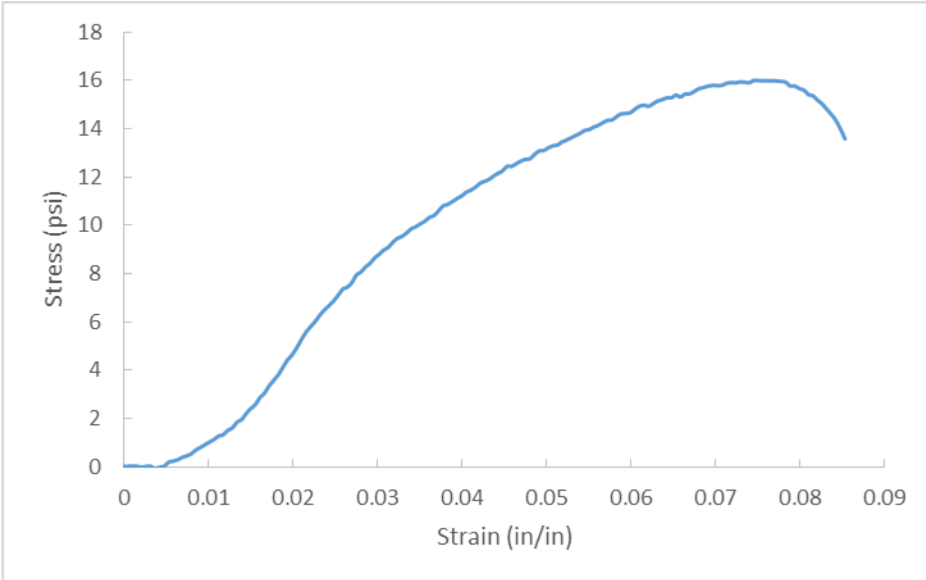

Figure A.183. D38-4, OC= 13.9%, CF= 200 pcf, T=28 Days.

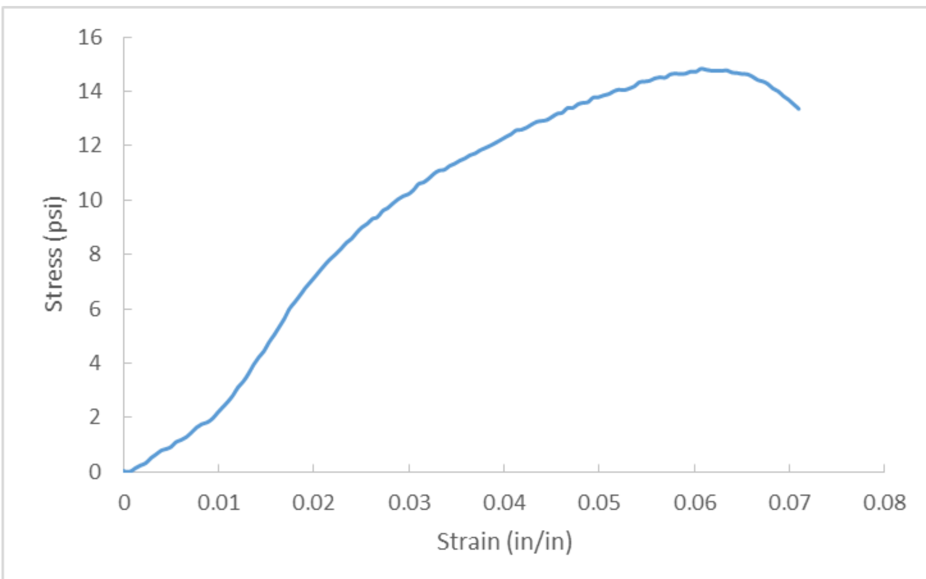

Figure A.184. D38-5, OC= 13.9%, CF= 200 pcf, T=28 Days.

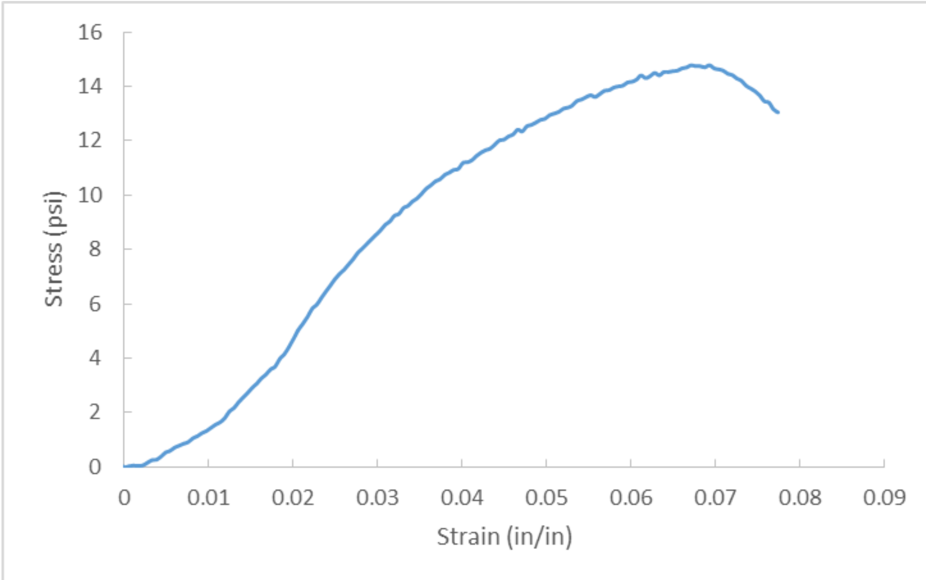

Figure A.185. D38-6, OC= 13.9%, CF= 200 pcf, T=28 Days.

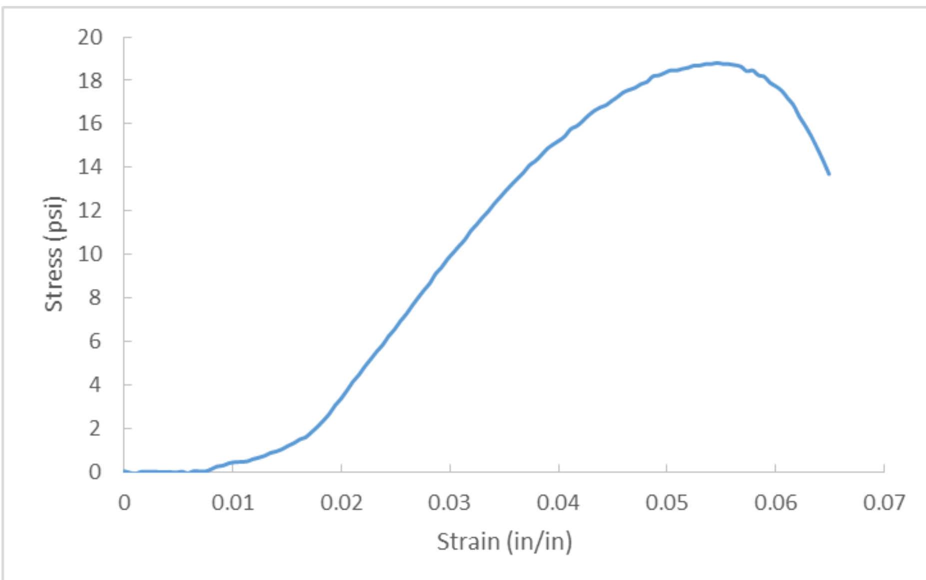

Figure A.186. D39-4, OC= 3.9%, CF= 200 pcf, T=28 Days.

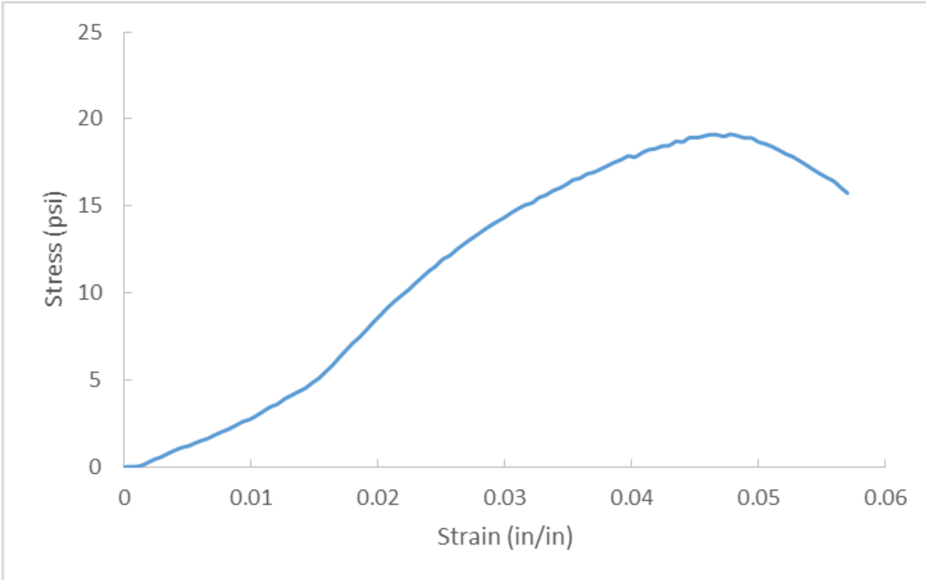

Figure A.187. D39-6, OC= 3.9%, CF= 200 pcf, T=28 Days.

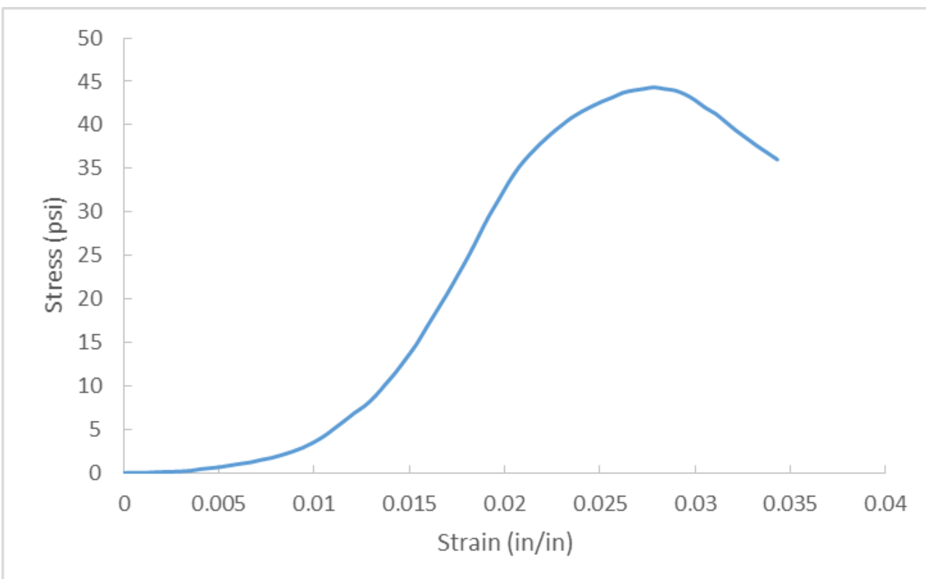

Figure A.188. D44-1, OC= 4.1%, CF= 200 pcf, T=28 Days.

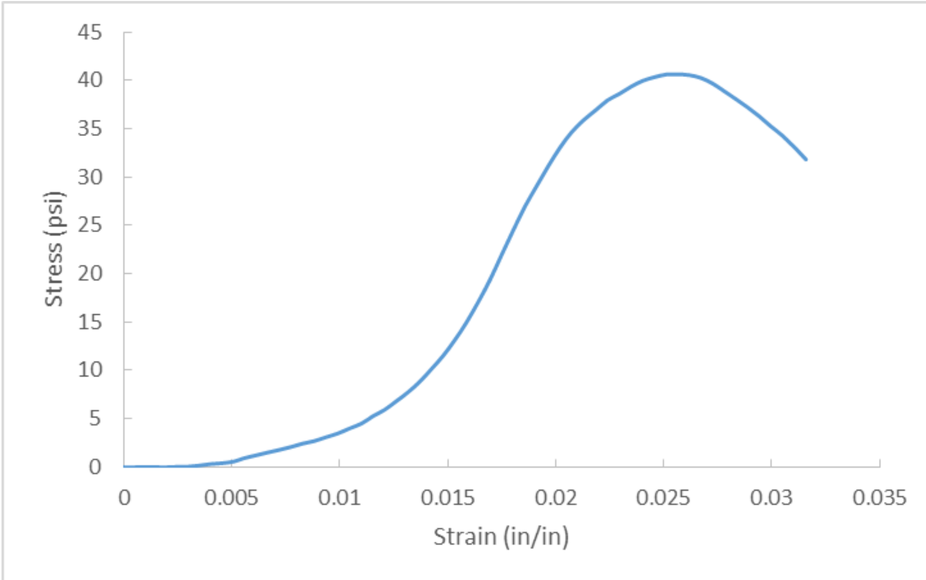

Figure A.189. D44-2, OC= 4.1%, CF= 200 pcf, T=28 Days.

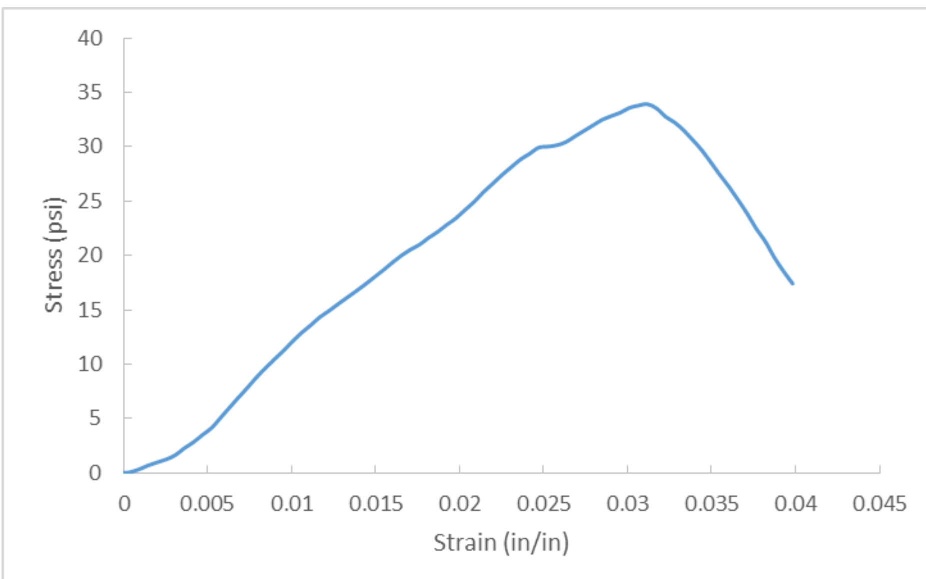

Figure A.190. D44-3, OC= 4.1%, CF= 200 pcf, T=28 Days.

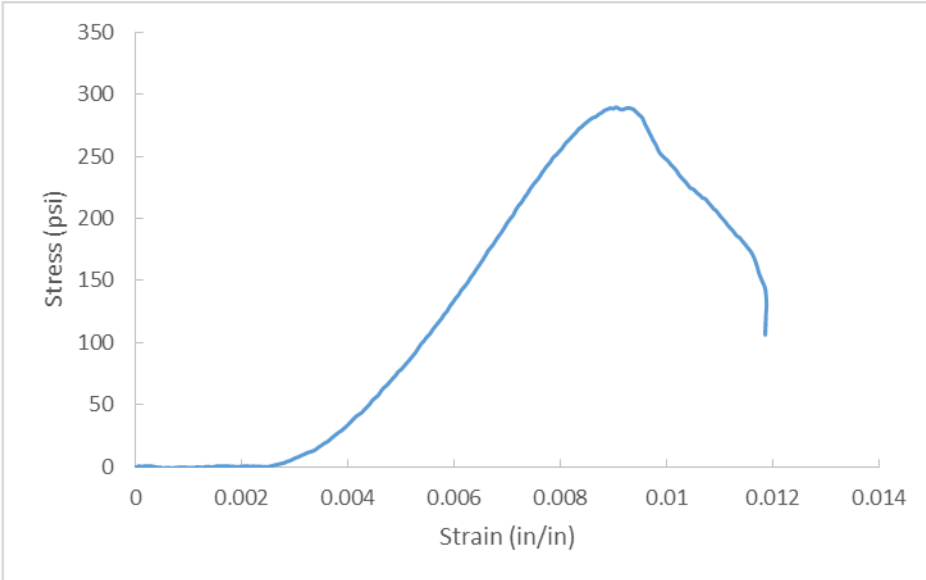

Figure A.191. D45-4, OC= 0.0%, CF= 400 pcf, T=28 Days.

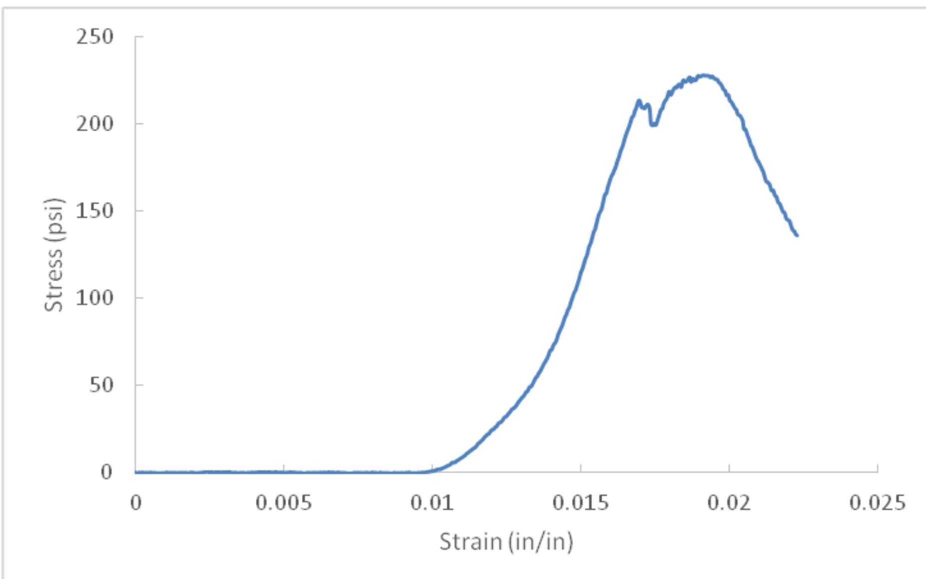

Figure A.192. D45-5, OC= 0.0%, CF= 400 pcf, T=28 Days.

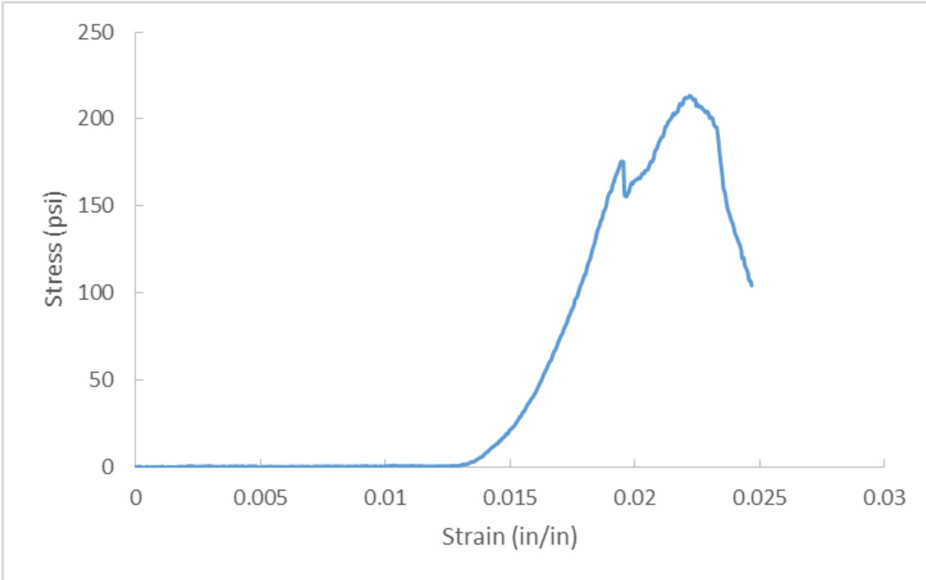

Figure A.193. D45-6, OC= 0.0%, CF= 400 pcf, T=28 Days.

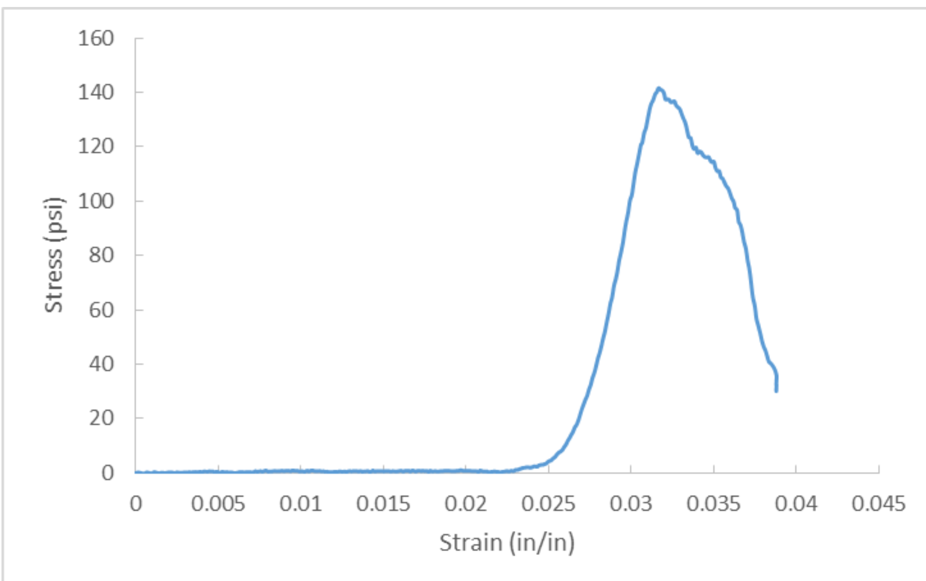

Figure A.194. D46-5, OC= 0.0%, CF= 300 pcf, T=28 Days.

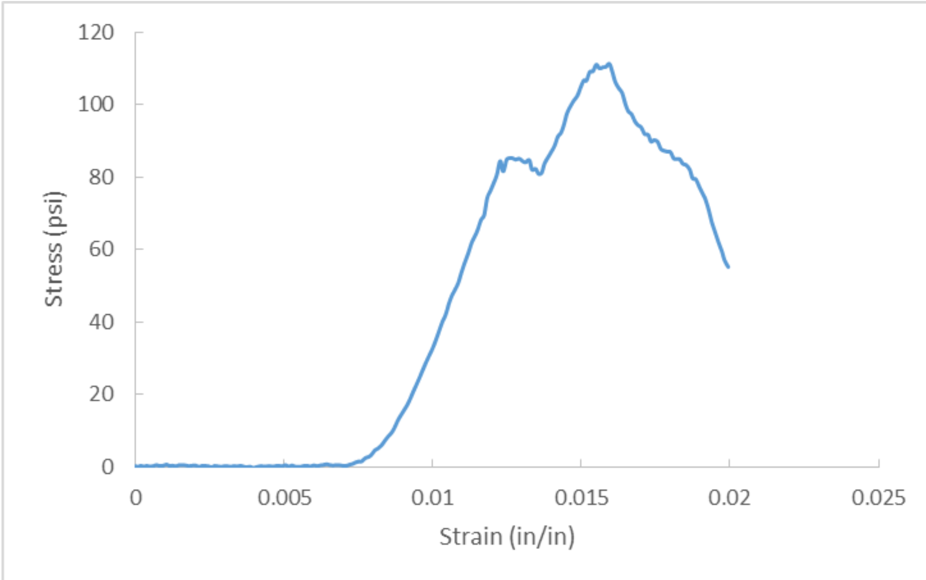

Figure A.195. D46-6, OC= 0.0%, CF= 300 pcf, T=28 Days.

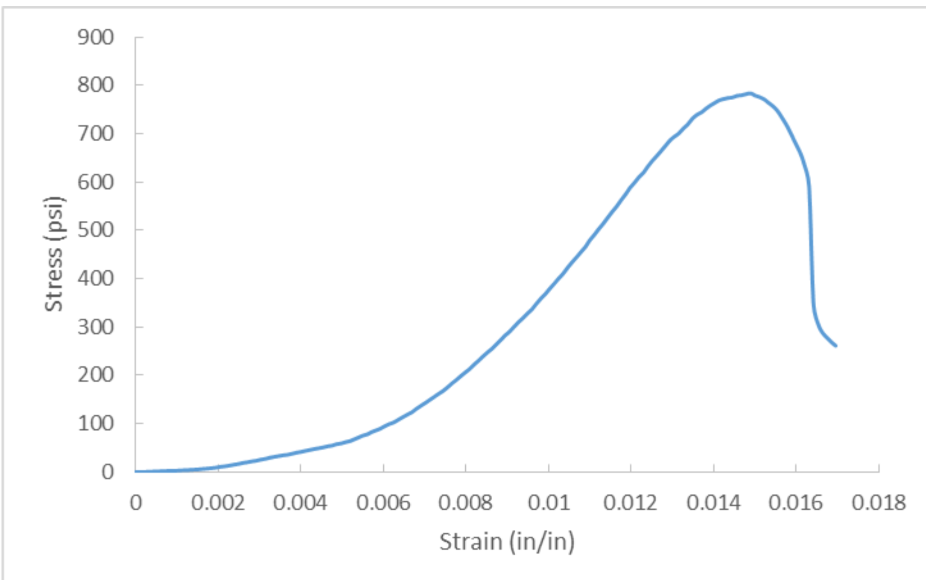

Figure A.196. D47-4, OC= 0.0%, CF= 400 pcf, T=28 Days.

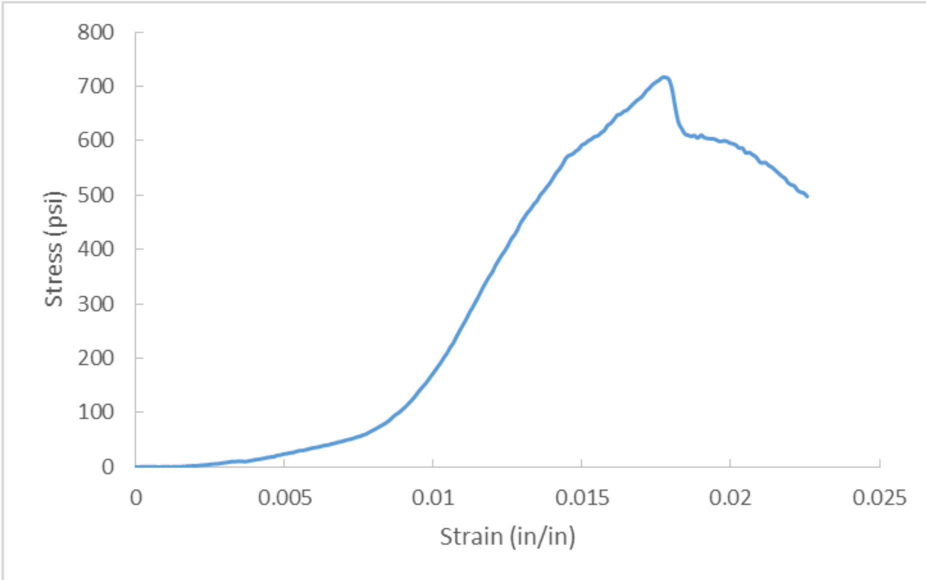

Figure A.197. D47-5, OC= 0.0%, CF= 400 pcf, T=28 Days.

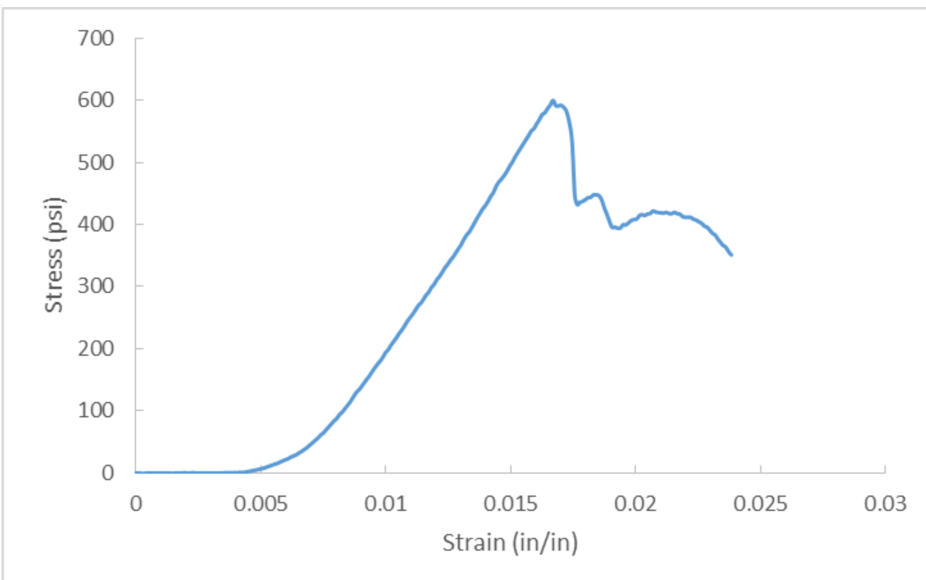

Figure A.198. D47-6, OC= 0.0%, CF= 400 pcf, T=28 Days.

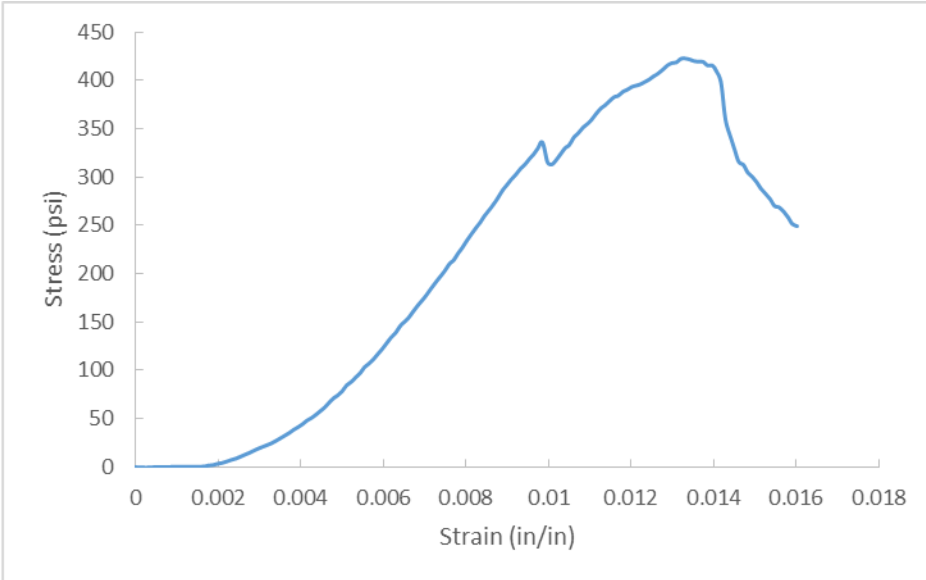

Figure A.199. D48-4, OC= 0.0%, CF= 300 pcf, T=28 Days.

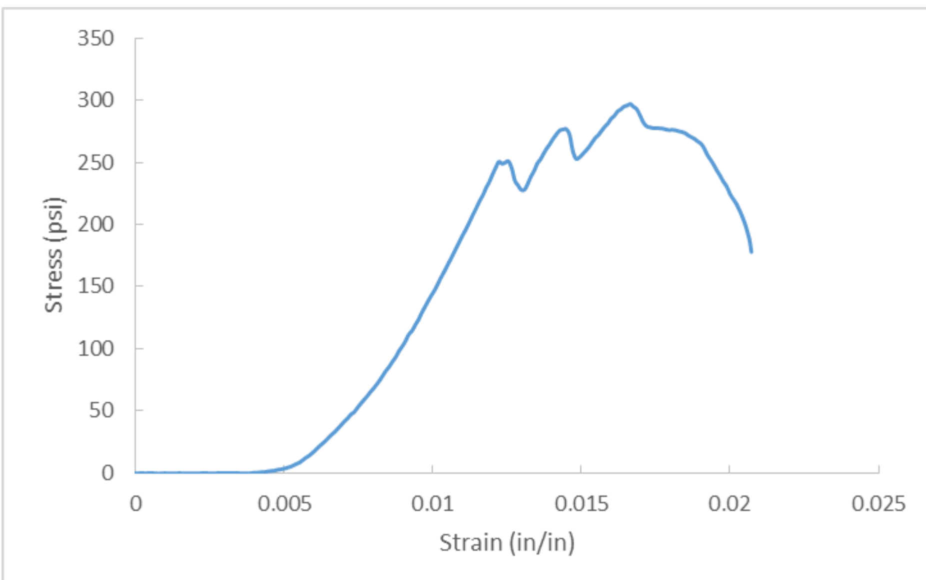

Figure A.200. D48-5, OC= 0.0%, CF= 300 pcf, T=28 Days.

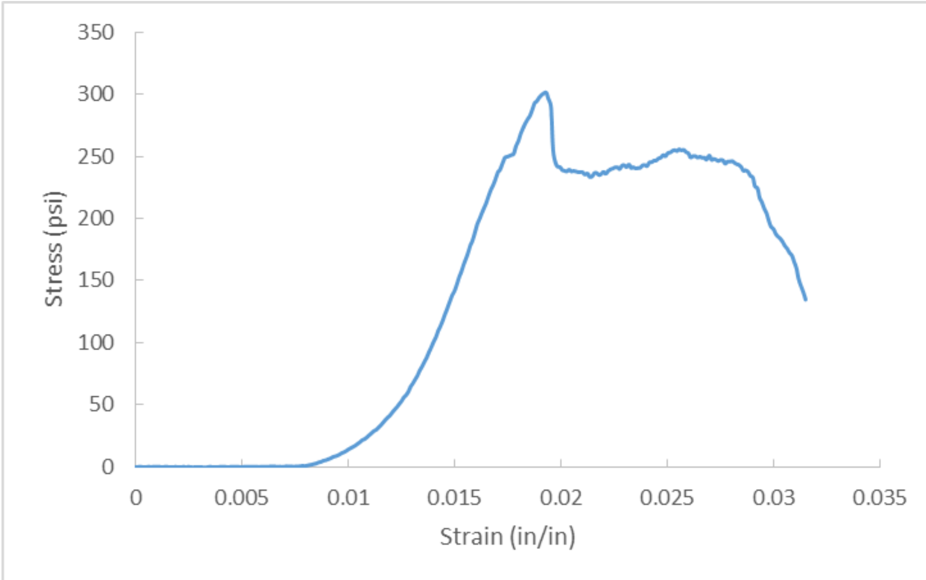

Figure A.201. D48-6, OC= 0.0%, CF= 300 pcf, T=28 Days.

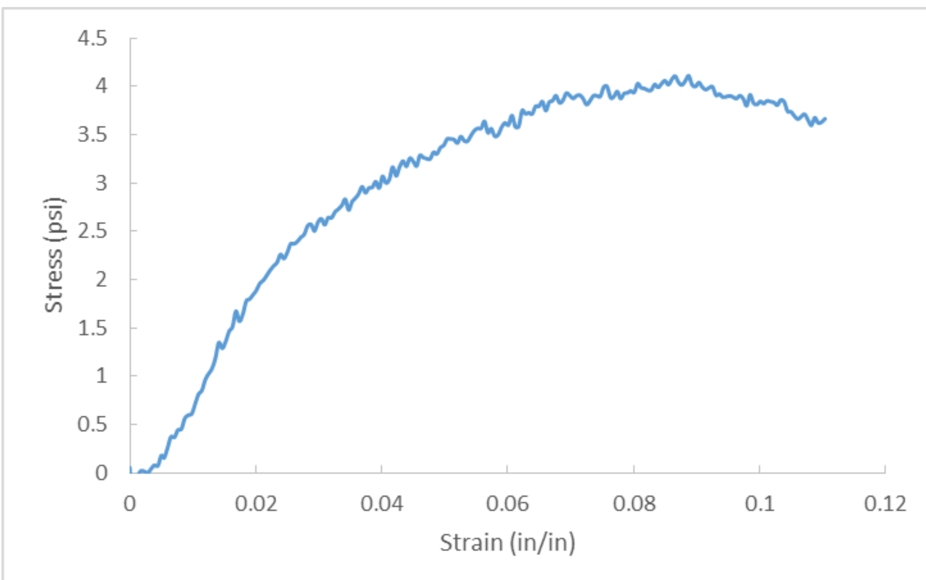

Figure A.202. W49-4, OC= 43.8%, CF= 300 pcf, T=28 Days.

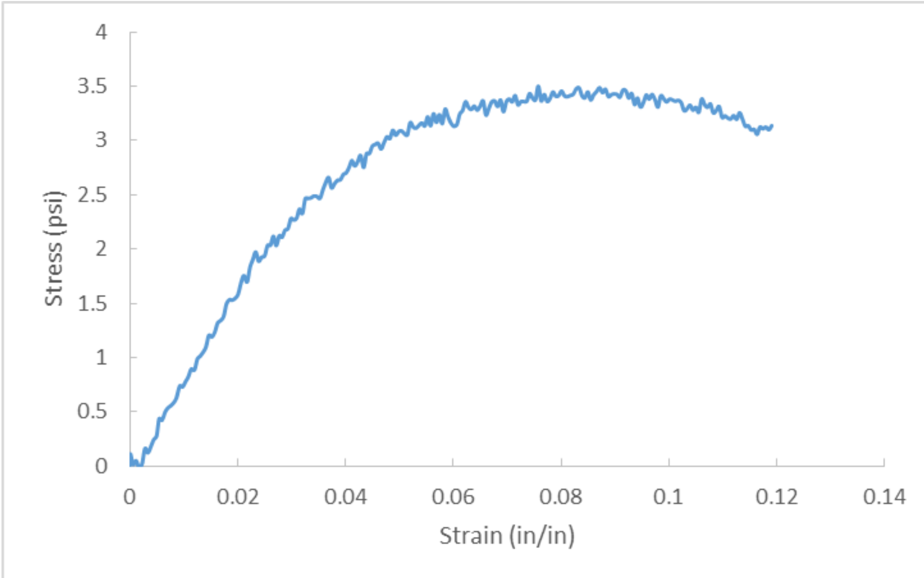

Figure A.203. W49-5, OC= 43.8%, CF= 300 pcf, T=28 Days.

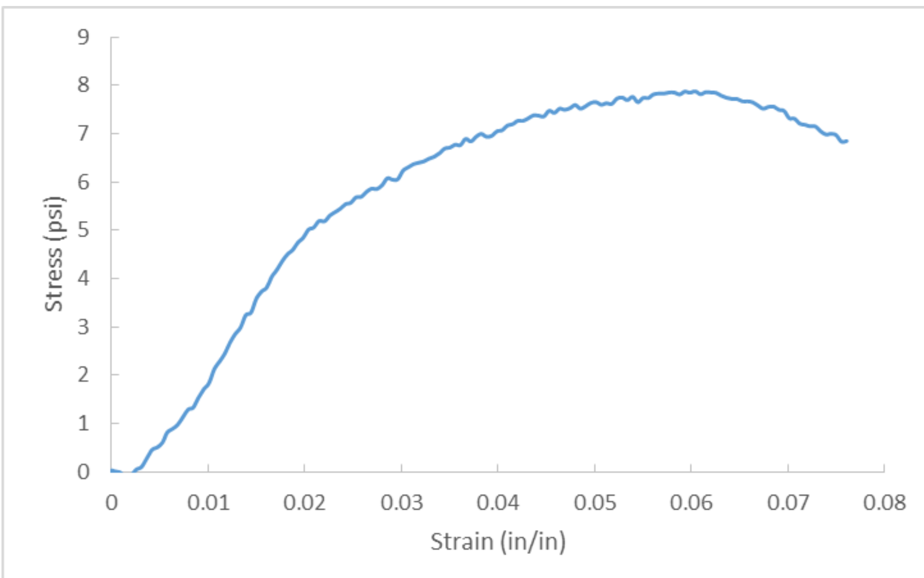

Figure A.204. W50-4, OC= 43.8%, CF= 400 pcf, T=28 Days.

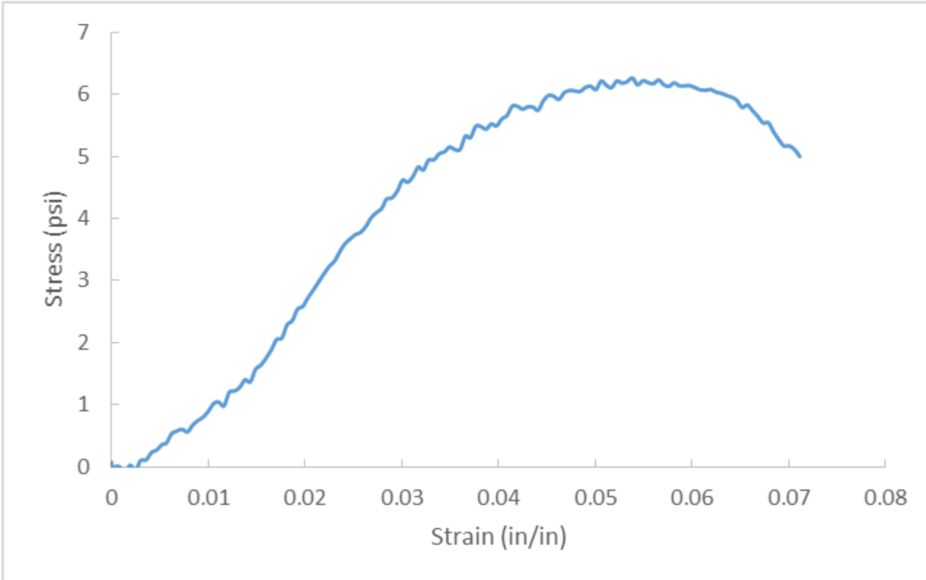

Figure A.205. W50-5, OC= 43.8%, CF= 400 pcf, T=28 Days.

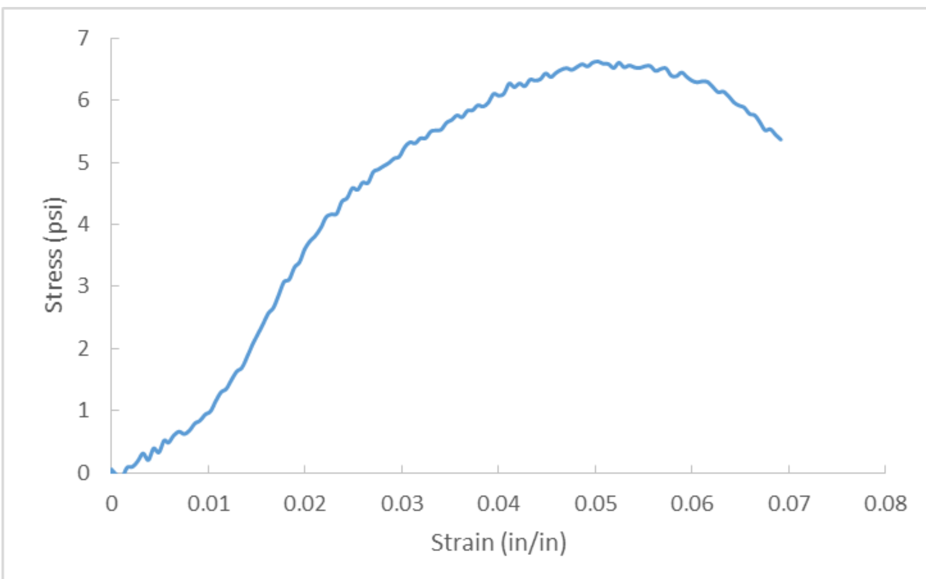

Figure A.206. W50-6, OC= 43.8%, CF= 400 pcf, T=28 Days.

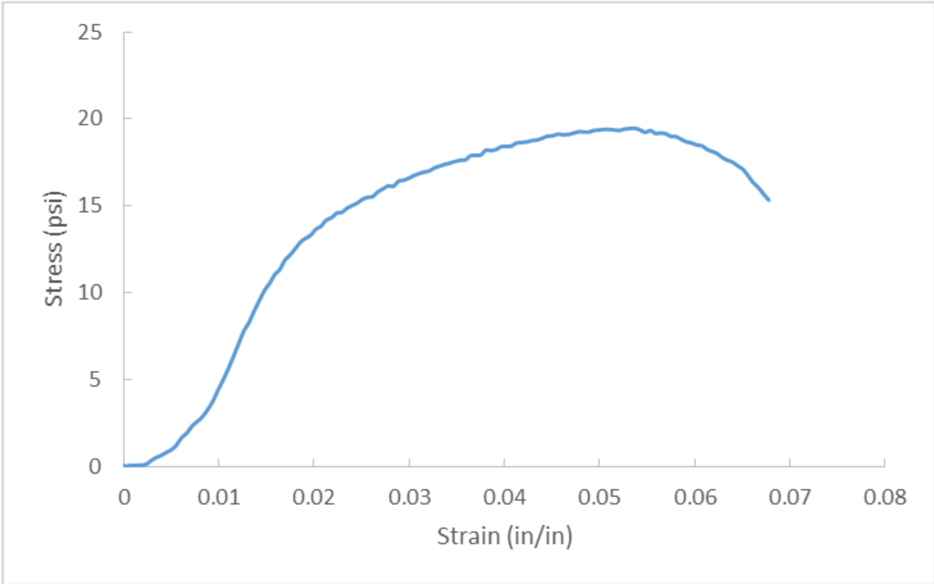

Figure A.207. W51.4, OC= 43.8%, CF= 500 pcf, T=28 Days.

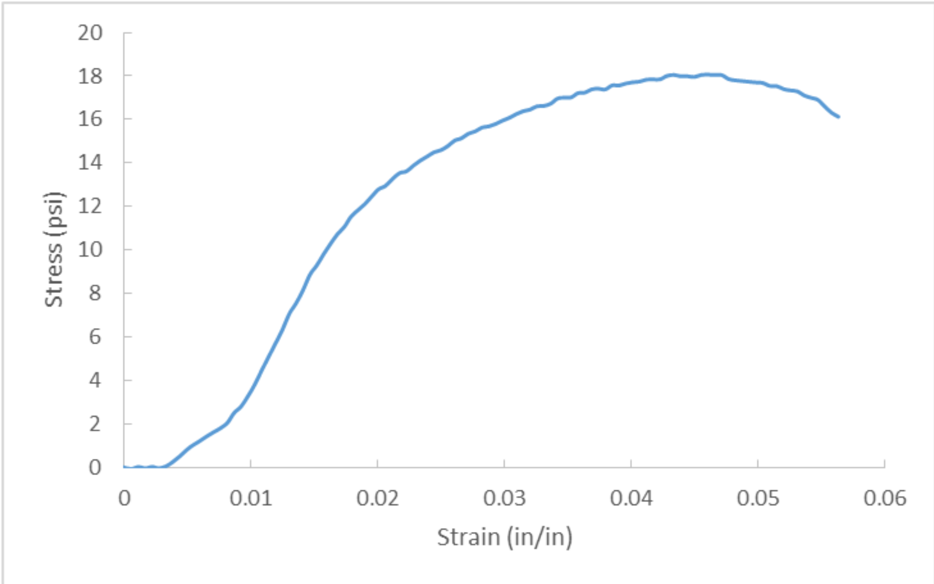

Figure A.208. W51.5, OC= 43.8%, CF= 500 pcf, T=28 Days.

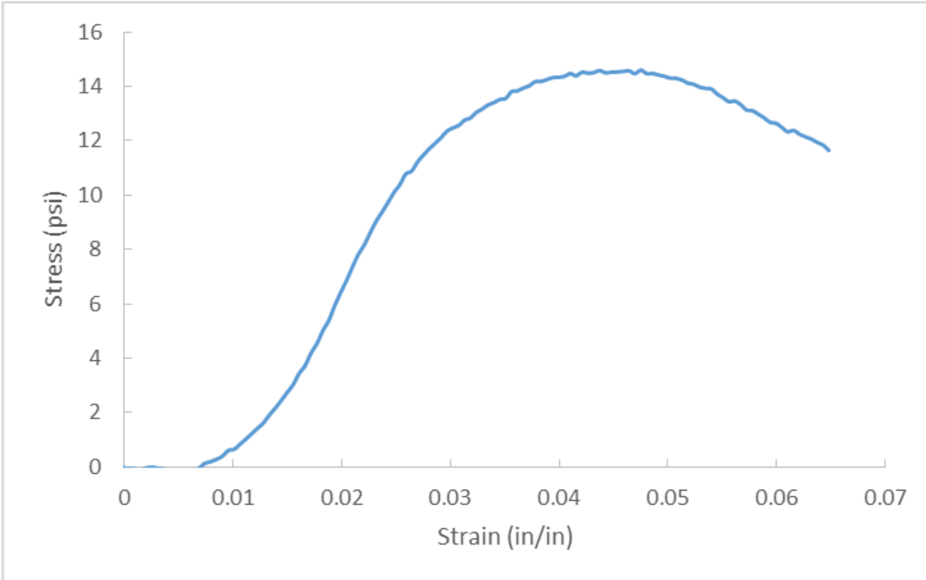

Figure A.209. W51.6, OC= 43.8%, CF= 500 pcf, T=28 Days.

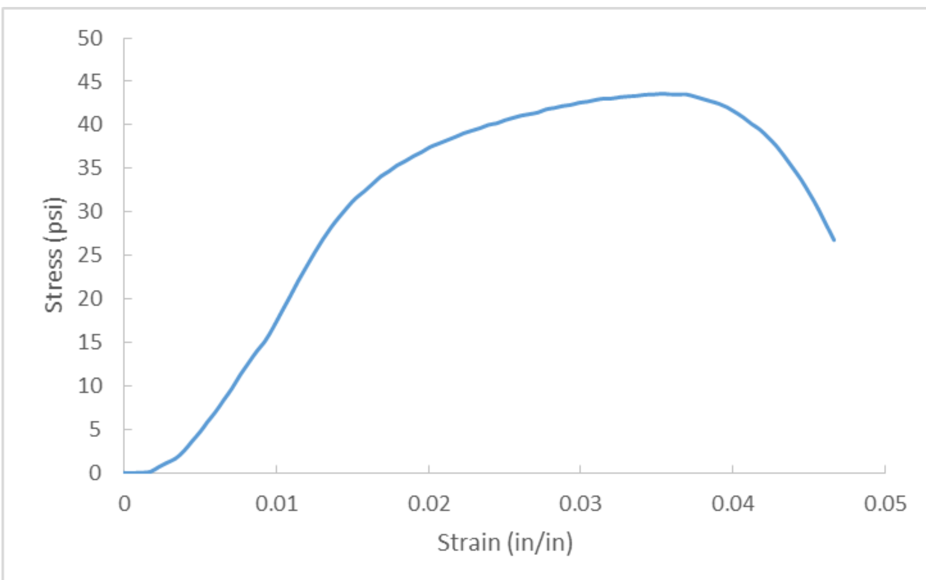

Figure A.210. D55.1, OC= 42.1%, CF= 500 pcf, T=28 Days.

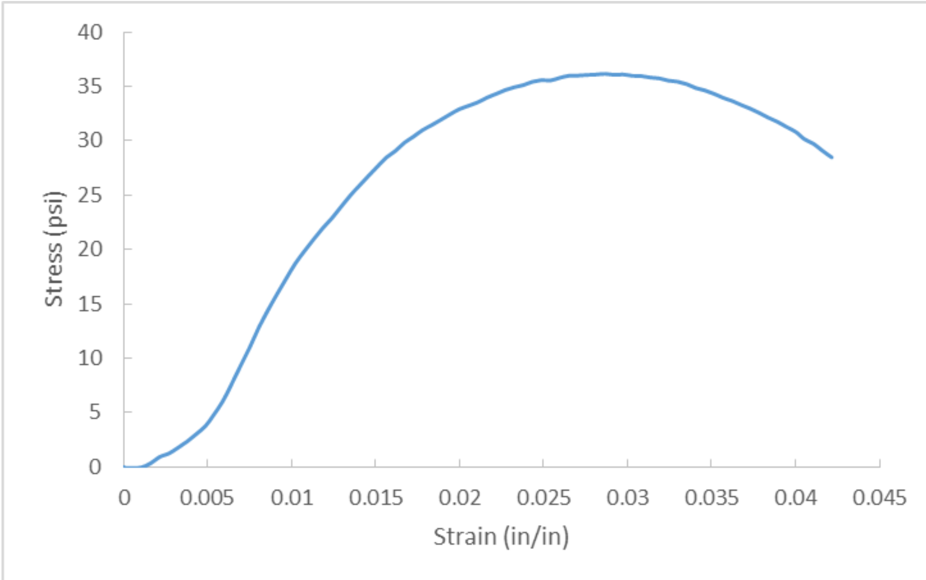

Figure A.211. D55.2, OC= 42.1%, CF= 500 pcf, T=28 Days.

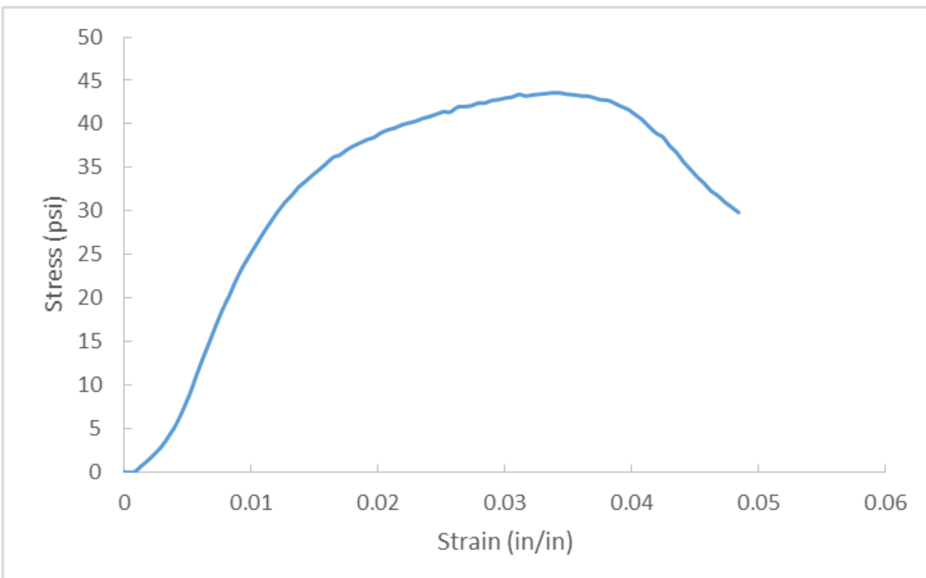

Figure A.212. D55.3, OC= 42.1%, CF= 500 pcf, T=28 Days.

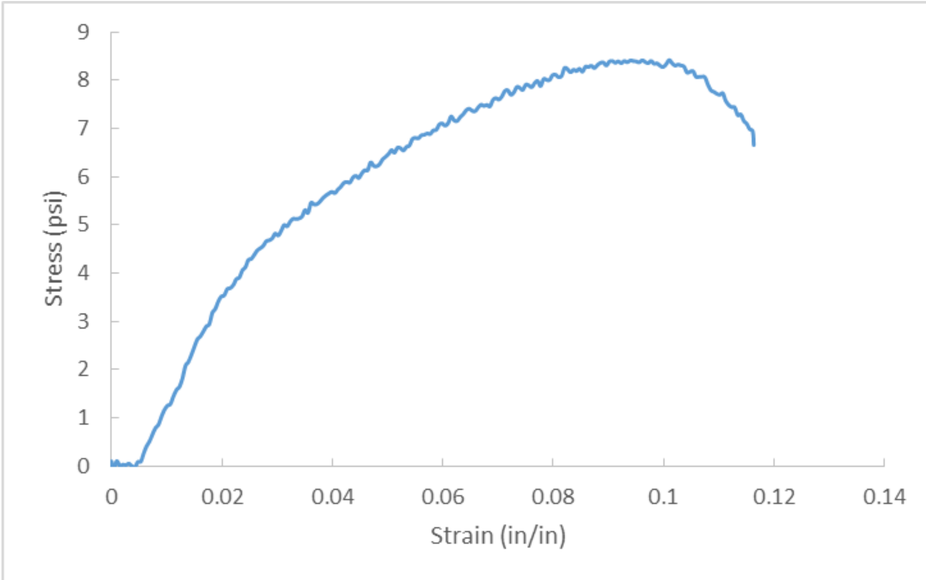

Figure A.213. D57.1, OC= 42.1%, CF= 500 pcf, T=28 Days.

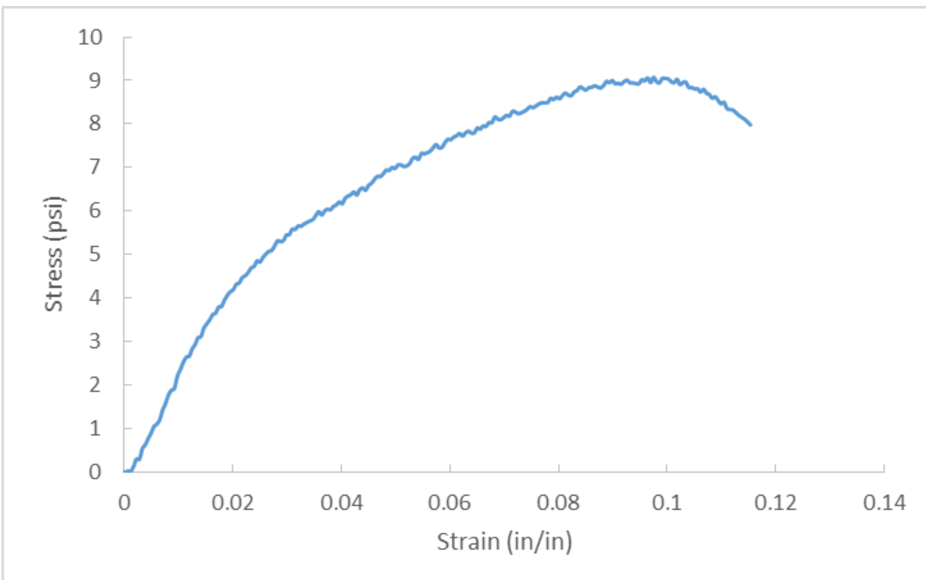

Figure A.214. D57.2, OC= 42.1%, CF= 500 pcf, T=28 Days.

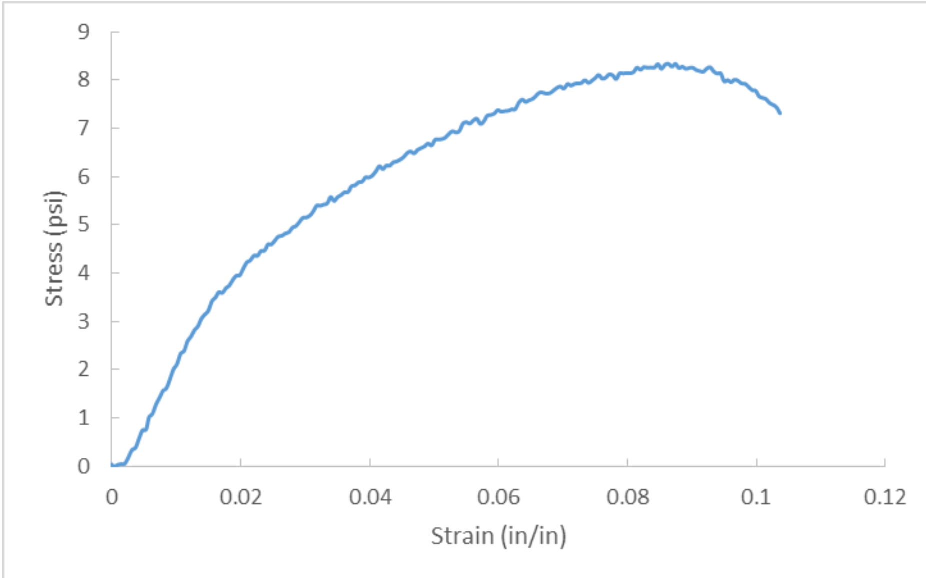

Figure A.215. D57.3, OC= 42.1%, CF= 500 pcf, T=28 Days.

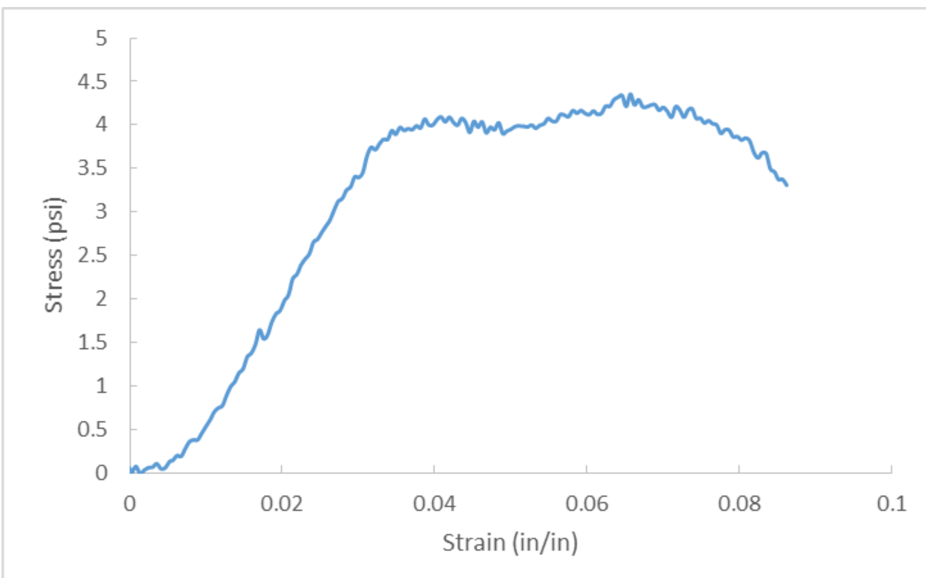

Figure A.216. D3-7, OC= 66.4%, CF= 400 pcf, T=56 Days.

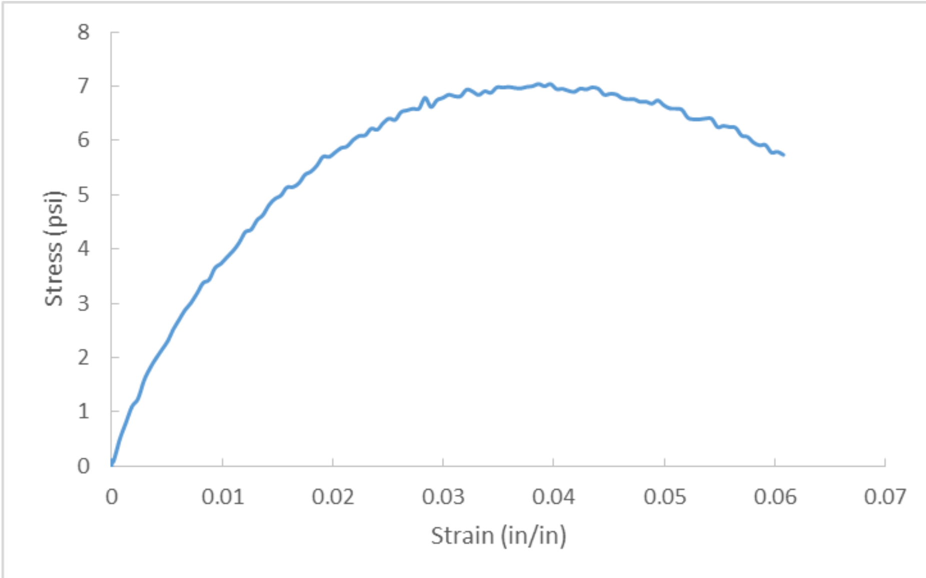

Figure A.217. D3-8, OC= 66.4%, CF= 400 pcf, T=56 Days.

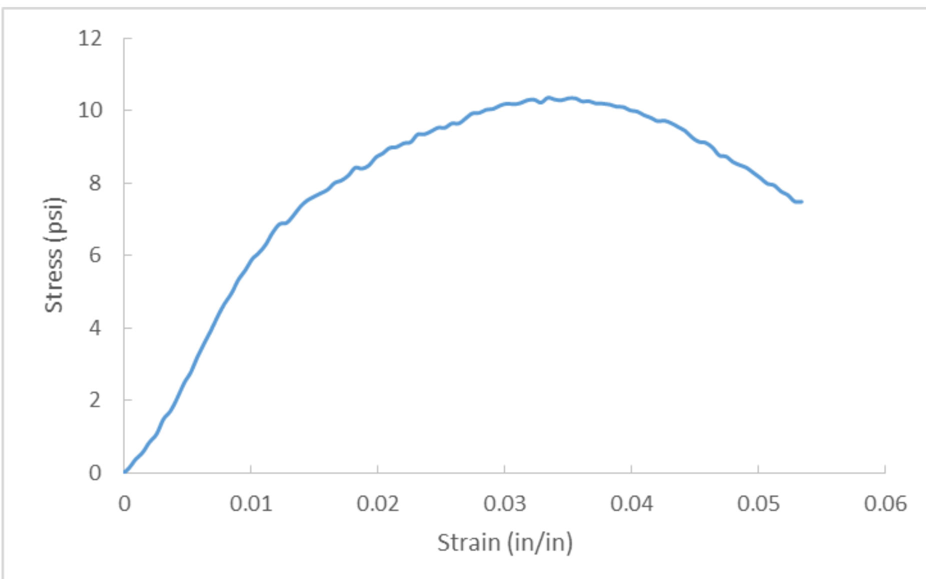

Figure A.218. D3-9, OC= 66.4%, CF= 400 pcf, T=56 Days.

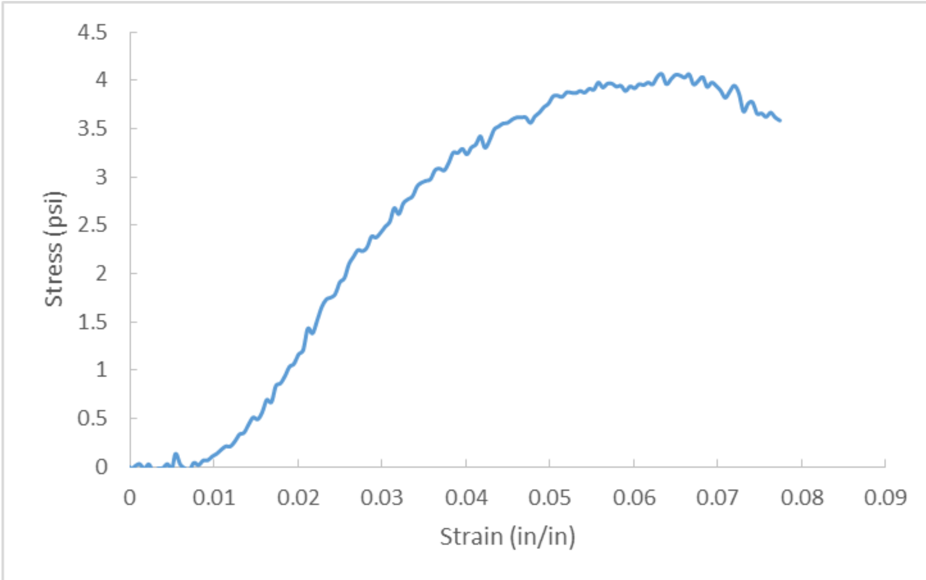

Figure A.219. D4-7, OC= 66.4%, CF= 200 pcf, T=56 Days.

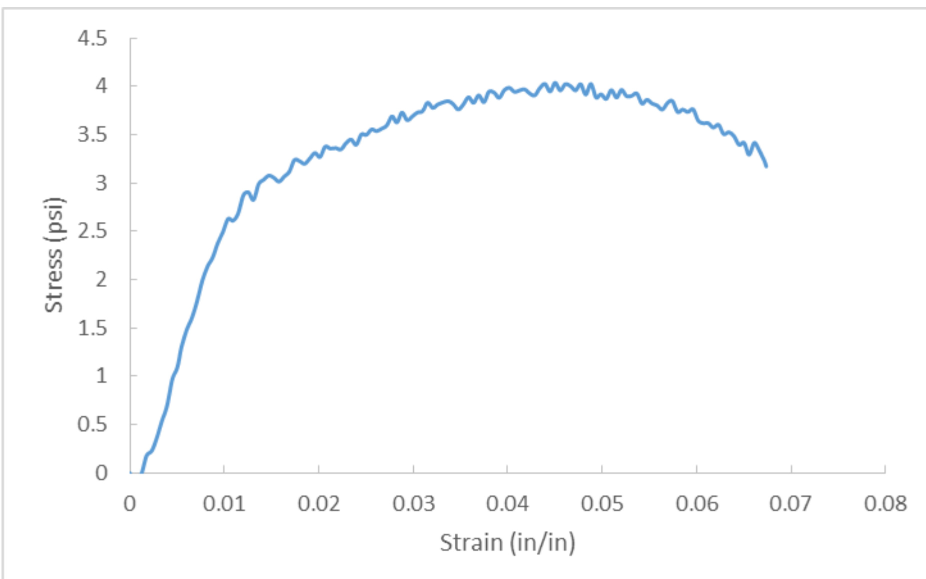

Figure A.220. D4-9, OC= 66.4%, CF= 200 pcf, T=56 Days.

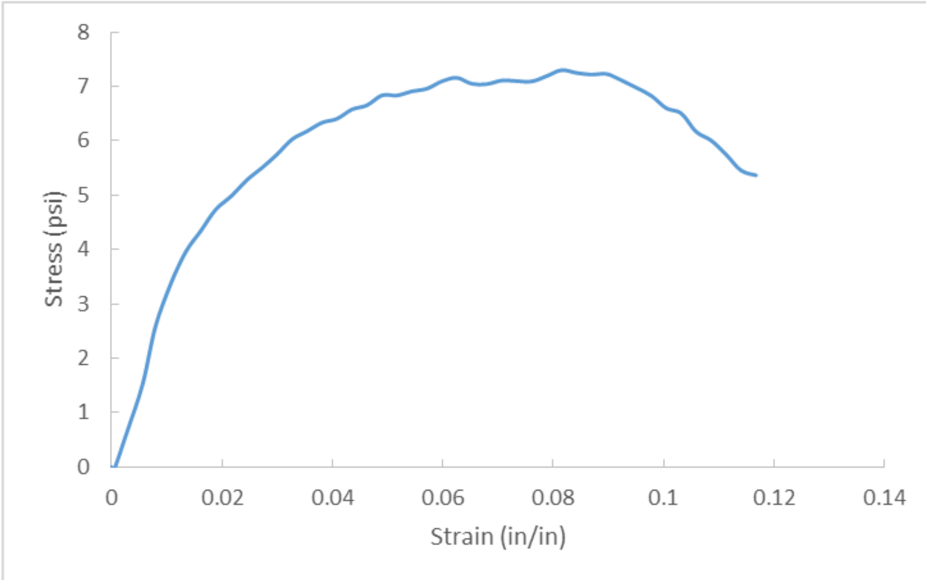

Figure A.221. D8-7, OC= 65.9%, CF= 300 pcf, T=56 Days.

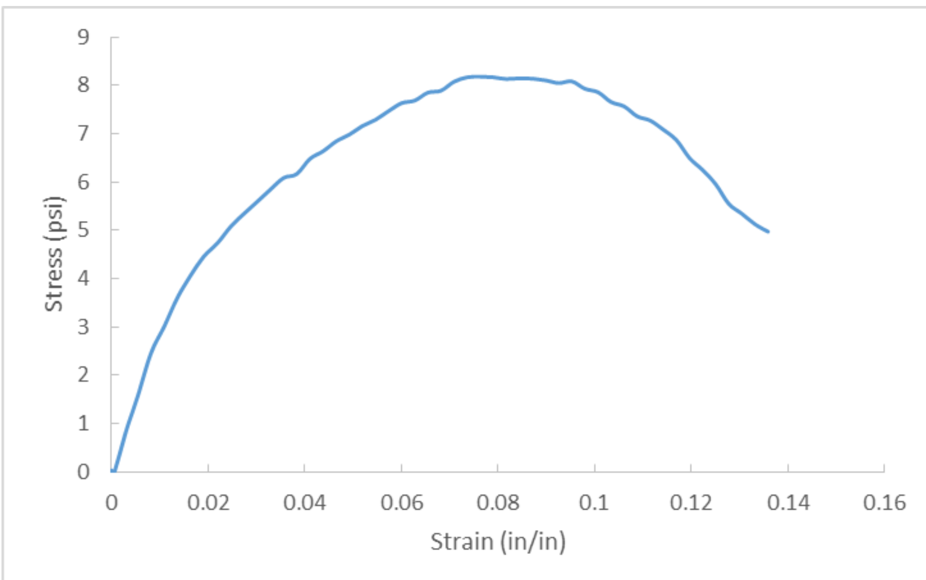

Figure A.222. D8-8, OC= 65.9%, CF= 300 pcf, T=56 Days.

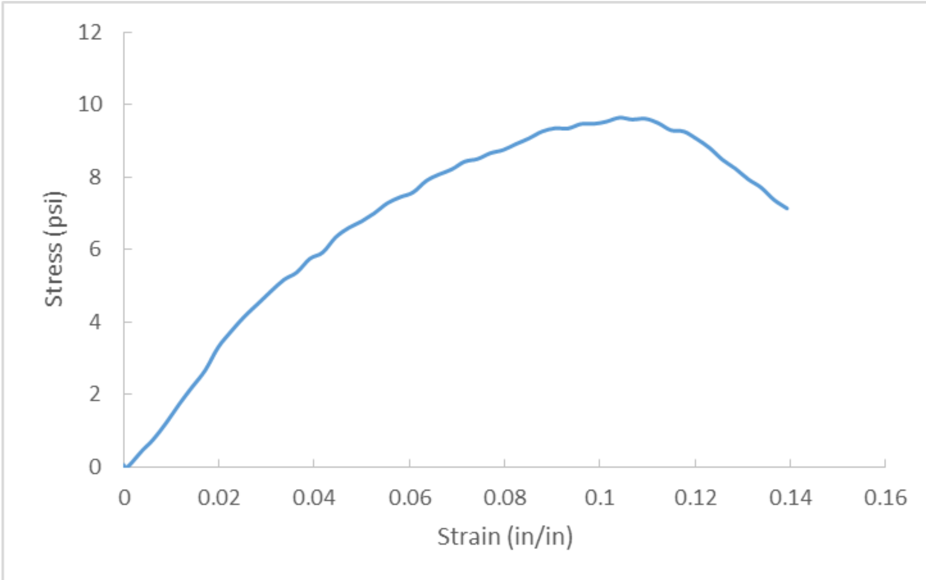

Figure A.223. D8-9, OC= 65.9%, CF= 300 pcf, T=56 Days.

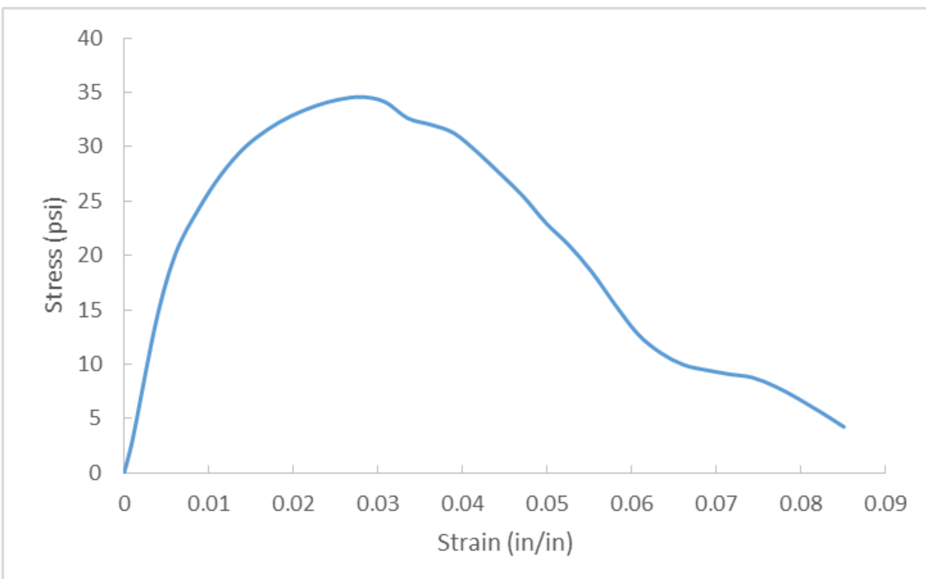

Figure A.224. D9-7, OC= 65.9%, CF= 400 pcf, T=56 Days.

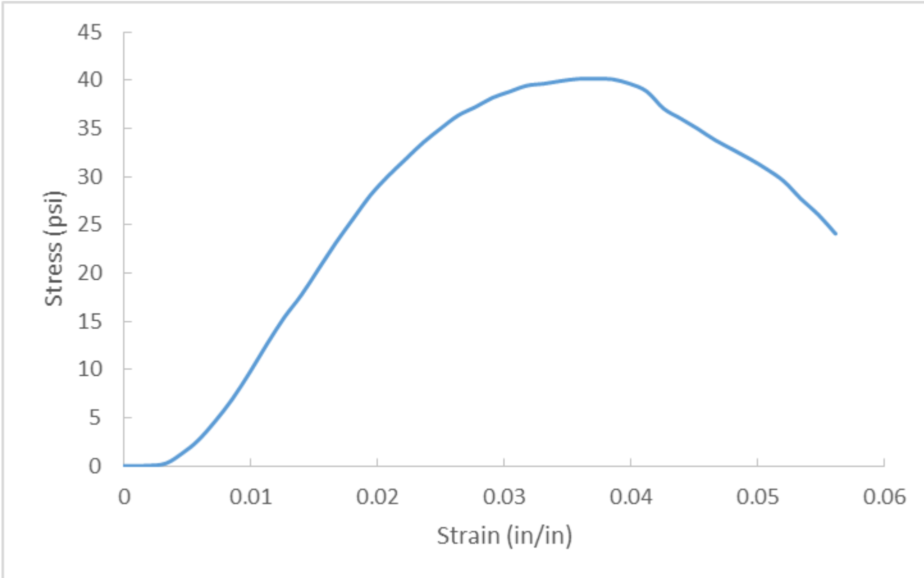

Figure A.225. D9-8, OC= 65.9%, CF= 400 pcf, T=56 Days.

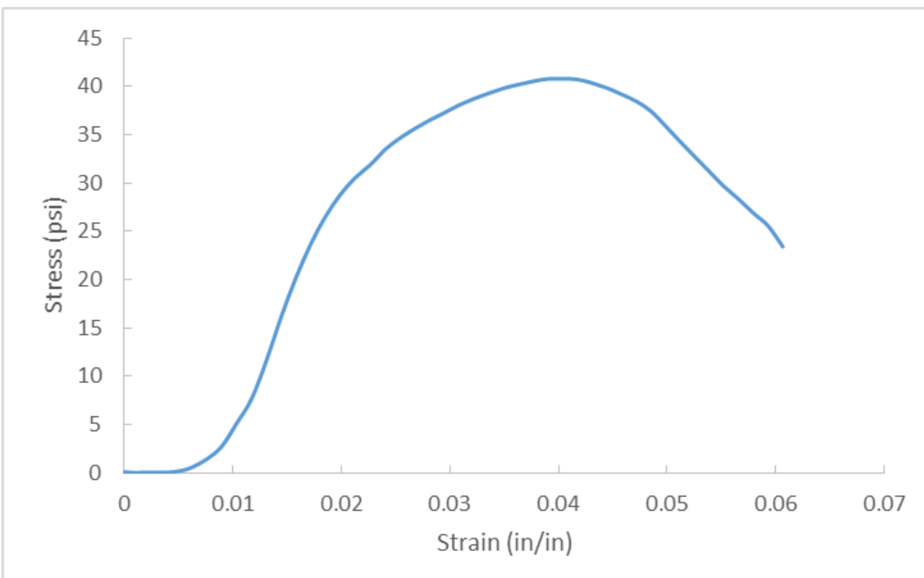

Figure A.226. D9-9, OC= 65.9%, CF= 400 pcf, T=56 Days.

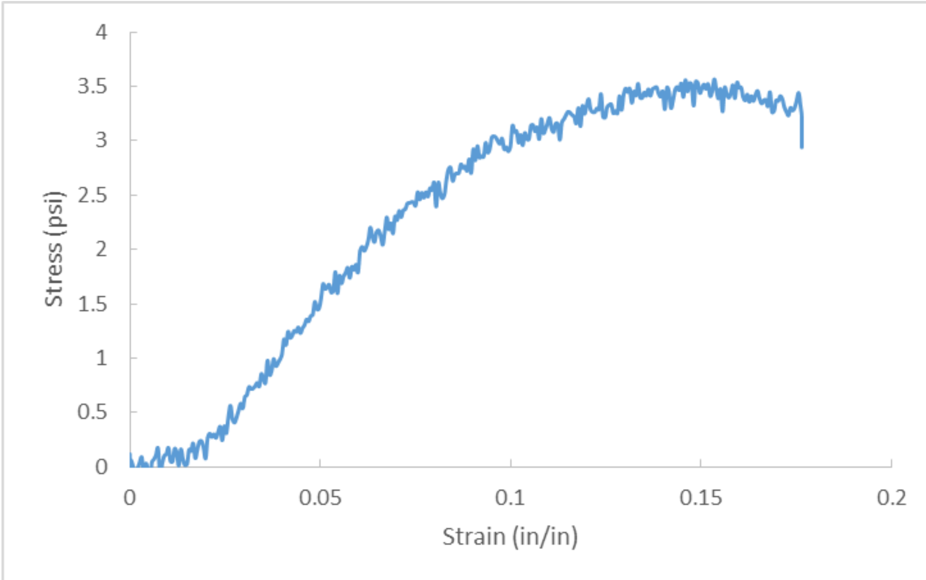

Figure A.227. D10-7, OC= 41.3%, CF= 300 pcf, T=56 Days.

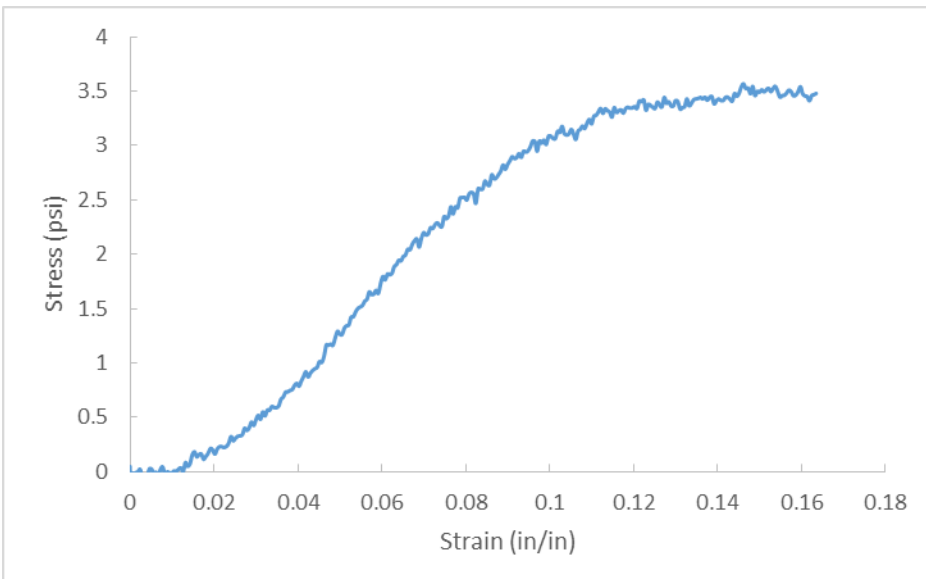

Figure A.228. D10-8, OC= 41.3%, CF= 300 pcf, T=56 Days.

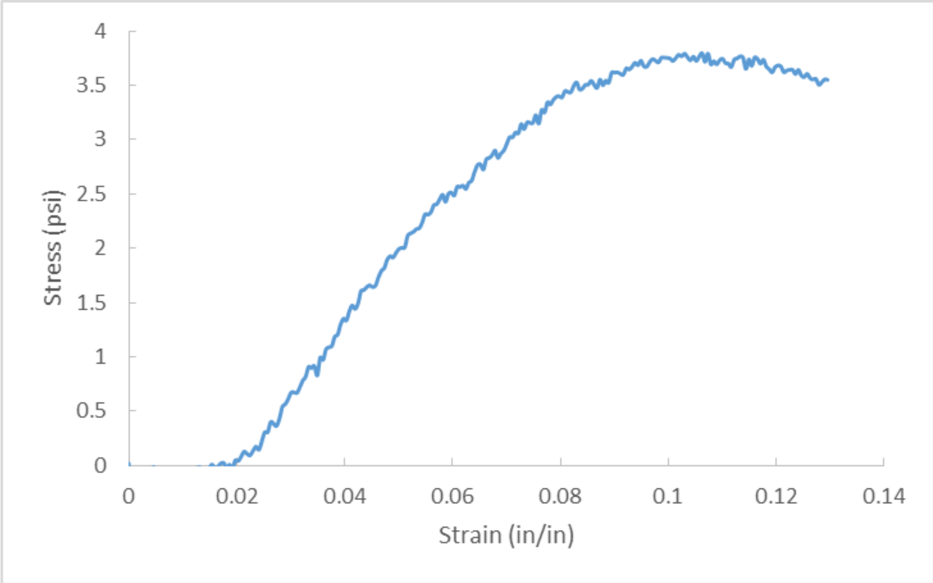

Figure A.229. D11-7, OC= 27.8%, CF= 300 pcf, T=56 Days.

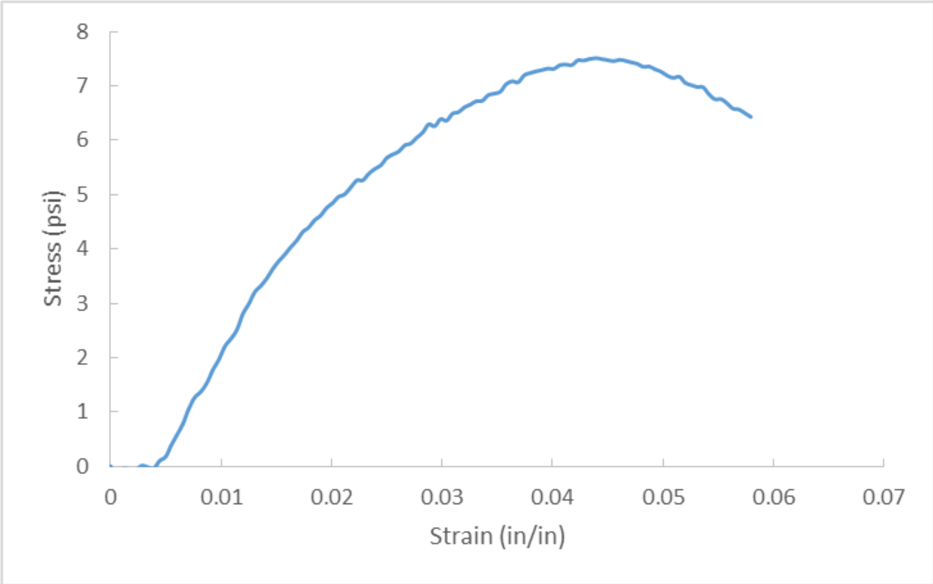

Figure A.230. D12-7, OC= 24.1%, CF= 300 pcf, T=56 Days.

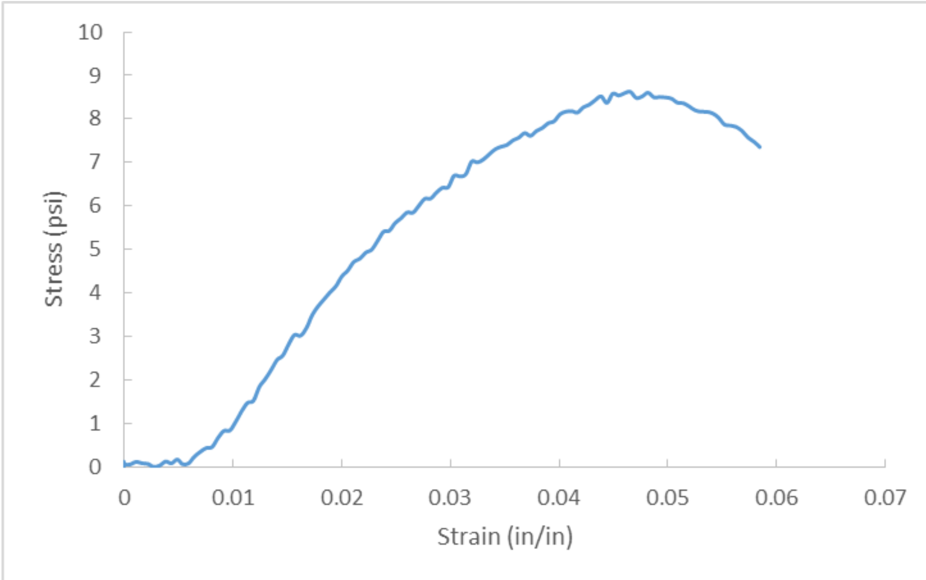

Figure A.231. D12-8, OC= 24.1%, CF= 300 pcf, T=56 Days.

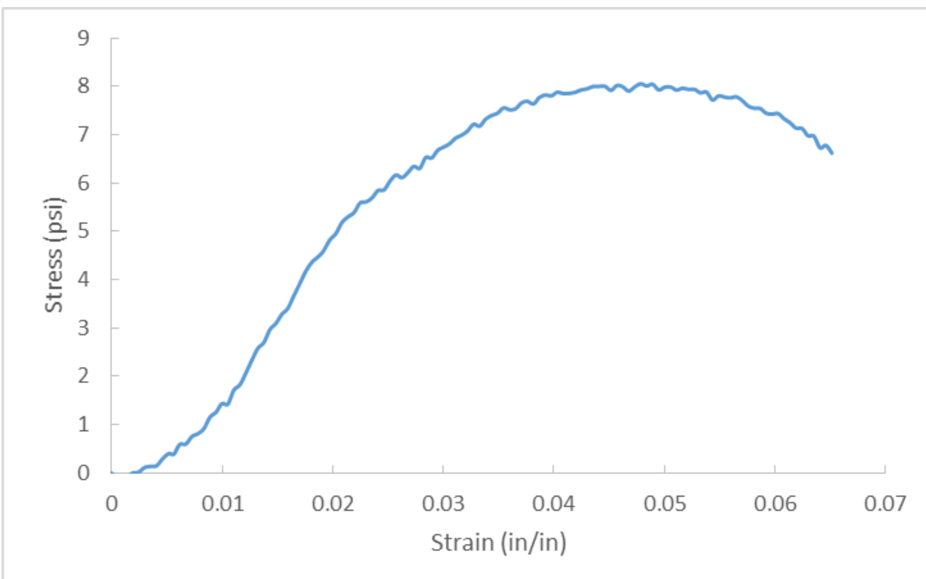

Figure A.232. D12-9, OC= 24.1%, CF= 300 pcf, T=56 Days.

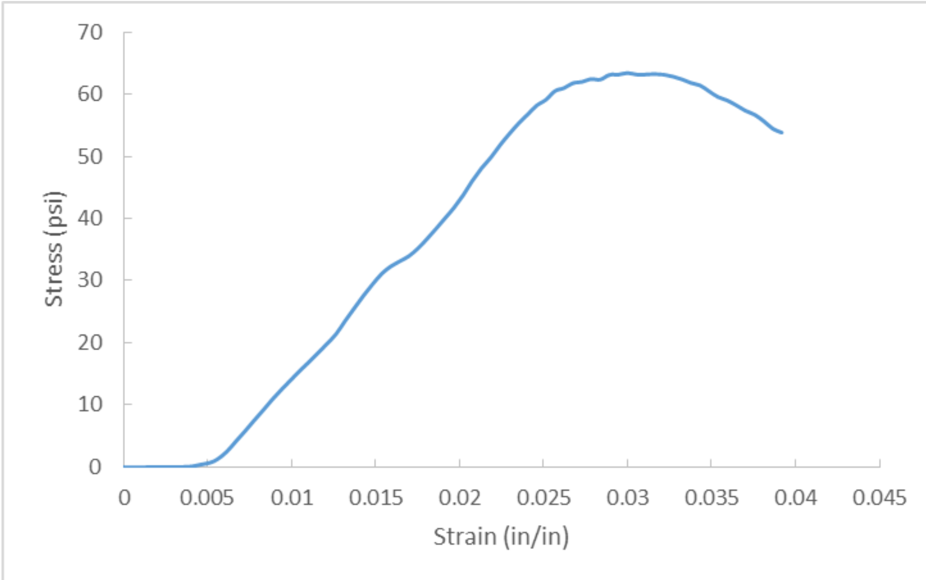

Figure A.233. D13-7, OC= 11.2%, CF= 300 pcf, T=56 Days.

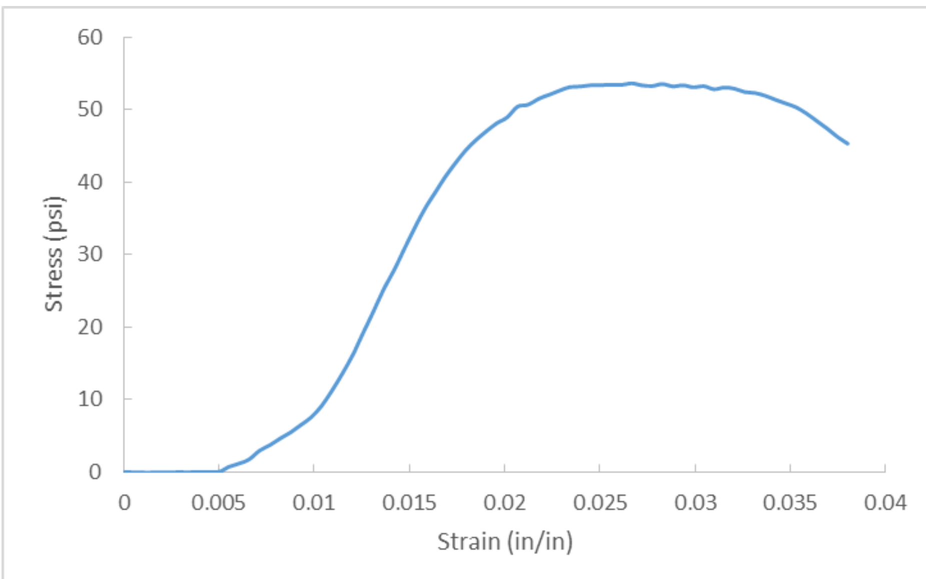

Figure A.234. D13-8, OC= 11.2%, CF= 300 pcf, T=56 Days.

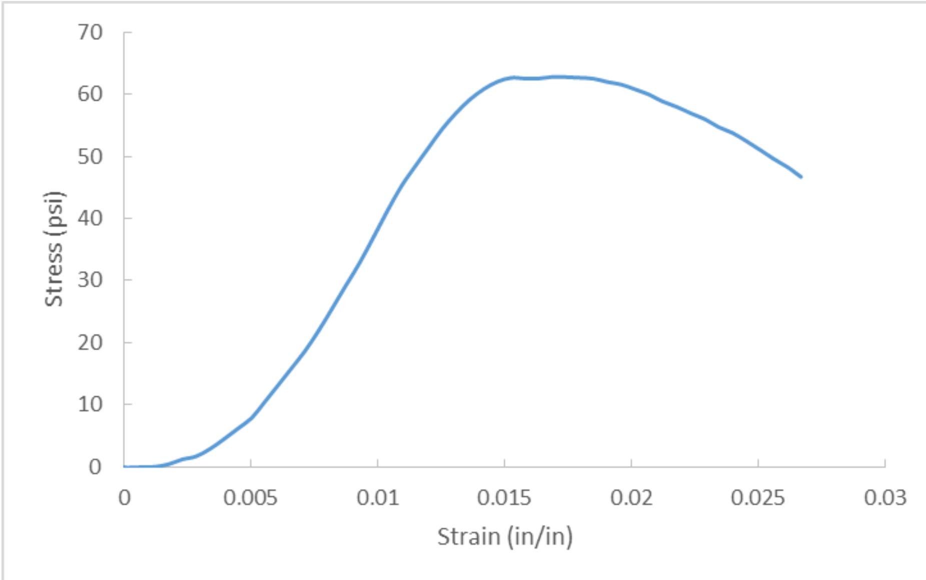

Figure A.235. D13-9, OC= 11.2%, CF= 300 pcf, T=56 Days.

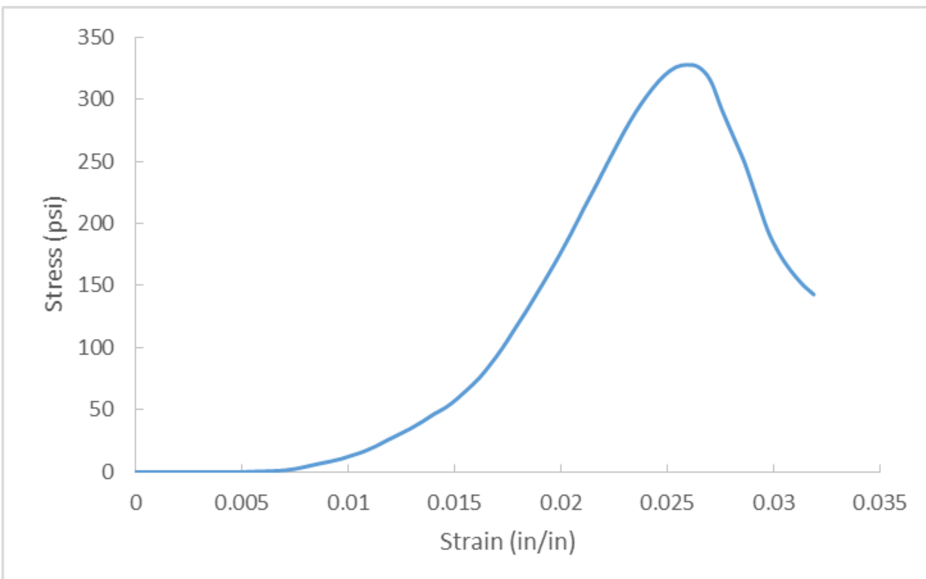

Figure A.236. D14-7, OC= 4.6%, CF= 300 pcf, T=56 Days.

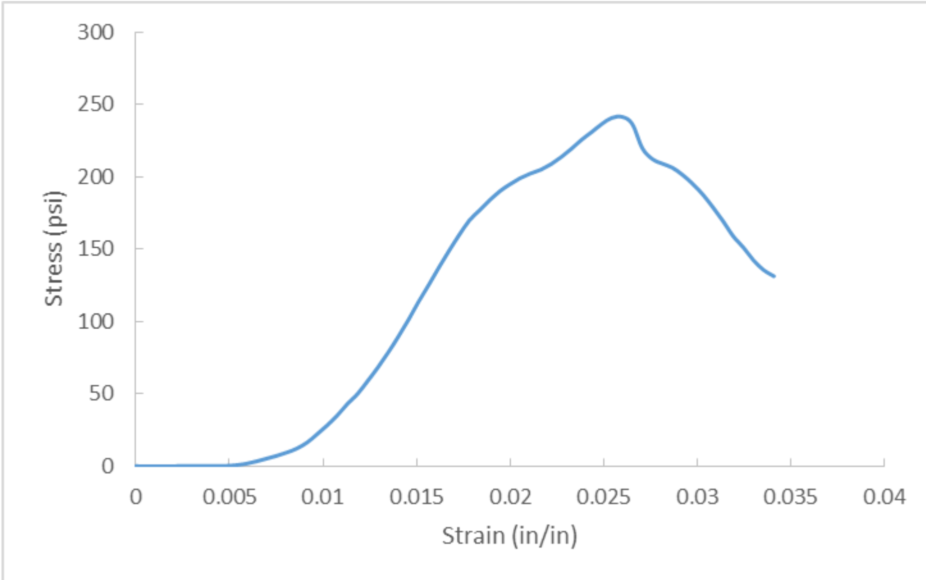

Figure A.237. D14-8, OC= 4.6%, CF= 300 pcf, T=56 Days.

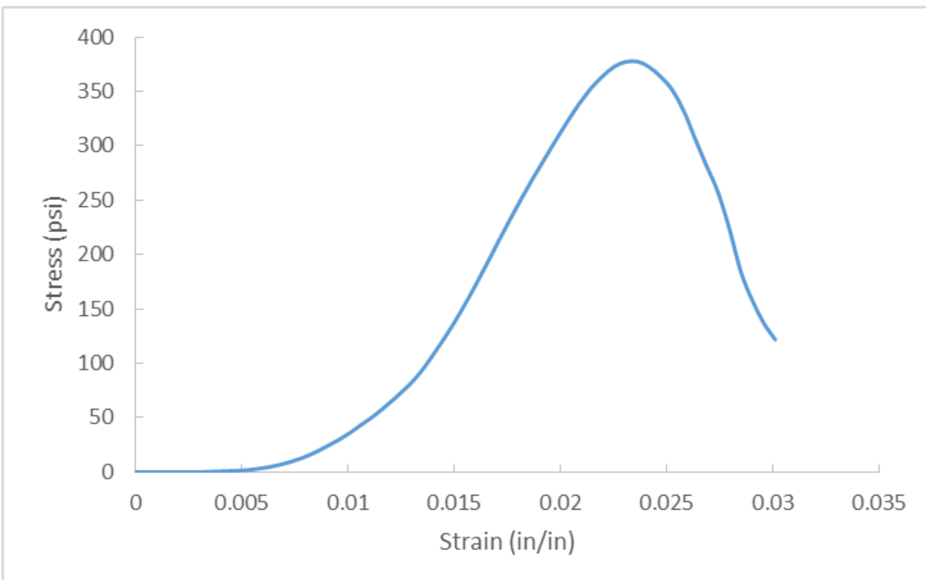

Figure A.238. D14-9, OC= 4.6%, CF= 300 pcf, T=56 Days.

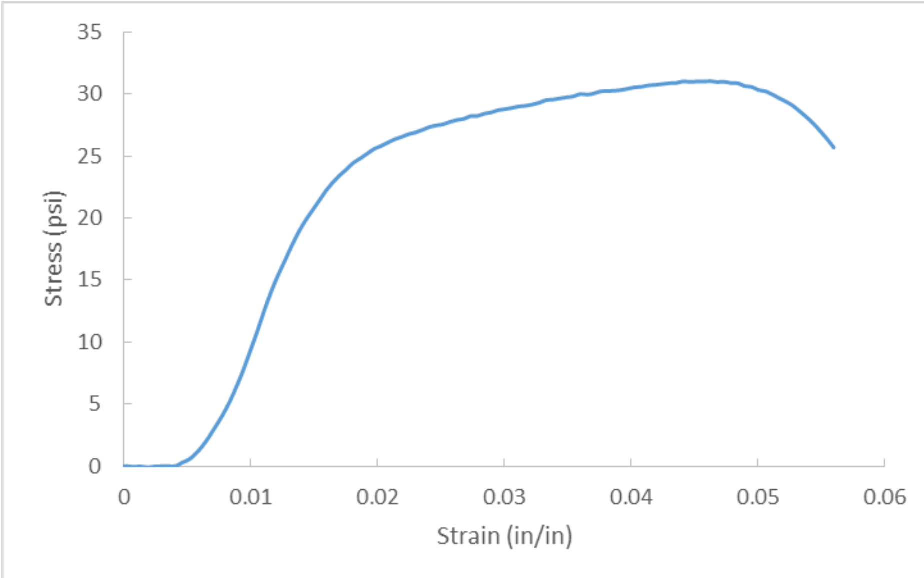

Figure A.239. D15-7, OC= 40.5%, CF= 300 pcf, T=56 Days.

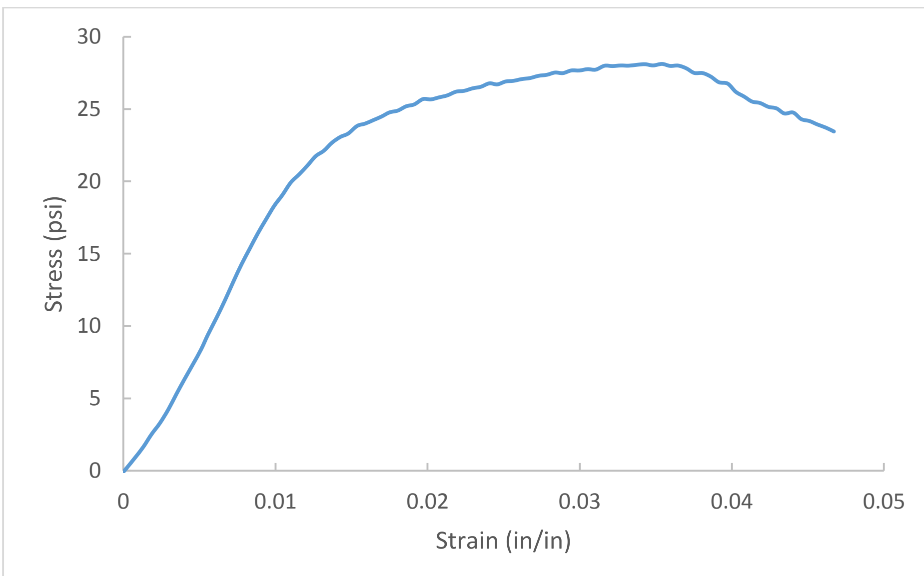

Figure A.240. D15-8, OC= 40.5%, CF= 300 pcf, T=56 Days.

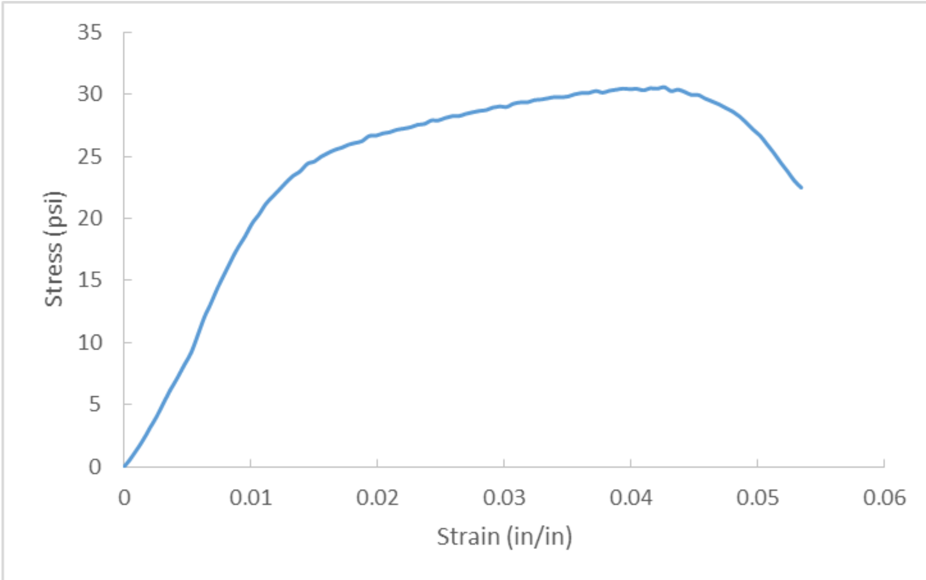

Figure A.241. D15-9, OC= 40.5%, CF= 300 pcf, T=56 Days.

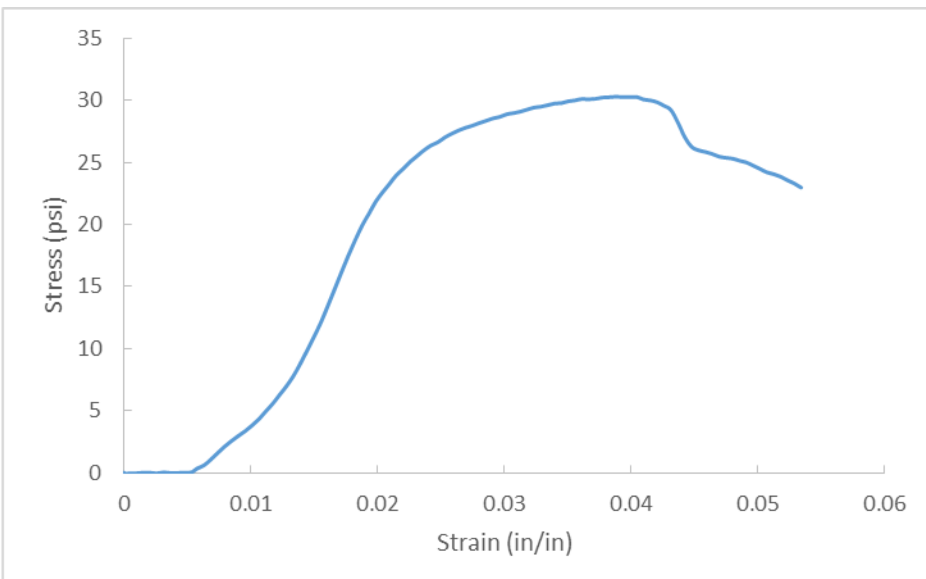

Figure A.242. D16-7, OC= 34.7%, CF= 300 pcf, T=56 Days.

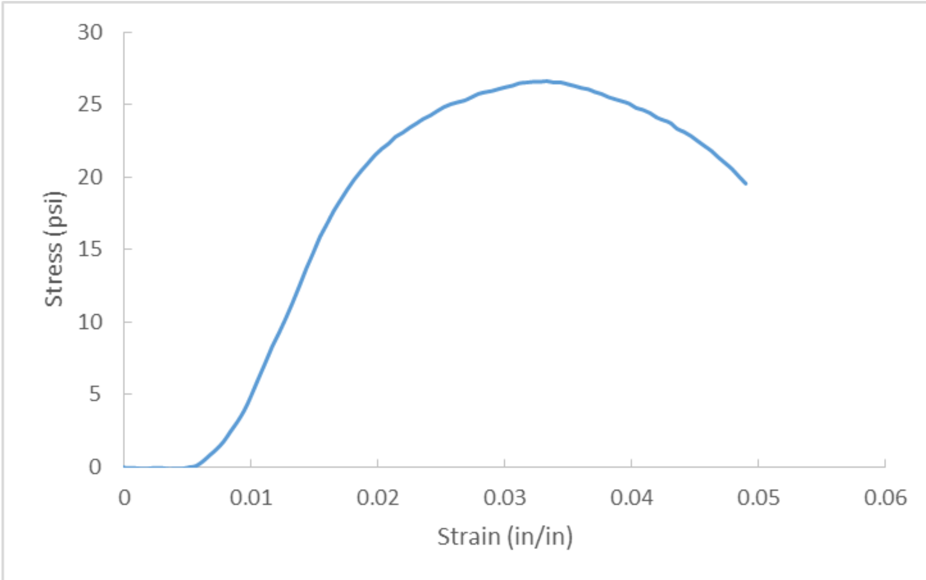

Figure A.243. D16-8, OC= 34.7%, CF= 300 pcf, T=56 Days.

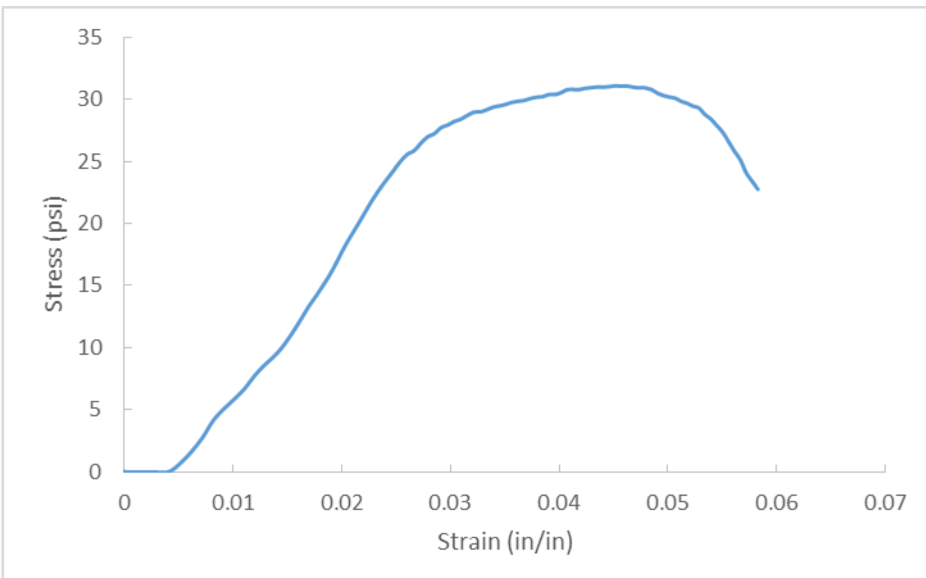

Figure A.244. D16-9, OC= 34.7%, CF= 300 pcf, T=56 Days.

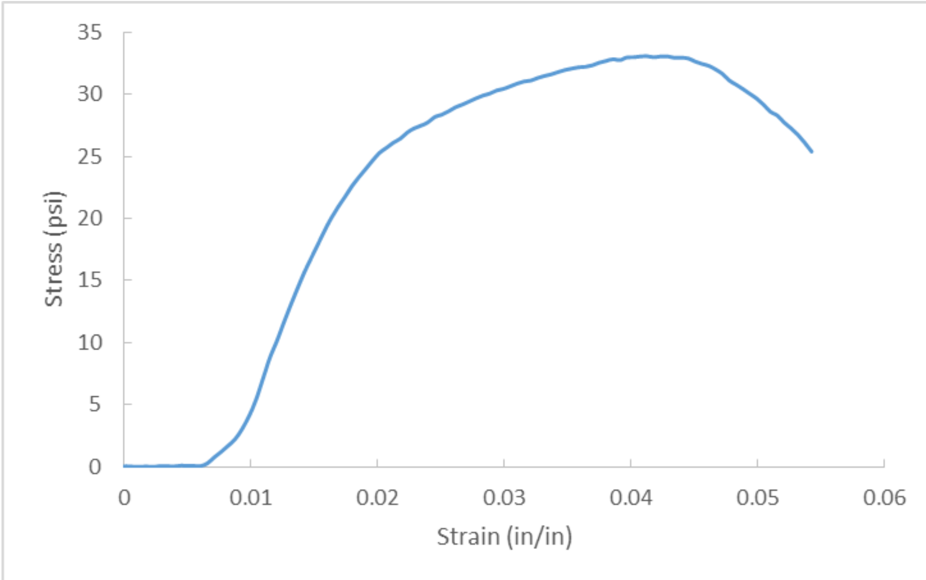

Figure A.245. D17-7, OC= 19.2%, CF= 300 pcf, T=56 Days.

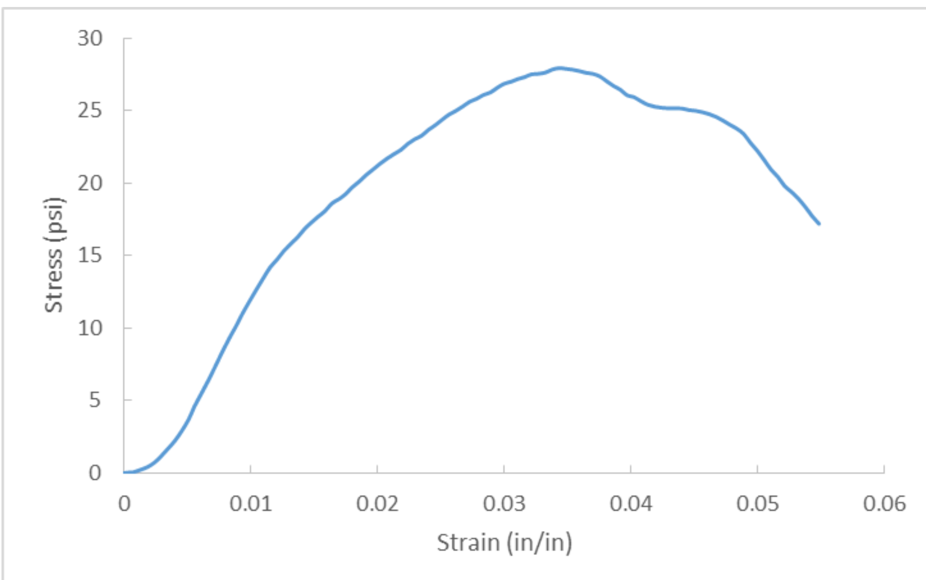

Figure A.246. D17-8, OC= 19.2%, CF= 300 pcf, T=56 Days.

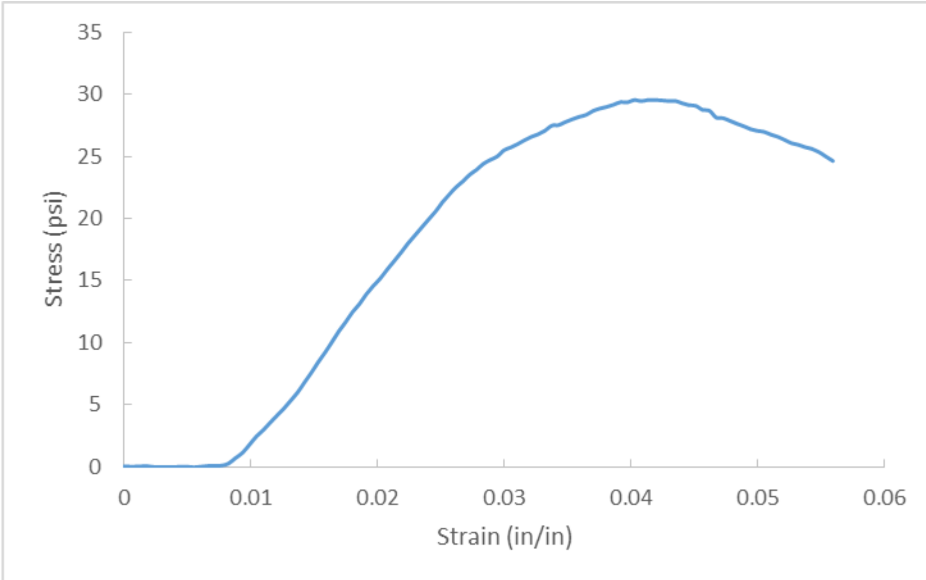

Figure A.247. D17-9, OC= 19.2%, CF= 300 pcf, T=56 Days.

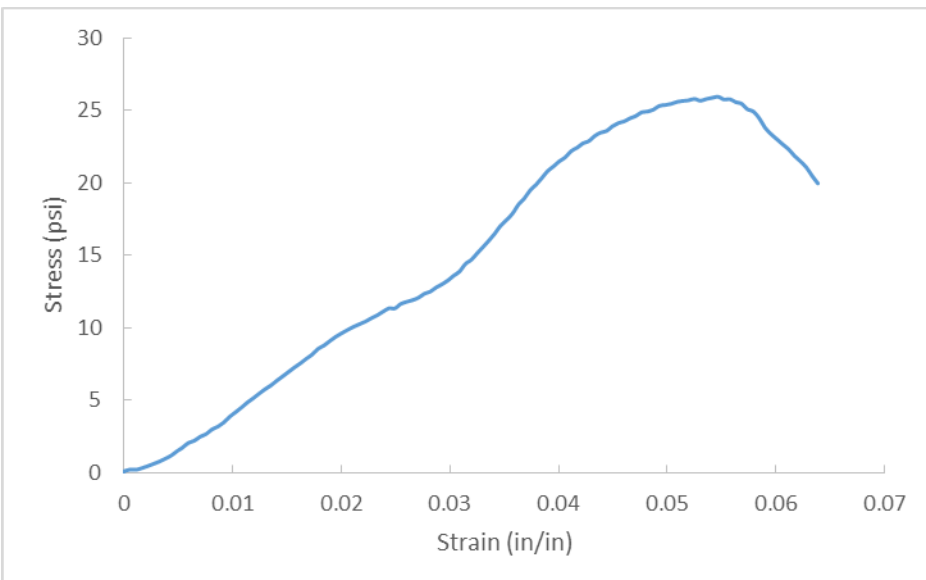

Figure A.248. D18-7, OC= 18.9%, CF= 300 pcf, T=56 Days.

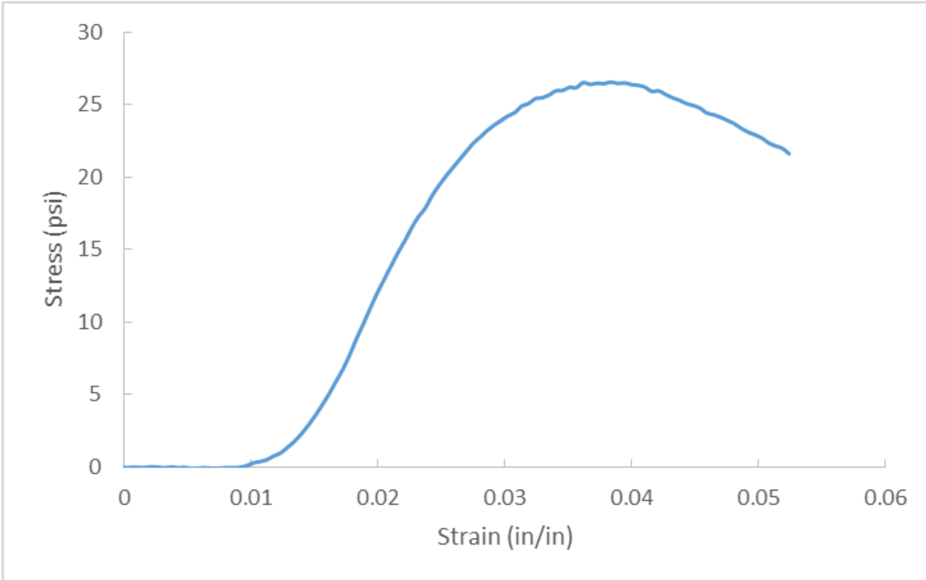

Figure A.249. D18-8, OC= 18.9%, CF= 300 pcf, T=56 Days.

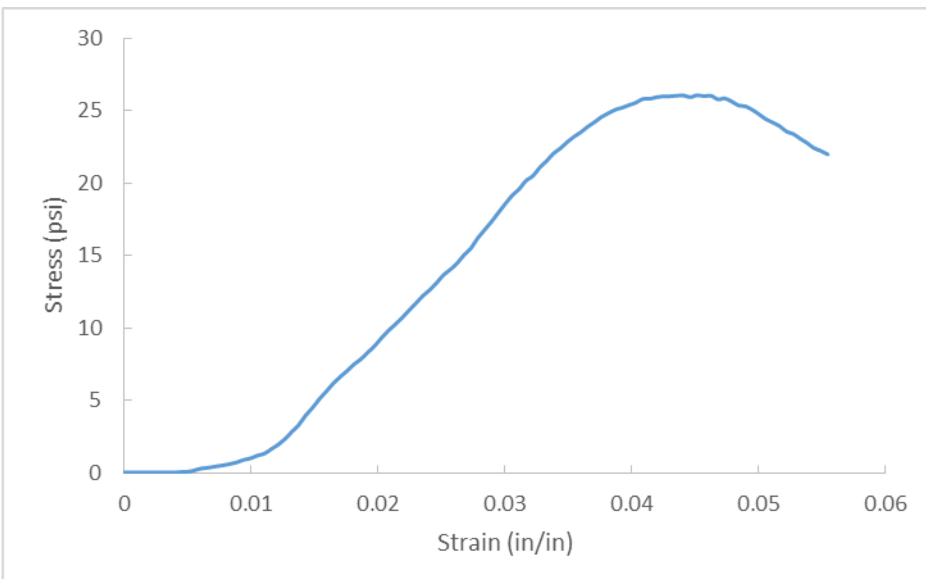

Figure A.250. D18-9, OC= 18.9%, CF= 300 pcf, T=56 Days.

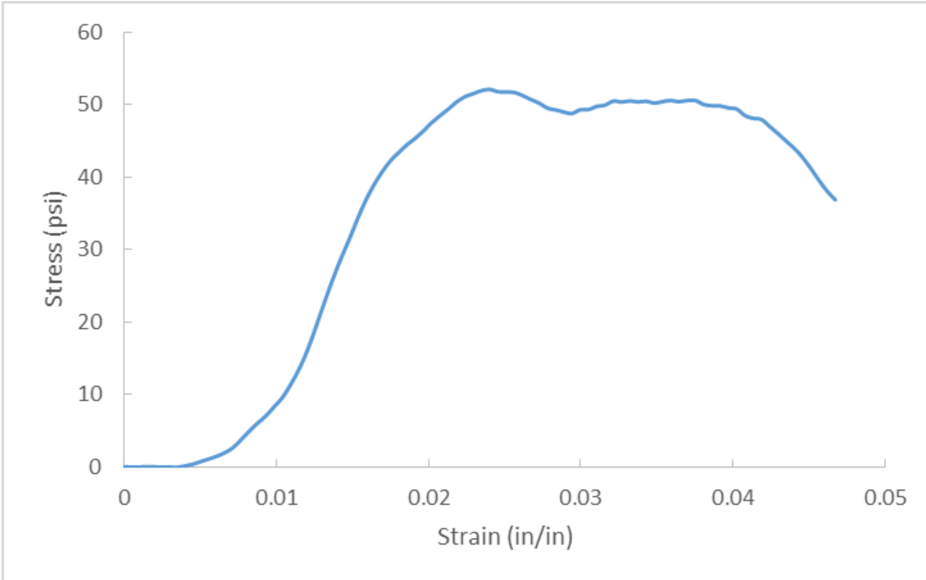

Figure A.251. D19-7, OC= 8.5%, CF= 300 pcf, T=56 Days.

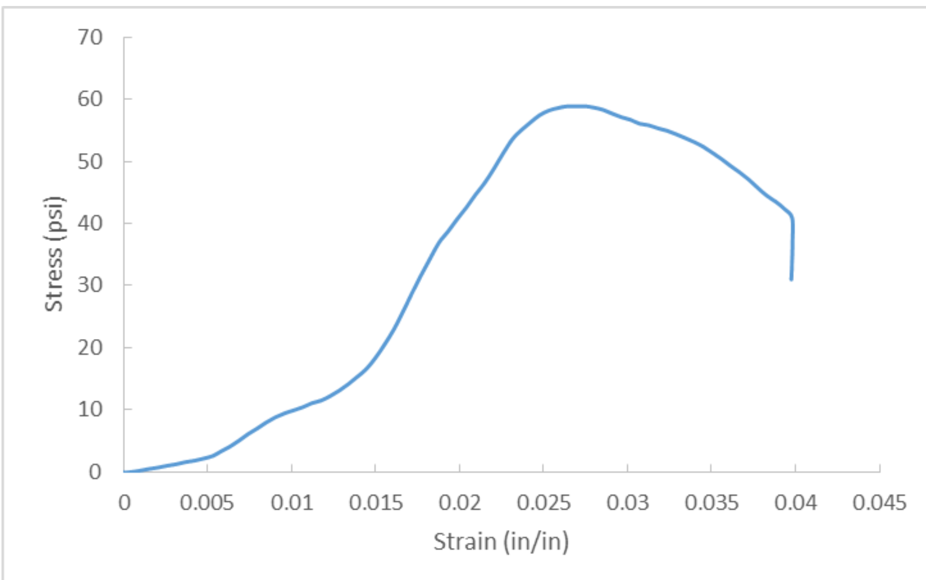

Figure A.252. D19-8, OC= 8.5%, CF= 300 pcf, T=56 Days.

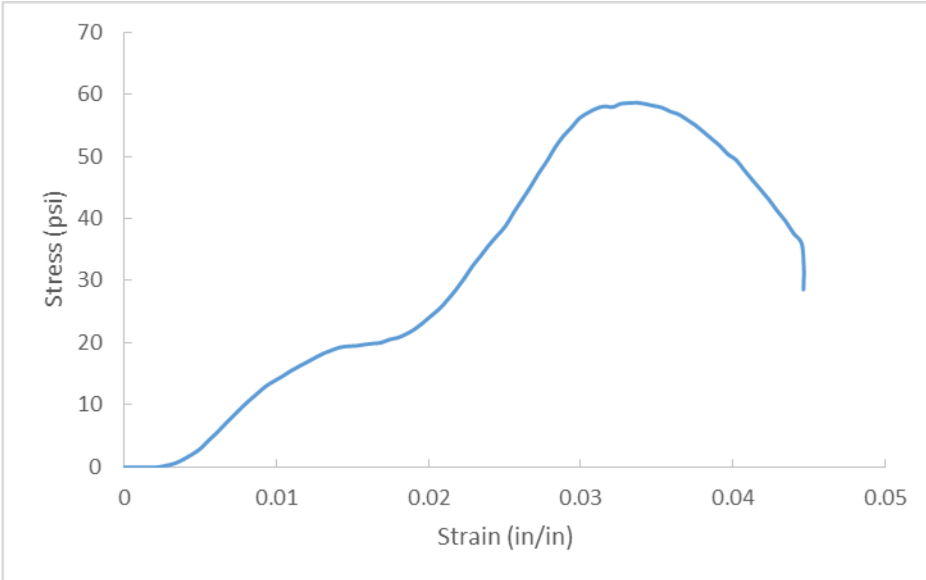

Figure A.253. D19-9, OC= 8.5%, CF= 300 pcf, T=56 Days.

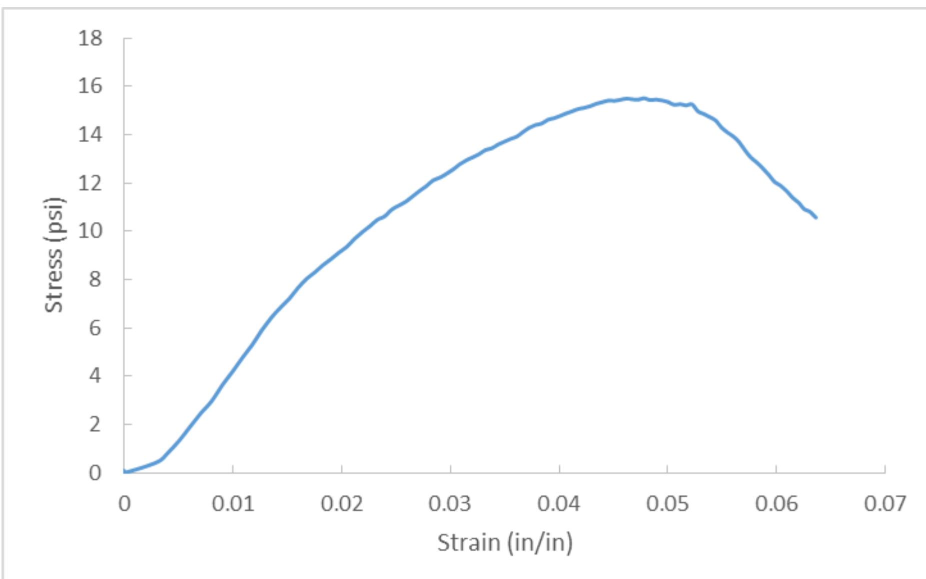

Figure A.254. D25-8, OC= 41.2%, CF= 400 pcf, T=56 Days.

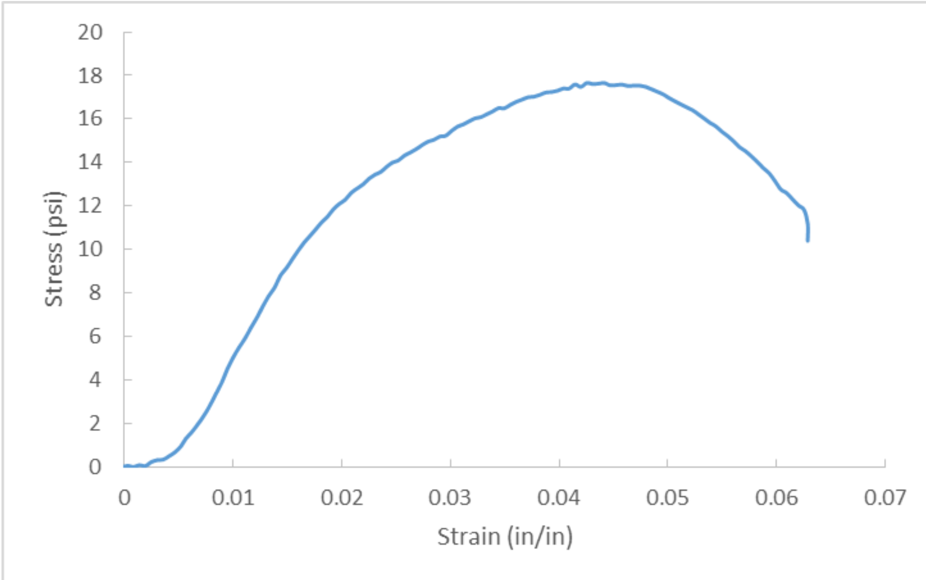

Figure A.255. D25-9, OC= 41.2%, CF= 400 pcf, T=56 Days.

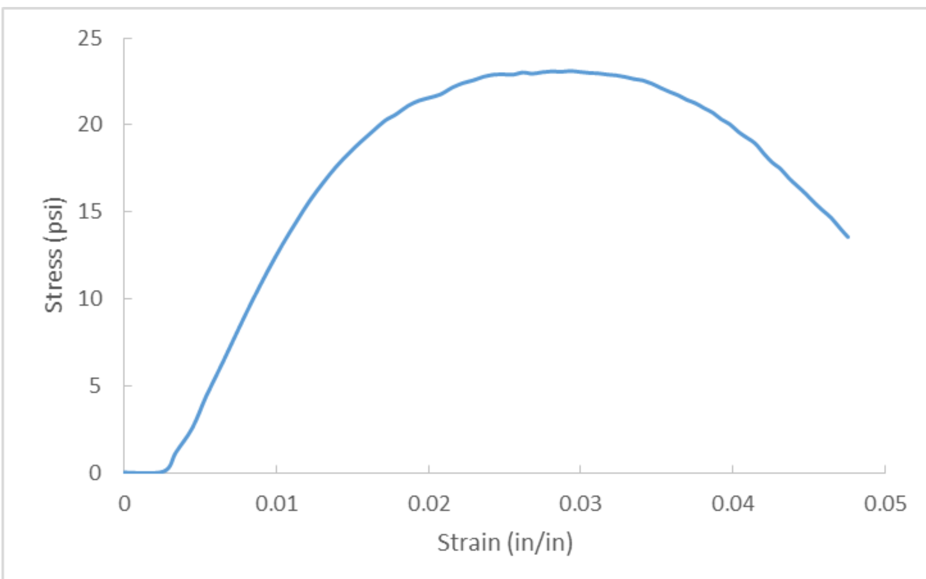

Figure A.256. D26-7, OC= 29.8%, CF= 400 pcf, T=56 Days.

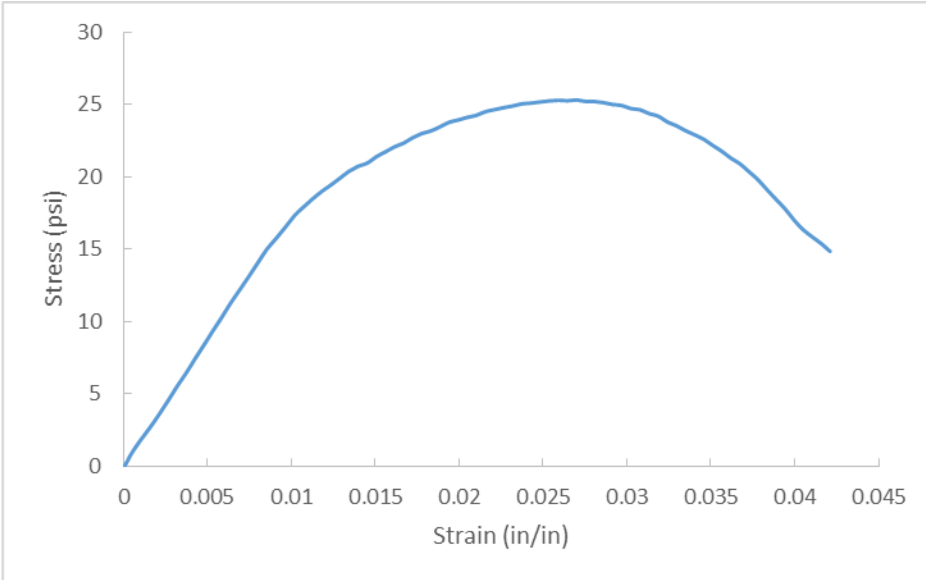

Figure A.257. D26-8, OC= 29.8%, CF= 400 pcf, T=56 Days.

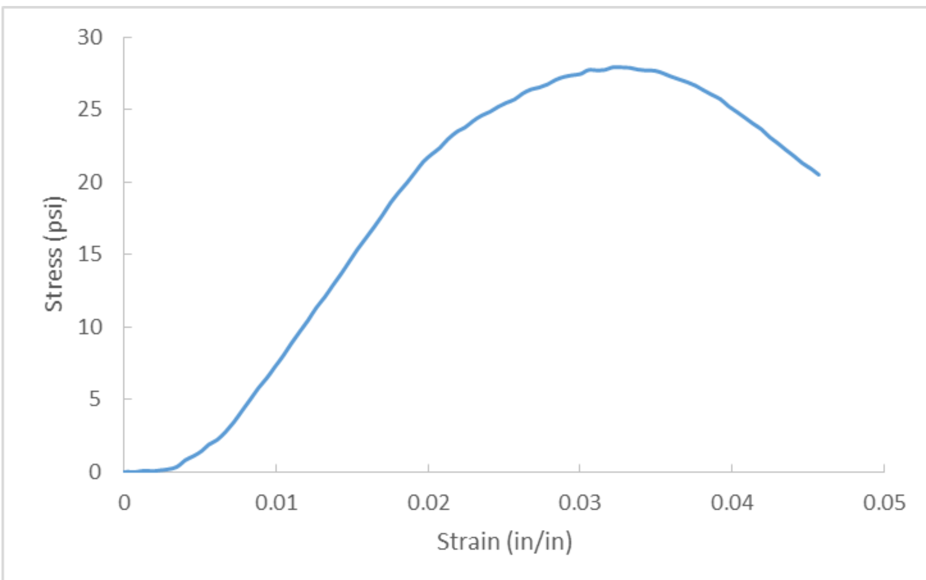

Figure A.258. D26-9, OC= 29.8%, CF= 400 pcf, T=56 Days.

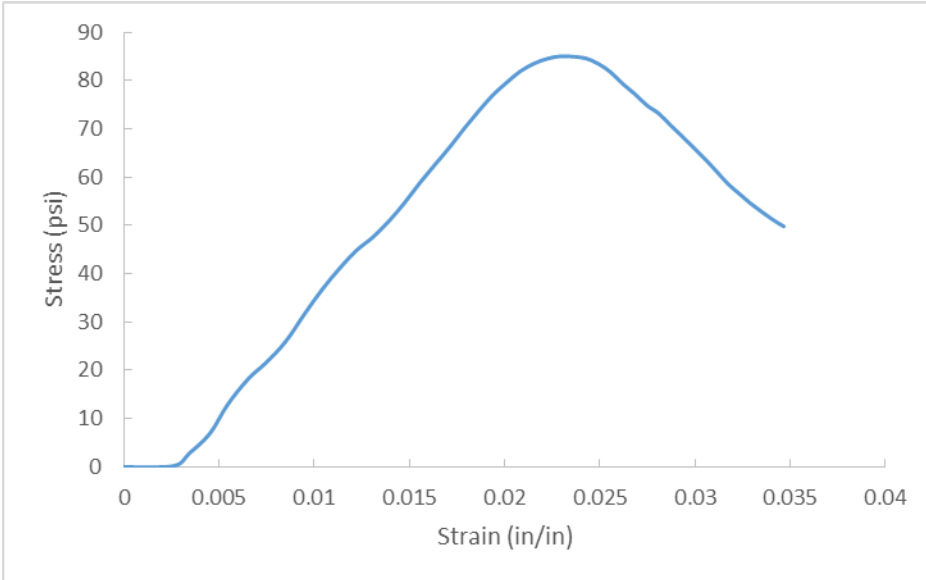

Figure A.259. D27-7, OC= 21.1%, CF= 400 pcf, T=56 Days.

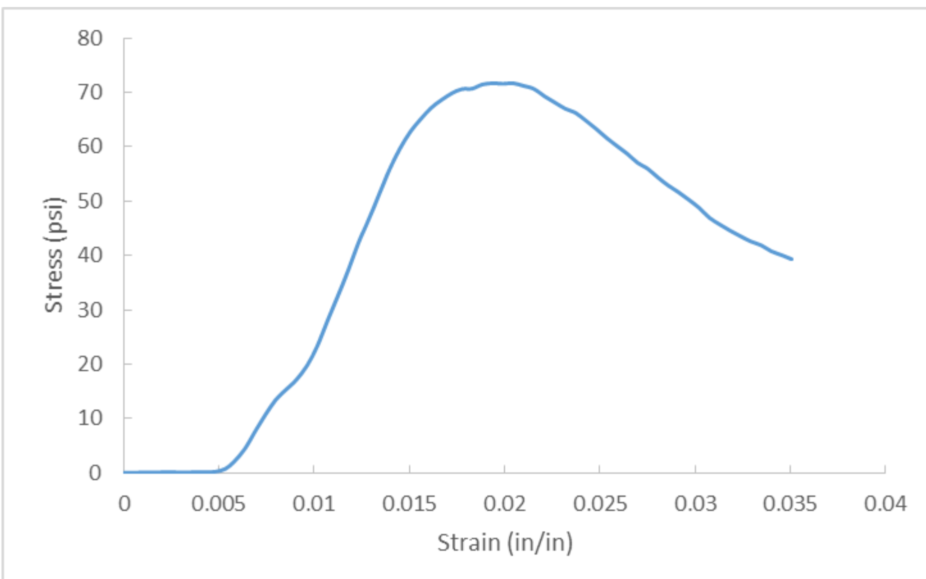

Figure A.260. D27-8, OC= 21.1%, CF= 400 pcf, T=56 Days.

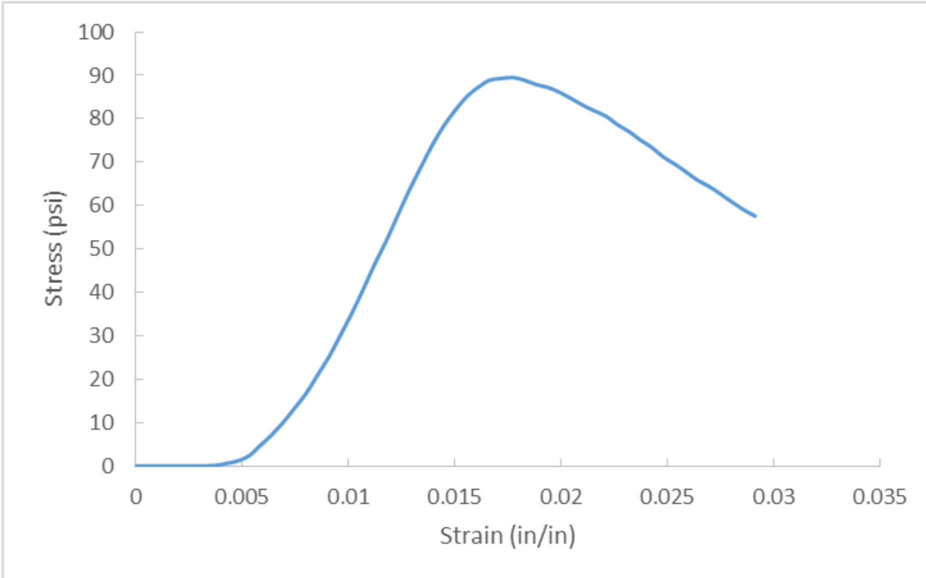

Figure A.261. D27-9, OC= 21.1%, CF= 400 pcf, T=56 Days.

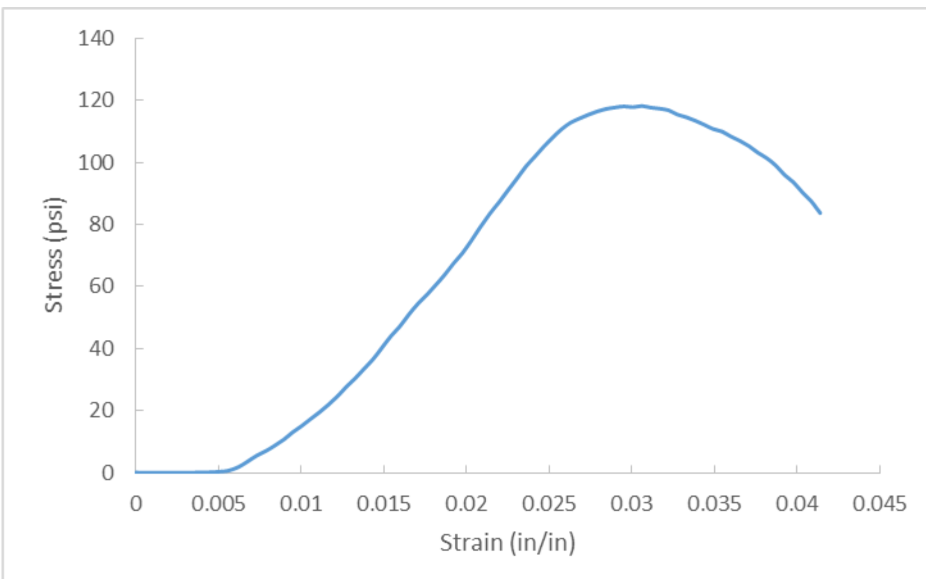

Figure A.262. D28-7, OC= 12.6%, CF= 400 pcf, T=56 Days.

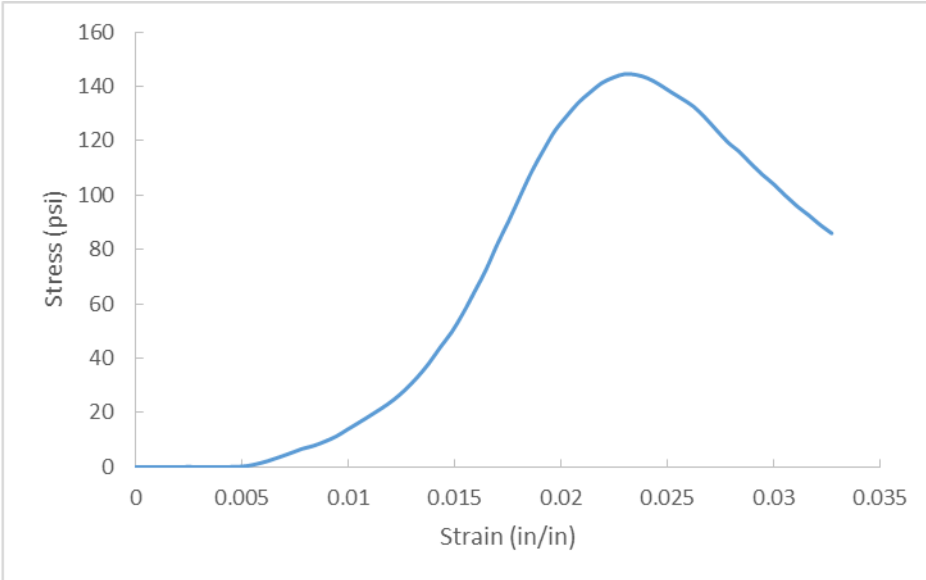

Figure A.263. D28-8, OC= 12.6%, CF= 400 pcf, T=56 Days.

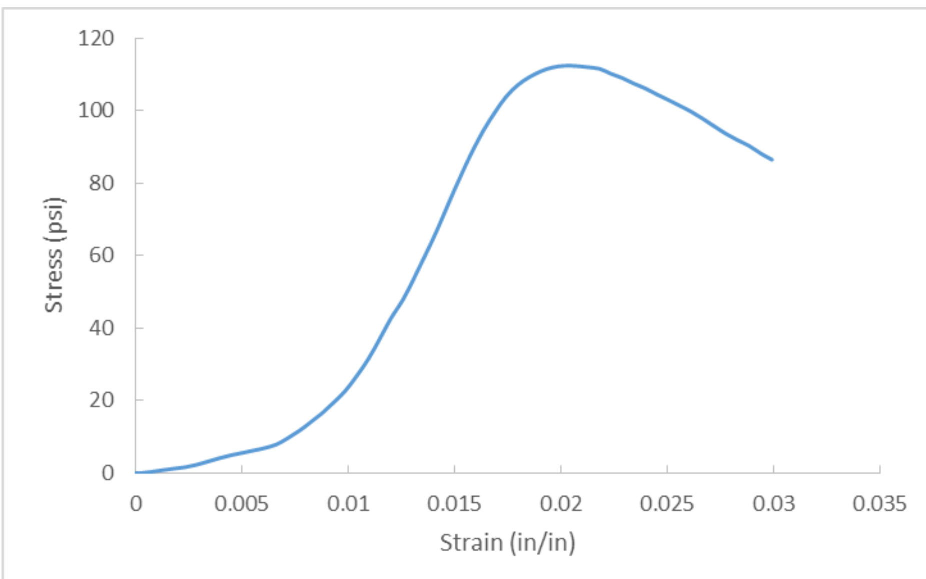

Figure A.264. D28-9, OC= 12.6%, CF= 400 pcf, T=56 Days.

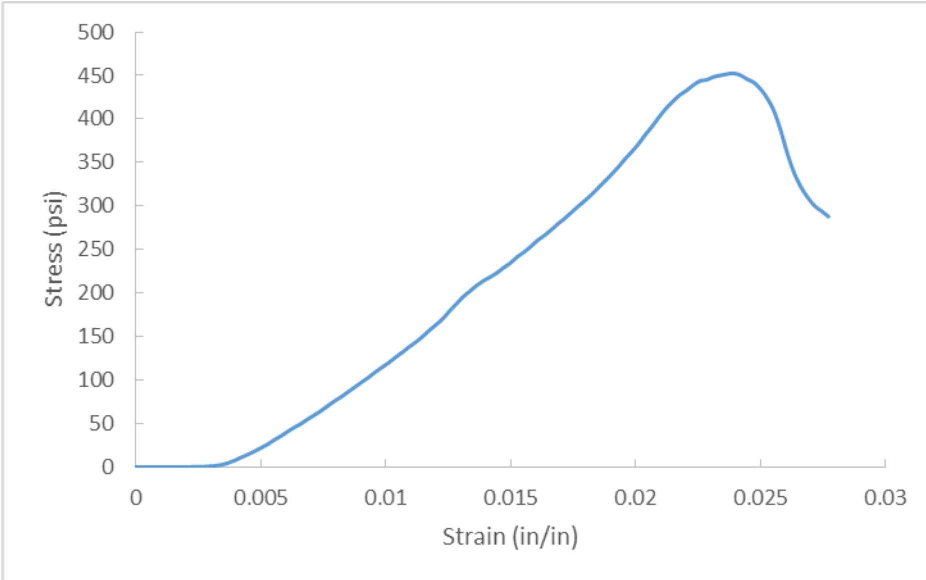

Figure A.265. D29-7, OC= 4.2%, CF= 400 pcf, T=56 Days.

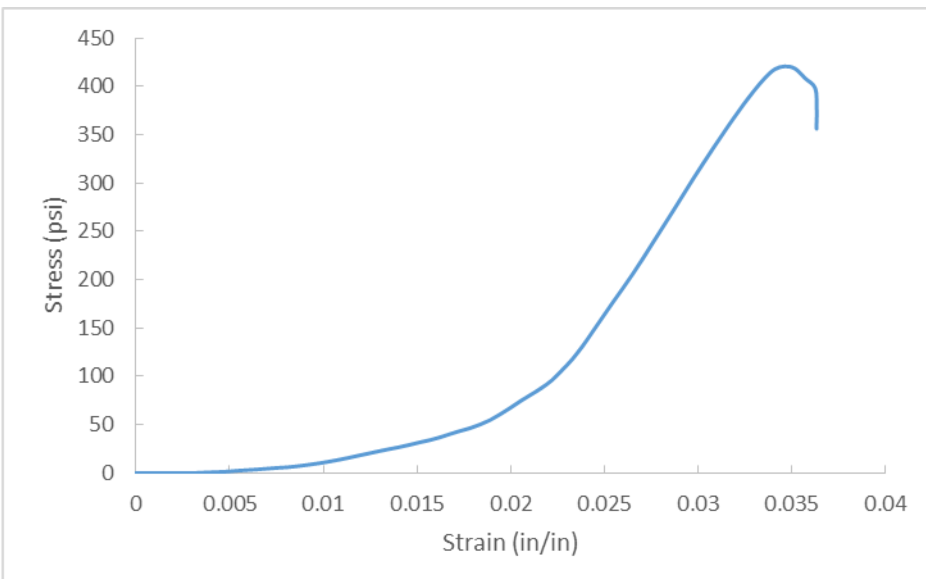

Figure A.266. D29-8, OC= 4.2%, CF= 400 pcf, T=56 Days.

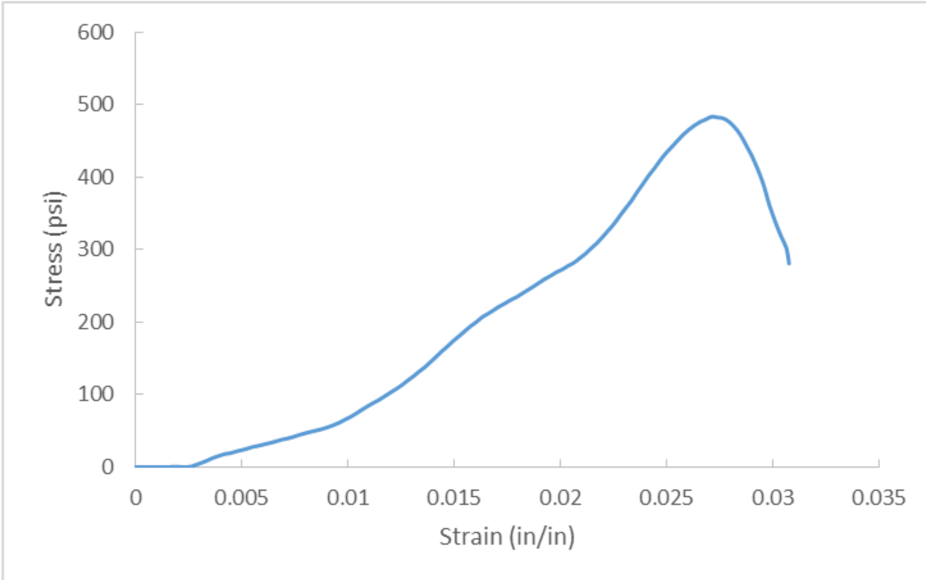

Figure A.267. D29-9, OC= 4.2%, CF= 400 pcf, T=56 Days.

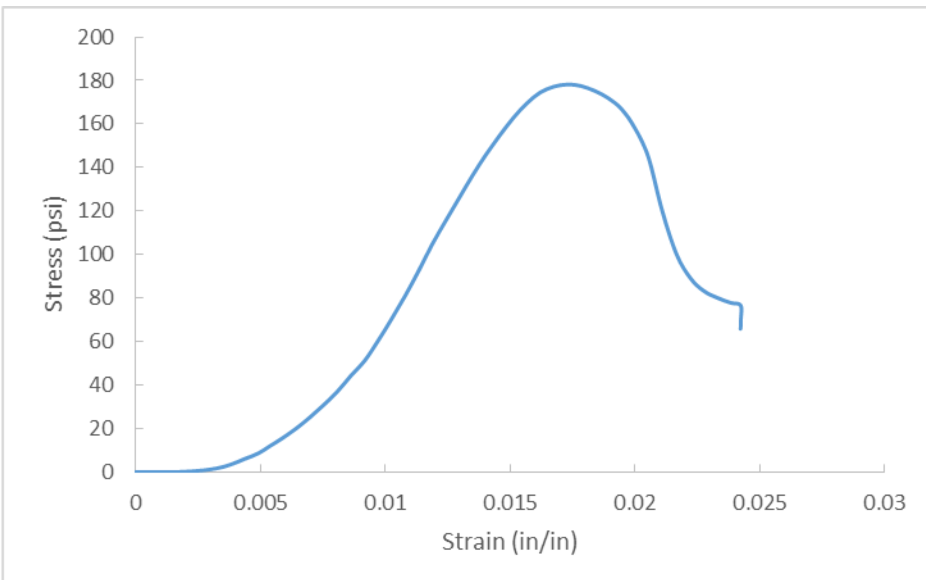

Figure A.268. D30-7, OC= 4.9%, CF= 400 pcf, T=56 Days.

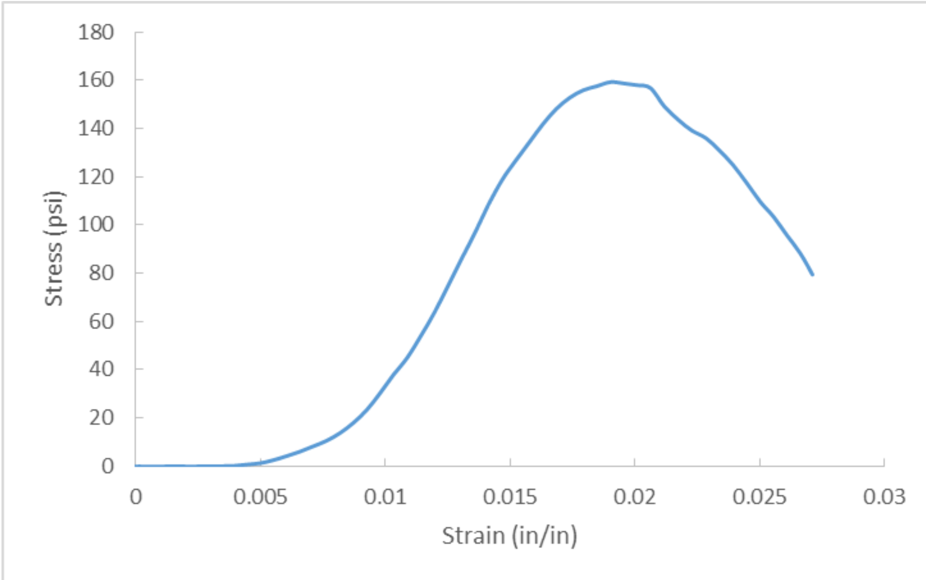

Figure A.269. D30-8, OC= 4.9%, CF= 400 pcf, T=56 Days.

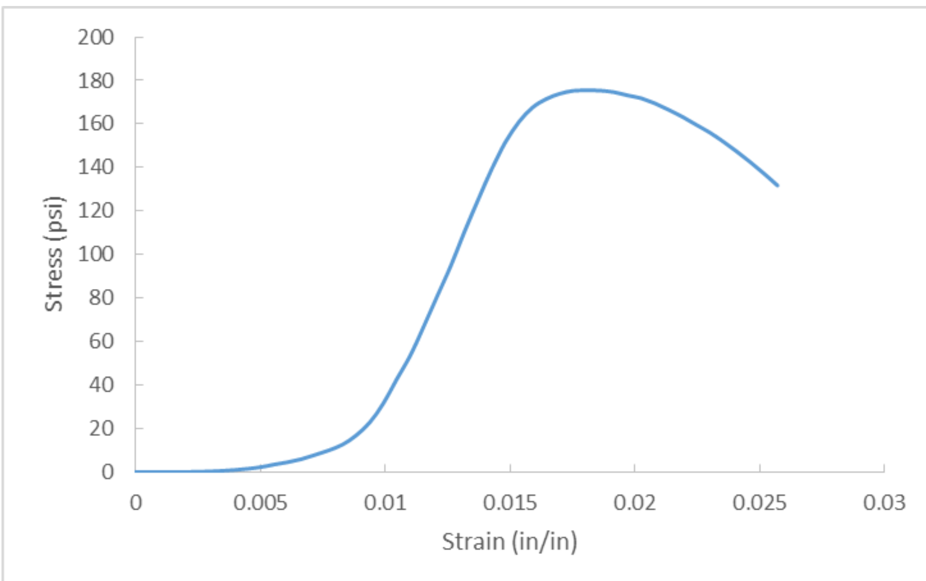

Figure A.270. D30-9, OC= 4.9%, CF= 400 pcf, T=56 Days.

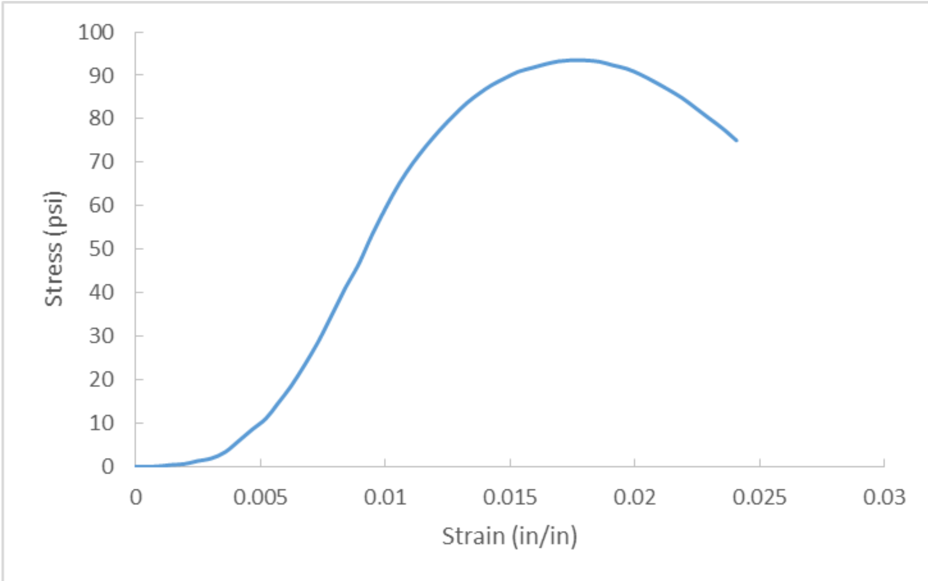

Figure A.271. D31-7, OC= 11.8%, CF= 400 pcf, T=56 Days.

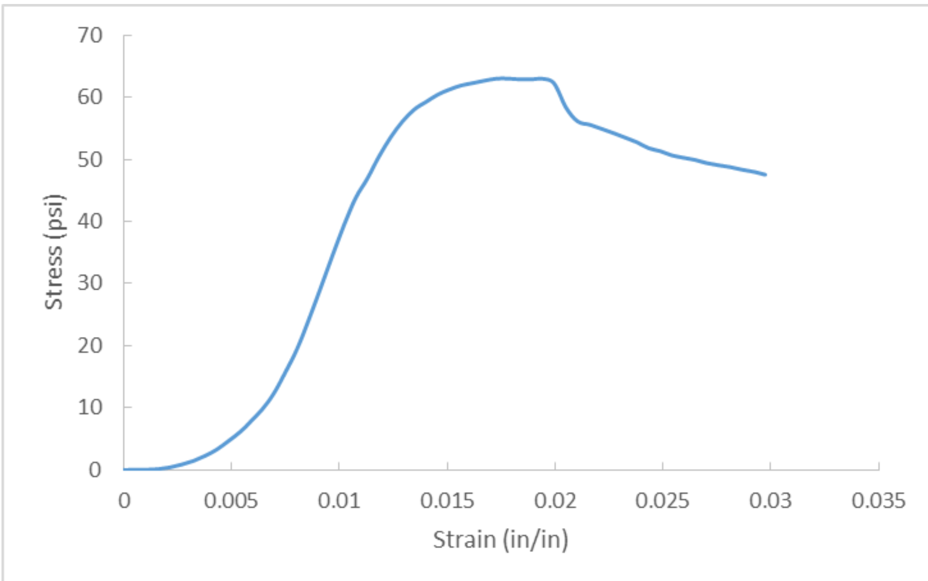

Figure A.272. D31-8, OC= 11.8%, CF= 400 pcf, T=56 Days.

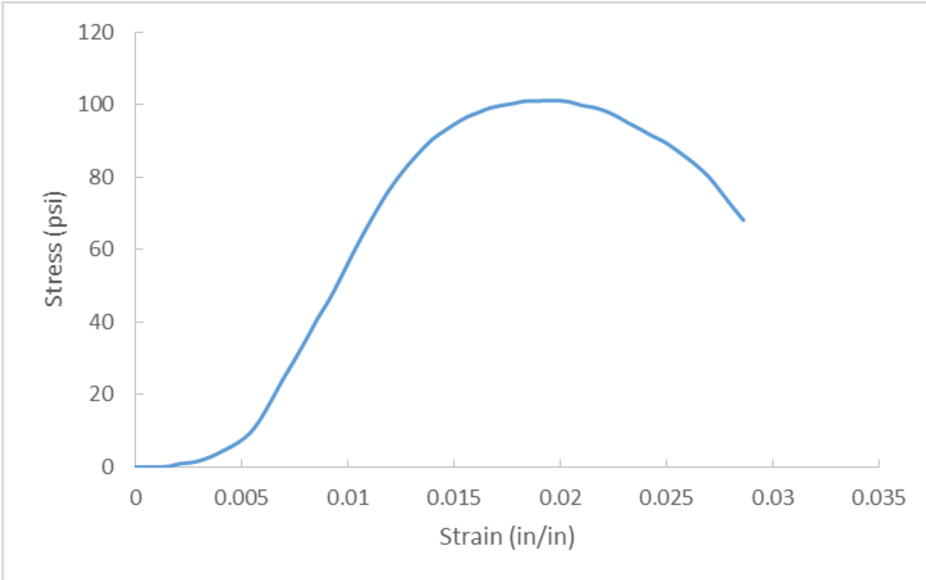

Figure A.273. D31-9, OC= 11.8%, CF= 400 pcf, T=56 Days.

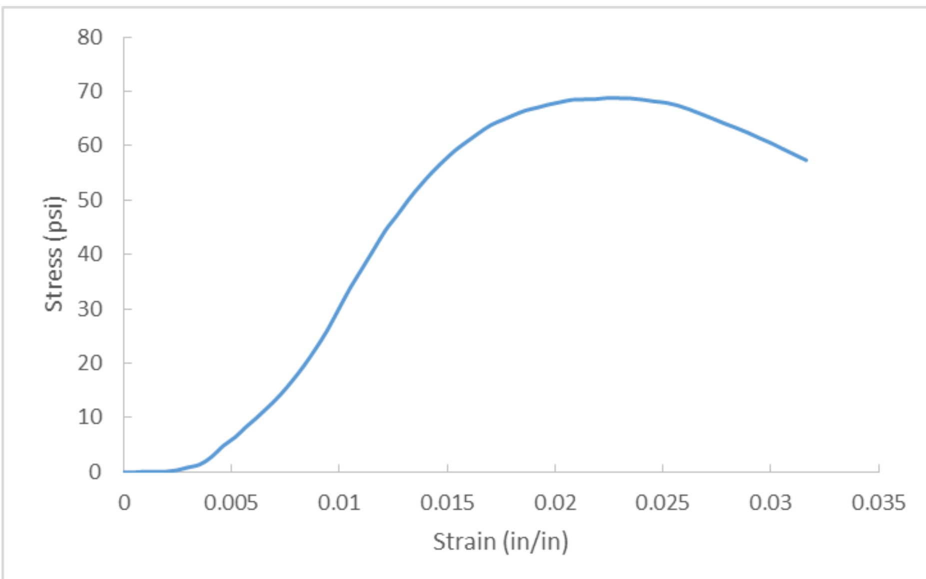

Figure A.274. D32-7, OC= 17.2%, CF= 400 pcf, T=56 Days.

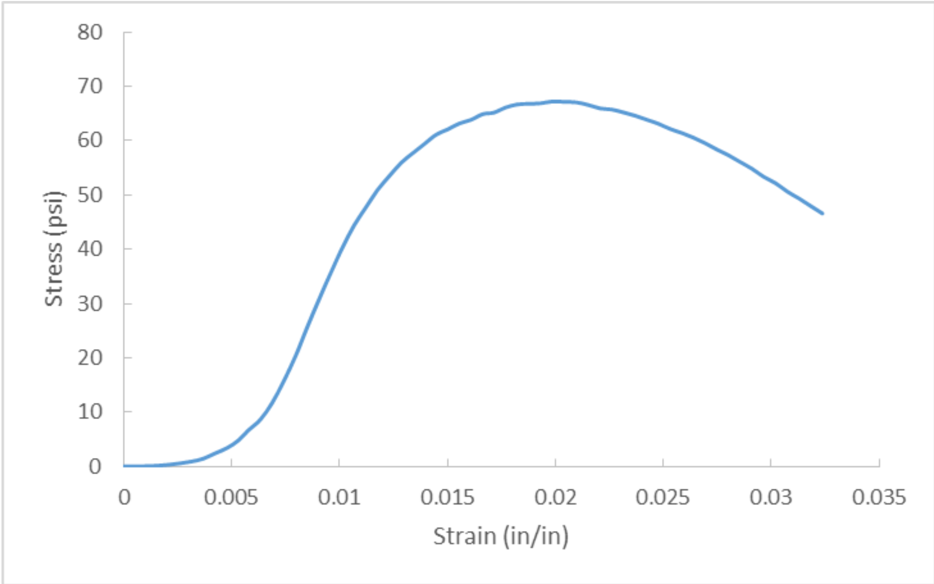

Figure A.275. D32-8, OC= 17.2%, CF= 400 pcf, T=56 Days.

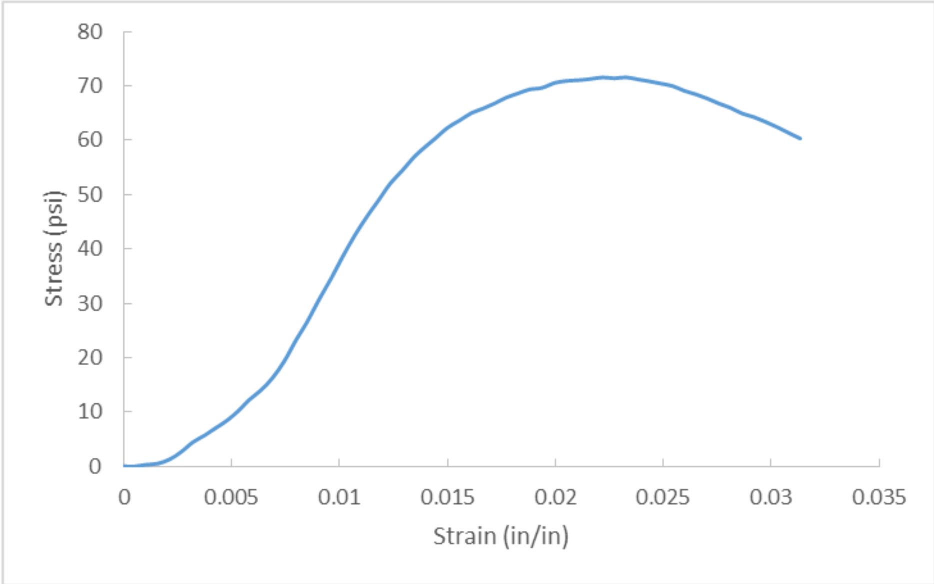

Figure A.276. D32-9, OC= 17.2%, CF= 400 pcf, T=56 Days.

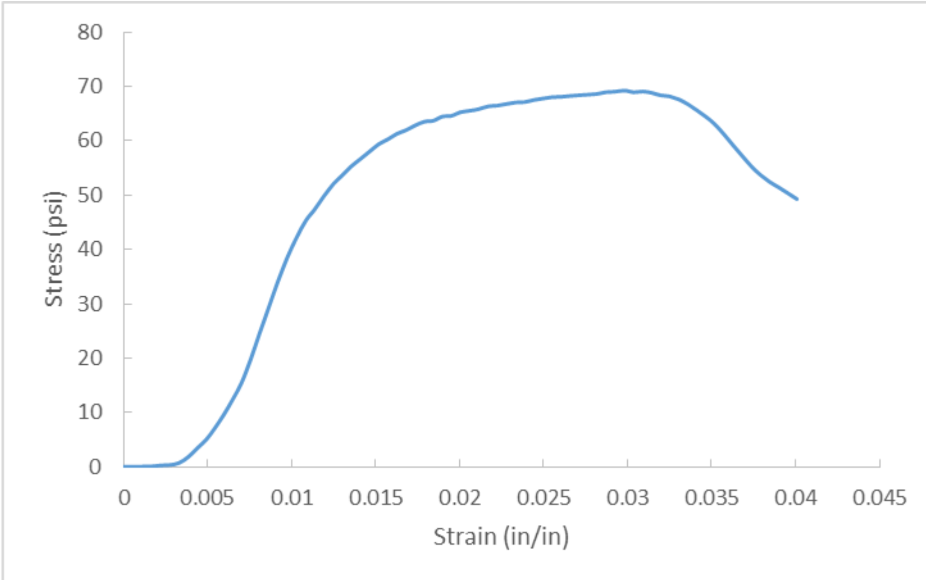

Figure A.277. D33-7, OC= 25.1%, CF= 400 pcf, T=56 Days.

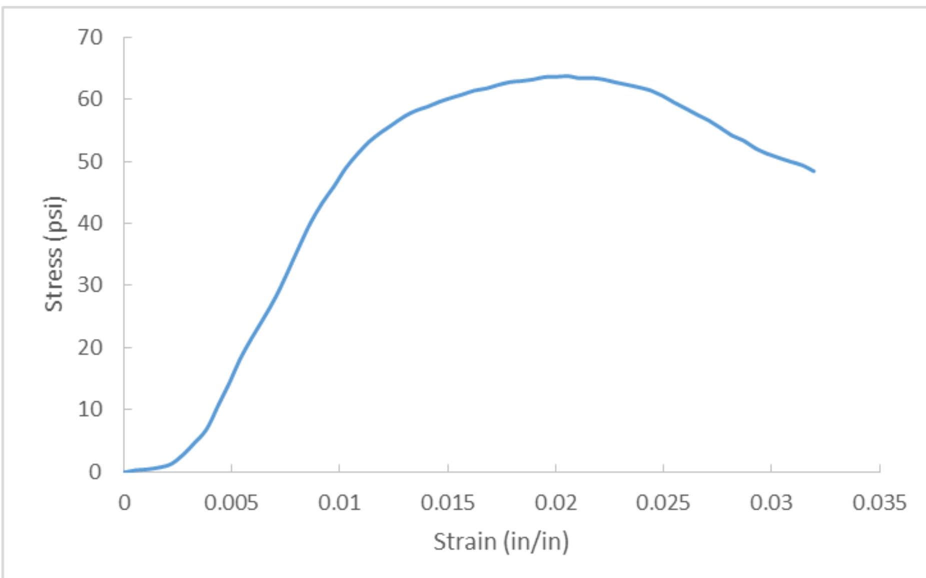

Figure A.278. D33-8, OC= 25.1%, CF= 400 pcf, T=56 Days.

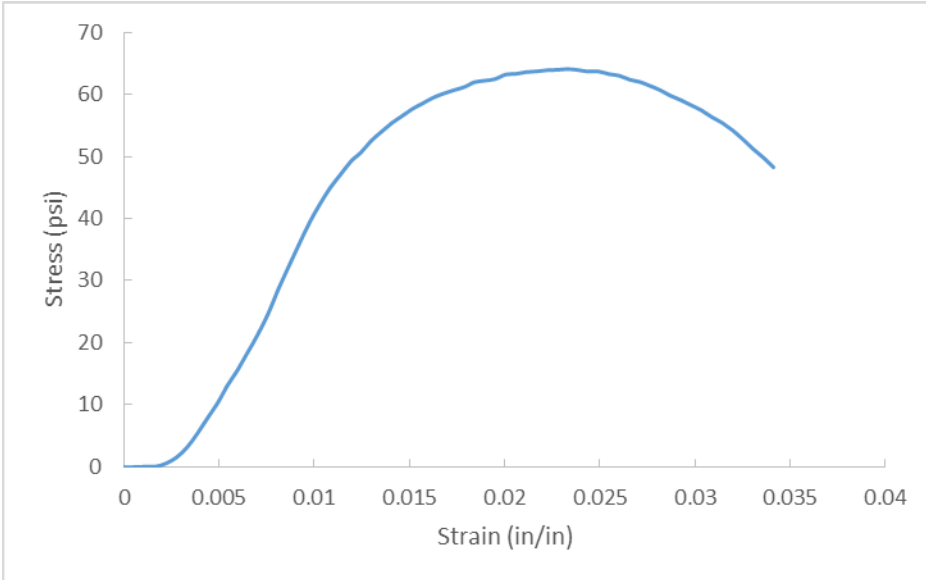

Figure A.279. D33-9, OC= 25.1%, CF= 400 pcf, T=56 Days.

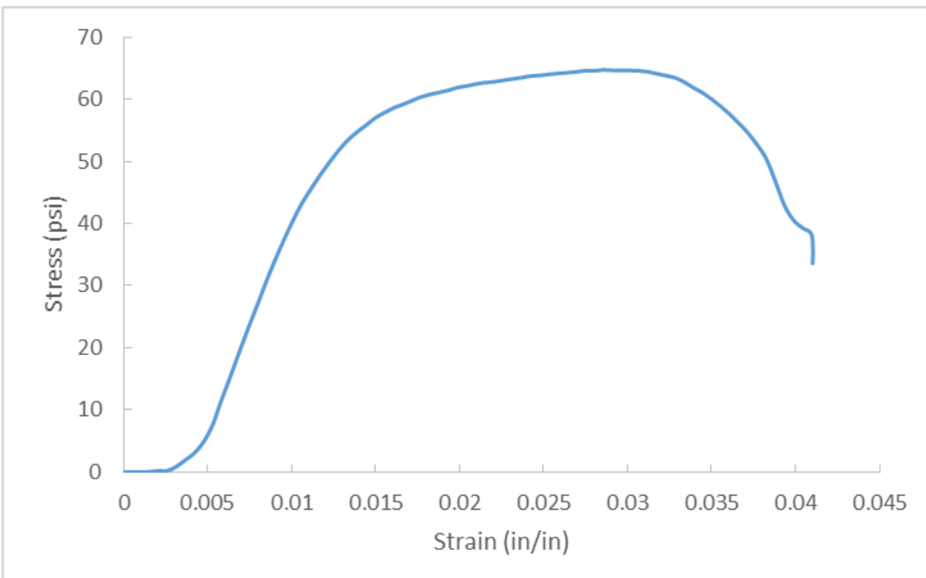

Figure A.280. D34-7, OC= 40.9%, CF= 400 pcf, T=56 Days.

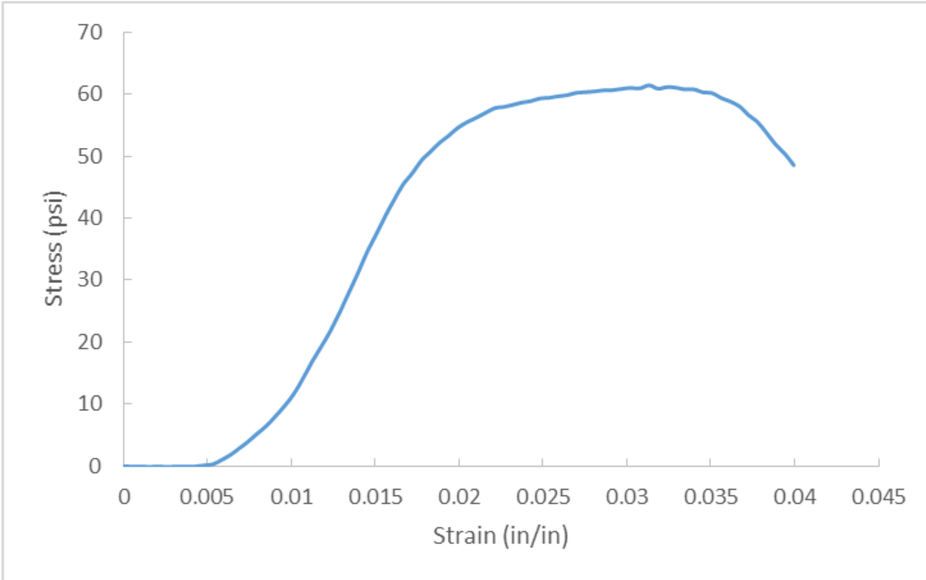

Figure A.281. D34-8, OC= 40.9%, CF= 400 pcf, T=56 Days.

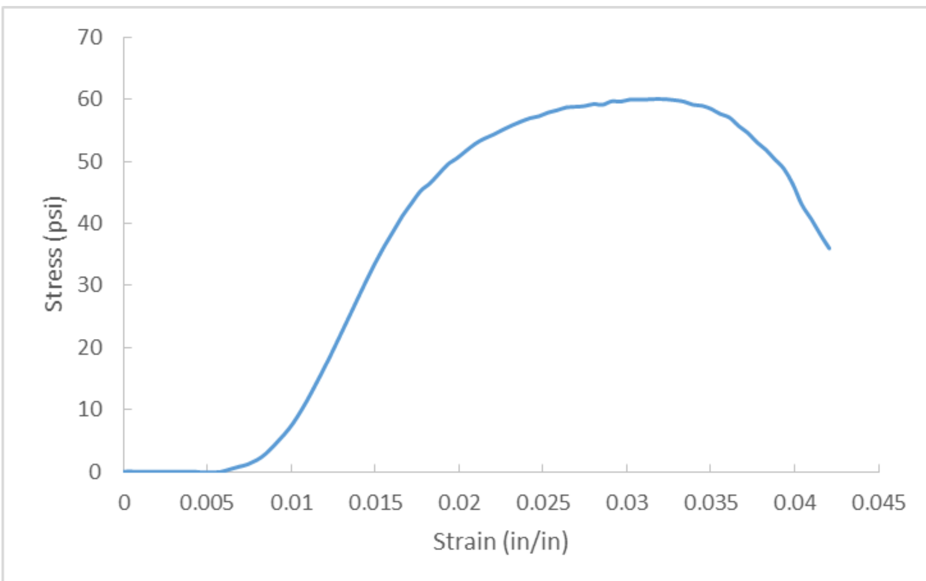

Figure A.282. D34-9, OC= 40.9%, CF= 400 pcf, T=56 Days.

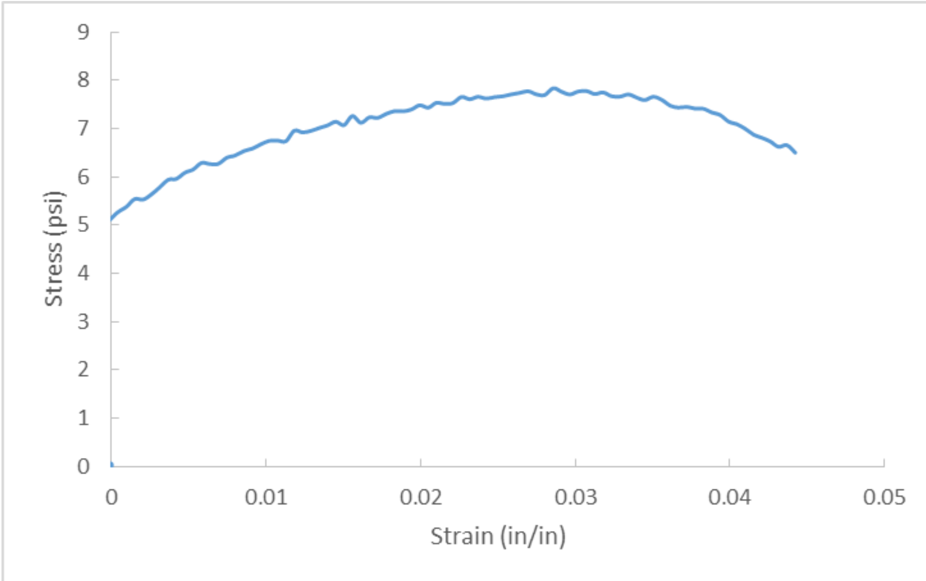

Figure A.283. D35-8, OC= 39.6%, CF= 200 pcf, T=56 Days.

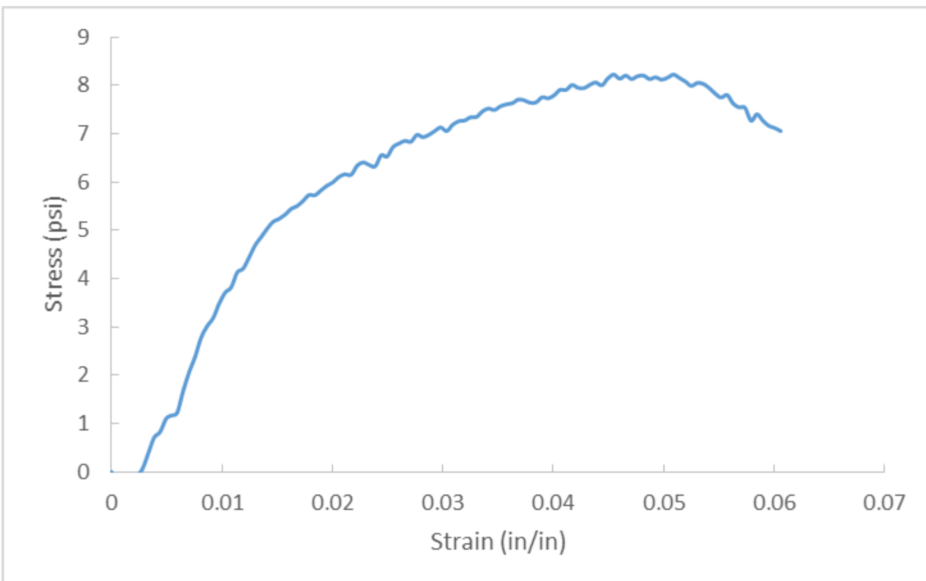

Figure A.284. D35-9, OC= 39.6%, CF= 200 pcf, T=56 Days.

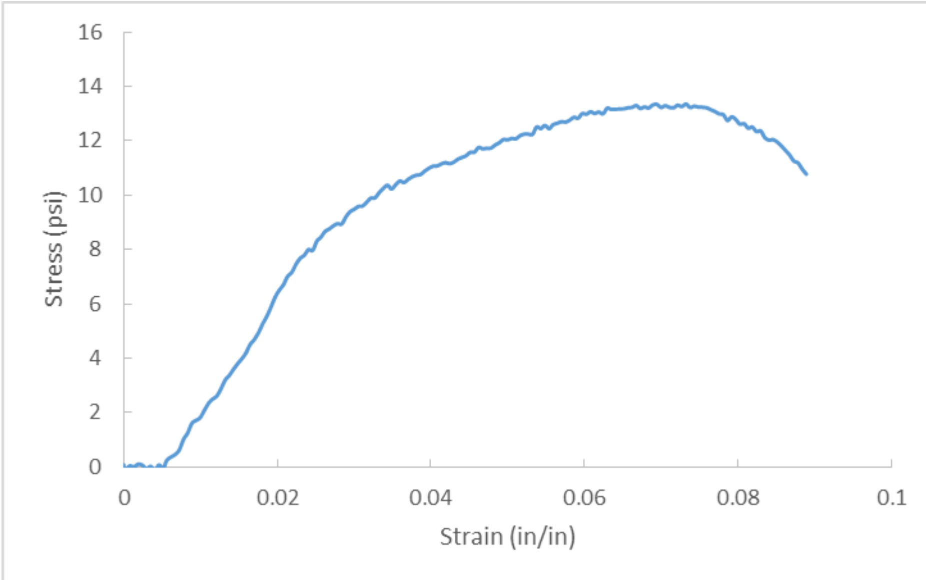

Figure A.285. D36-7, OC= 25.6%, CF= 200 pcf, T=56 Days.

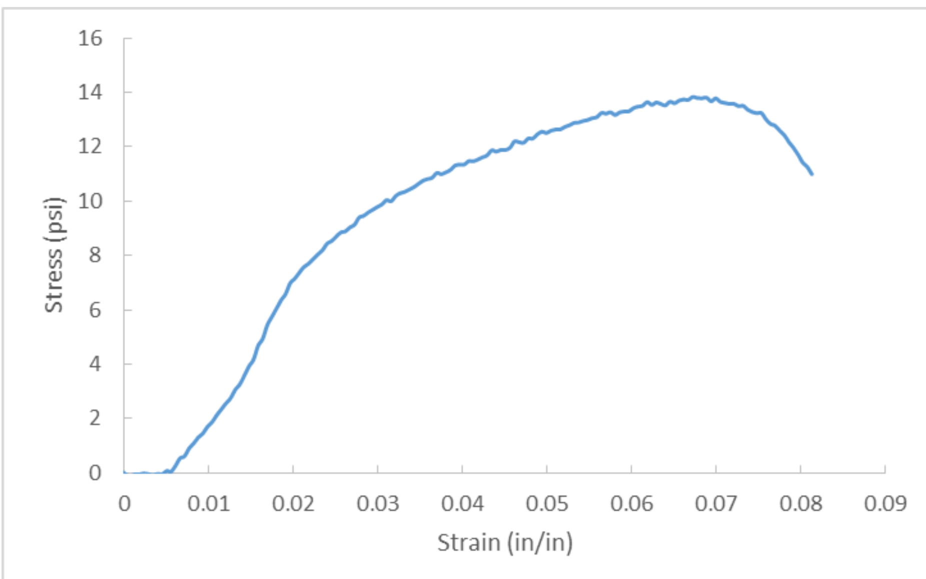

Figure A.286. D36-8, OC= 25.6%, CF= 200 pcf, T=56 Days.

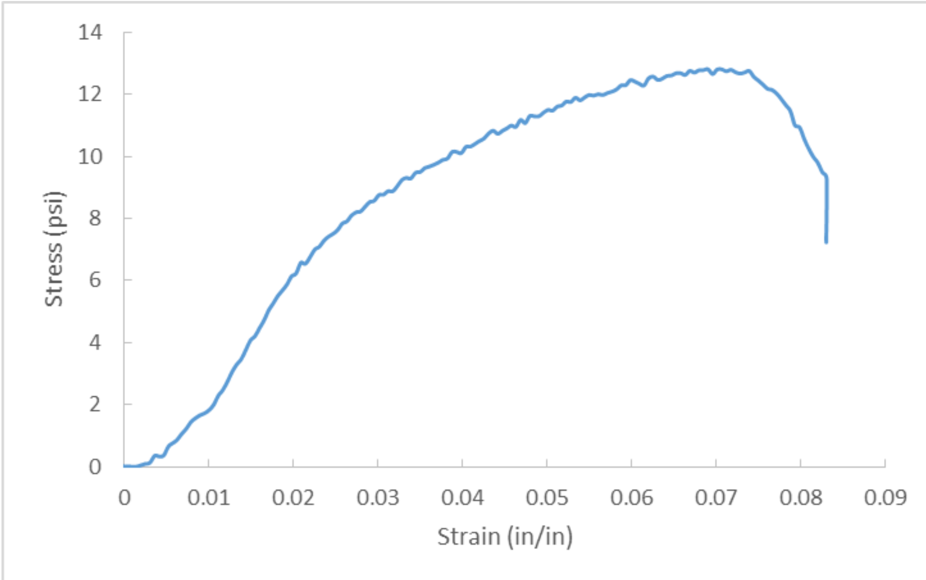

Figure A.287. D36-9, OC= 25.6%, CF= 200 pcf, T=56 Days.

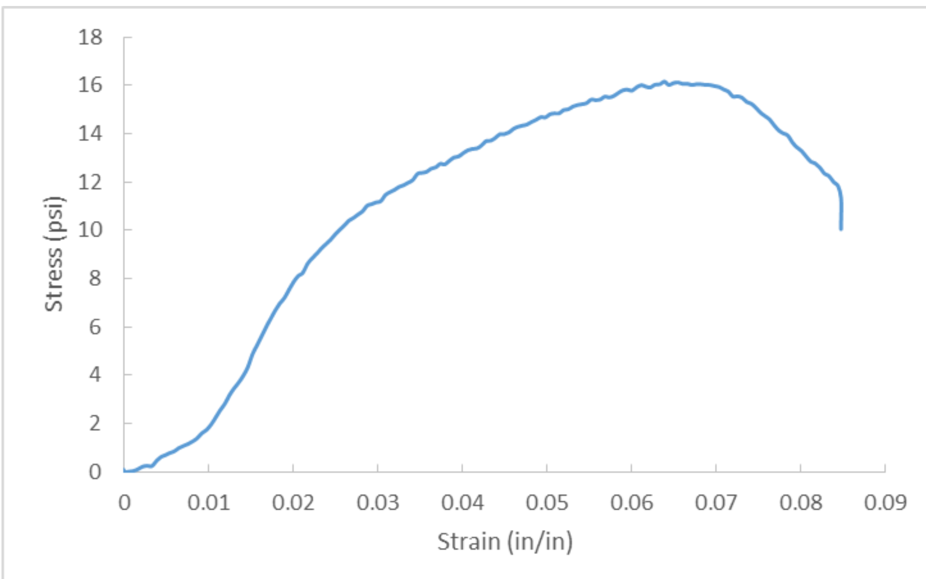

Figure A.288. D37-7, OC= 18.9%, CF= 200 pcf, T=56 Days.

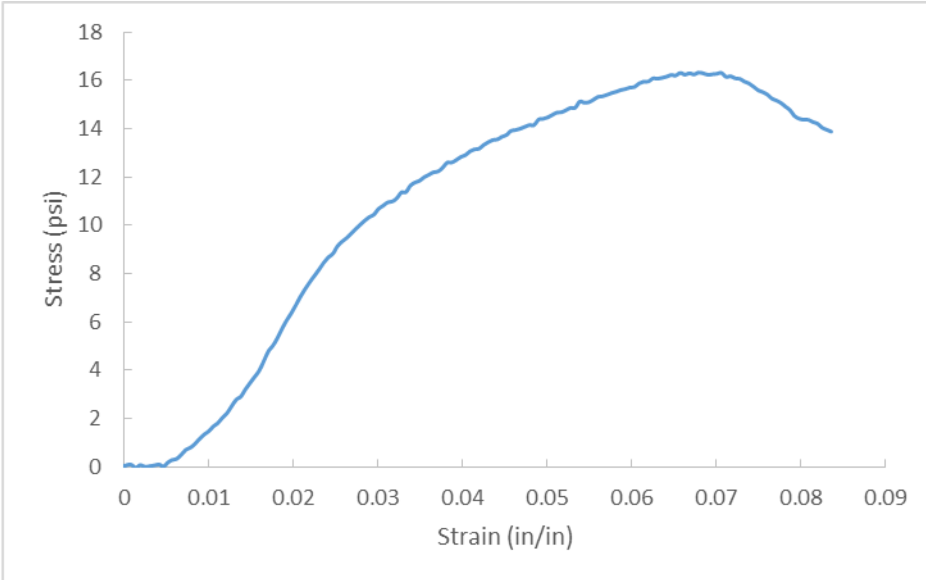

Figure A.289. D37-8, OC= 18.9%, CF= 200 pcf, T=56 Days.

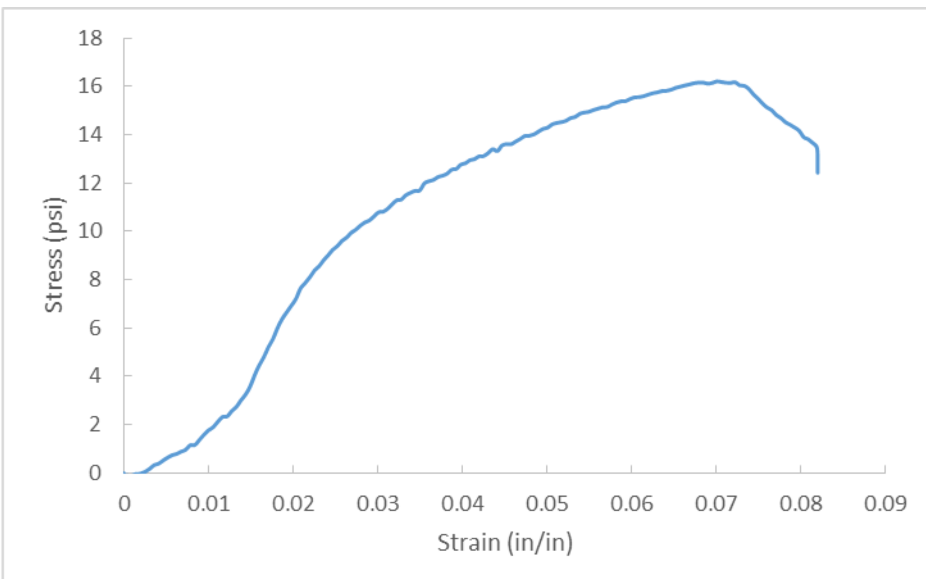

Figure A.290. D37-9, OC= 18.9%, CF= 200 pcf, T=56 Days.

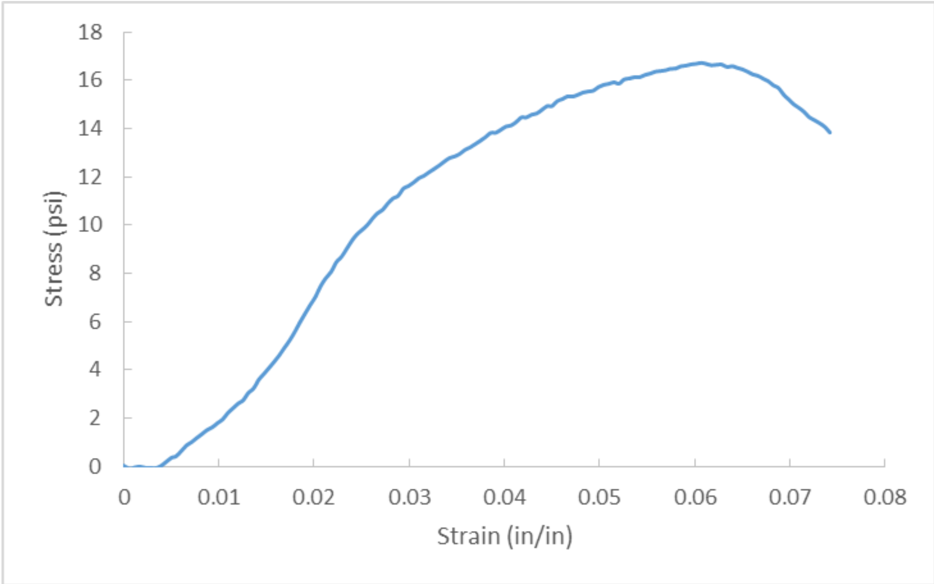

Figure A.291. D38-7, OC= 13.9%, CF= 200 pcf, T=56 Days.

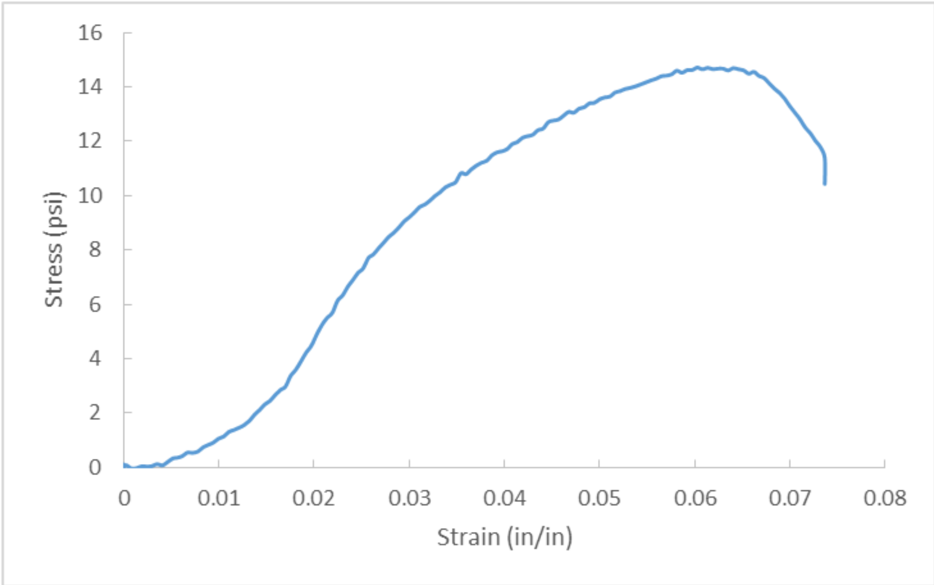

Figure A.292. D38-8, OC= 13.9%, CF= 200 pcf, T=56 Days.

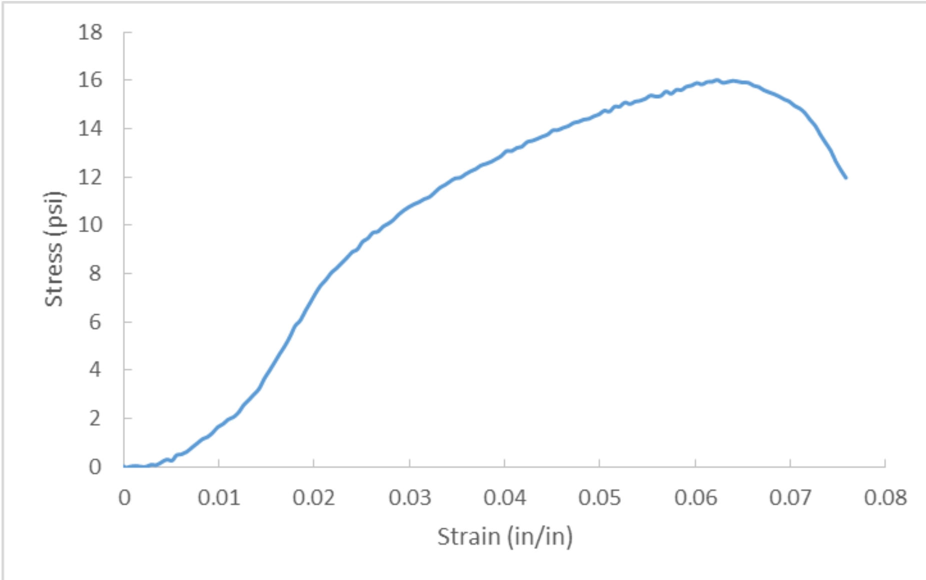

Figure A.293. D38-9, OC= 13.9%, CF= 200 pcf, T=56 Days.

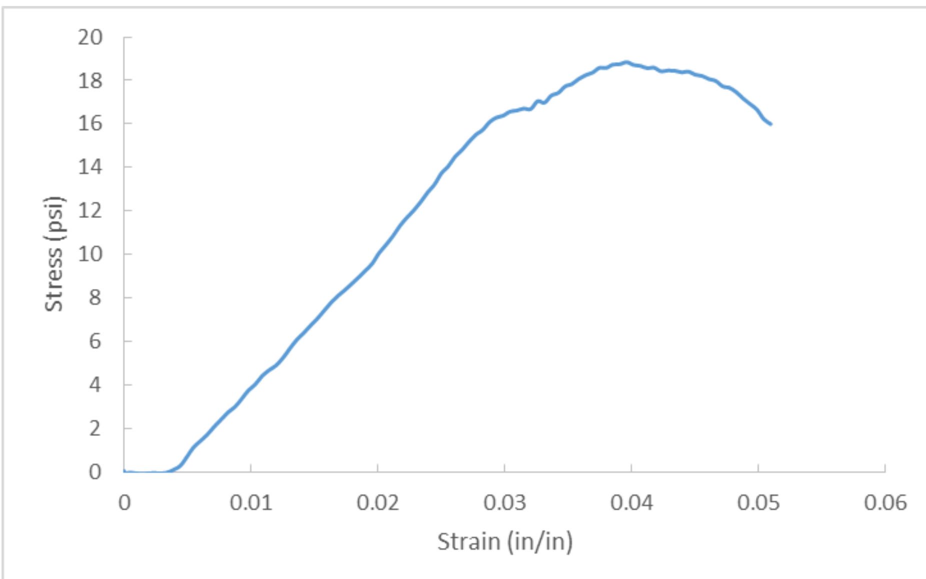

Figure A.294. D39-7, OC= 3.9%, CF= 200 pcf, T=56 Days.

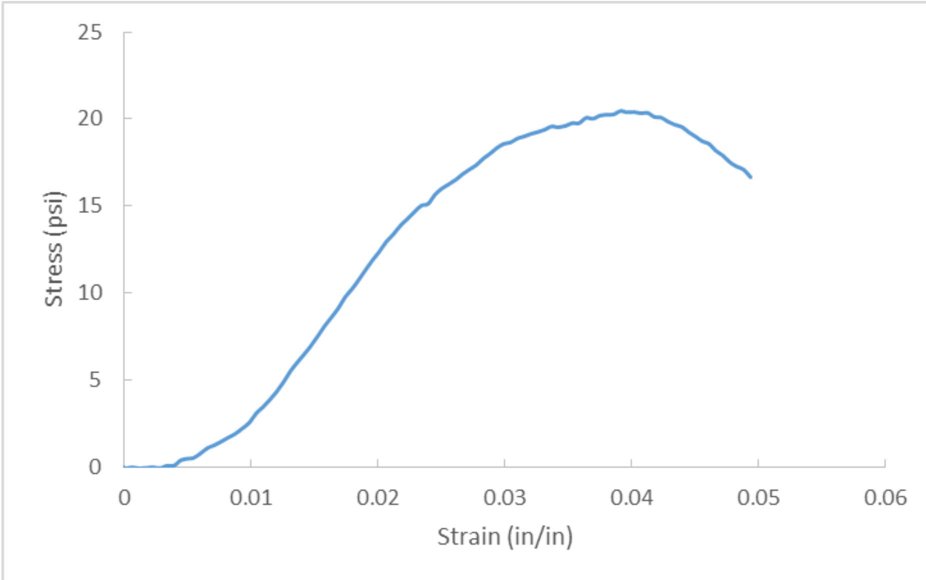

Figure A.295. D39-8, OC= 3.9%, CF= 200 pcf, T=56 Days.

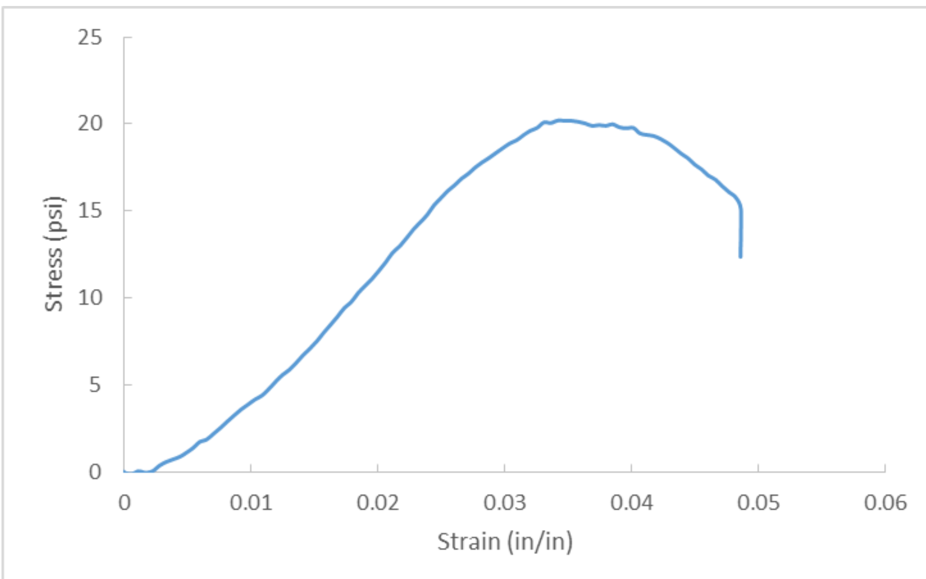

Figure A.296. D39-9, OC= 3.9%, CF= 200 pcf, T=56 Days.

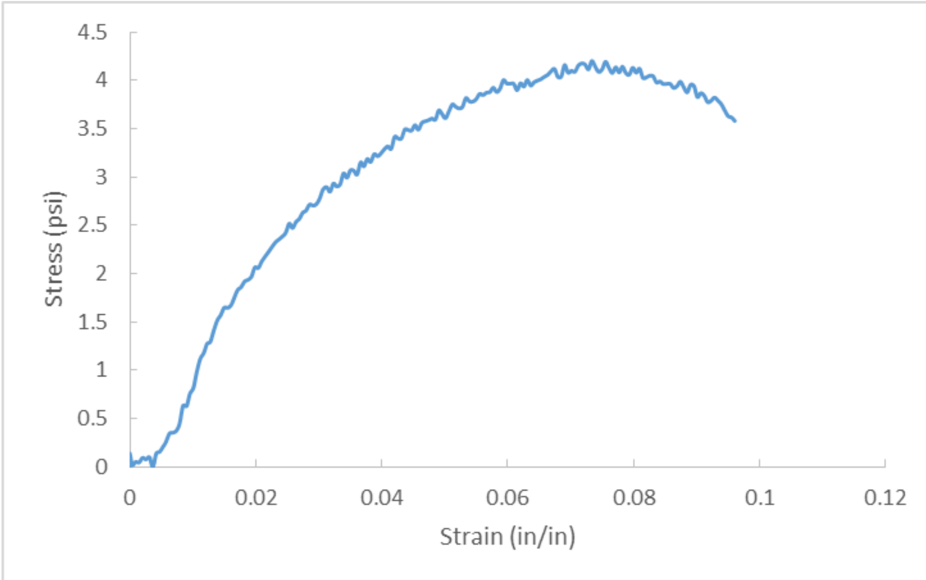

Figure A.297. D43-3, OC= 11.3%, CF= 200 pcf, T=56 Days.

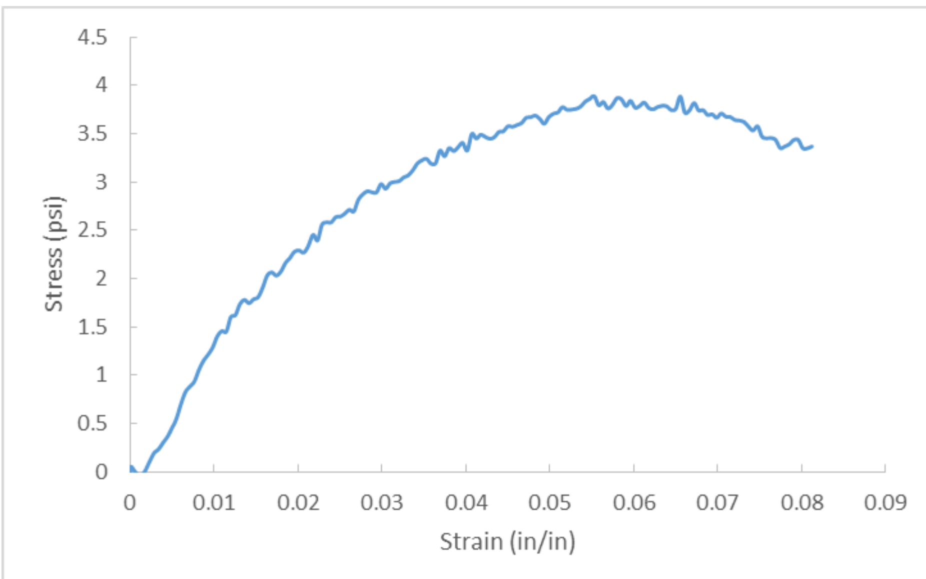

Figure A.298. D43-4, OC= 11.3%, CF= 200 pcf, T=56 Days.

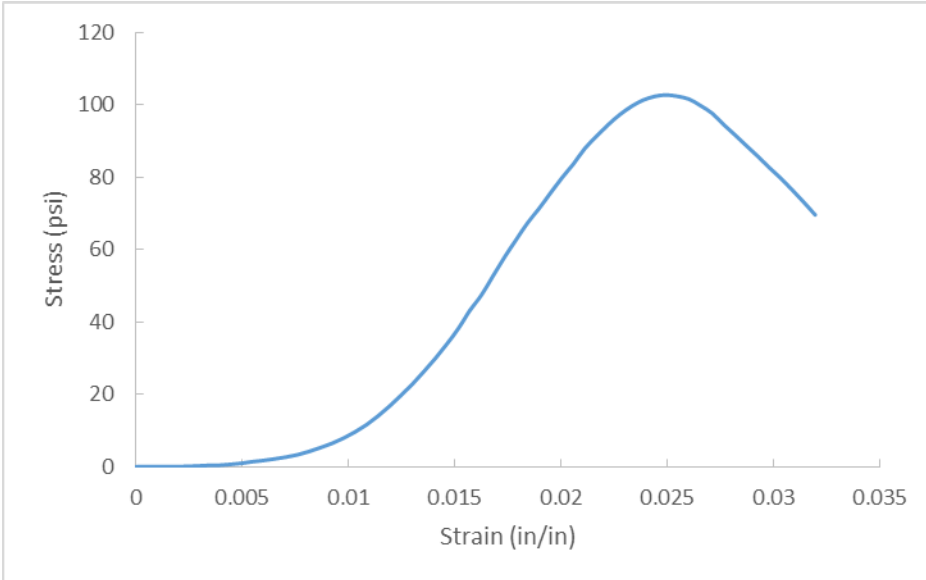

Figure A.299. D44-4, OC= 4.1%, CF= 200 pcf, T=56 Days.

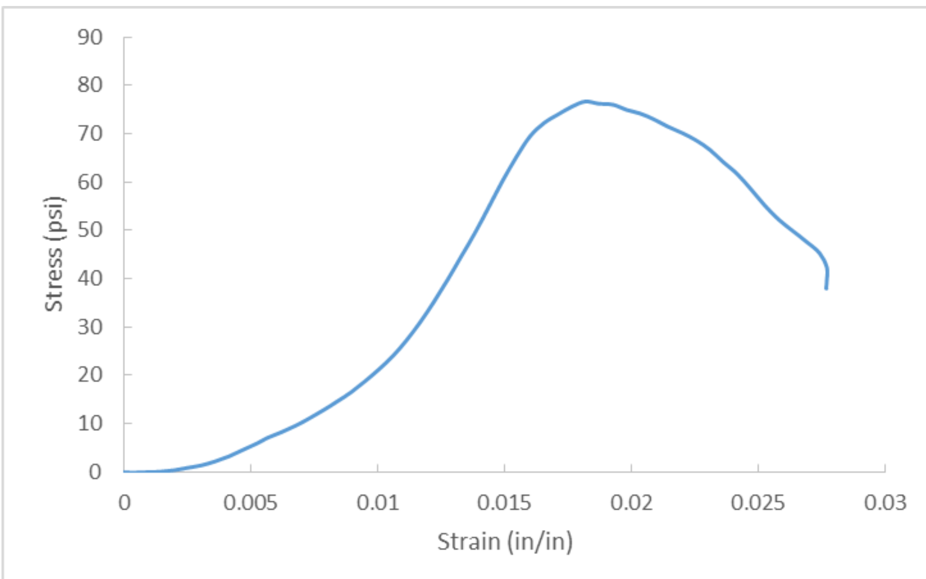

Figure A.300. D44-5, OC= 4.1%, CF= 200 pcf, T=56 Days.

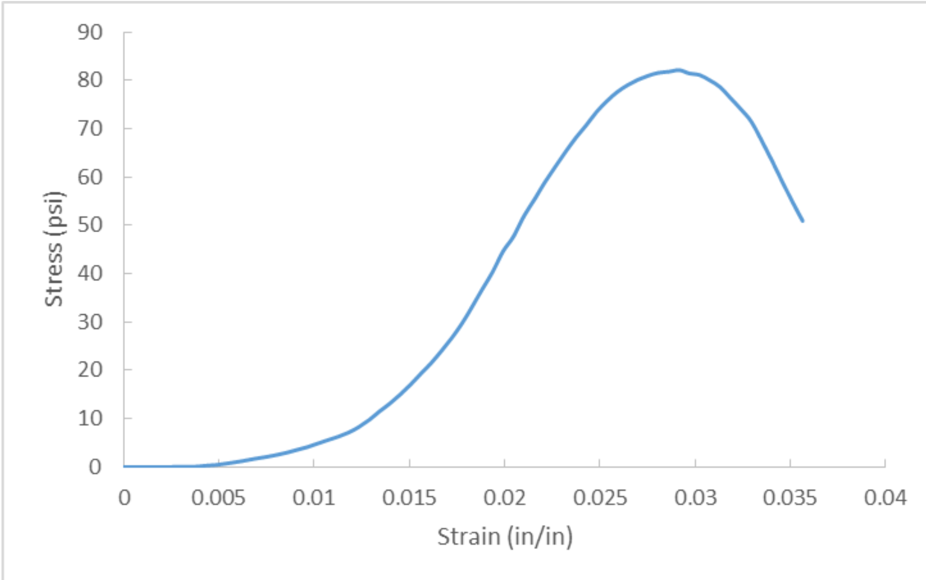

Figure A.301. D44-6, OC= 4.1%, CF= 200 pcf, T=56 Days.

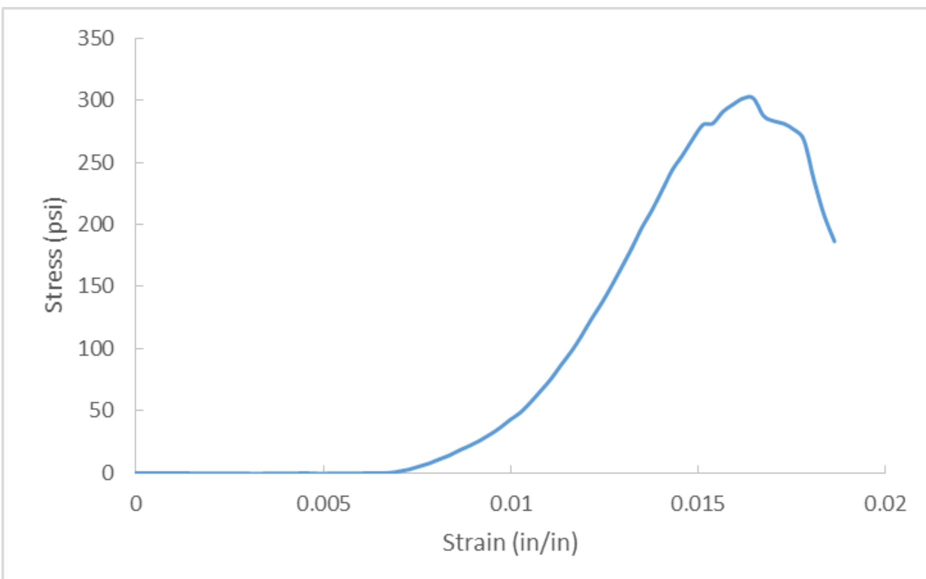

Figure A.302. D45-7, OC= 0.0%, CF= 400 pcf, T=56 Days.

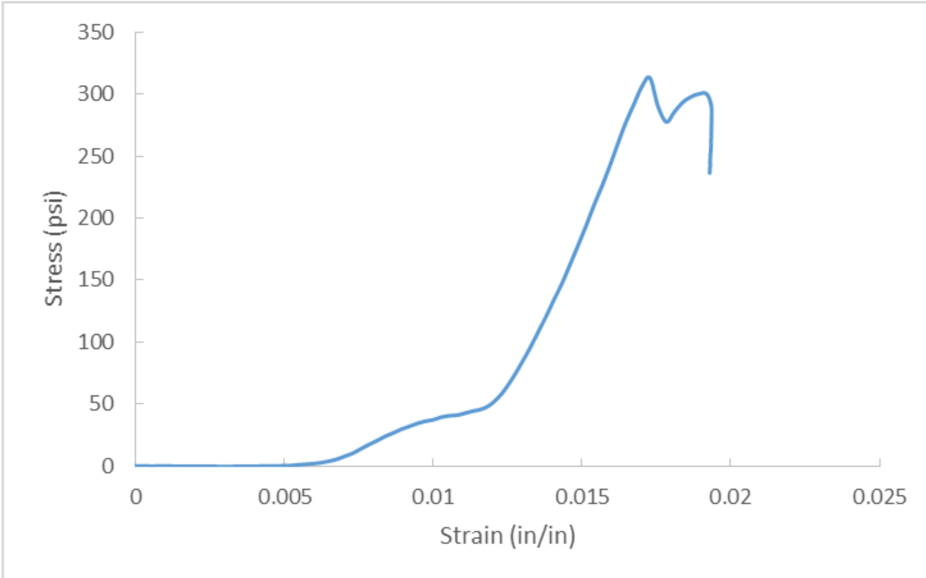

Figure A.303. D45-8, OC= 0.0%, CF= 400 pcf, T=56 Days.

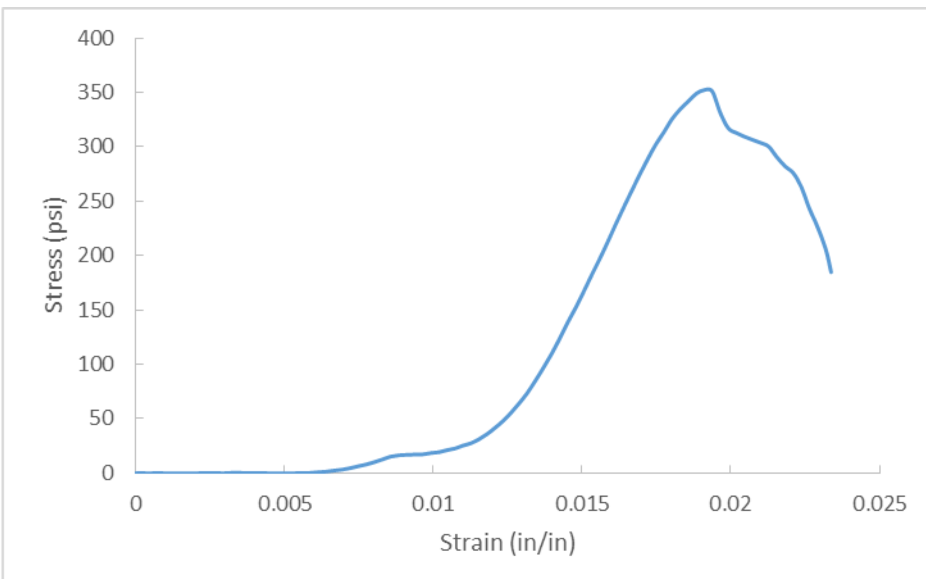

Figure A.304. D45-9, OC= 0.0%, CF= 400 pcf, T=56 Days.

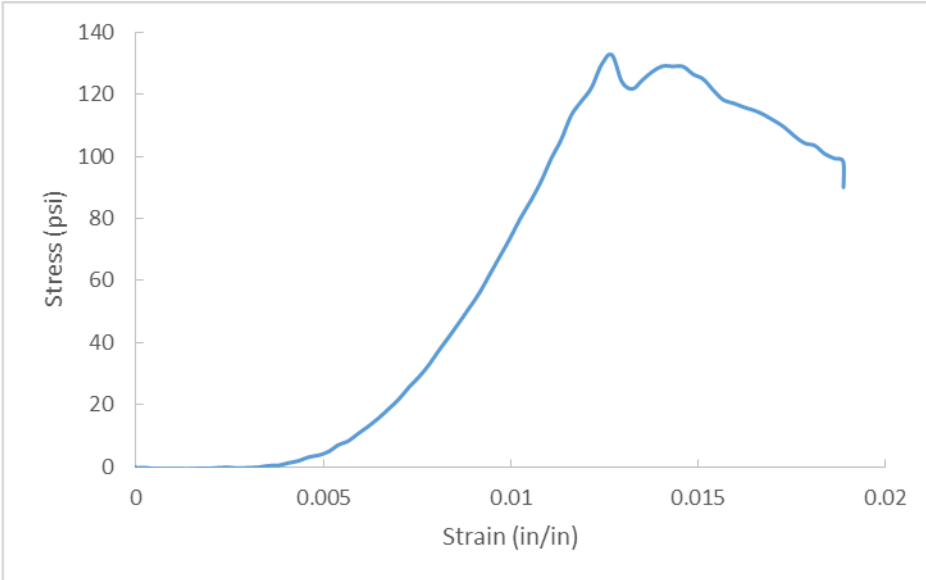

Figure A.305. D46-7, OC= 0.0%, CF= 300 pcf, T=56 Days.

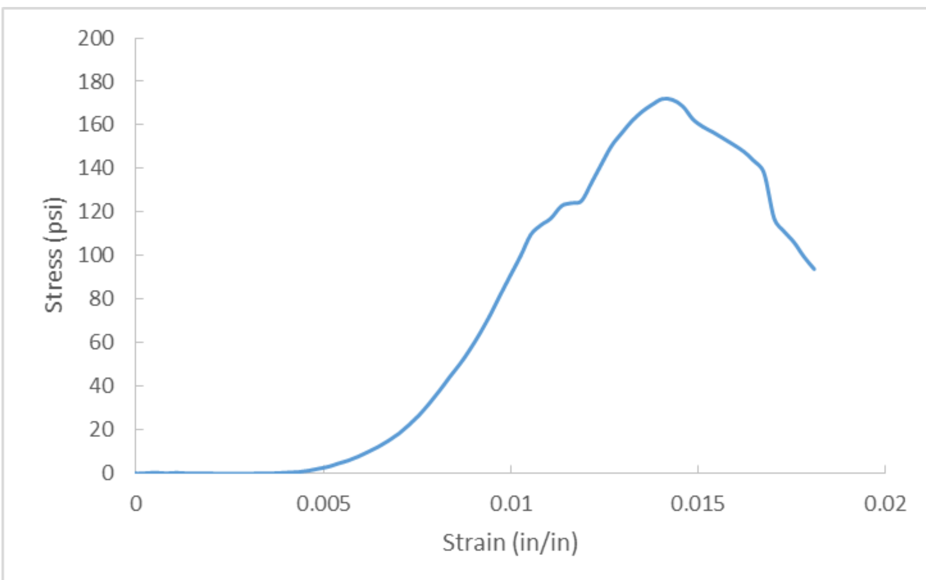

Figure A.306. D46-8, OC= 0.0%, CF= 300 pcf, T=56 Days.

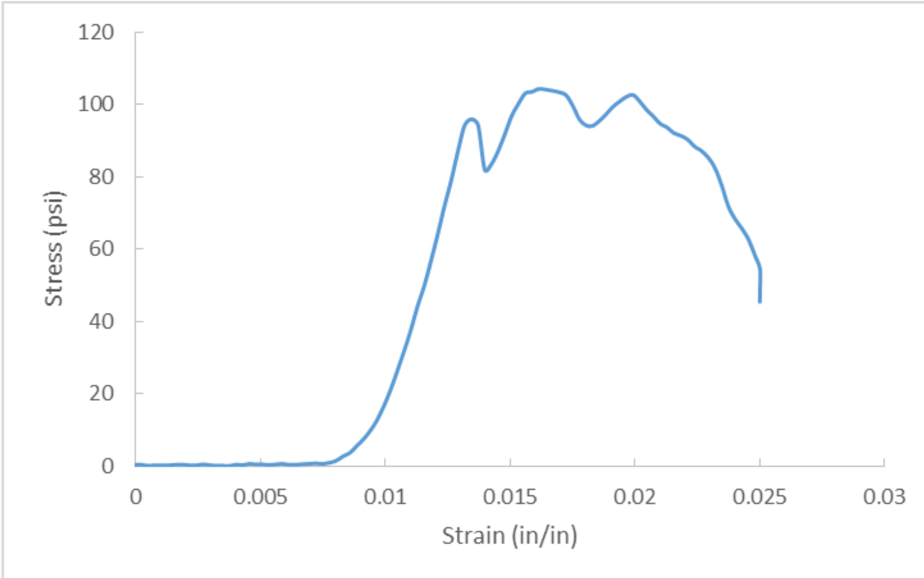

Figure A.307. D46-9, OC= 0.0%, CF= 300 pcf, T=56 Days.

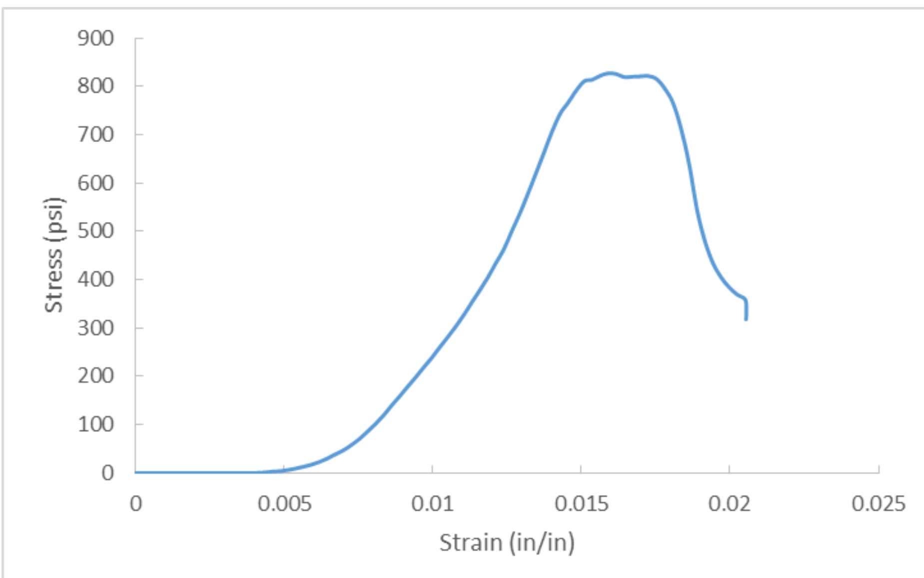

Figure A.308. D47-7, OC= 0.0%, CF= 400 pcf, T=56 Days.

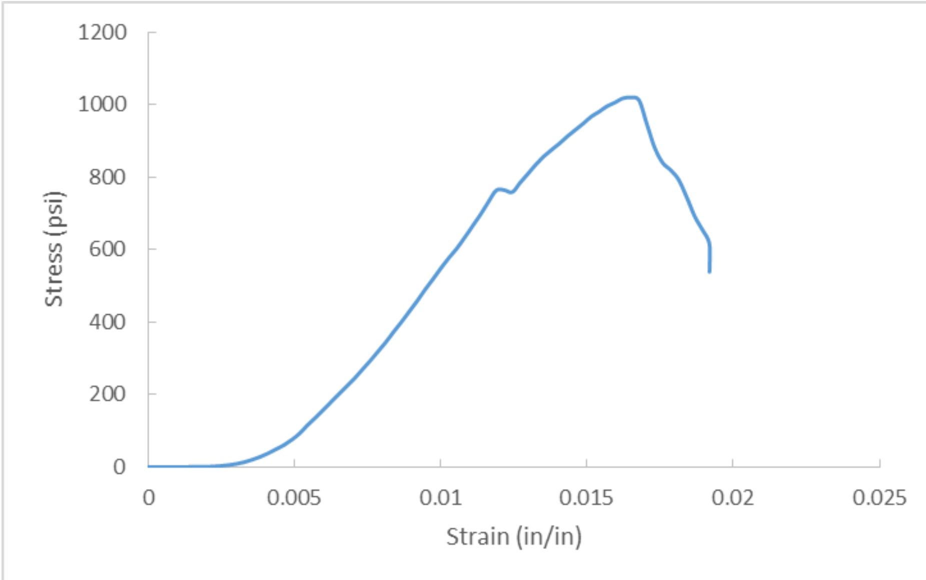

Figure A.309. D47-8, OC= 0.0%, CF= 400 pcf, T=56 Days.

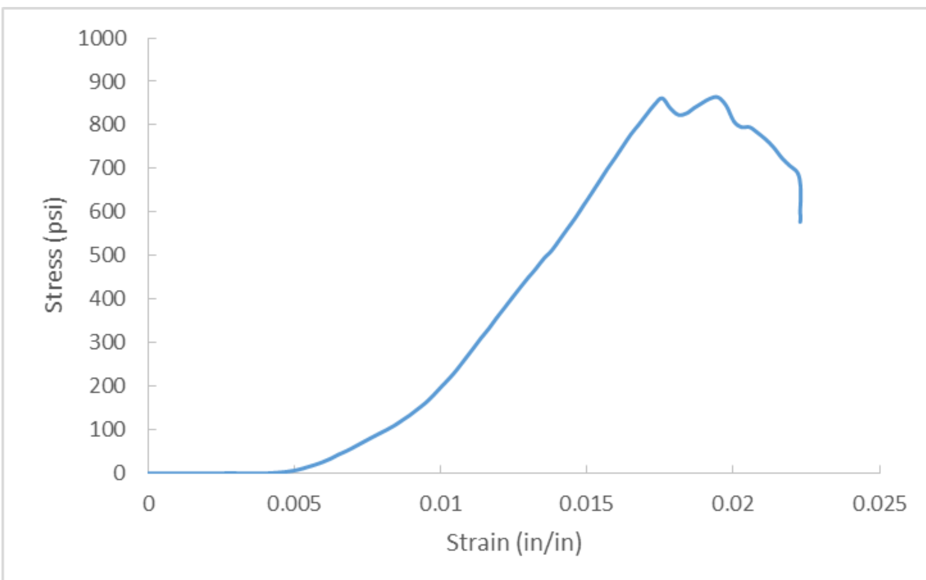

Figure A.310. D47-9, OC= 0.0%, CF= 400 pcf, T=56 Days.

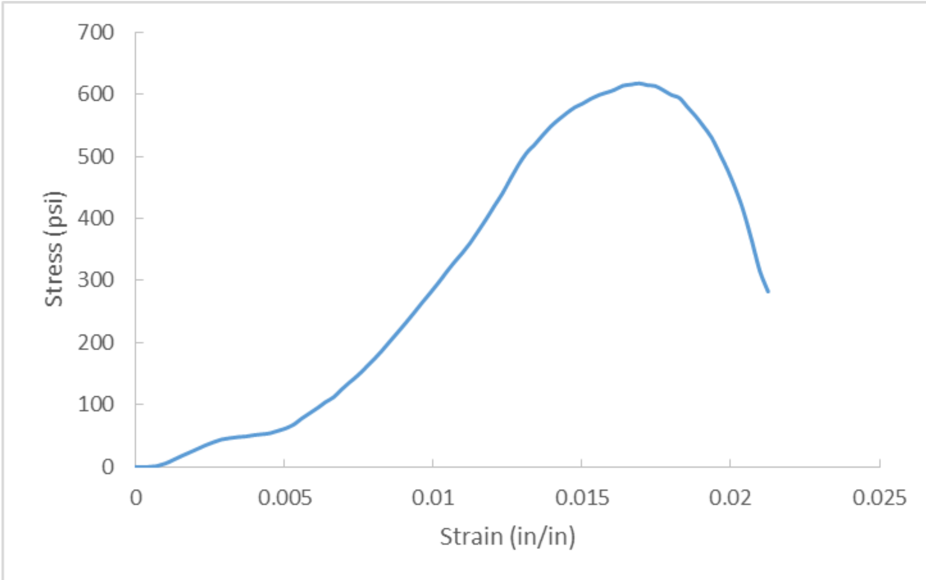

Figure A.311. D48-7, OC= 0.0%, CF= 300 pcf, T=56 Days.

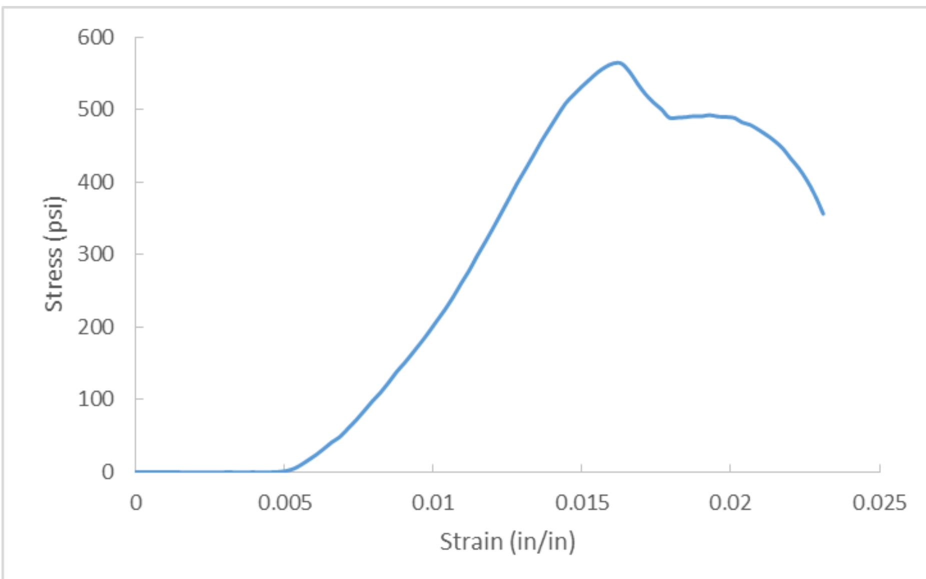

Figure A.312. D48-8, OC= 0.0%, CF= 300 pcf, T=56 Days.

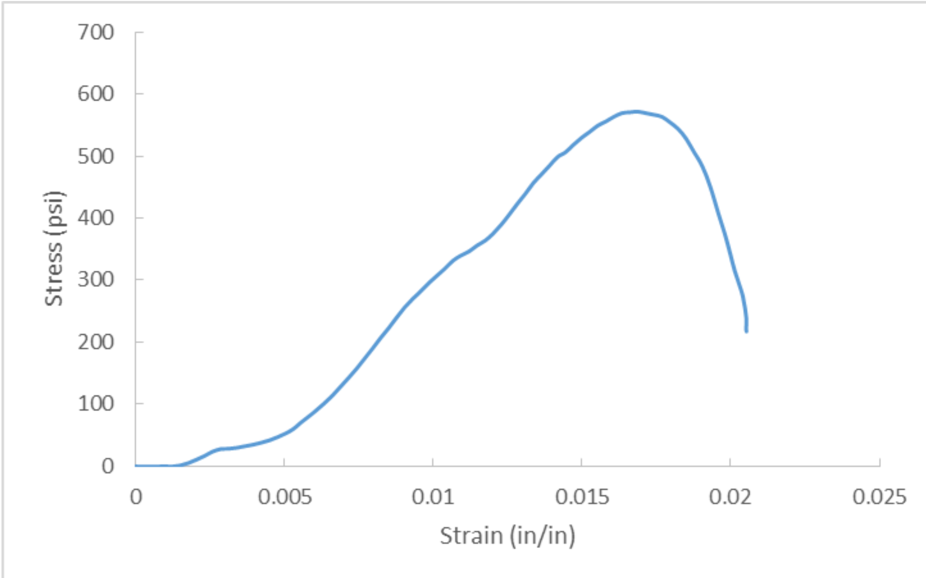

Figure A.313. D48-9, OC= 0.0%, CF= 300 pcf, T=56 Days.

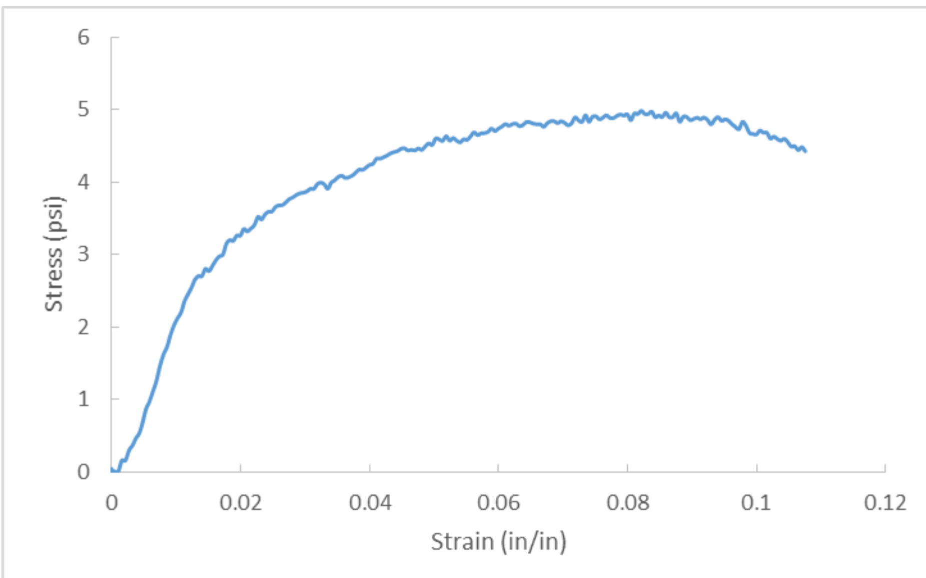

Figure A.314. W49-7, OC= 43.8%, CF= 300 pcf, T=56 Days.

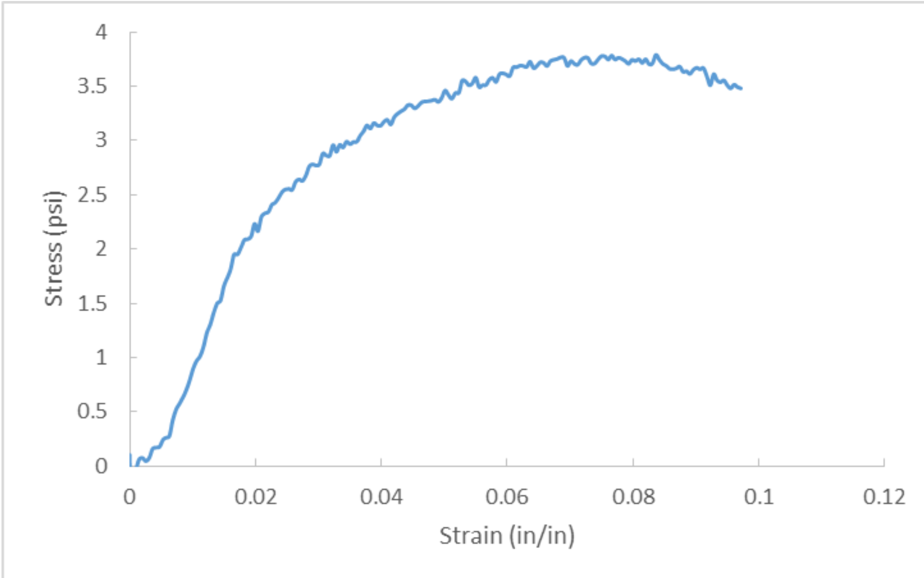

Figure A.315. W49-9, OC= 43.8%, CF= 300 pcf, T=56 Days.

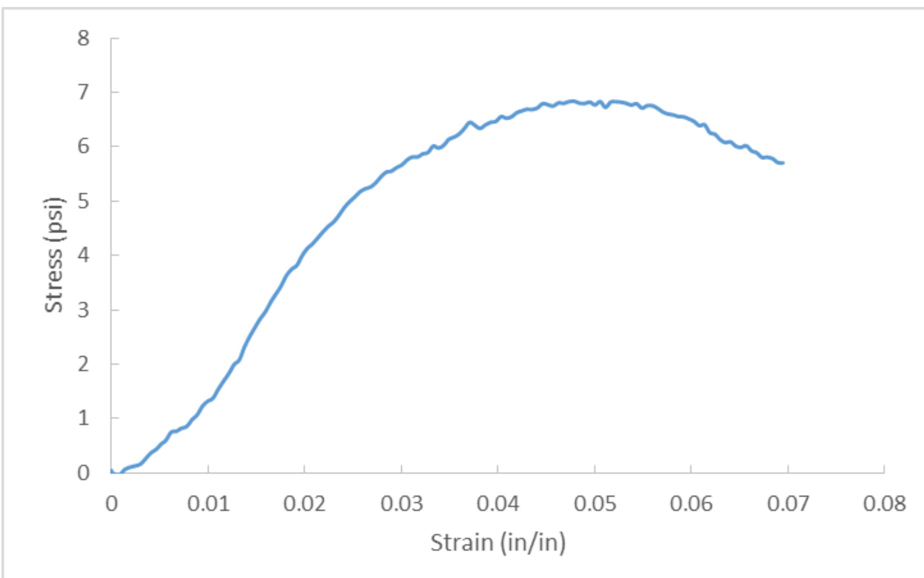

Figure A.316. W50-7, OC= 43.8%, CF= 400 pcf, T=56 Days.

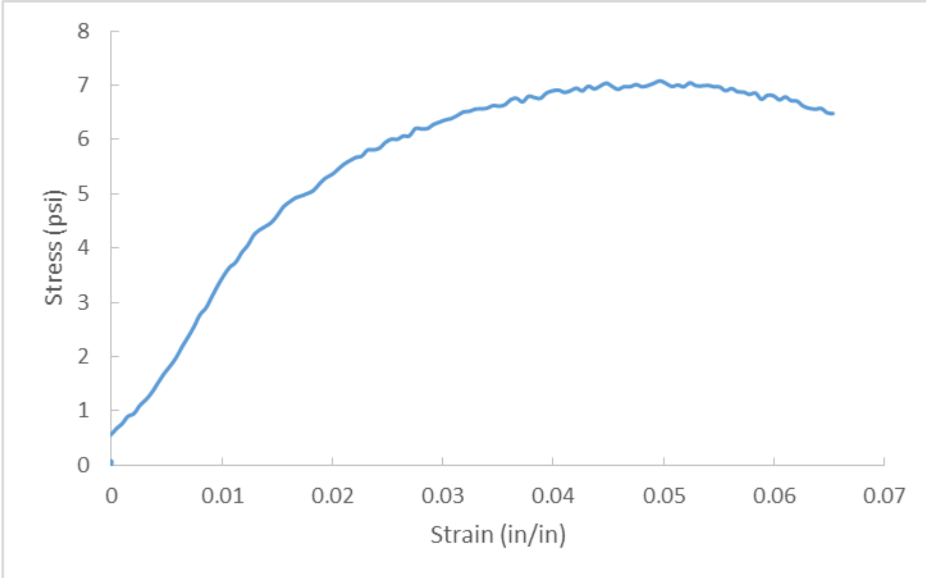

Figure A.317. W50-8, OC= 43.8%, CF= 400 pcf, T=56 Days.

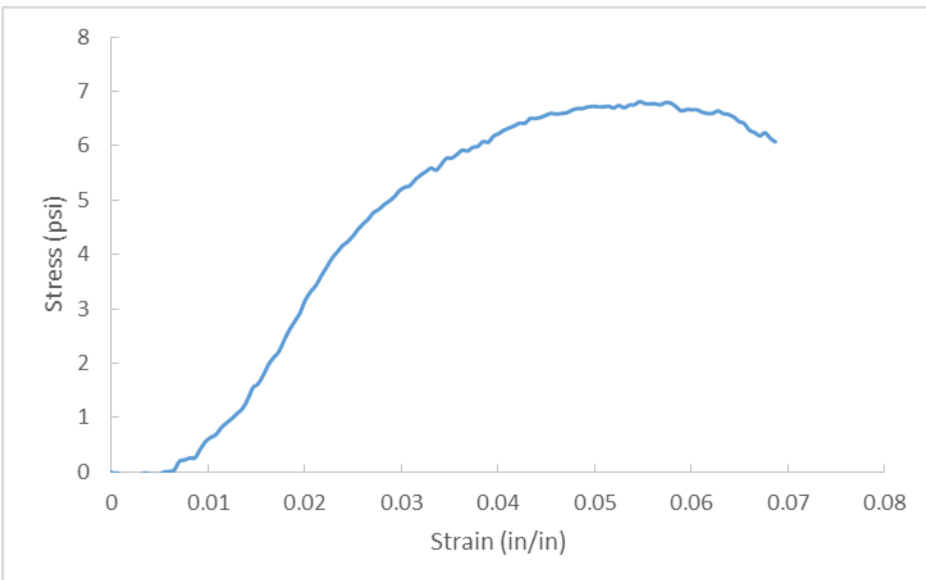

Figure A.318. W50-9, OC= 43.8%, CF= 400 pcf, T=56 Days.

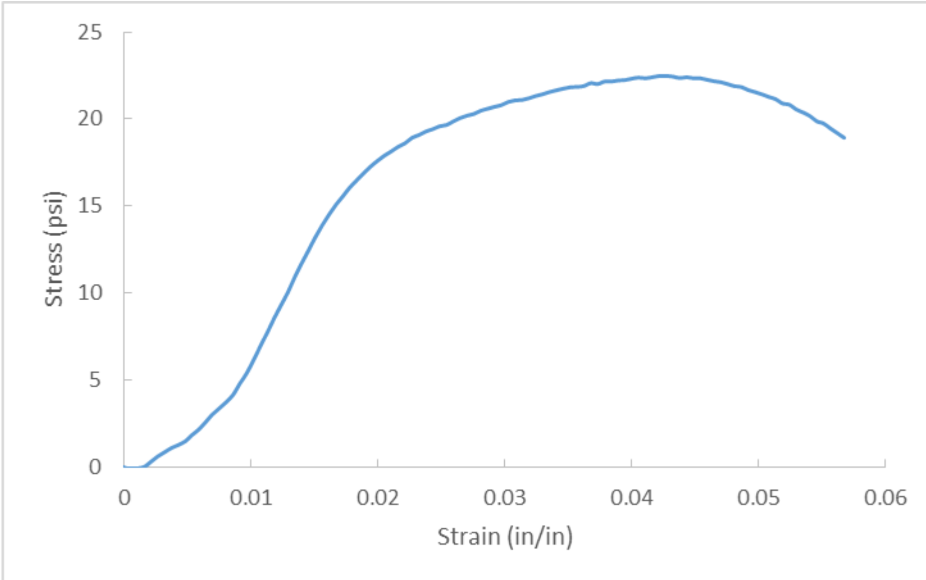

Figure A.319. W51-7, OC= 43.8%, CF= 500 pcf, T=56 Days.

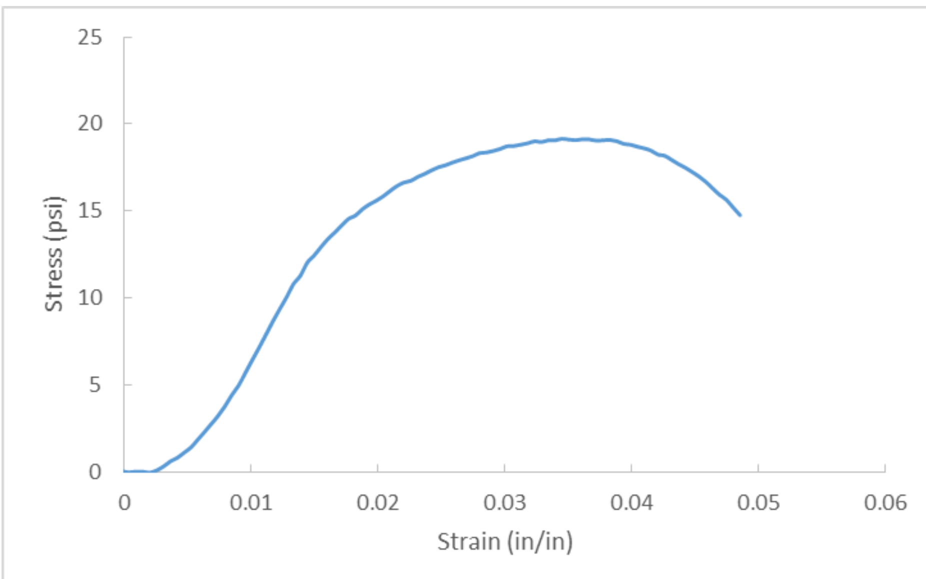

Figure A.320. W51-9, OC= 43.8%, CF= 500 pcf, T=56 Days.

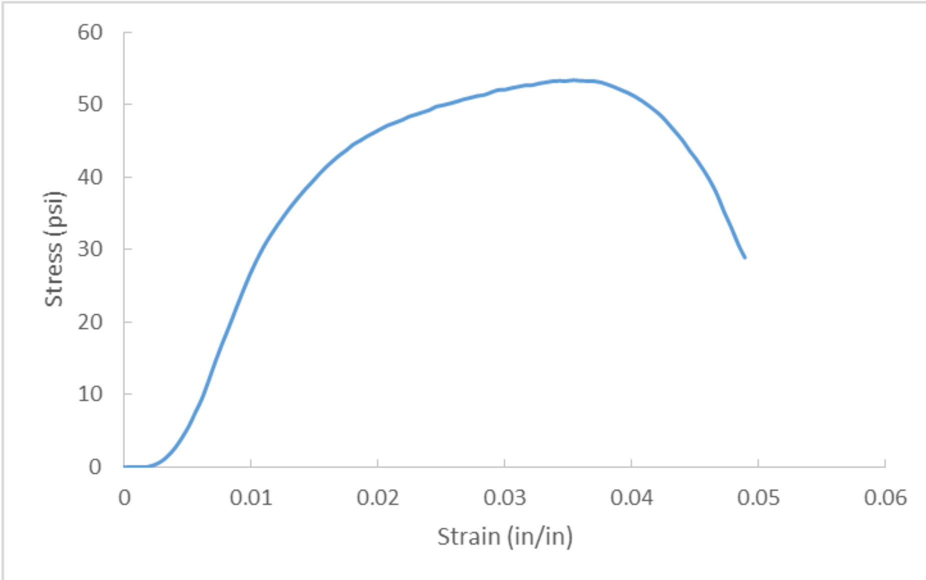

Figure A.321. D55-4, OC= 42.1%, CF= 500 pcf, T=56 Days.

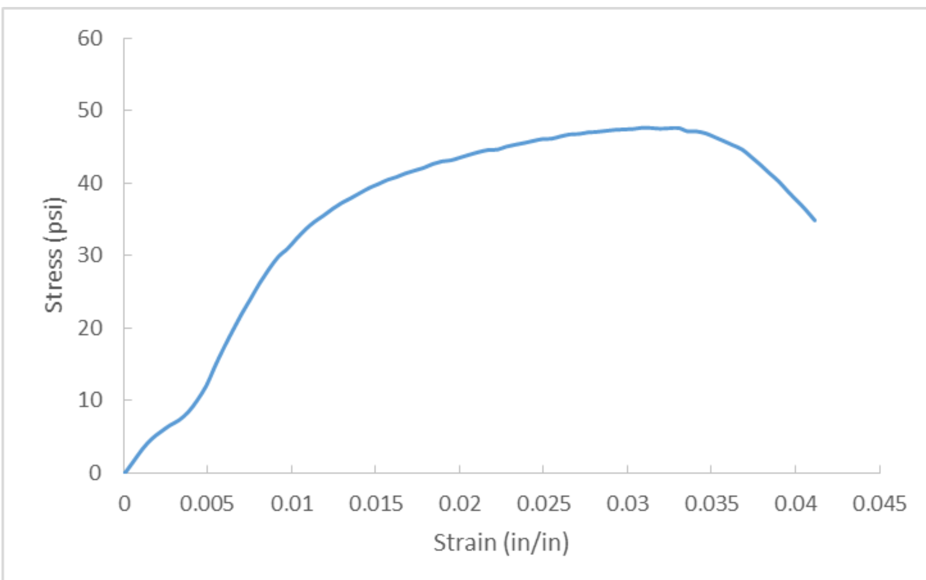

Figure A.322. D55-5, OC= 42.1%, CF= 500 pcf, T=56 Days.

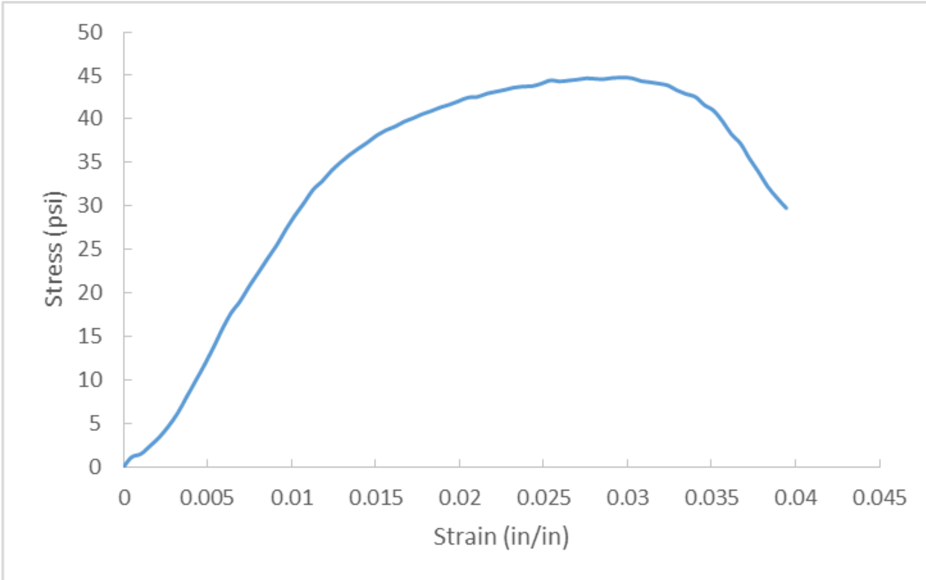

Figure A.323. D55-6, OC= 42.1%, CF= 500 pcf, T=56 Days.

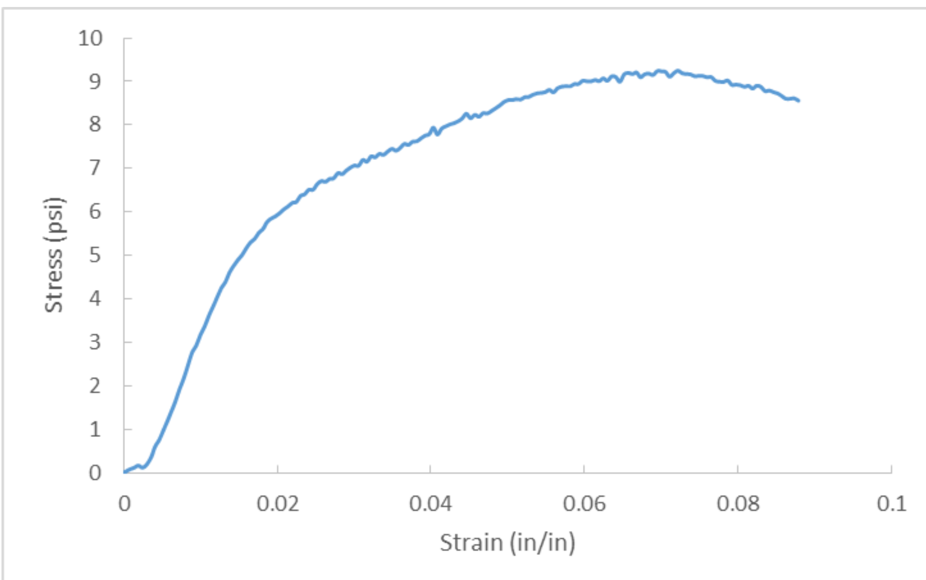

Figure A.324. D57-4, OC= 42.1%, CF= 500 pcf, T=56 Days.

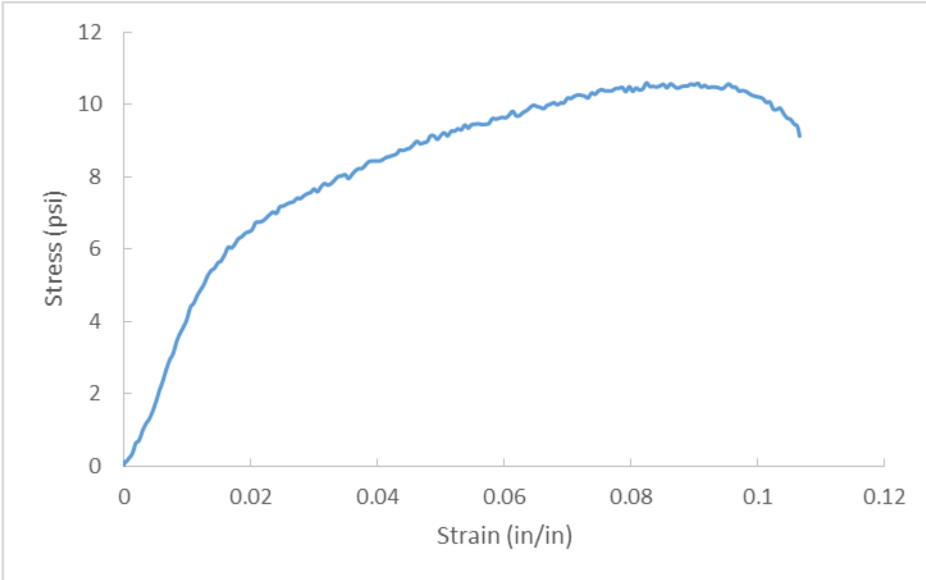

Figure A.325. D57-5, OC= 42.1%, CF= 500 pcf, T=56 Days.

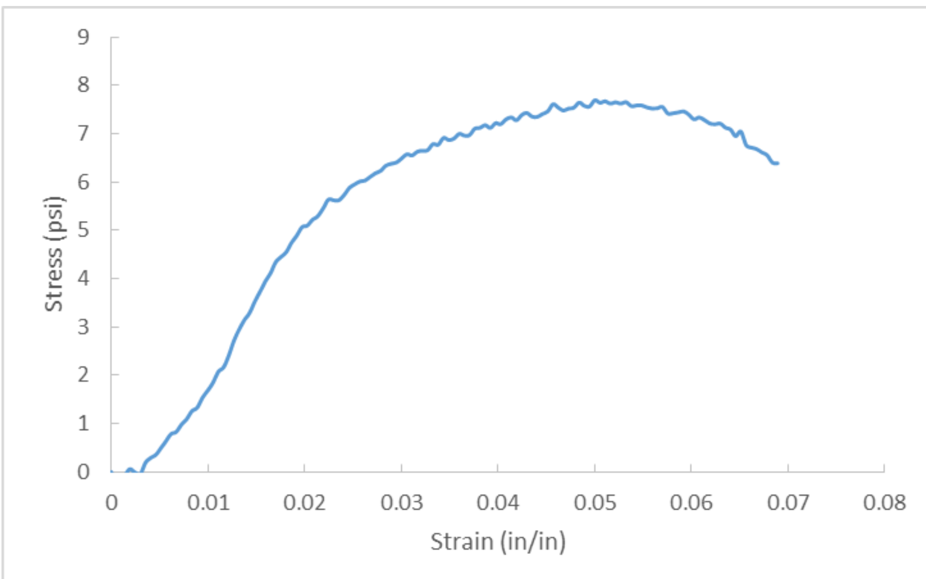

Figure A.326. D57-6, OC= 42.1%, CF= 500 pcf, T=56 Days.

Appendix B: Cement Factor Volume Correction

When applying small scale laboratory data to the field, it is important to account for the increased volume due to mixing. In the laboratory, mix designs were created using a unit volume design. This was then scaled down to the appropriate volume. Within the unit volume, the amount of binder (cement) was fixed for each mix design. Enough soil was then added to fill the unit volume. Therefore a cement content of 300 pcy was exactly 300 pcy in the mix because the volume was fixed. This is not as simple outside of the laboratory.

When cement or grout is added to a system in the field, the volume of the system increases. Therefore the equation that one might use to determine the amount of cement needed is no longer valid. This equation is shown below.

$$C = CF * V_s$$

Where,

- C = Cement needed (lb)
- V_s = Volume of soil to be treated (ft^3)

Realistically, the cement factor should be multiplied by the total volume of the system after treatment. The following equation was derived and accounts for the final volume of the system.

$$C = \frac{V_s * CF}{1 - \frac{CF}{62.4}\left(\frac{1}{3.15} + w/c\right)}$$

Where,

- C = Cement needed (lb)
- V_s = Volume of soil to be treated (ft^3)
- CF = Cement factor (lb/ft^3)
- w/c = Water to cement ratio (lb/lb)

This equation can be used for both wet and dry mix applications. If dry mixing is used, the w/c may be taken as 0. The example below compares the results of both equations and demonstrates the error of the one mentioned first.

Given:

- CF = 12 lb/ft^3
- V_s = 100 (ft^3)
- w/c = 0.8

Table B.1. Cement needed example.

Equation	$C = CF * V_s$	$C = \dfrac{V_s * CF}{1 - \dfrac{CF}{62.4}\left(\dfrac{1}{3.15} + w/c\right)}$
Cement Needed	$C = 12 * 100 = 1200\ lb$	$C = \dfrac{100 * 12}{1 - \dfrac{12}{62.4}\left(\dfrac{1}{3.15} + 0.8\right)} = 1528.5\ lb$
Volume Added	$V_{Grout} = \dfrac{1200}{62.4 * 3.15} + \dfrac{1200 * 0.8}{62.4}$ $V_{Grout} = 21.5\ ft^3$	$V_{Grout} = \dfrac{1528.5}{62.4 * 3.15} + \dfrac{1528.5 * 0.8}{62.4}$ $V = 27.37\ ft^3$
System Volume	$V = V_s + V_{Grout} = 121.5\ ft^3$	$V = V_s + V_{Grout} = 127.37\ ft^3$
Check	$CF = \dfrac{Cement}{Total\ Volume} = \dfrac{1200}{121.5} = \mathbf{9.88}\ \dfrac{\boldsymbol{lb}}{\boldsymbol{ft^3}}$	$CF = \dfrac{Cement}{Total\ Volume} = \dfrac{1528.5}{127.37} = \mathbf{12}\ \dfrac{\boldsymbol{lb}}{\boldsymbol{ft^3}}$

As shown above, it is important to account for the increased volume of the system.

Appendix C: State Road 33 CPT Soundings

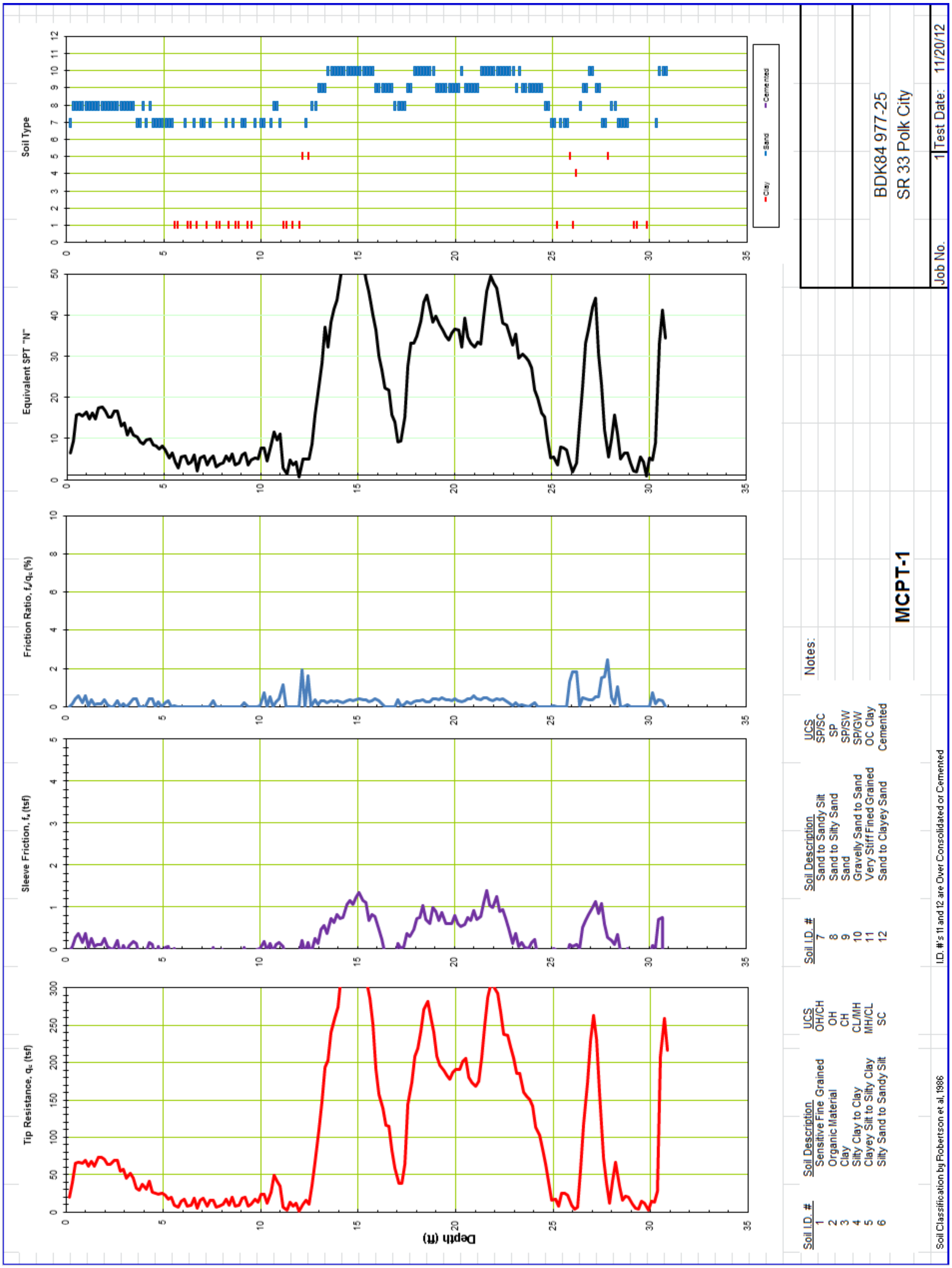

Figure C.1. CPT 1 along SR-33.

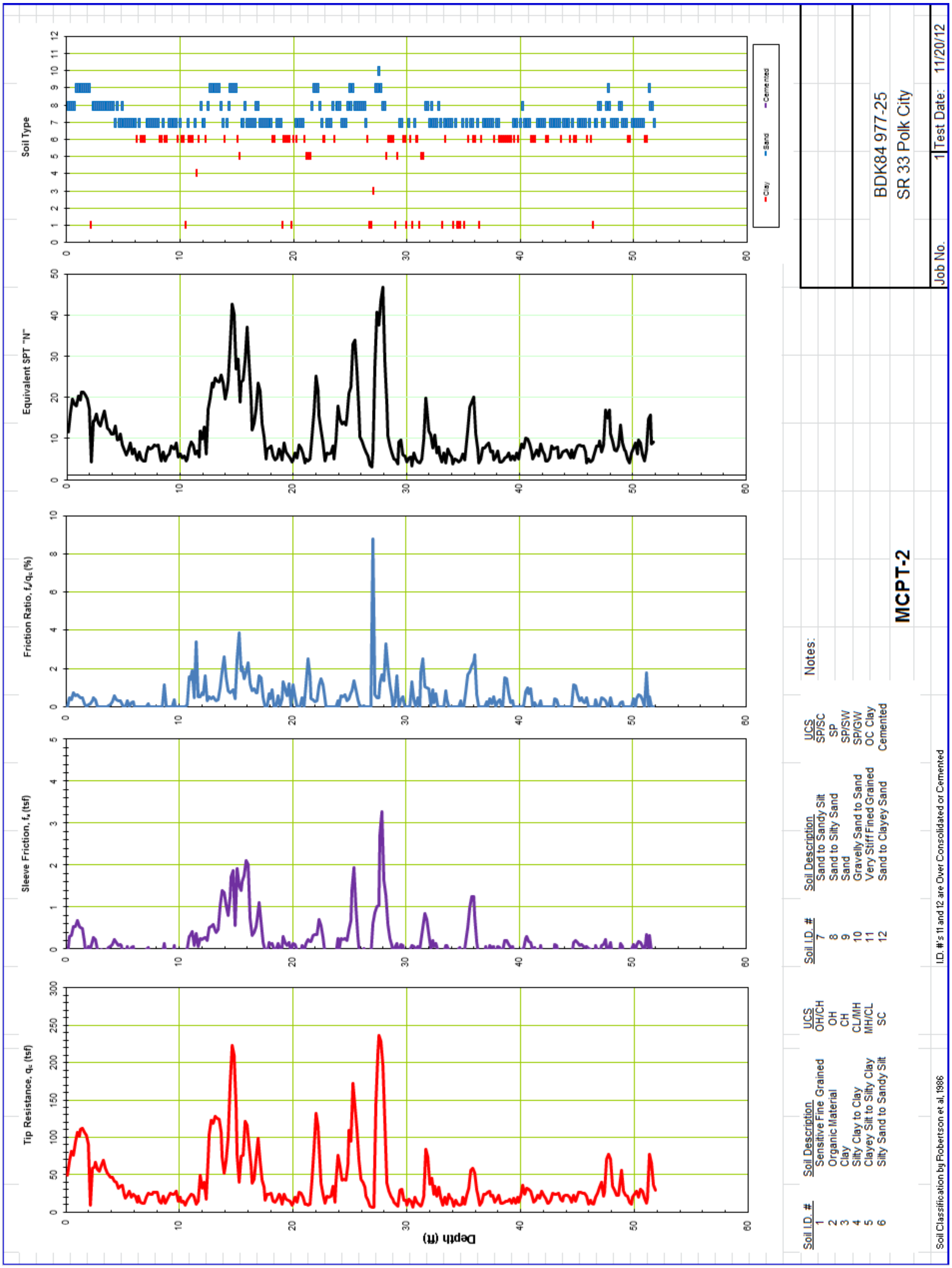

Figure C.2. CPT 2 along SR-33.

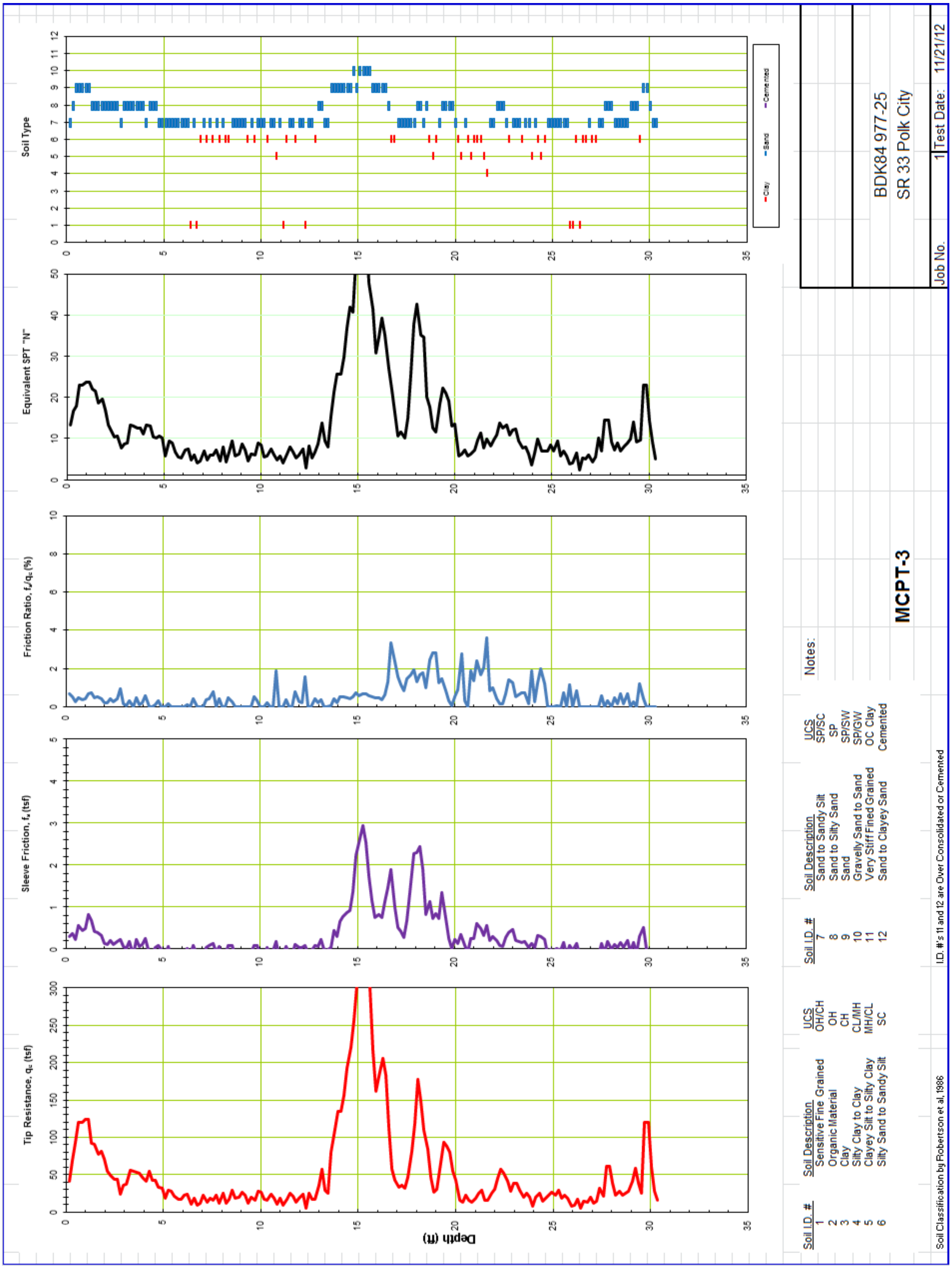

Figure C.3. CPT 3 along SR-33.

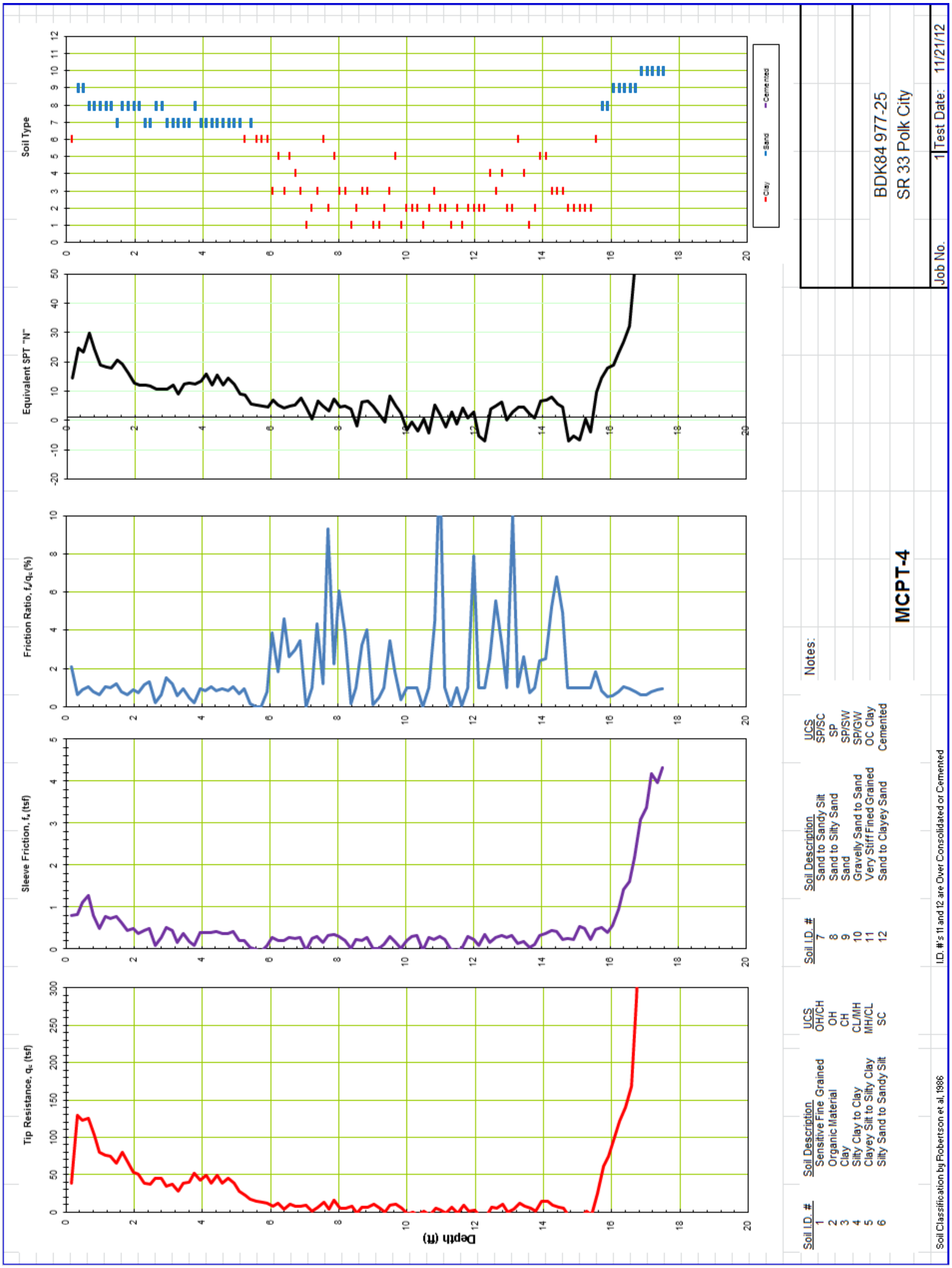

Figure C.4. CPT 3 along SR-33.

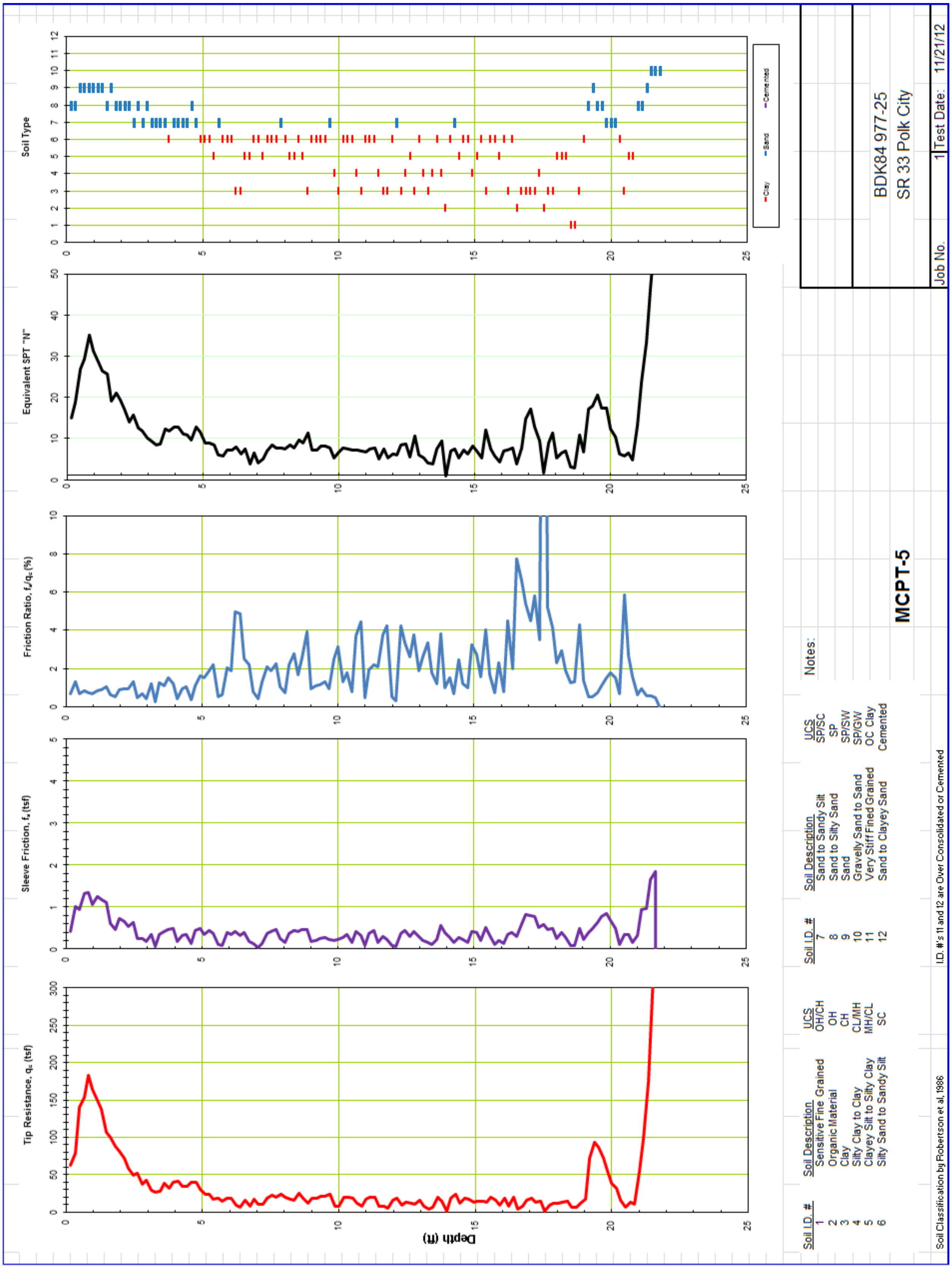

Figure C.5. CPT 5 along SR-33.

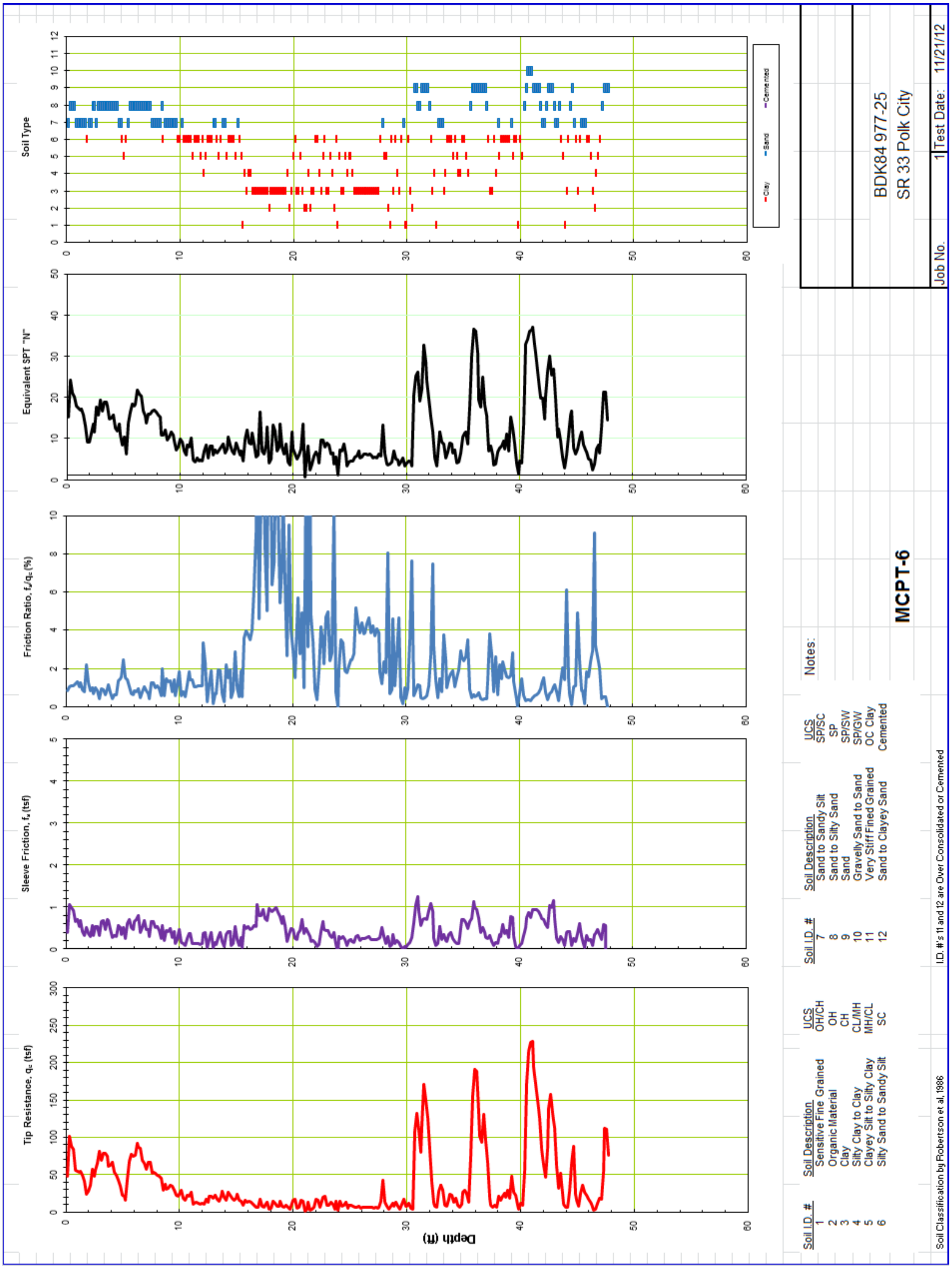

Figure C.6. CPT 6 along SR-33.

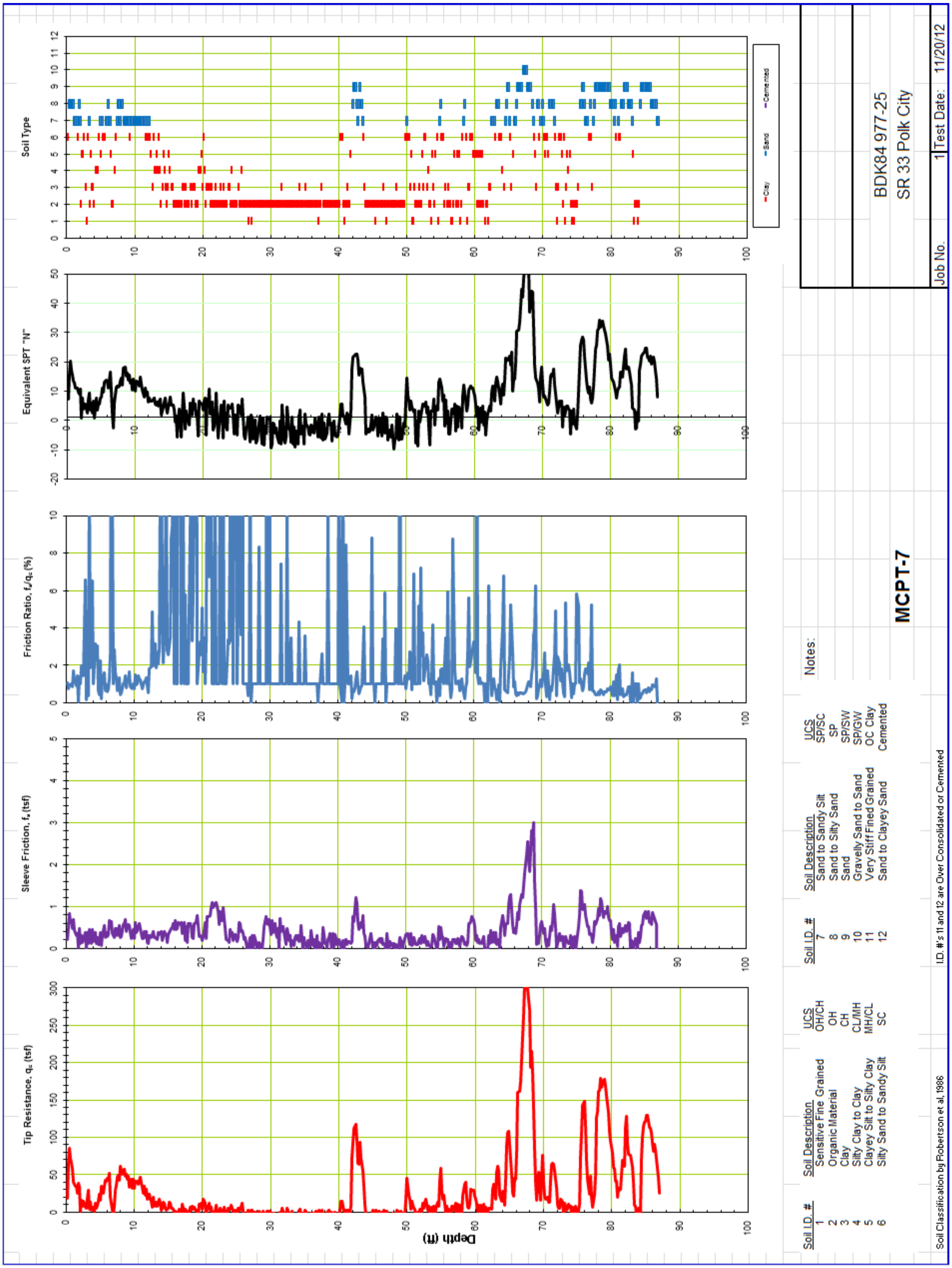

Figure C.7. CPT 7 along SR-33.

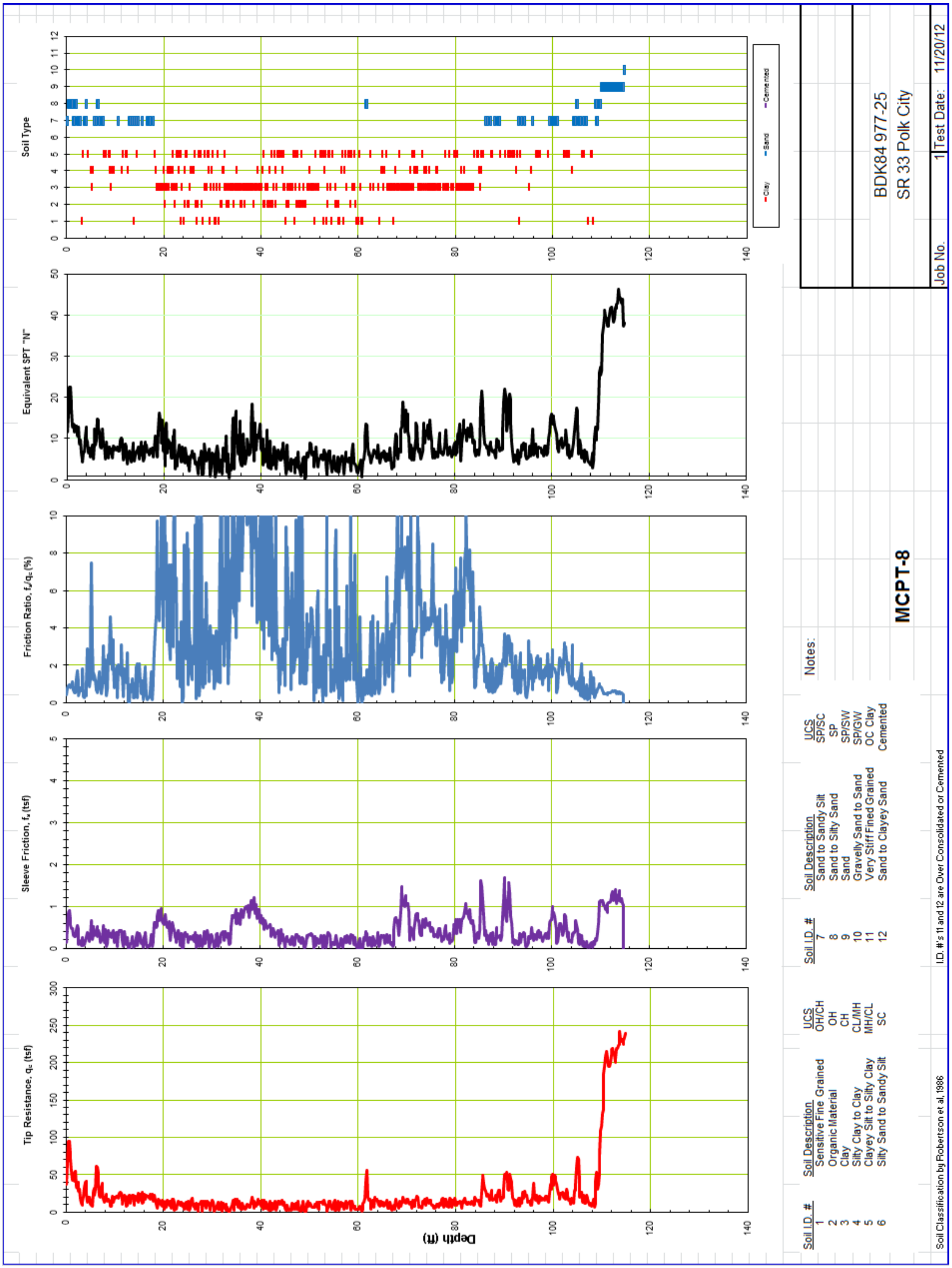

Figure C.8. CPT 8 along SR-33.

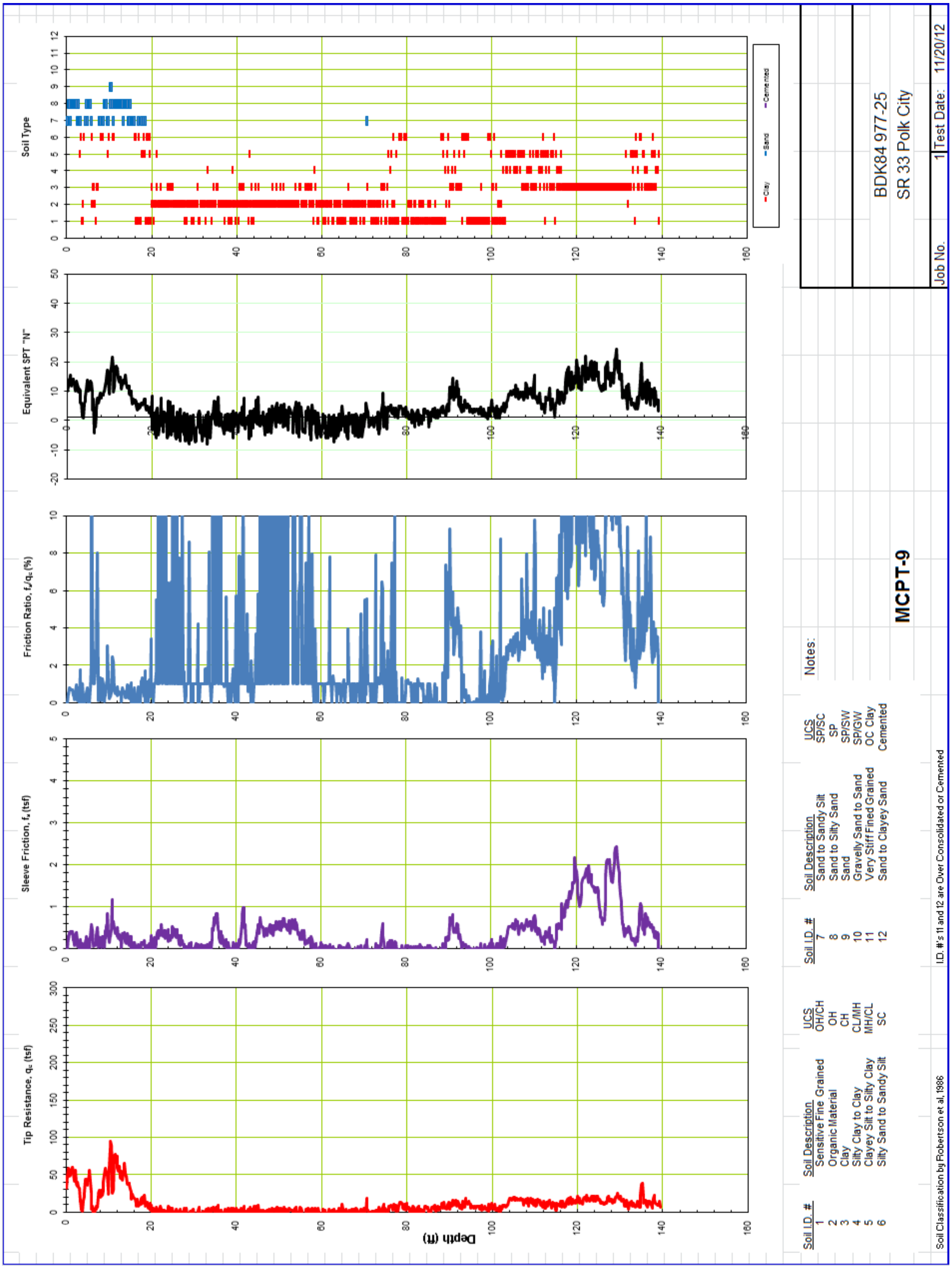

Figure C.9. CPT 9 along SR-33.

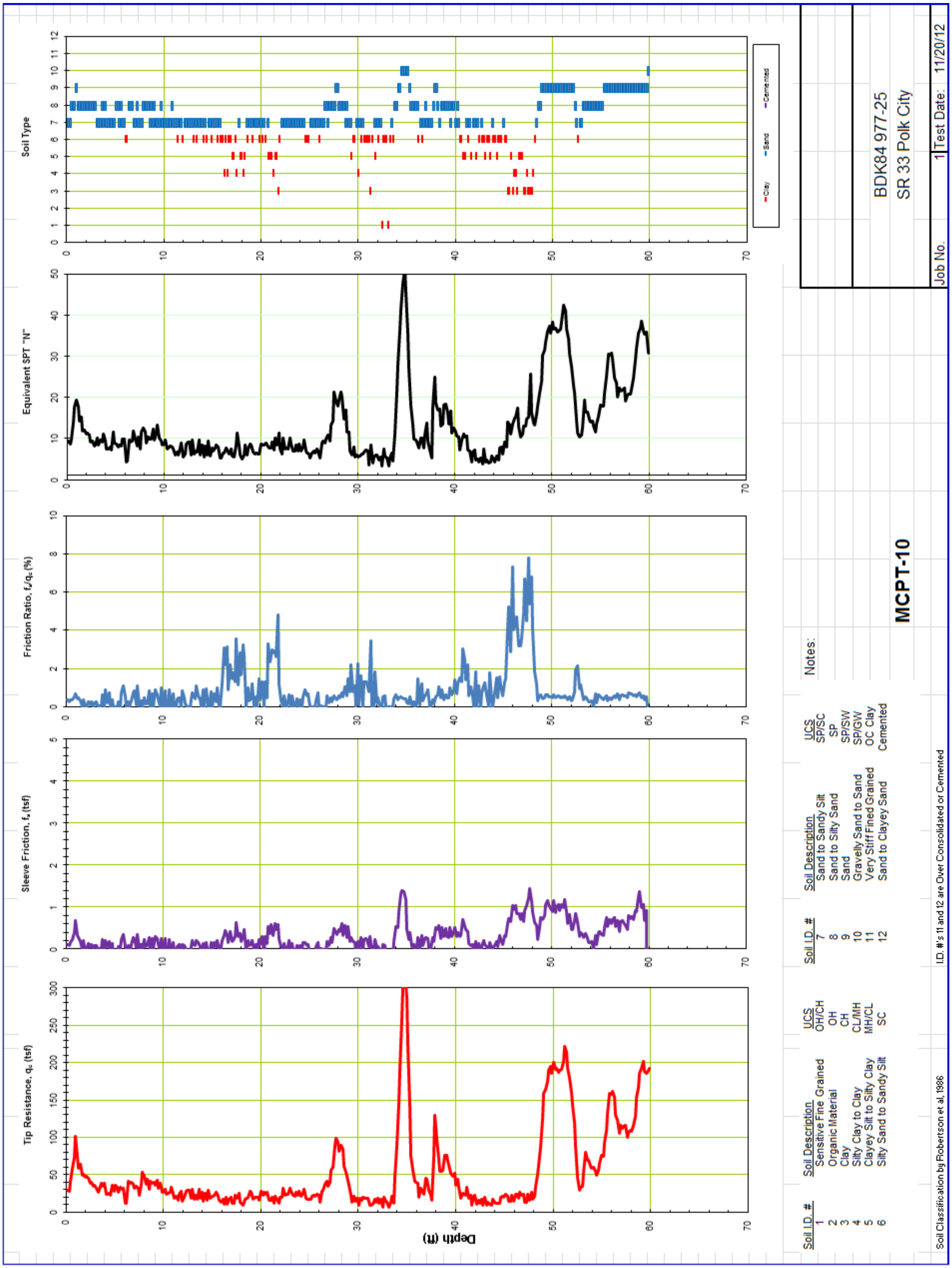

Figure C.10. CPT 10 along SR-33.

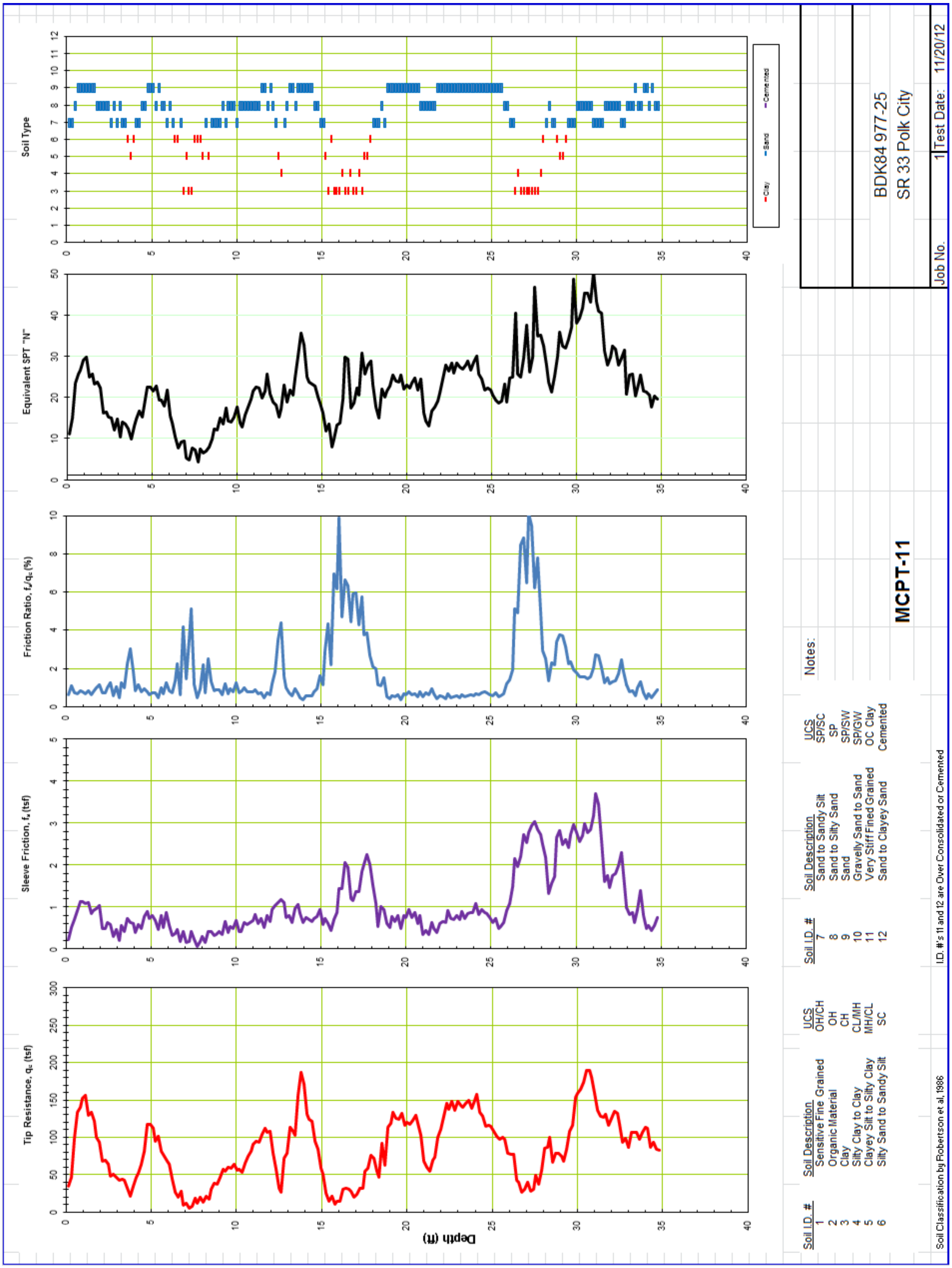

Figure C.11. CPT 11 along SR-33.