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Full Scale Evaluation of Organic Soil Mixing

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by

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ABSTRACT

Soil mixing is a procedure that has proven to be effective for loose or soft compressible soils. The method stabilizes the soil in-place using specialized augers, tillers, or paddles that inject grout or dry cementitious powders as part of the mixing process. The Federal Highway Administration design manual for soil mixing helps to estimate the required amount of cementitious binder to produce a target design strength. However, it is biased towards inorganic soils and only mentions caution when confronting organic soils which usually come with a high water table, moisture content and void volume.

The Swedish Deep Stabilization Research Centre cited studies with highly organic soils in regards to soil mixing and suggested that organic soils may need to reach a 'threshold' of cement content before strength gain can occur. The University of South Florida also conducted a study on highly organic soils and was able to confirm this concept. USF also proposed a threshold selection curve based on the organic content. This thesis extends this concept to the bench scale testing of multiple full scale field studies.

This thesis will conclude with the presentation of new threshold curves based on the new data from the added field case studies. Given that there were variable binders and soil types used in the data analyzed, these threshold curves are dependent upon soil type and binder type, thus expanding upon the curve previously suggested.

CHAPTER 1: INTRODUCTION

Virtually all structures depend on the support from a sound foundation to both resist the loads and control long-term settlement. Lighter structures like homes typically use shallow foundations only a few feet deep and where loads only affect the soils within close proximity to the ground surface (e.g. 5 to 10ft deep). Heavier structures like bridges and tall buildings are rarely founded on shallow foundations and the near surface soils are mostly disregarded in design. In these cases, deep structural elements are either driven or drilled into the ground where the loads are then transferred to deep competent soils or rock.

Pavement used to support highway traffic is actually a shallow foundation that may be supported by a wide range of soil types over the length of a given road. The design of a roadway "foundation" is primarily concerned with the strength of soil in the upper 5ft but stop-checks deeper soils to around 15ft periodically to ensure no unusually weak material is encountered. When soft soils are found that could lead to subsidence or long-term stability issues, some form of ground stabilization is employed. Historically, this has involved complete removal of the troublesome material, in-place strengthening using stone or sand columns, or direct mixing of the material with a chemical binder (i.e. lime, cement, etc.). The latter, known as *soil mixing*, is the topic of this thesis with specific focus on applications involving perhaps the weakest and most problematic of all soil types, organic soils.

1.1 Background

Organic soils are the by-product of decomposing plant life and are often encountered in low-lying regions that hold rainwater. As there is very little mineral structure to these soils, they tend to be very compressible and make a poor foundation/base for roads. As a result, common practices have been to completely remove these materials and replace them with competent soils capable of withstanding highway loads.

Figure 1.1. Organic soil replacement along Interstate I-4 between Tampa and Plant City, FL.

For over 40 years, the method of soil mixing (via jet grouting, wet mixing or dry mixing methods) has been used to improve the strength characteristics of insitu soil. Therein, an additive such as lime, cement, slag or flyash is mixed with weak soils in-place to stabilize the material making it strong enough to withstand anticipated loads. The equipment used for this procedure ranges from full length multi-auger systems to huge blenders with paddles oriented vertically or horizontally.

Soil mixing has been largely successful in inorganic sand and clays, but organic soil has historically been problematic requiring alternate stabilization methods such as long term surcharging or excavation and replacement (Figure 1.1, Figure 1.2). Today, these methods, while still being used, are being taken over by soil mixing. While this option may seem more desirable, there is still much to learn in order to make it a practical method for organics.

Figure 1.2. Before and after organic soil replacement on I-4.

Currently, the FHWA Design Manual for Deep Soil Mixing does not provide much recommendation in regards to organic soils. It states multiple times in varying sections that a greater amount of binder should be used…

> "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." (Bruce et al., 2013)

> “…soils with high organic content may require large amounts of binder to achieve suitable strength.” (Bruce et al., 2013)

> “Organic soils tend to require more binder than inorganic soils, and sandy soils require less binder than clay soils.” (Bruce et al., 2013)

However no definitive recommendations are provided to address the issue. FHWA then goes on to describe how organics may interfere with cementation…

> “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.” (Bruce et al., 2013)

...again without recommendations, only caution.

The issue is presented again in regards to curing time. The effect of curing time is to increase mixture strength. Equations typically used to predict time dependent strength gain provide “…a conservative estimate of the strength increase with time for cement and cement-slag treatment, except for some highly organic soils” (Bruce et al., 2013).

Clearly there is a lack of what-to-do, if faced with the issue. As mentioned above, soil mixing is now a competitive alternative to replacement. The reason being primarily cost, as replacement can be an expensive endeavor. However, in the midst of all this uncertainty concerning organic soils, FHWA warns that “…stiffer/denser cohesive soils and soils containing organics/peat are more costly to mix" (Bruce et al., 2013). Making it critical to know which method is cost effective for the project at hand.

1.2 Organization of Thesis

This thesis addresses the complications associated with design and/or stabilizing organic soils by firstly performing a literature review of previous studies in Chapter 2. In this section, many of the laboratory case studies used to develop the current FHWA guidelines will be summarized and compared in detail. Results will then be compared to the University of South Florida's recent laboratory scale research performed with highly organic soils.

Chapter 3 presents results from a large scale laboratory test bed, intended to simulate field conditions. This tested the effectiveness of both dry and wet soil mixing methods. It will briefly discuss the fabrication of the test bed and methods used for mixing, provide load-settlement relationships, time-settlement graphs, and results from excavated and tested columns from the wet mixed section.

Chapter 4 presents the findings from multiple 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, and a soil mixing project that ran concurrent to this thesis and was tracked by the research team.

Lastly, Chapter 5 will present an analysis of the data from the bench scale studies in Chapters 2 and 4. Once conclusions have been made on this data, recommendations for future testing will be provided. Chapter 5 will also include an analysis of the large scale test bed findings.

CHAPTER 2: LITERATURE REVIEW

2.1 Wet Soil Mixing

Wet soil mixing is most commonly performed through injection of a wet binder slurry into the soil strata using mechanical equipment that closely resembles that which is used for drilled shafts (Figure 2.1). A multi-paddle large diameter tool gains depth in the soil by injecting slurry while slowly spinning. This process is critical as blade rotation / mixing effectiveness can affect the soil strength outcome.

Figure 2.1. Wet soil mixing equipment (Garbin and Mann, 2012).

This method forms soil columns, whose strength (like concrete) depend on the amount of cement used, water to cement ratio, and aggregate (soil type). Typically repeating column patterns (i.e. hexagonal or rectangular configurations) are used to provide coverage to the entire area in need of strengthening. Given the soil itself usually contains water, injection of additional water in the grout/slurry restricts wet mixing methods to soils having a moisture content of 60% or less.

2.2 Dry Soil Mixing

There are two basic mixing techniques regarding dry soil mixing (DSM). The first technique, extremely similar to wet soil mixing, uses a pattern layout where vertical columns are distributed throughout the treatment area. The second uses a horizontally aligned axis tilling-like tool head that blends an entire area (not just columns), but is restricted to shallow soil deposits within reach of a backhoe type arm (Figure 2.2). The second method is also known as mass stabilization. Dry methods use high pressure air to inject into the soil a dry binder in powder form such as cement, lime, flyash or slag. Depending on the equipment, a tiller or paddle then mixes the dry powder with the existing ground water and soil.

Figure 2.2. Tilling type mixing tool for DSM mass stabilization, (Baker, 2015).

Dry soil mixing is ideal for wetter soils with moisture content above 40%. Organic soils tend to have extremely high moisture contents relative to other soil types (300 to 1000% for organic soil compared to 20 to 40% for inorganic soils) which then necessitates large amounts of cement to produce a reasonable w/c ratio and the necessary strength.

2.3 FHWA Laboratory Soil Mixing Case Studies

Literature cited laboratory studies have been performed that form the basis of the latest FHWA Manual design curve (Figure 2.22) for soil mixing applications (Bruce, 2013). Different types of soils are presented within this compilation as well as the utilization of different mixing methods (wet or dry), mixing procedures, tamping style, and curing conditions.

2.3.1 Case Study 1: Jacobson et al. 2003[1]

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.

The soil from I-95/Route 1 interchange had a range of organic contents varying from 1.8% to 46.4% with an average of 10.5%. By USGS classification the soil was 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 6%, and the average pH was 6.6. The average liquid limit of the samples was 67. Table 2.1 shows results of this soil when mixed with 100% cement at different mix ratios.

The soil from State Route 33 in West Point, VA consisted mostly of marsh deposits of soft, organic clays with moisture contents varying from 15% to 200%, as well as organic contents of

[1] Section 2.3.1 references "Factors affecting strength gain in lime-cement columns and development of a laboratory testing procedure." By: Jacobson, J.R., G.M. Filz and J.K. Mitchell (2003).

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

0% to 40%, respectively. Above the soft clay was a variable amount of fill material, below was 3 to 6 m of loose to firm sand, and below that was moderately stiff silty clay.

Zone 1 of State Route 33 was taken at a depth of 4.5 to 7.5 m and was a more uniform zone. 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 shows the results of this soil when mixed with 100% cement.

Zone 2 of State Route 33 was taken at a depth of 11.0 to 14.5 m and was of greater variance than zone 1. 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 shows results of this soil when mixed with 100% cement.

These batches were mixed using the dry mixing method. A 4-liter capacity KitchenAid™ stand mixer, using the dough hook attachment, was used. This capacity permitted the manufacture of eight samples. During production firstly the soil was homogenized, then the weight of the batch was taken along with two moisture samples. Using a microwave the time needed for the moisture

Table 2.2. Results from Zone 1 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

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

samples to dry was accelerated. 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 diameter by 100mm tall.

The main findings from this case study were:

1. If the soil was allowed to dry out and then rewetted to reinstate the previous soil moisture, strength of the mixture decreased.
2. The addition of lime both increased or decreased strength depending on soil type.

3. As the soil water to cement ratio increased, strength of the mixture decreased (for 100% cement soil mixtures).

2.3.2 Case Study 2: Miura et al. 2002[2]

This study analyzed 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 below 30 kPa (4.4 psi). (Yin & Lai, 1998)

Table 2.4. Soil characteristics of marine deposits used (Yin & 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

[2] Section 2.3.2 references "Engineering behavior of cement stabilized clay at high water content." By: Miura, N., S. Horpibulsuk and T.S. Nagaraj (2003).

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 2.3 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.

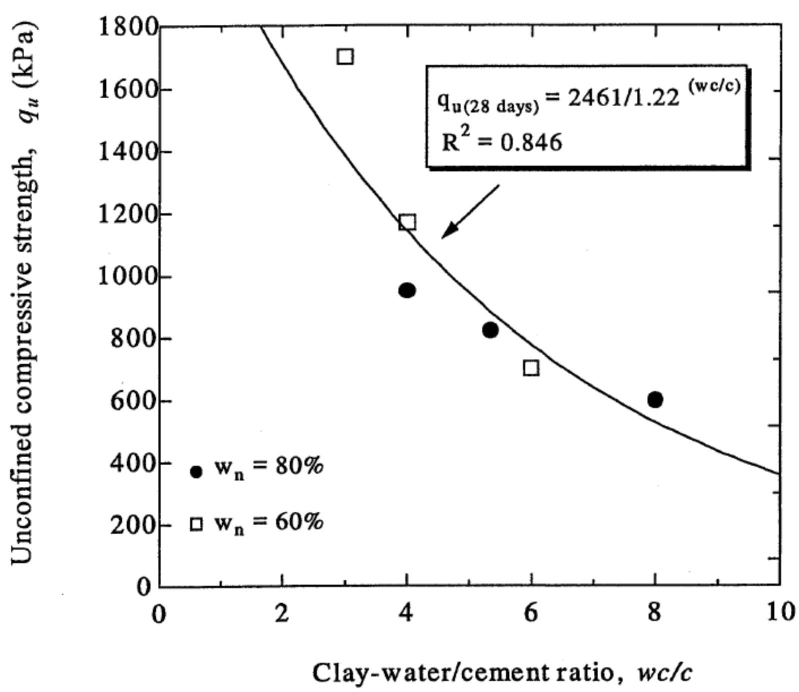

Figure 2.3. 28-day results from Yin and Lai's study (Miura, Horpibulsuk, & Nagaraj, 2002).

2.3.3 Case Study 3: Horpibulsuk et al. 2003[3]

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, A.S., & D.T, 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

[3] Section 2.3.3 references "Assessment if strength development in cement-admixed high water content clays with Abrams' law as a basis." By: Horpibulsuk, S., N. Miura and T.S. Nagara (2003).

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
Sulphur 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)	3000 cm^2

After mixing, the product was put into steel molds with dimensions of 75mm diameter and 90mm 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 200mm. Curing of the samples was then done in a humid room. (Uddin, A.S., & D.T, 1997)

The graph shown in Figure 2.4 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*.

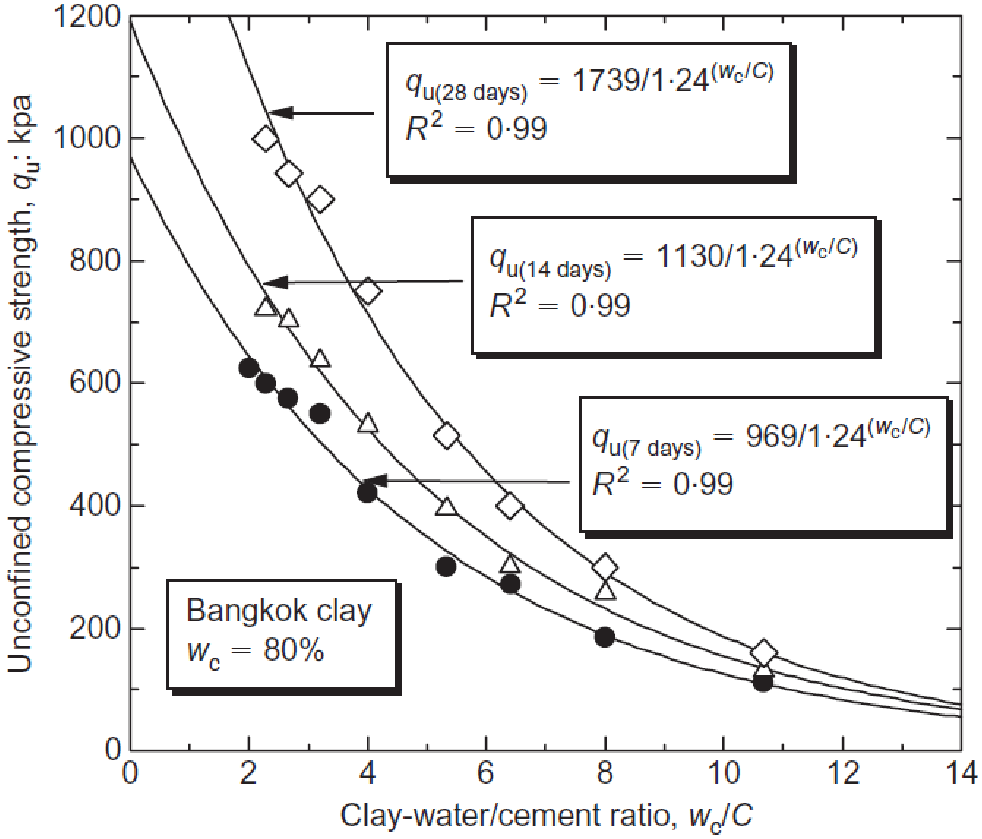

Figure 2.4. Unconfined compressive strength results of the study (Horpibulsuk, Miura, & Nagara, 2003).

2.3.4 Case Study 4: Lorenzo and Bergado 2006[4]

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 was typical soft Bangkok clay. The sample was taken from a depth of 4 to 5m at the Asian Institute of Technology (AIT) campus in Klong Luang, Pathumthani, Thailand and contained the properties shown in Table 2.7.

[4] Section 2.3.4 references "Fundamental characteristics of cement-admixed clay in deep mixing." By: Lorenzo, G.A and D.T. Bergado (2006).

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 50mm with a height of 100mm. The temperature and humidity of the curing room were 25°C and 97%, respectively. Figure 2.5 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).

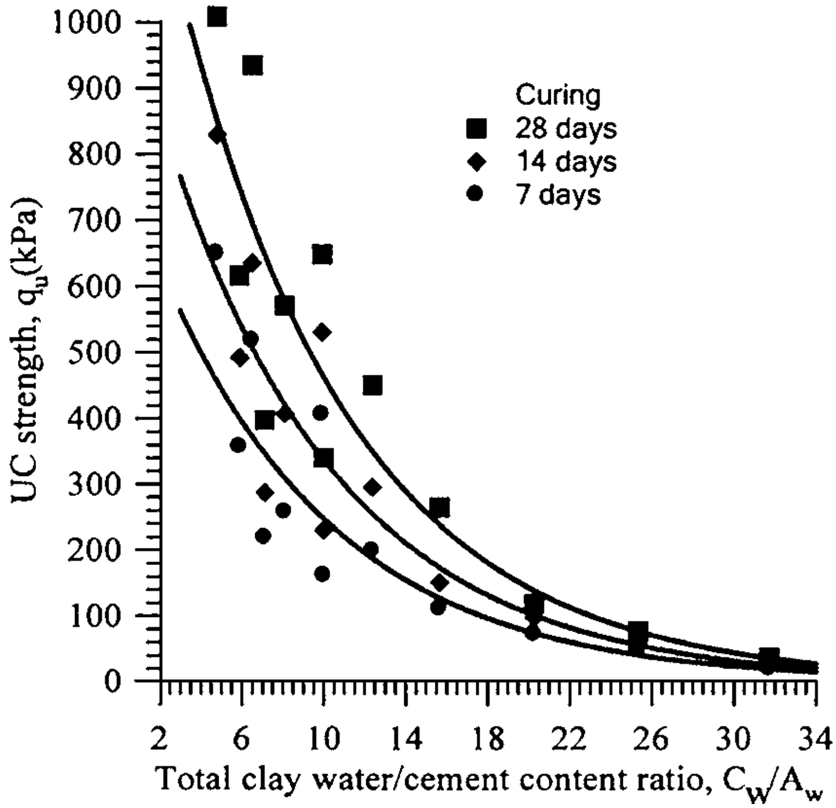

Figure 2.5. Unconfined compressive strength versus total clay water to cement ratio (Lorenzo & Bergado, 2006).

2.3.5 Case Study 5: Hodges et al. 2008[5]

Completed in 2008, this study analyzed 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 relatively easy to mix soils.

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

[5] Section 2.3.5 references *Laboratory mixing, curing, and strength testing of soil-cement specimens applicable tothe wet method of deep mixing,* by Hodges, D.K., G.M. Filz and D.E. Weatherby (2008).

acquired from beneath the ground water table. For testing the soils were oven dried, then the amount of water needed for that condition was calculated and added in.

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

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.

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 of a kitchen stand mixer and hand mixing were attempted, however 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™ 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 fines 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 combination was rather fluid and if it was on the stiffer side the sample was rodded (Hodges, Filz, & Weatherby, 2008). Once filled, the overflow on the tops on 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 2.6 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.

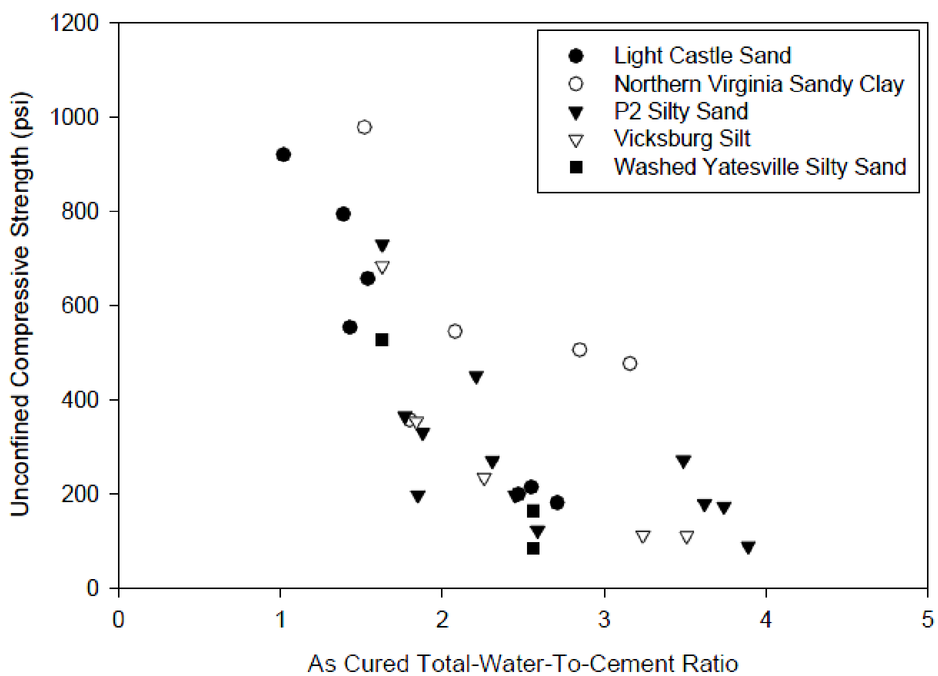

Figure 2.6. 28-day strength versus as cured total water to cement ratio (Hodges, Filz, & Weatherby, 2008).

2.3.6 Interpretation of Literature Findings

Figure 2.7 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 2.8 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. This is not unexpected as w/c ratio is not addressed and is the factor that is most responsible for strength in soil mixed with cement. This is due to the existing moisture content which must be overcome in order to achieve a sufficient w/c ratio and overall cement content. This differs from concrete mixes where moisture can be limited to control both w/c ratio and cement content to achieve a suitable mix design.

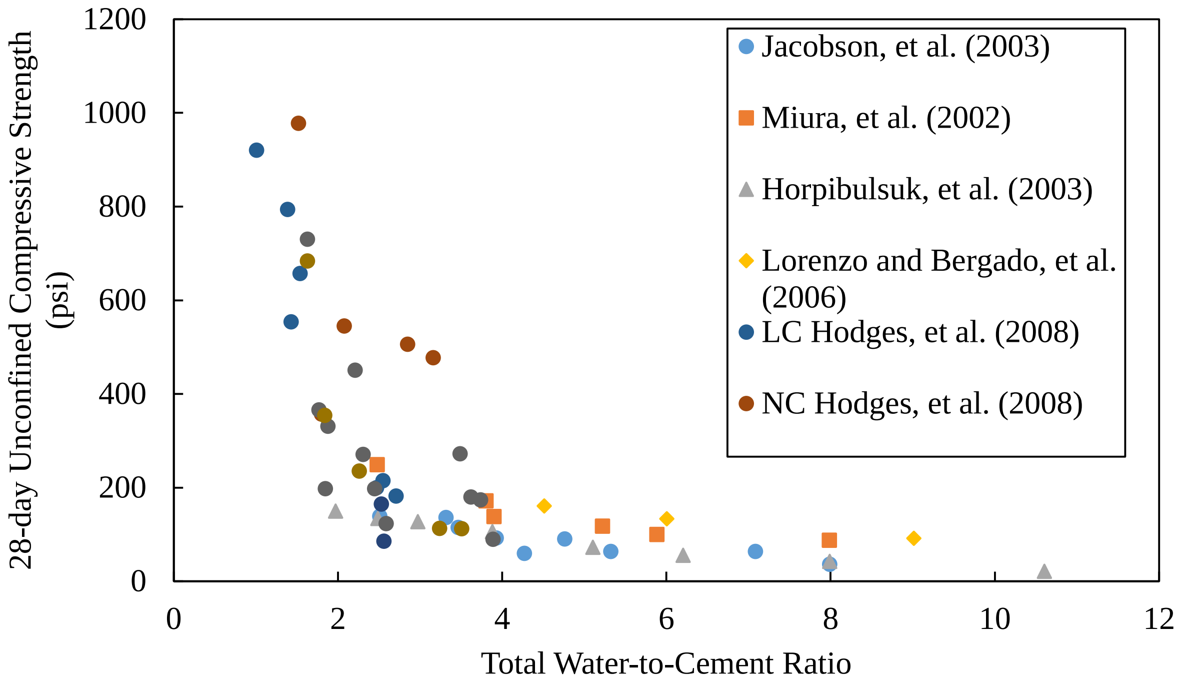

Figure 2.7. Overall results seen from case studies (Hodges, Filz, & Weatherby, 2008) (Horpibulsuk, Miura, & Nagara, 2003) (Jacobson, Filz, & Mitchell, 2003) (Lorenzo & Bergado, 2006) (Miura, Horpibulsuk, & Nagaraj, 2002).

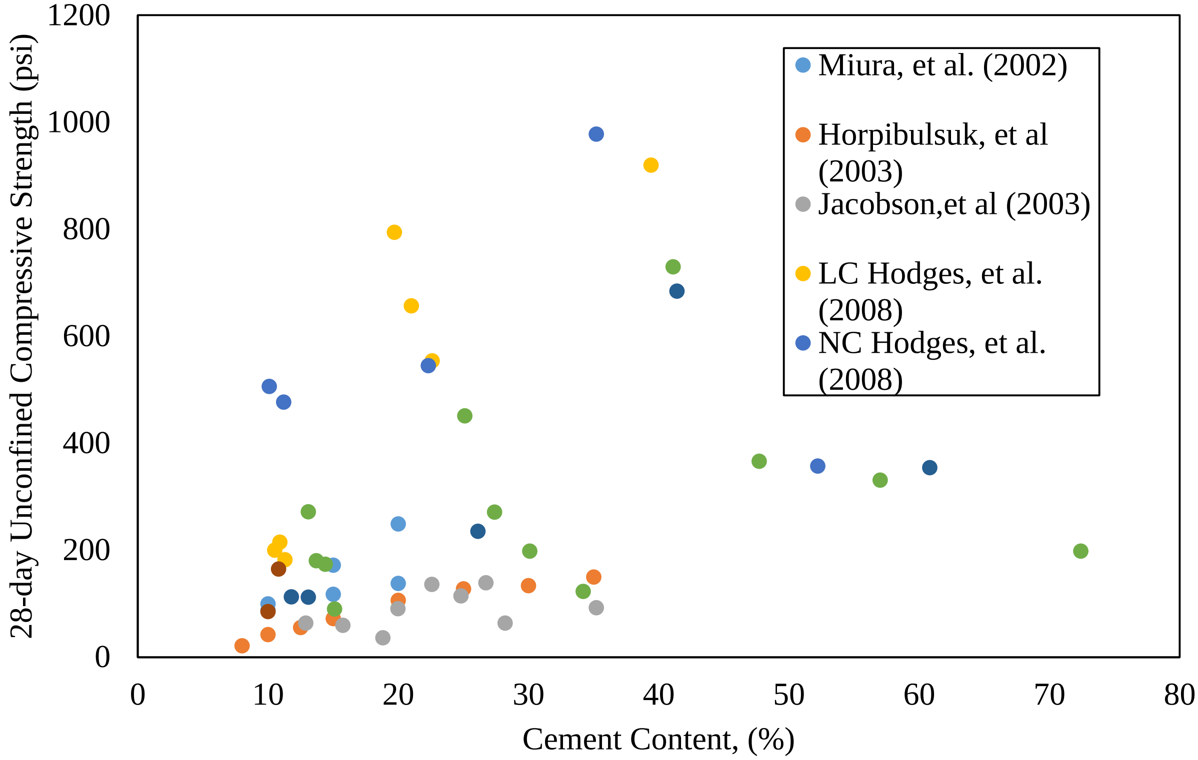

Figure 2.8. Overall results seen from case studies in terms of cement content, excluding Lorenzo and Bergado (Hodges, Filz, & Weatherby, 2008) (Horpibulsuk, Miura, & Nagara, 2003) (Jacobson, Filz, & Mitchell, 2003) (Miura, Horpibulsuk, & Nagaraj, 2002).

Figure 2.9 shows the same relationship as Figure 2.8, but with cement factor (weight of the cement divided by the volume of the mix). The trend of this graph correlates increasing strength with an increasing amount of cement a bit better, however is still very broad.

Considering cement factor is typically used instead of cement content Figure 2.8 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

$W_c = weight\ of\ cement$

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

$A_w = cement\ content, \%$

$CF = cement\ factor$

While the weight of cement was unknown for many case studies, it was irrelevant. The weight of cement appears in the formula in such a way that allows for it to cancel itself out and therefore it is not a needed variable. If the case study provided a cement factor already that was used.

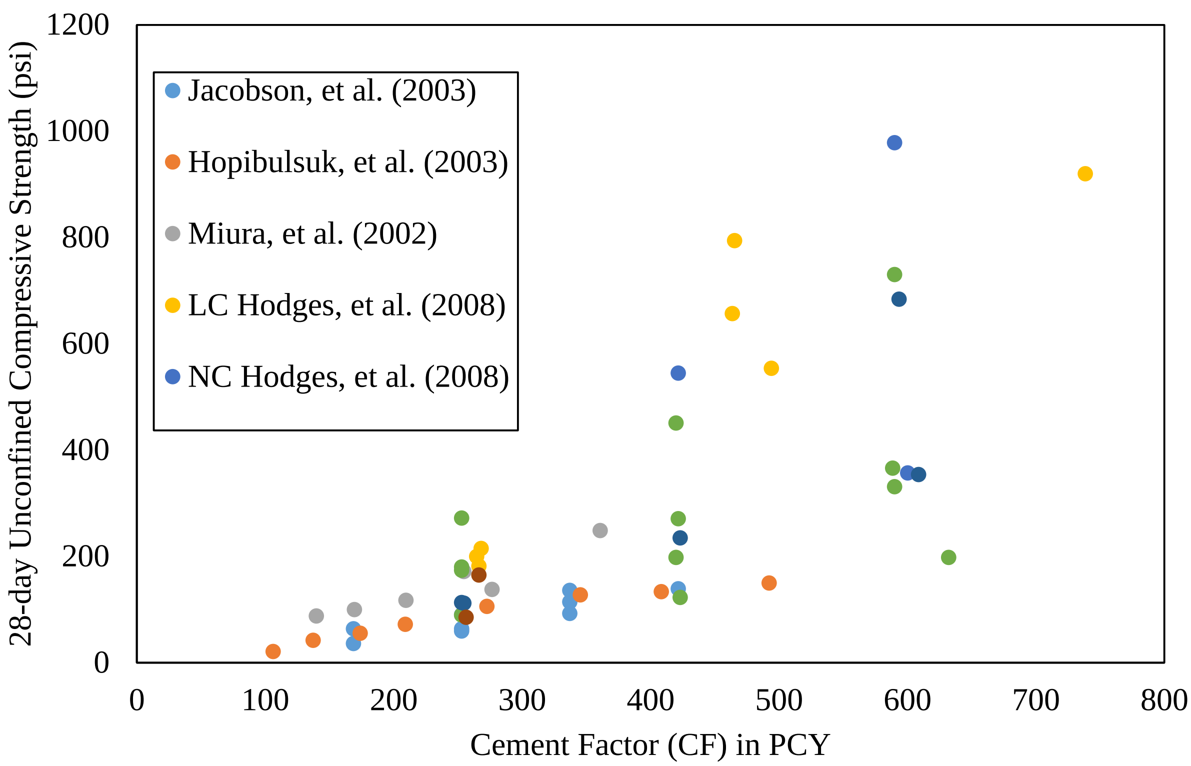

Figure 2.9. Overall results seen from case studies in terms of cement factor, excluding Lorenzo and Bergado (Hodges, Filz, & Weatherby, 2008) (Horpibulsuk, Miura, & Nagara, 2003) (Jacobson, Filz, & Mitchell, 2003) (Miura, Horpibulsuk, & Nagaraj, 2002).

2.3.6.1 Breakdown of Literation Findings by Water-to-Cement Ratio

Since the figures seen in Section 2.3.6 are so wide-ranging Figure 2.7 and Figure 2.9 are shown again (now as Figure 2.10 and Figure 2.11 respectively), this time instead of distinguished by case study, they are shown in three ranges of water-to-cement ratio (1-2.49, 2.5-3.99, 4-11). When these sections are separated, the inconsistencies are more apparent as can specifically be seen in Figure 2.14 where practically no trend is seen. Figure 2.13 and Figure 2.15 show more of an expected trend. To take this process one step further, because of the abnormalities seen in Figure 2.14, the data has been broken out by case study to confirm that individually each case does promote the wanted correlation.

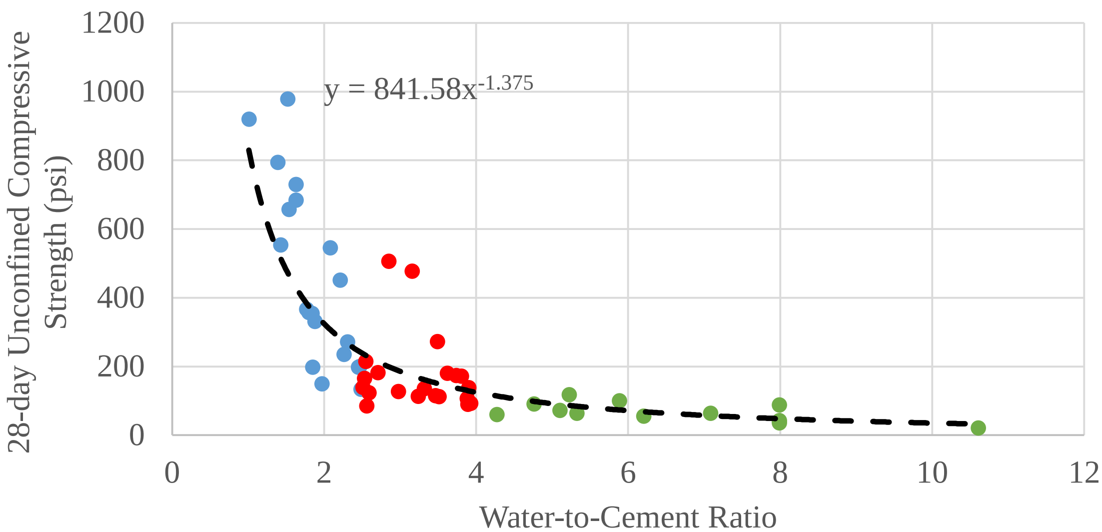

Figure 2.10. Overall results seen from case studies grouped by color into different water-to-cement ratio categories. Blue is 1-2.49, Red is 2.5-3.9, Green is 4-11. (Hodges, Filz, & Weatherby, 2008) (Horpibulsuk, Miura, & Nagara, 2003) (Jacobson, Filz, & Mitchell, 2003) (Miura, Horpibulsuk, & Nagaraj, 2002).

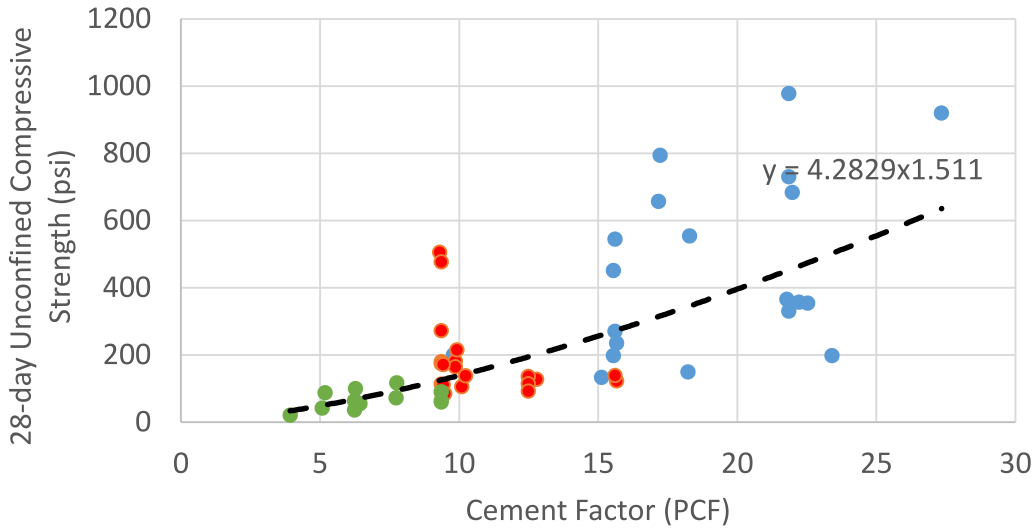

Figure 2.11. Overall results seen from case studies in terms of cement factor, separated by water-to-cement ratio. The grouping is the same as explained in Figure 2.10. (Hodges, Filz, & Weatherby, 2008) (Horpibulsuk, Miura, & Nagara, 2003) (Jacobson, Filz, & Mitchell, 2003) (Miura, Horpibulsuk, & Nagaraj, 2002).

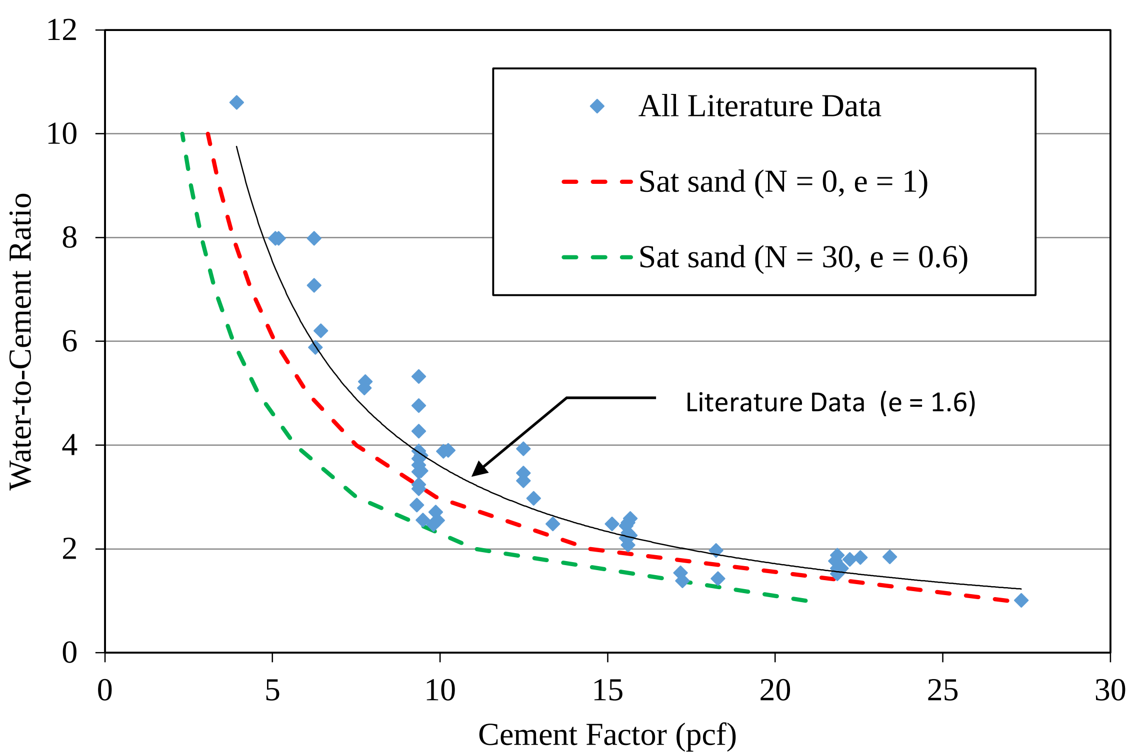

Figure 2.12. Water-to-cement ratio versus cement factor.

When the data is separated out by case study it is clear that the reason for the broadness of Figure 2.7-Figure 2.9 is because of the range of soil types being compared. Aside from Hodges Northern Virginia sandy clay, the other case studies all follow the overall trends noted by the dashed lines generated from all data sets together.

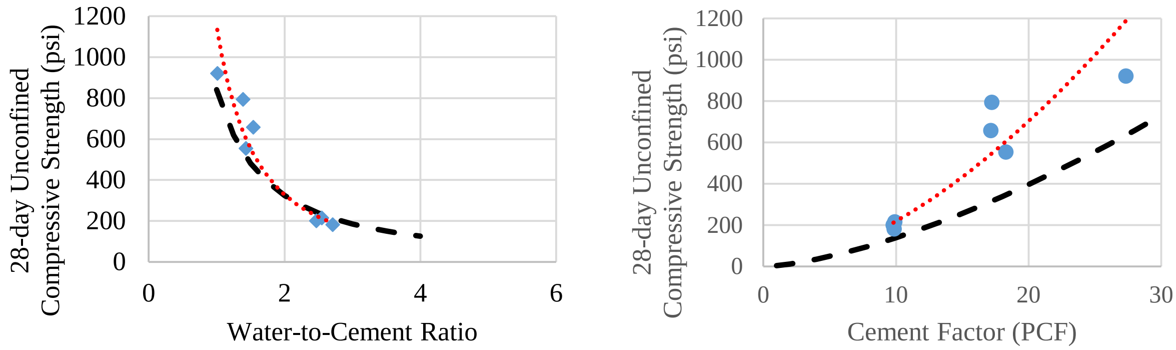

Figure 2.13. Hodges Light Castle sand. (Hodges, Filz, & Weatherby, 2008)

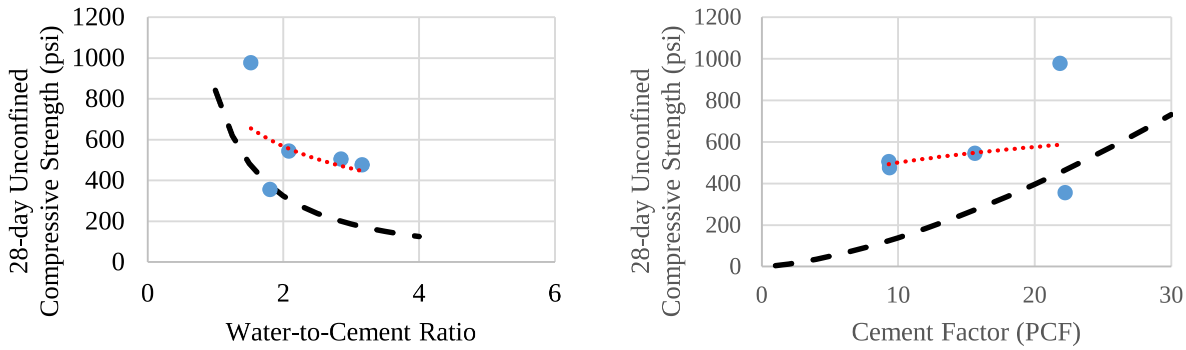

Figure 2.14. Hodges Northern Virginia sandy clay. (Hodges, Filz, & Weatherby, 2008)

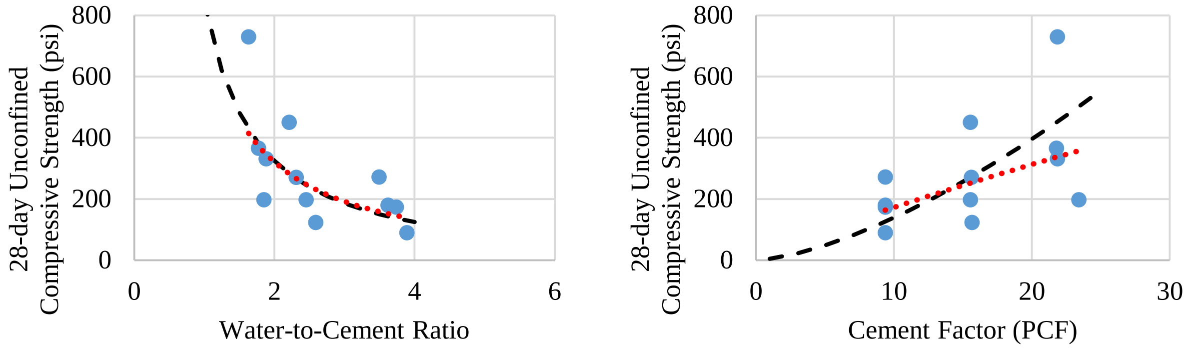

Figure 2.15. Hodges P2 silty sand. (Hodges, Filz, & Weatherby, 2008)

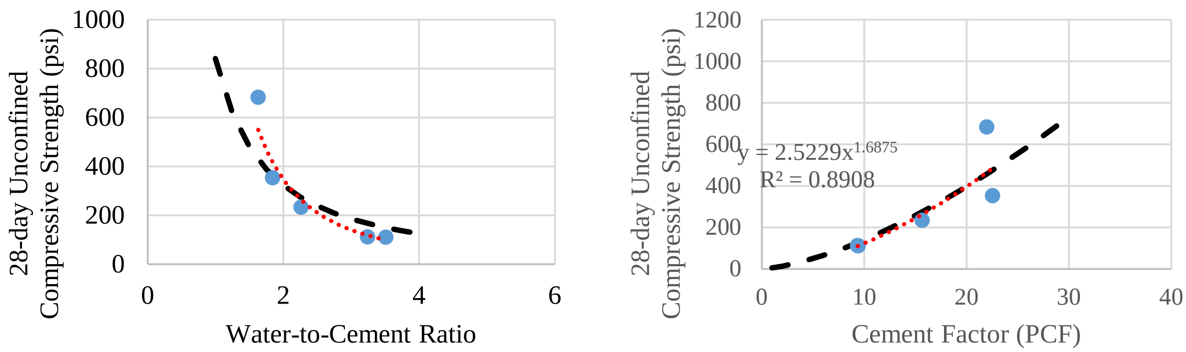

Figure 2.16. Hodges Vicksburg silt. (Hodges, Filz, & Weatherby, 2008)

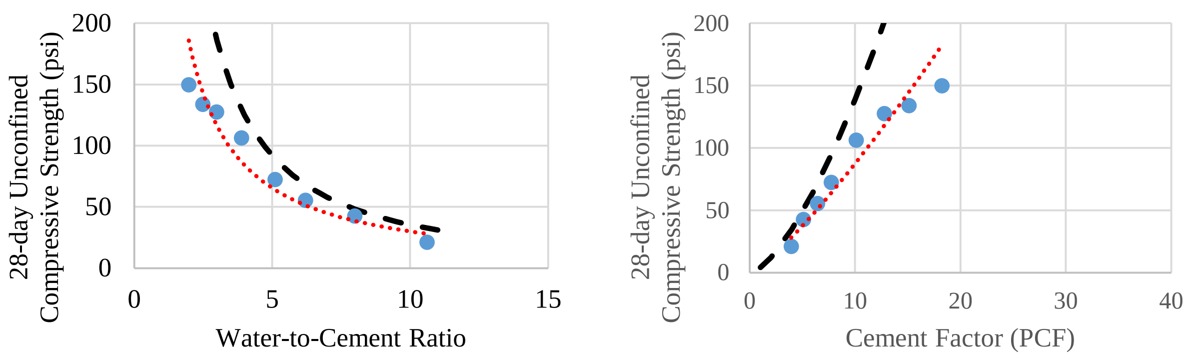

Figure 2.17. Horpibulsuk soft Bangkok clay. (Horpibulsuk, Miura, & Nagara, 2003)

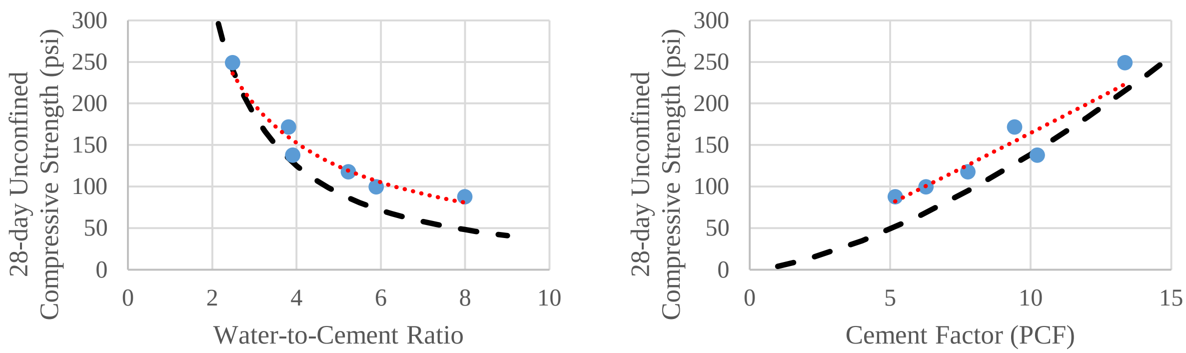

Figure 2.18. Miura soft marine deposit of silty clay. (Miura, Horpibulsuk, & Nagaraj, 2002)

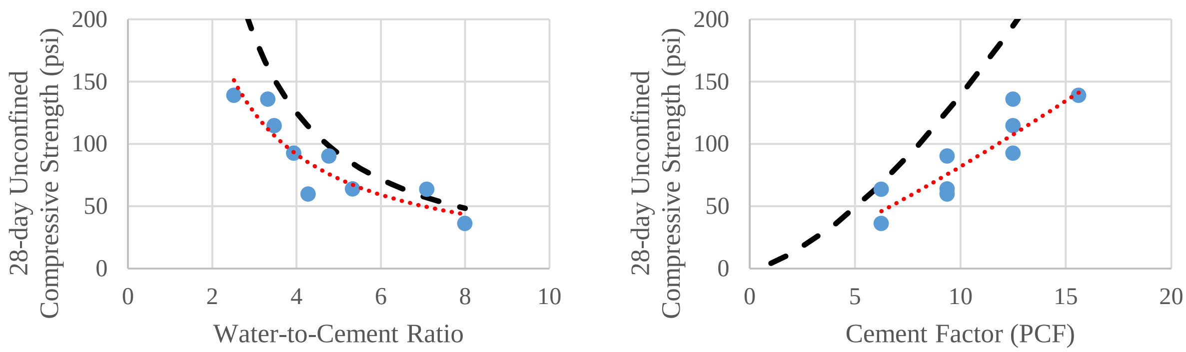

Figure 2.19. Jacobson organic silt. (Jacobson, Filz, & Mitchell, 2003)

2.3.6.2 Breakdown of Literature Findings by Dry Mixing versus Wet Mixing

As a final check, the cases were separated by wet and dry mixing methods, seen in Figure 2.20 and Figure 2.21. Both charts demonstrate the promotion of the expected trends and therefore no unusual issue was seen here.

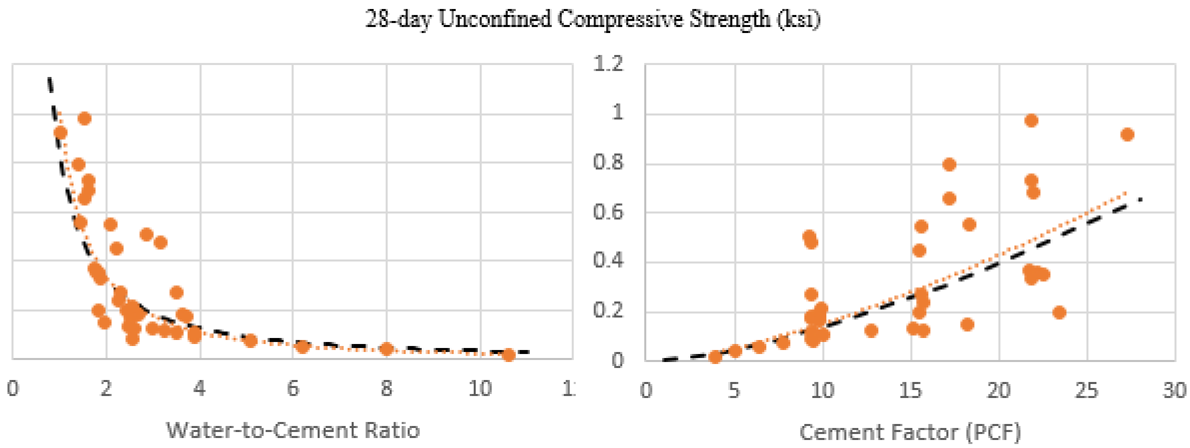

Figure 2.20. Separation of Figure 2.7 and Figure 2.9 by wet mixing method.

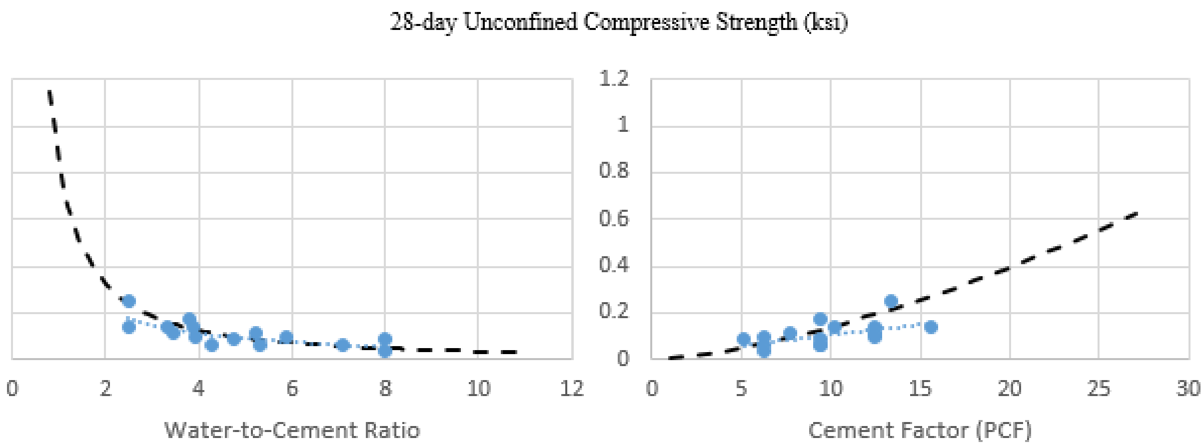

Figure 2.21. Separation of Figure 2.7 and Figure 2.9 by dry mixing method.

2.3.6.3 FHWA Curve

After looking into the above resources which contributed to the FHWA curve provided for soil mixing, Figure 2.22, it was found that some data points were excluded in the preparation of that document. Figure 2.23 shows a modified curve based on all available data from the case studies. The modified FHWA design curve reflects slightly lower strengths than the published curve. It should also be noted that this curve is largely for inorganic soils. As noted previously, FHWA does not offer much guidance in regards to organic soil.

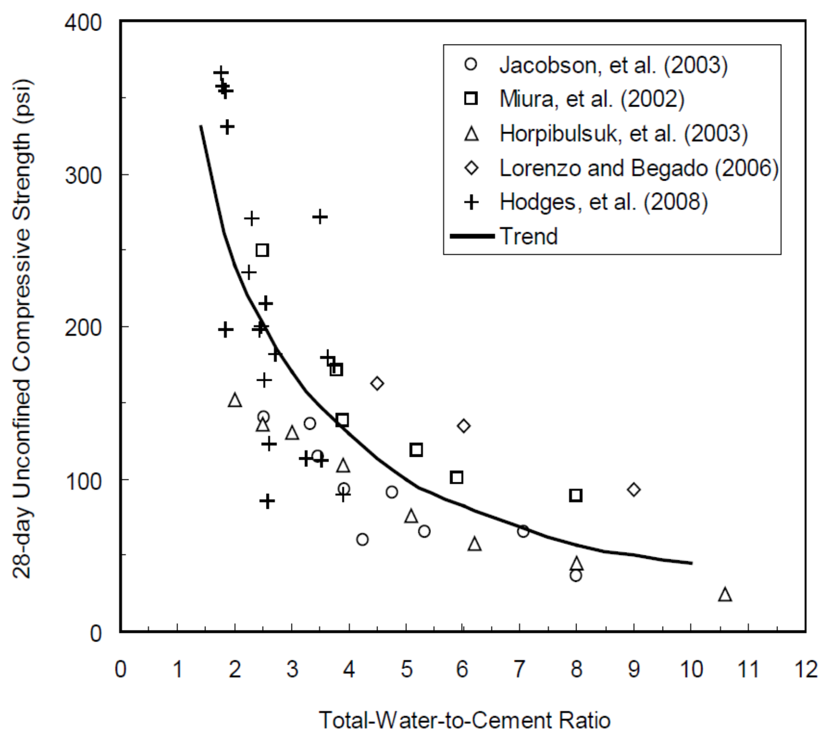

Figure 2.22. Strength vs. W/C ratio (Bruce et al., 2013).

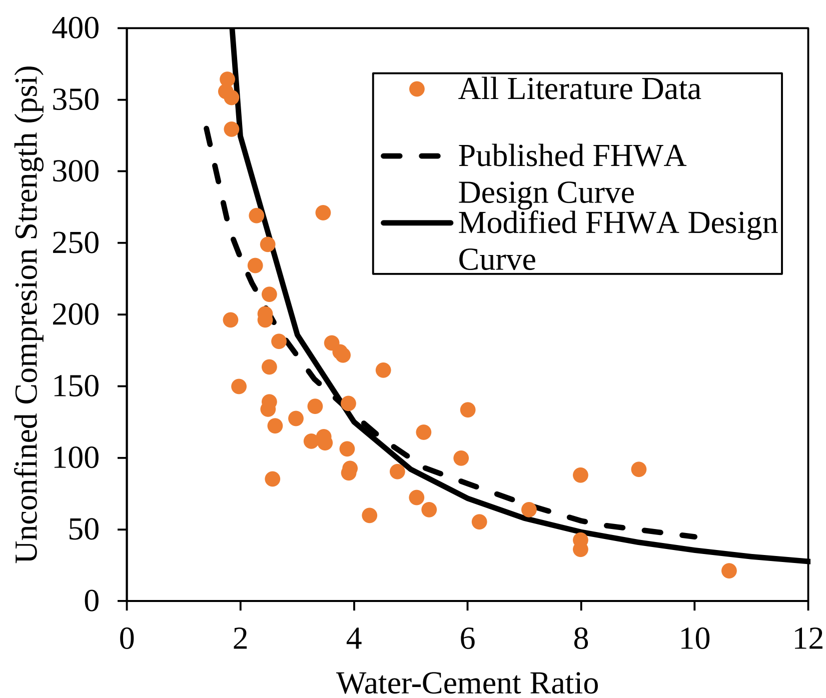

Figure 2.23. Modified FHWA curve using all data from case studies.

2.4 Threshold Concept

An earlier study proposed a threshold concept. Meaning, organic soil needs enough binder to surpass the threshold before any strength improvement. The Swedish Deep Stabilization Research Centre mentions this concept in their analysis of mud and peat and states a possible reasoning that with sufficient binder the humic acids are neutralized (Axxelsson et al., 2002).

> "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." (Axxelsson et al., 2002).

As part of this thesis work, a University of South Florida study, (Mullins and Gunaratne, 2015; Baker, 2015), also entertained and performed further development of this concept. The study was performed creating samples of varying organic content and binder factor (cement and slag were used). The initial results are shown in Figure 2.24 along with the above modified FHWA curve (Figure 2.23). The data seemed to follow a similar curve to FHWA, however at a much lower strength. All data plotted on the curve was from mixtures with cement binder only.

Slag was also tested, however many slag specimens proved too weak to test during this study using conventional unconfined compression tests. Slag-only specimens seemed to perform well in low organic contents (approximately 20% or less). More information on this can be found in Chapter 3 of Spencer Baker's thesis "Laboratory Evaluation of Organic Soil Mixing".

Analyzing the data further yielded Figure 2.25, which demonstrates a threshold for soils of varying organic contents. This shows the amount of cement that is required but essentially thrown away before strength gain can occur. Using this newfound technique Figure 2.26 was created, where all data then agreed well with the FHWA design curve generated from inorganic soils.

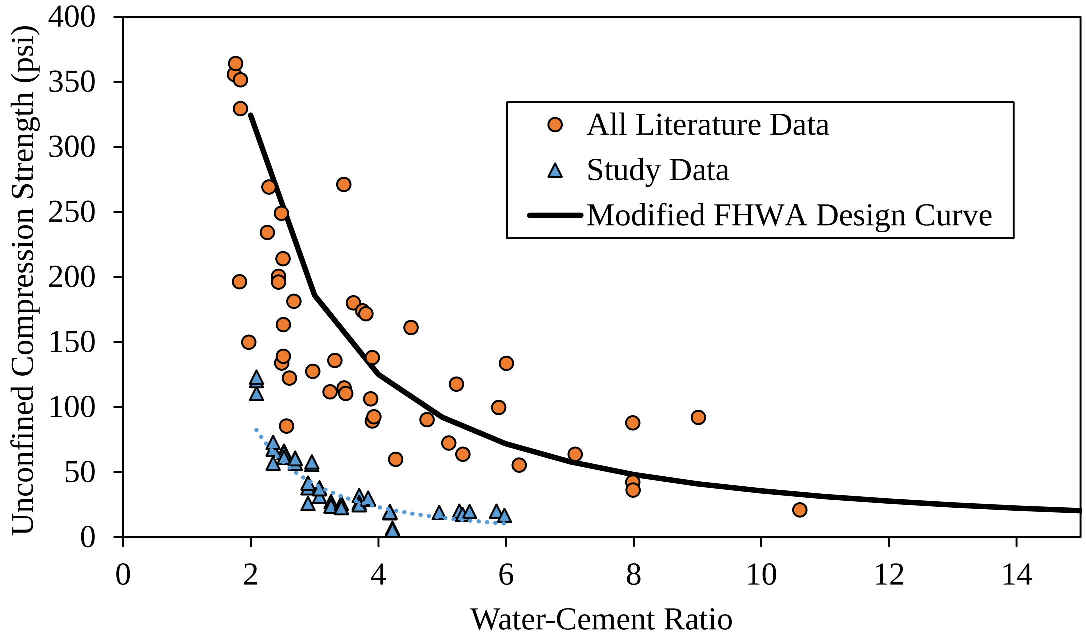

Figure 2.24. Results of laboratory unconfined compression tests along with literature values (Baker, 2015).

This thesis extends the laboratory findings to production soil mixing projects where laboratory bench tests were used to gain insight into expected performance before production commenced.

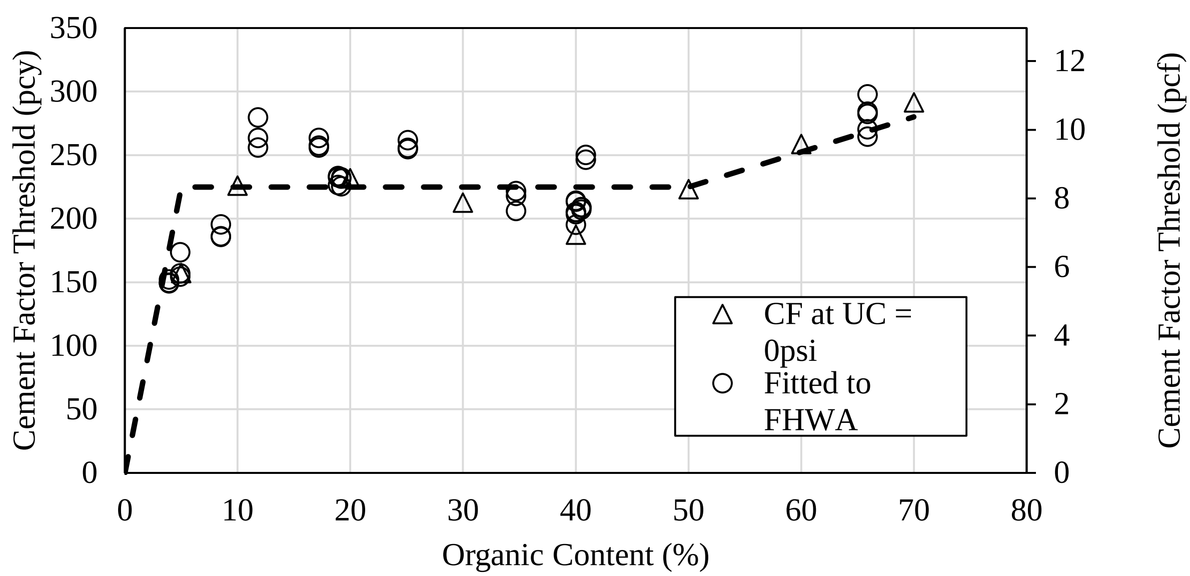

Figure 2.25. Cement factor threshold versus organic content (Baker, 2015).

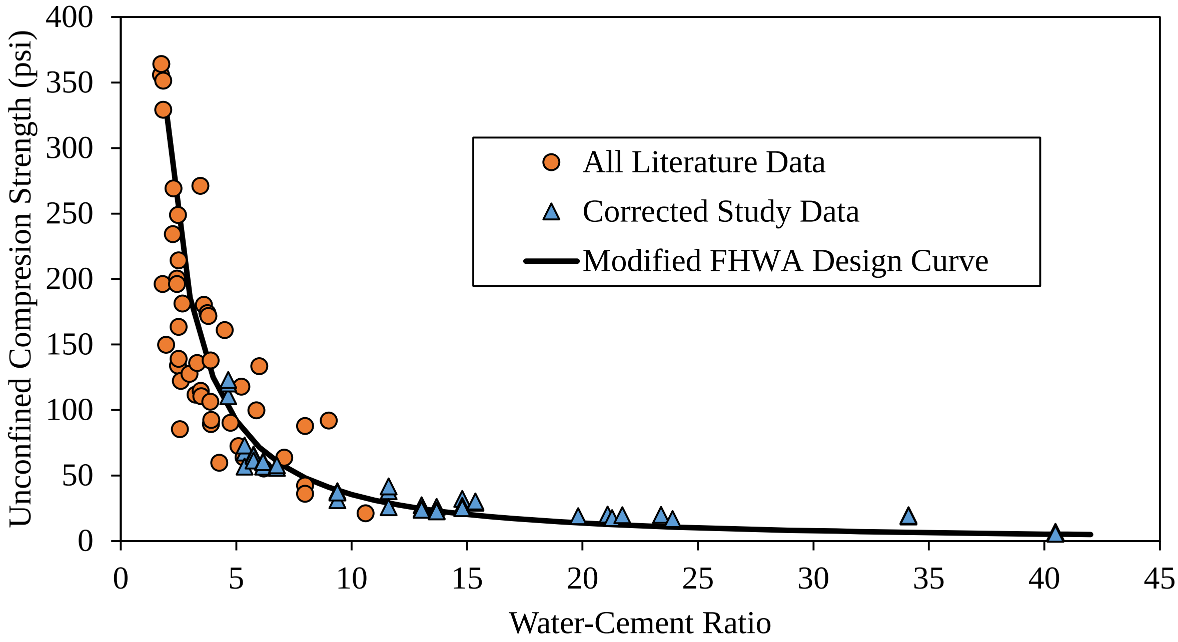

Figure 2.26. Unconfined compression strength versus w/c ratio corrected for cement factor threshold (Baker 2015).

CHAPTER 3: LARGE SCALE LABORATORY TESTING

As discussed at the end of Chapter 2, the USF study involved a wide range of laboratory tests performed utilizing different combinations of organic content, cement, and slag. From the results of these tests specific mix designs were chosen to test at a larger scale, (one-tenth scale of field conditions). This chapter will discuss the fabrication of the test bed, the type of soil tested and prepared, wet and dry mixing, loading of the test bed and results, and compression tests from the wet mixed sections post loading. It should be noted cores were attempted for the dry mixed section, however they were unsuccessful.

3.1 Fabrication of the Test Bed

The design of the test bed had several qualifications. Mainly, it needed to consist of three sections (wet mixing, drying mixing, and control; Figure 3.2), and be structurally capable of resisting deformations that could be caused by lateral soil pressures. Thus, a design of three 4ft by 4ft sections was decided upon, giving the tank final dimensions of 12ft long, 4ft wide, and 3ft high. The floor and walls were comprised of sheet metal, with I-sections as base supports. The bed was further reinforced with channel sections for wall support, which greatly added stiffness. The finished bed is shown below in Figure 3.1.

Figure 3.1. Bed during fabrication (left), finished bed with cover (right) (Baker, 2015).

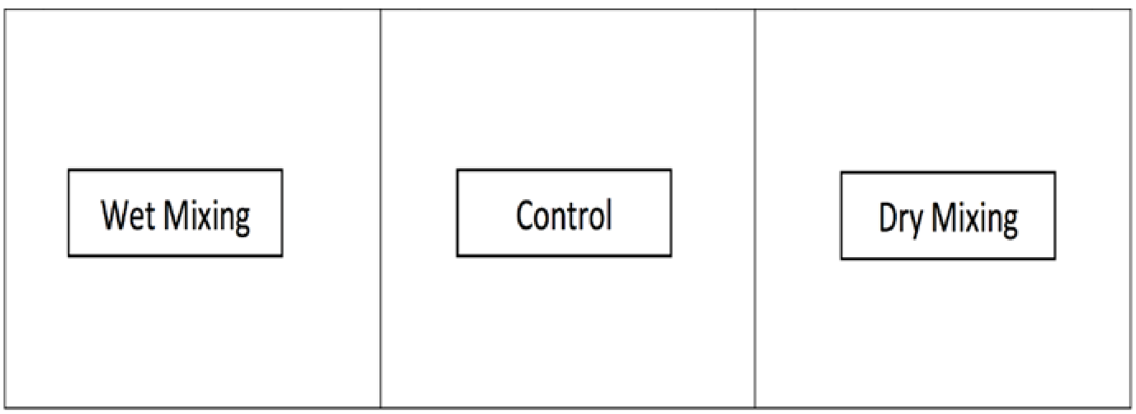

Figure 3.2. The soil bed layout (Baker, 2015).

3.2 Soil Properties and Preparation

The soil used for testing was taken from SR 33 just north of Polk City, Florida (Figure 3.3 and Figure 3.4). It had an organic content of 40% and moisture content of 167%. When the soil was received, a 24in layer of soil was placed into the test bed. The soil was allowed to sit in the tank until equilibrium conditions were established. Rainwater was periodically applied to help accomplish this and maintain the saturated soil conditions (Figure 3.5). Using rainwater mimicked what the soil would see in its natural habitat, versus using tap water, which may contain contaminants that could lead to chemical issues.

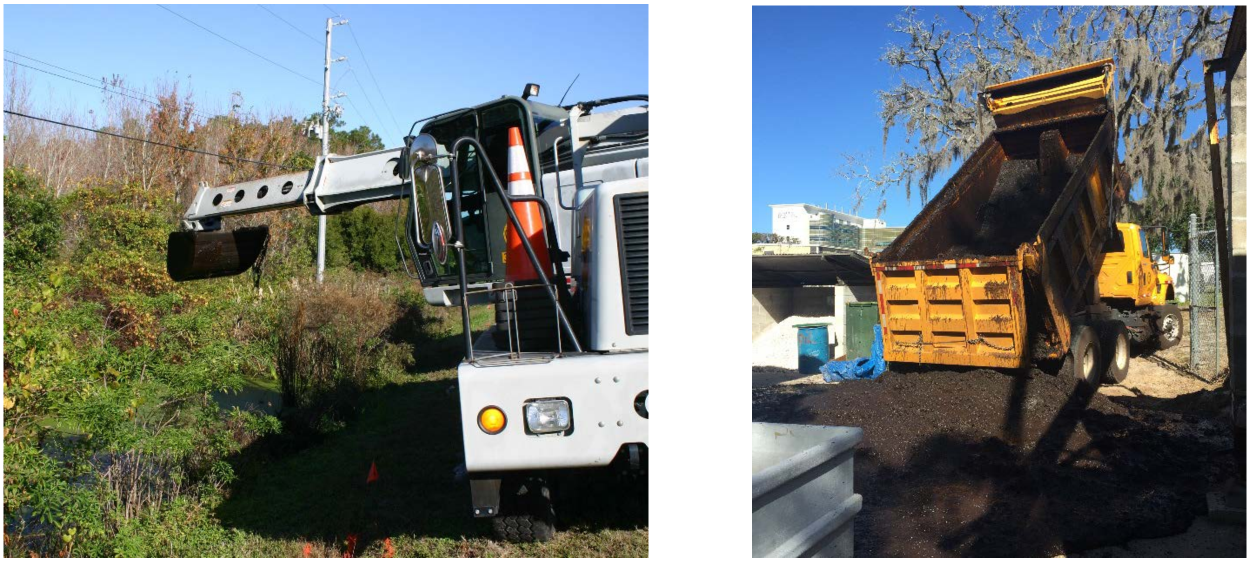

Figure 3.3. Excavation of the soil and initial delivery.

Figure 3.4. Transfer of the soil to the test bed.

Figure 3.5. Adding rainwater to soil bed to preserve natural conditions.

3.3 Soil Mixing Design

This section briefly describes the mix design for each method (wet and dry) as well as mixing technique. The initial condition for the mix design was it had to accommodate the load from a 5ft roadway embankment (a loading of 600psf).

The laboratory study showed that slag did not perform well for the sandy organic soil from SR 33. Long after the laboratory study was completed, vane shear tests were performed on the leftover all-slag and slag-cement cylinders deemed incapable of being tested using an unconfined compression test (Figure 3.6). After 100+ days, these cylinders were still showing almost no signs of strength gain. Test results ranged from 1.37psi to 6.12psi which in essence was deemed unstabilized. For further information on the results of the all-slag and slag-cement cylinder vane shear tests please reference Appendix B.

Figure 3.6. Vane shear test performed on the slag-cement cylinders.

3.3.1 Wet Mixing Design

In Chapter 2 it was presented that for wet mixing, typically soil columns are used. Mullins (1996) discussed methods that produce columns of higher strength material (e.g. DR, stone columns, deep soil mixing, or rigid inclusions) often use a proportional strength method to design the strength of a treated soft soil region which depends on the number, diameter, and strength of columns installed using the following expressions:

$$q_{des} = a_s(q_{col} - q_{sur}) + q_{sur} \tag{3-1}$$

where

q_{des} = 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 is simply the ratio of the column area to unit cell area (equation 3-2) and the unit cell area depends on the treatment pattern and spacing as shown in Figure 3.7. This in turn depends on the column layout configuration (hexagonal or rectangular).

$$a_s = \frac{a_{col}}{a_{unitcell}} \tag{3-2}$$

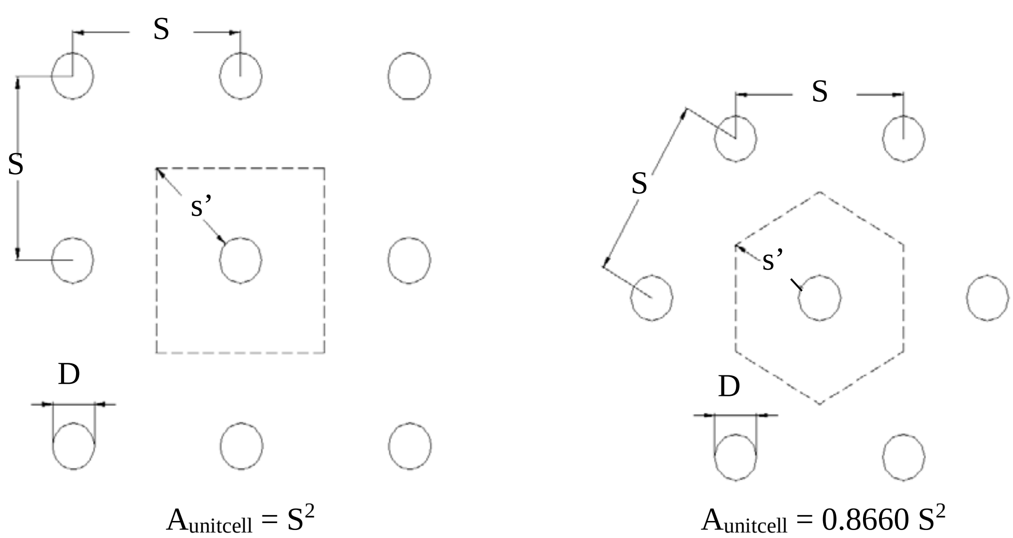

Figure 3.7. Unit cell adapted from Filz, (2012).

For this case, a design strength of 600psf was required and a 4.25in column diameter was selected based on available equipment. This meant that the column spacing, configuration geometry and column strength remained to be determined. In soft soils like highly plastic clay or organics, the insitu soil can be assumed to have zero strength relative to the columns, $q_{sur} = 0$.

Using a 20% area replacement ratio and a 9in center-to-center column spacing, the column strength (for a 4.25in diameter column) would need to be 20.83psi (3000psf). In order to determine the mix design that would yield this strength, data from the lab tests was extrapolated (the highest strength seen in wet mixing was 17.37psi) (Figure 3.8). While the cement factor chart suggests using 535pcy, factoring in an assumed moisture content of 200% yielded a required cement content of 590pcy. In terms of what each column sees, a 4.25in diameter column of 24in depth would then require 13lbs of grout with a w/c of 0.8.

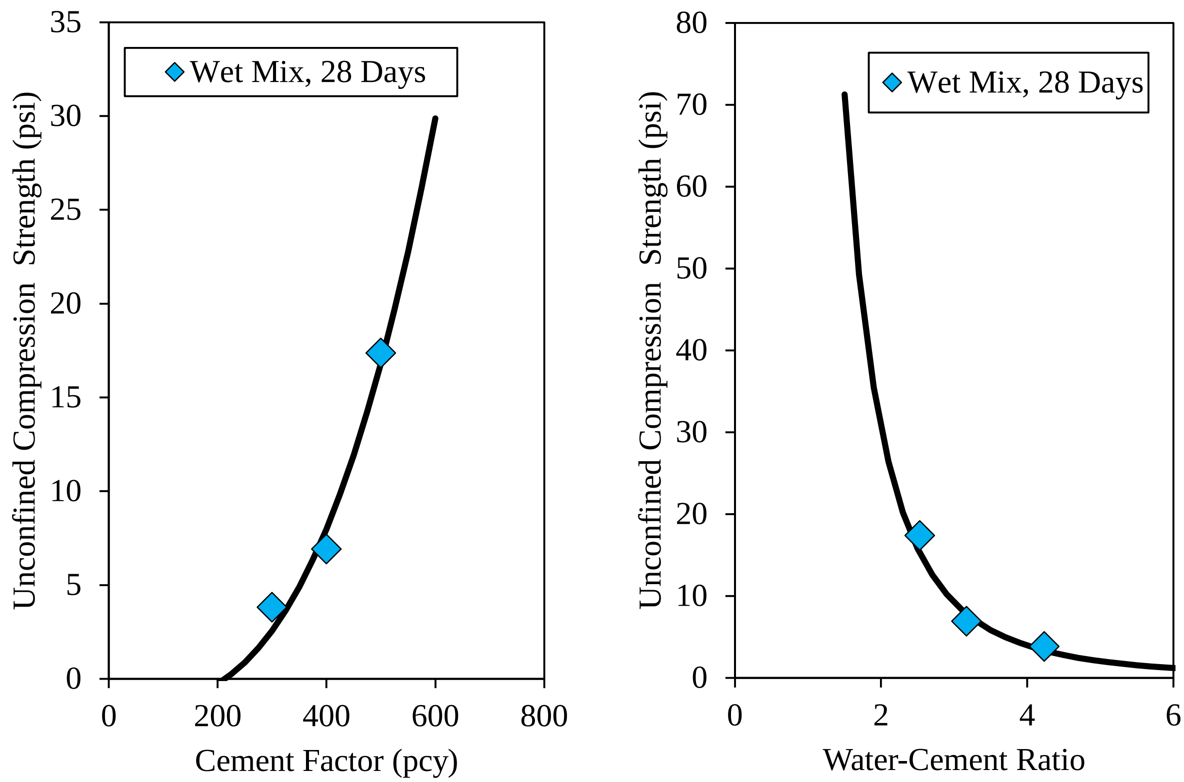

Figure 3.8. Extrapolated wet mixing data for a grout w/c ratio of 0.8 (Baker, 2015).

The system to implement this mixing technique is rendered in Figure 3.9. The mixing tool was a hollow-core auger that had an engine attachment so it would spin as it dispensed the 0.8 w/c grout. To pump the grout to the auger a pressure pot was used. This involved first mixing a fixed volume of grout on the side and then transferring the grout to the pressure pot, which connected to the auger by grout hose. To achieve flow, air was pressurized over the grout and thus grout flowed from the hoses to the hollow core section of the auger which released the grout into the soil while spinning. As seen in Figure 3.9 the auger and pressure pot were suspended from load cell held by a crane hoist so that the system could be easily translated.

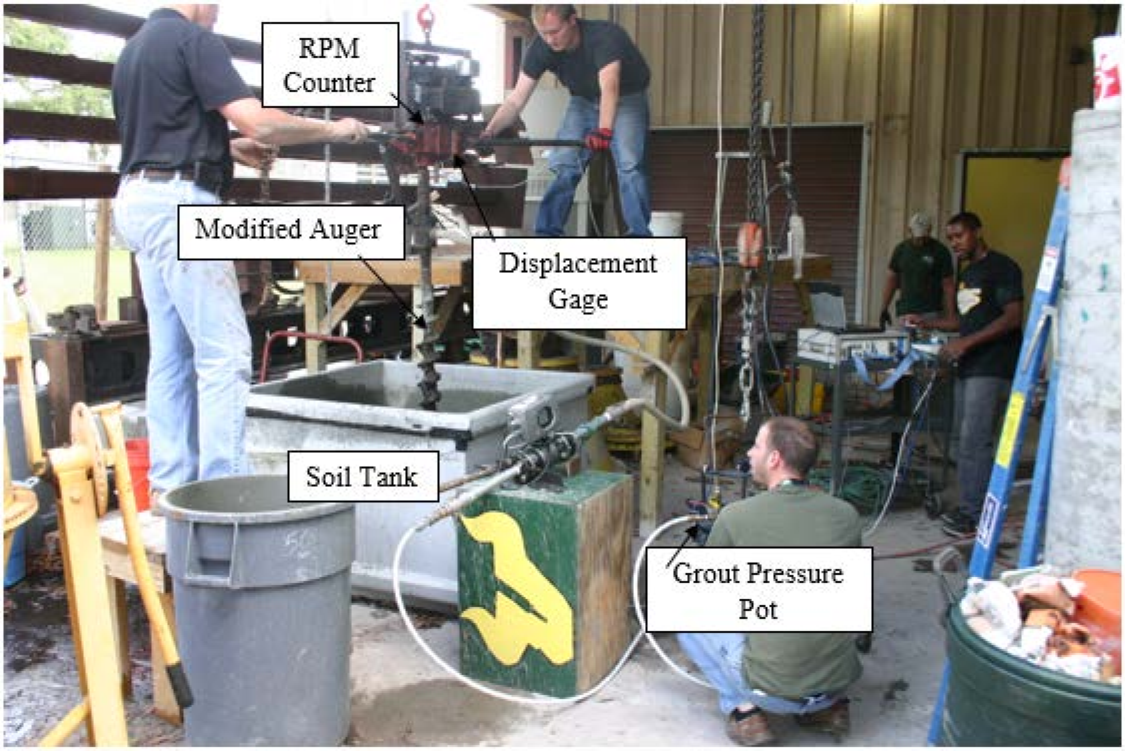

Figure 3.9. Wet mixing machine design (Baker, 2015).

Column injection was done using a hexagonal pattern (Figure 3.10), using column diameters of 4.25in as stated above, and spacing of 9in center to center. This yielded a pattern of 19 columns to accommodate the loading area of a 2ft diameter bearing plate. While technically only the center 7 columns were loaded, the outer columns were produced to provide intended confinement. Numbered flags were placed in the soil to label column locations for mixing (Figure 3.11). The final mix design asked for 16lbs of grout in each column.

Figure 3.12 shows the amount of grout pumped into each column in relation to the amount needed. Most columns received a sufficient amount of grout or were not far from the quantity needed. The overall process of wet mixing is shown in Figure 3.13, where the grout was mixed to the side, the grout flow rate, weight, penetration/position of the auger tip was monitored, and the soil mixing column was installed (Figure 3.14).

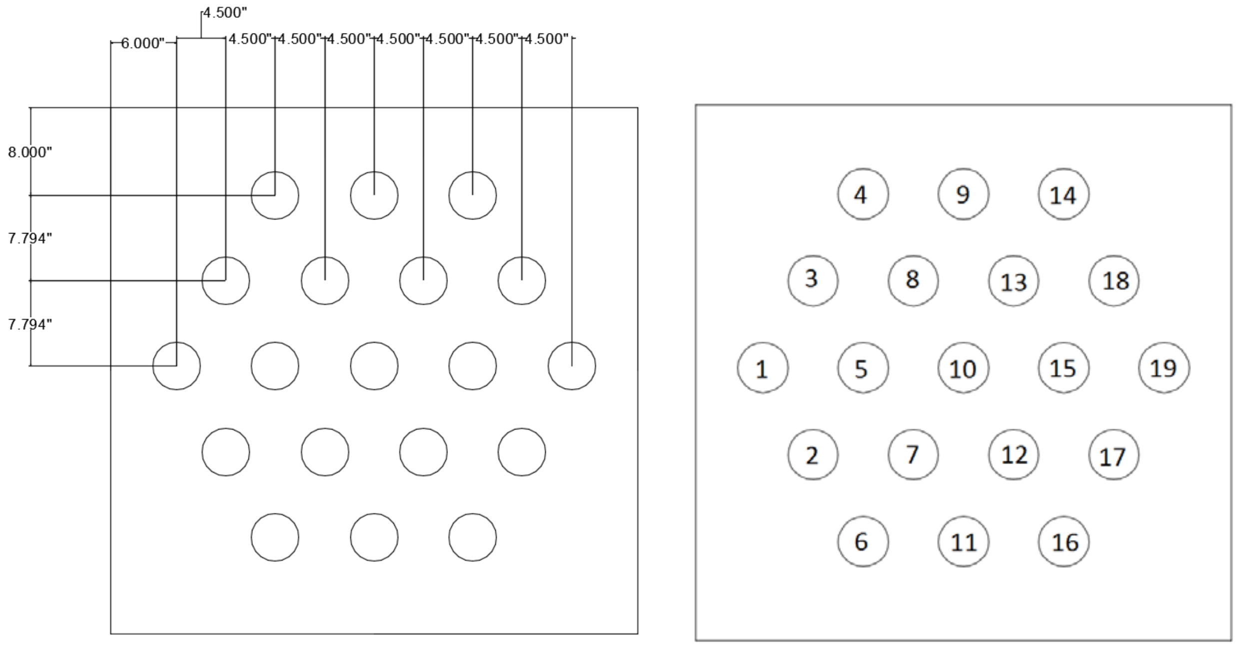

Figure 3.10. Hexagonal column pattern and spacing, north right (Baker, 2015).

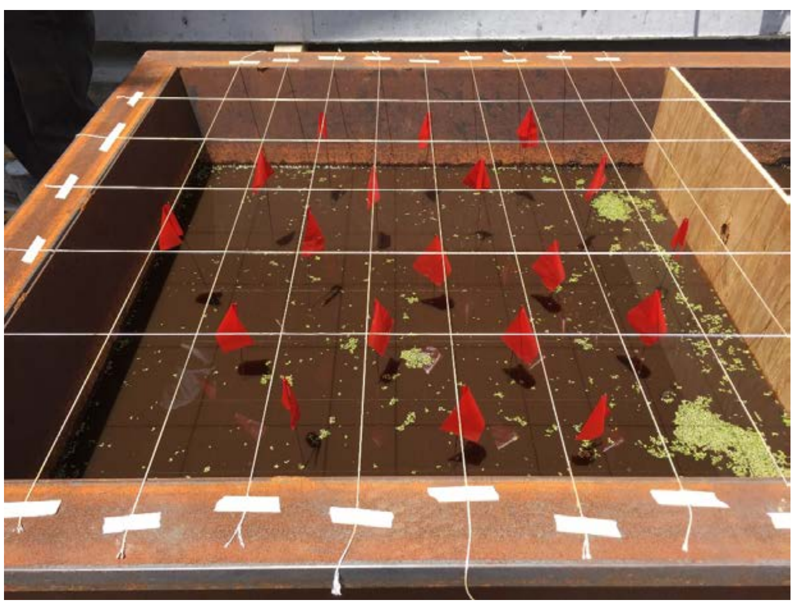

Figure 3.11. Hexagonal column pattern laid out before mixing, north right.

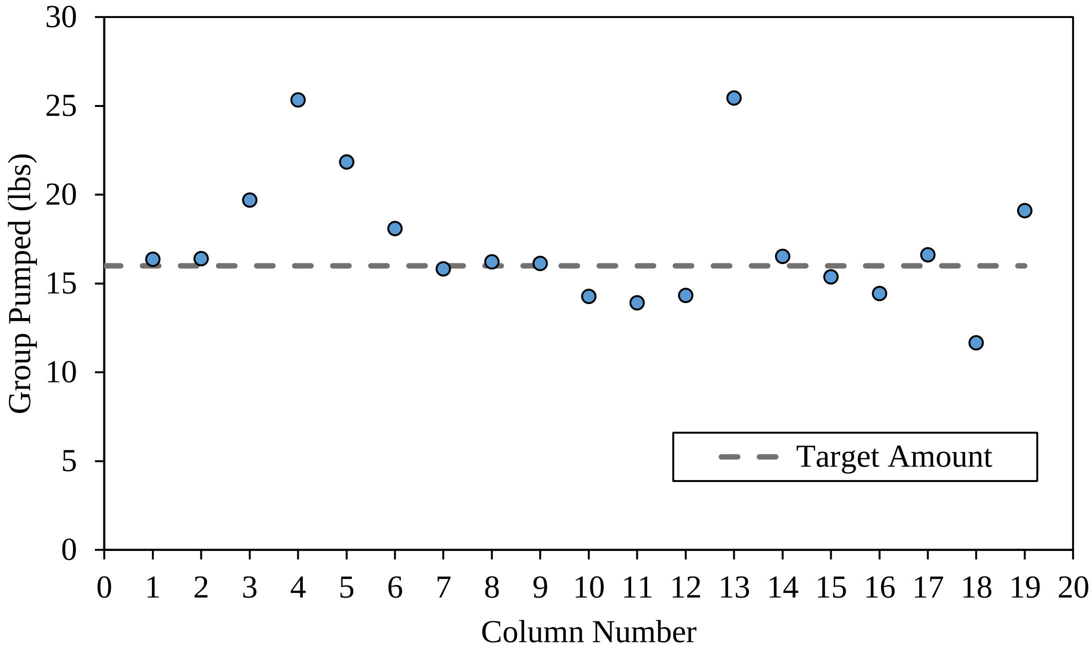

Figure 3.12. Amount of grout injected into each column (Baker, 2015).

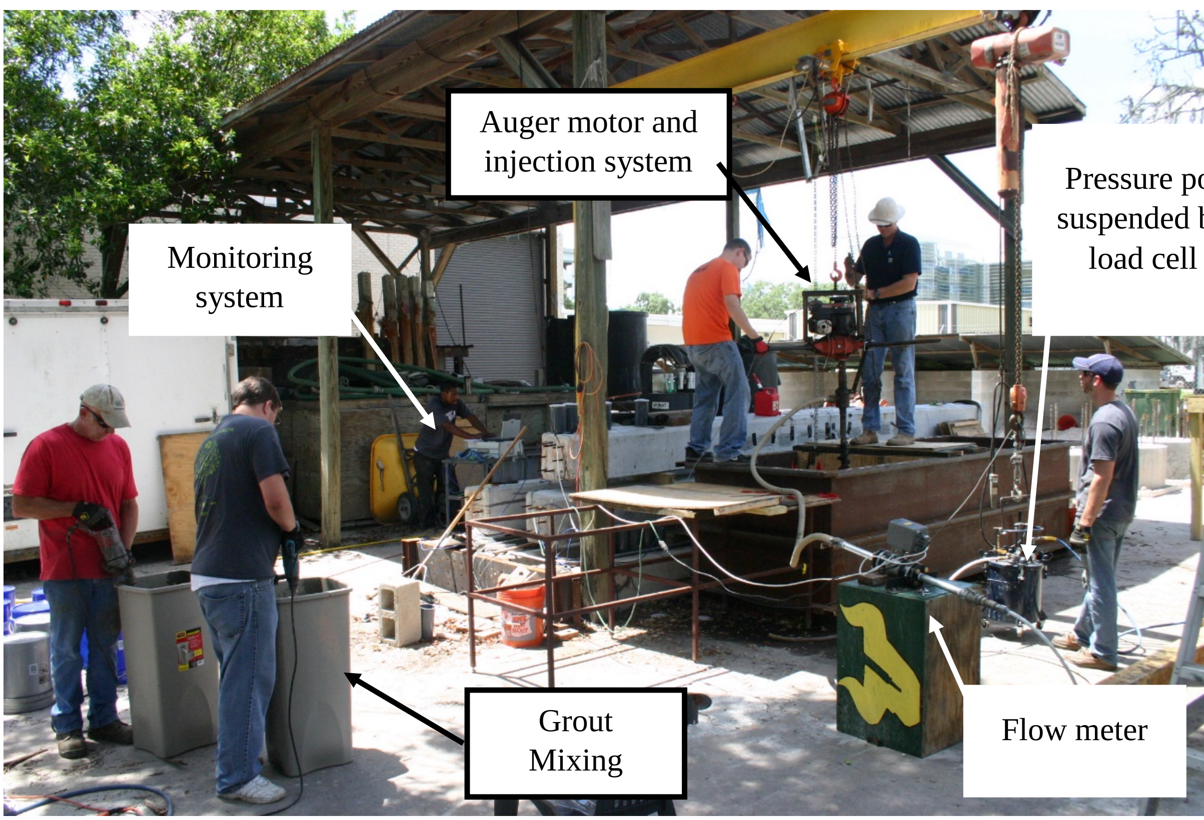

Figure 3.13. Mixing process of wet soil columns.

Figure 3.14. Wet mixing process up close.

3.3.2 Dry Mixing

The dry mixing technique differed from wet mixing. Initially, column construction was attempted with this method. Seeing as dry powder cannot be pumped like grout, the idea came about to place the powder into a PVC pipe with a removable cap on the bottom to release the cement powder into the soil, (the same amount, 8.9lbs, as wet mixing was used). Once the cement was in the soil the auger was used to mix the dry cement into the soil, with the intent to form a column similar to that of wet mixing (Figure 3.15). This was extremely unsuccessful as there was no evidence that cement had even been placed into the soil. Thus, a technique closer to what was discussed in Chapter 2 (mass stabilization) was performed.

The soil moisture was first brought back to a fully saturated state through the addition of rainwater and a horizontally spinning tiller was used to check if the soil could be mixed mechanically. This also loosened the soil to a state that could be reasonably well penetrated during

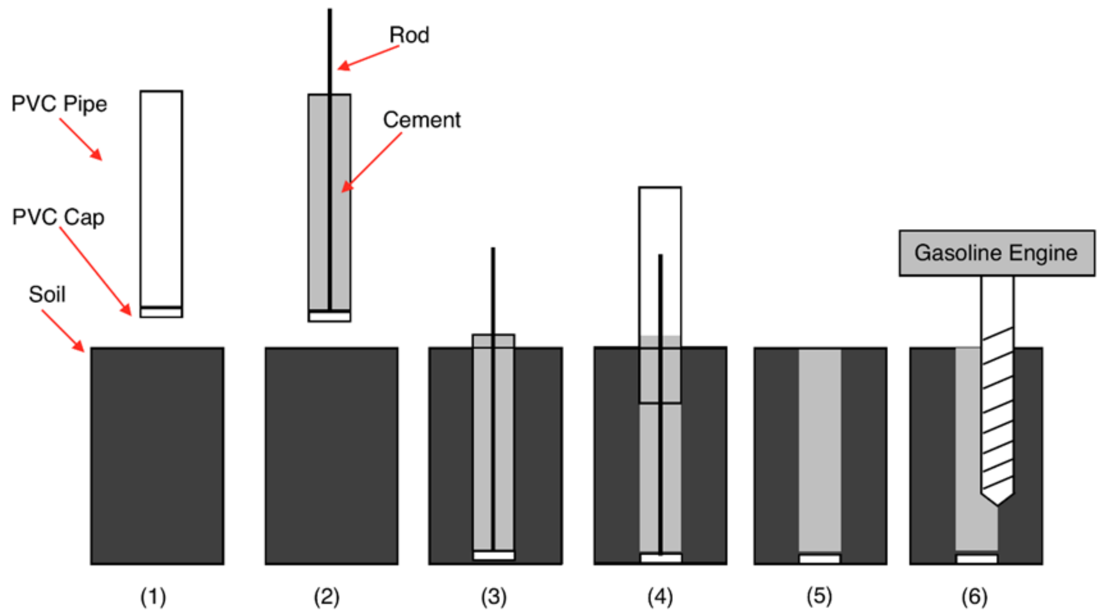

Figure 3.15. Dry mixing concept procedure (Baker, 2015).

the soil mixing process. Two vertically aligned mixing paddles were used as well to assist. This was done until the soil achieved a fluid like mixing motion (Figure 3.16). Then, dry cement powder (288lbs) was evenly distributed across the top of the soil and mixed thoroughly with the tiller,

Figure 3.16. Initial breakup of the saturated organic soil prior to adding cement.

again with the assistance of the paddles (Figure 3.17 to 3.20). During this entire process the tiller experienced several issues mixing below the surface. For this reason the two outer blades were removed, which made the tiller operable at greater depths. Once the section appeared homogenously mixed the top was leveled (Figure 3.21). It should be noted that the addition of cement greatly reduced the workability of the soil immediately. This method was much more difficult to implement than the wet mixing method.

Figure 3.17. Right before cement was added (left), the initial addition of cement (right).

Figure 3.18. Adding of cement (left), and making sure it was equally distributed (right).

Figure 3.19. Initial mixing with the tiller.

Figure 3.20. Mixing with the tiller and mixing paddles.

Figure 3.21. Finished dry mixing soil bed.

3.4 Loading

Once the soil cured for 28-days a frame to support the loading system was fabricated and welded to the test bed. A small amount of sand was placed directly on the soil to form a leveling pad and aid in any unevenness the bearing plate might experience. Next 2ft diameter steel bearing plates were centered in each of the three test sections (Figure 3.22, Figure 3.23). On top of each of the three bearing plates a 300 gallon plastic tank was placed that was filled with water as a load source. As water is easy to add/remove, loading could be added gradually. The support frame kept the tanks aligned vertically and stopped any translation horizontally or tipping (Figure 3.24). An additional frame was also erected on which to mount the instrumentation used to measure vertical displacement (Figure 3.25).

Figure 3.22. Bearing plate assembly.

Figure 3.23. Closer look at bearing plate set up (sand, wood, then steel plate).

Figure 3.24. Loading system on the test bed.

Figure 3.25. String line transducer above water tank (Baker, 2015).

The diameter of the tanks were larger than the loading plate such that the height of the water in the tanks produced proportionally more pressure on the soil than at the water at the base of the tank. Load was applied in 50psf increments per day up to the design load of 600psf. However, load was not increased if the rate of settlement had not fallen to an acceptable level (i.e. 0.001in/min or less) per ASTM plate load test standards.

3.5 Results

Both the wet and dry soil mixed partitions outperformed the control section significantly. The loading was kept at the design load for the control section, however it was increased to 800psf for the wet and dry sections since they performed so well at the design load. Figure 3.26 shows the 50psf load increments and the number of days each load level was maintained. For the lower loads, multiple increments of loading could be applied on the same day. The wet mixed and control sections were loaded first, the dry mixed section a few days after so that the curing ages were equivalent for both sections.

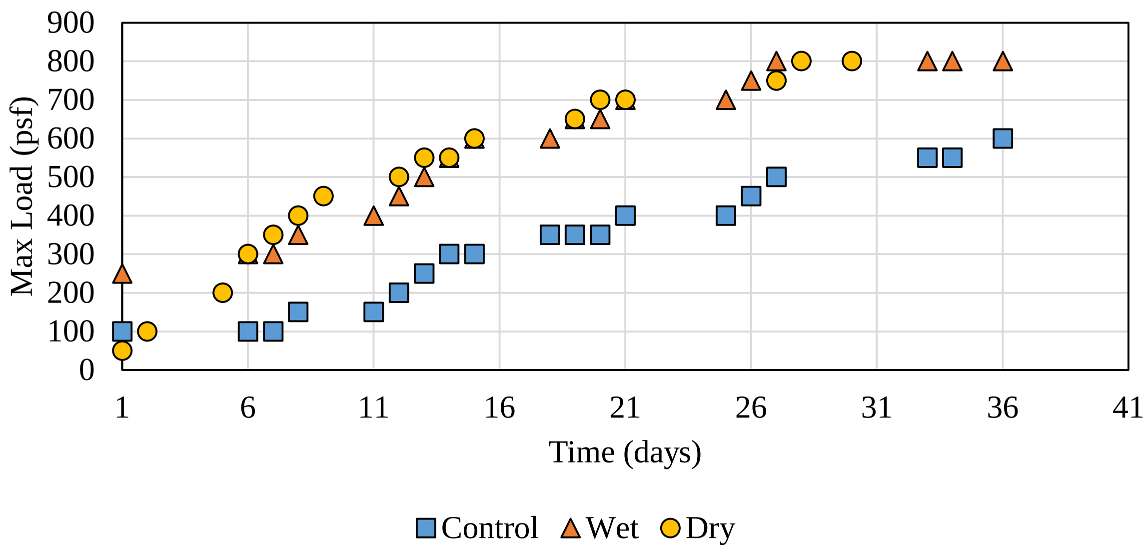

Figure 3.26. Loading schedule of the test bed (Baker, 2015).

Figure 3.27 shows the displacement of the soil versus the amount of pressure applied from both survey values taken and string line transducer data recorded. Figure 3.28 displays only the survey data in comparison to time. Due to a malfunction with the dry mix string line transducer only survey data is shown for that section. The survey data for the wet mixed and control sections were in close agreement with the string-line data.

When the tanks were unloaded it was planned to take off 25% of the load at a time. The wet and control sections were completed by removing 25% of the load on the first and second days, then 50% was removed on the third day. For the dry mix section, 25% of the load was removed in one day; the remaining load was removed on the second day in 25% increments.

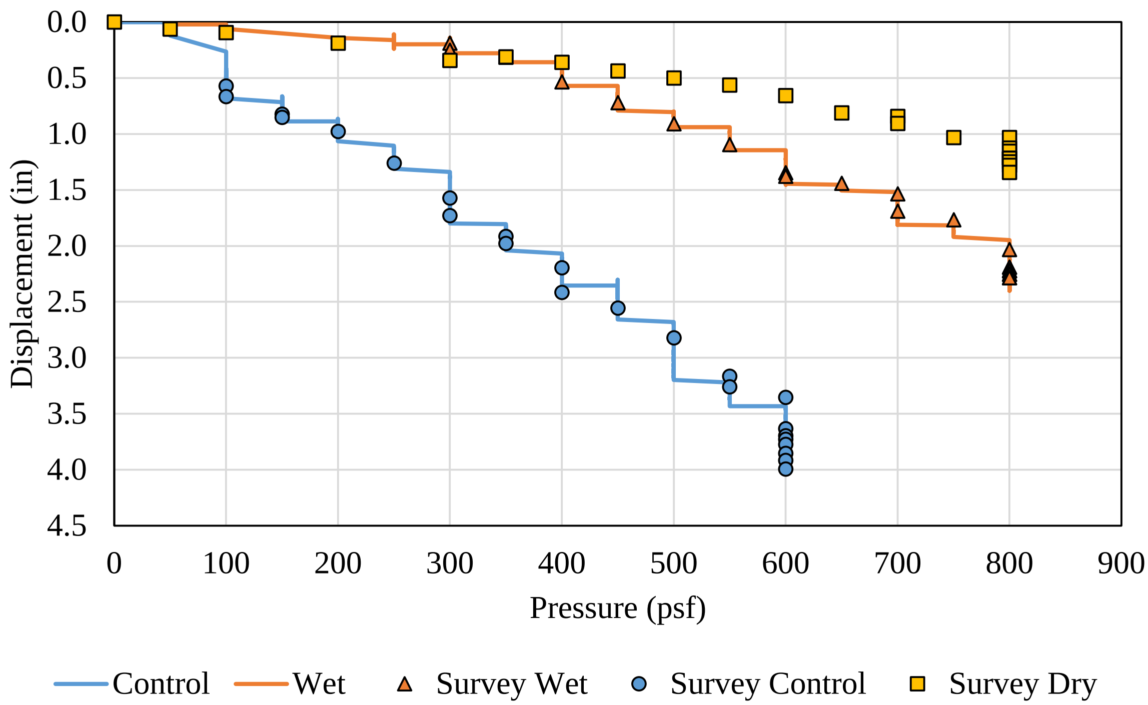

Figure 3.27. Displacement of the soil versus the applied load.

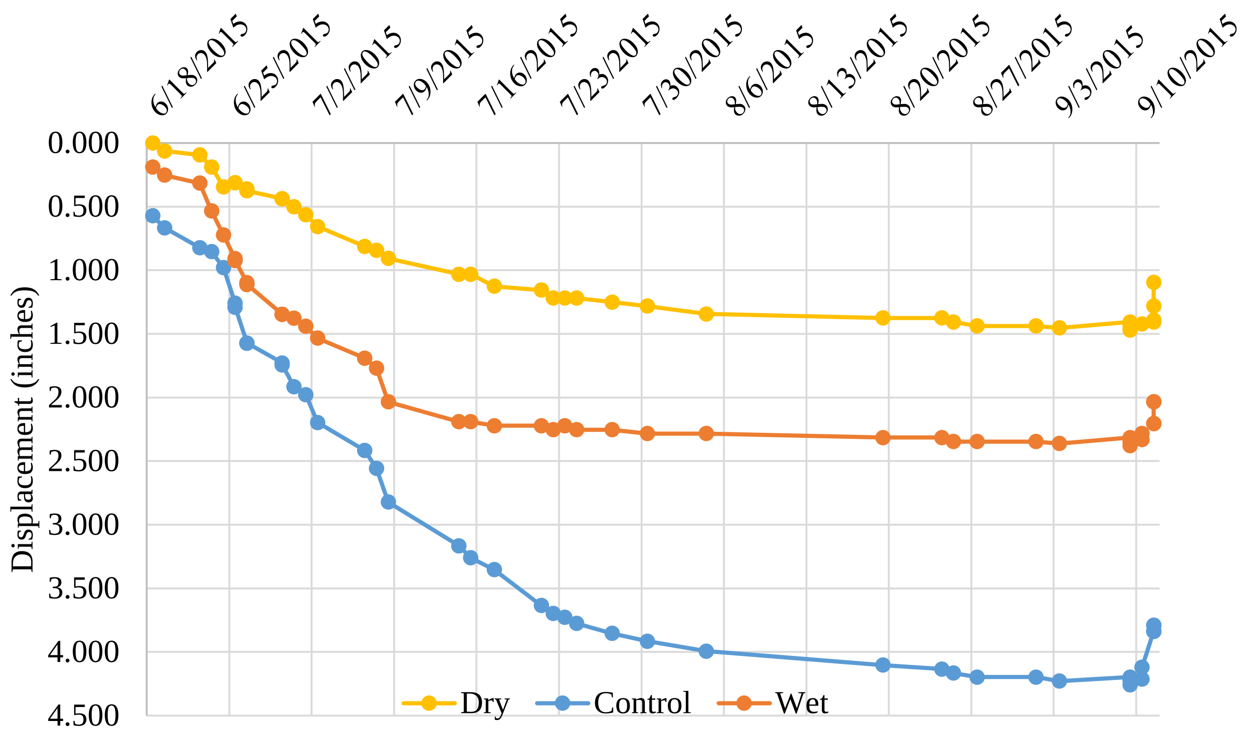

Figure 3.28. Displacement of the soil versus time (date) using survey data.

3.6 Individual Column Compression Test Results

Once the tanks were unloaded and the soil was done rebounding, the control partition was removed so the wet and dry sections could be excavated (Figure 3.29). Figure 3.30 shows the initial excavation of the soil surrounding the wet mixed columns giving an uncommon view of what the columns actually looked like once mixed. Initially the excavation was done by hand, with a small Kubota back hoe which removed excess soil from the center partition (untreated

Figure 3.29. Side view of the wet mixed section before exhuming.

control section). After the columns could be accessed from the side, a hose was used to wash away the rest of the soil surrounding the columns. The excess water had to be pumped out. The columns were extracted for compression tests to see how they compared to their expected performance.

Most columns held up well upon extraction, while others were not able to be salvaged (Figure 3.31). Figure 3.32 shows all columns after extraction, starting with column 19 in the top left corner (original pattern shown in Figure 3.10 and 3.11).

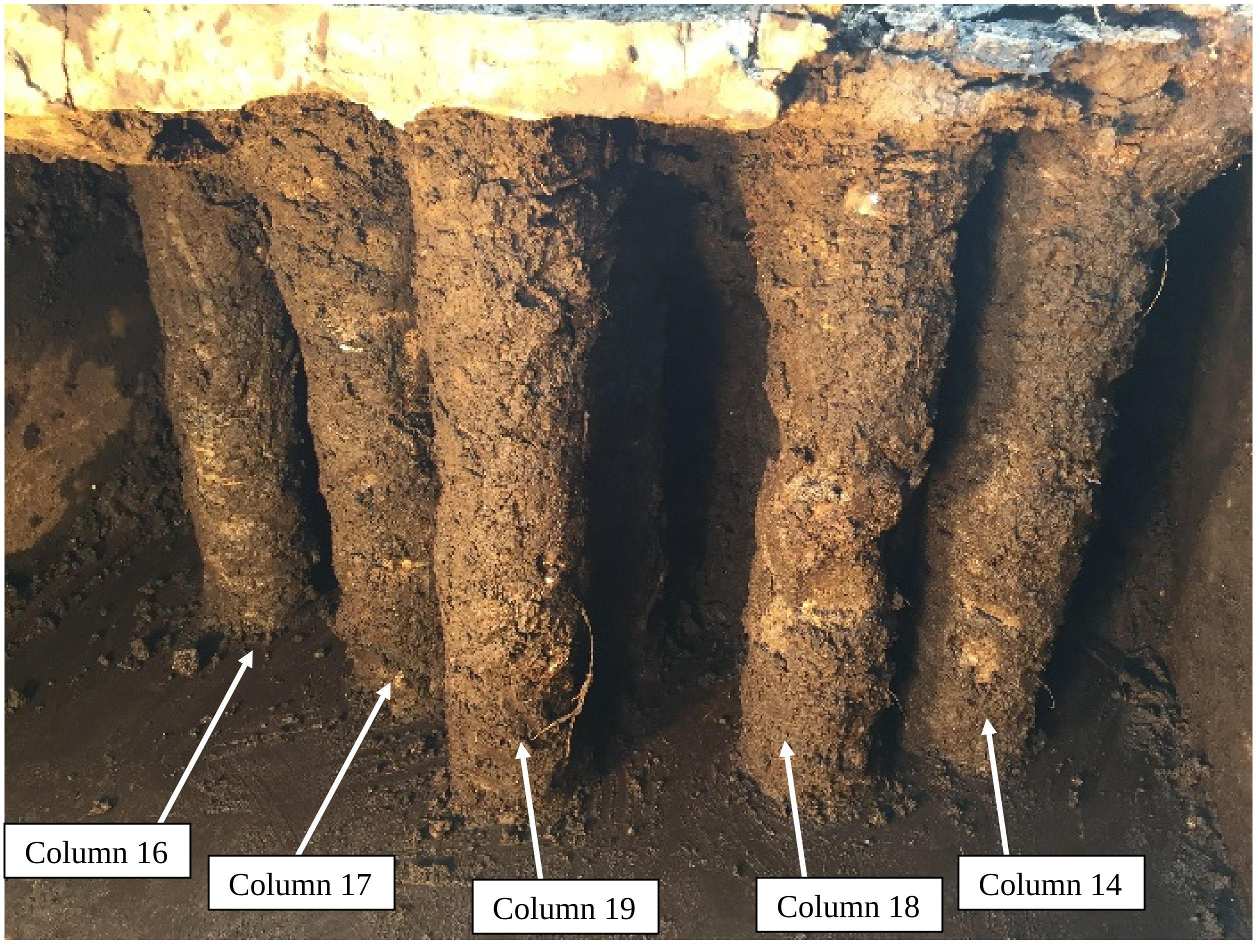

Figure 3.30. Side view of the wet mixed section after removing the surrounding soil.

After the excavation, the exhumed columns were cut into pieces with a length to diameter ratio of approximately 2. When the columns were created, the amount of grout pumped was recorded (Figure 3.12) and thus by exploring that data, the amount of grout pumped into the section of each column tested was acquired. By having this value the predicted compressive strength for the column section could then be calculated as a function of depth.

Figure 3.31. A column that was unable to be salvaged (left), versus columns that were able to be salvaged and tested (right).

Figure 3.32. The 19 wet-mixed columns after excavation.

Figure 3.33 compares the predicted compressive strength value with the column sections actual strength. The line drawn at the bottom represents the required amount of strength per column. Clearly, most columns surpassed the required strength. Only one of the 17 samples tested was reasonably close to that predicted from laboratory findings. The column sections predicted strengths that are 2 to 28 times the actual, about 9 times on average. This variability, discussed later in Chapter 5, implies that another variable was likely affecting strength results.

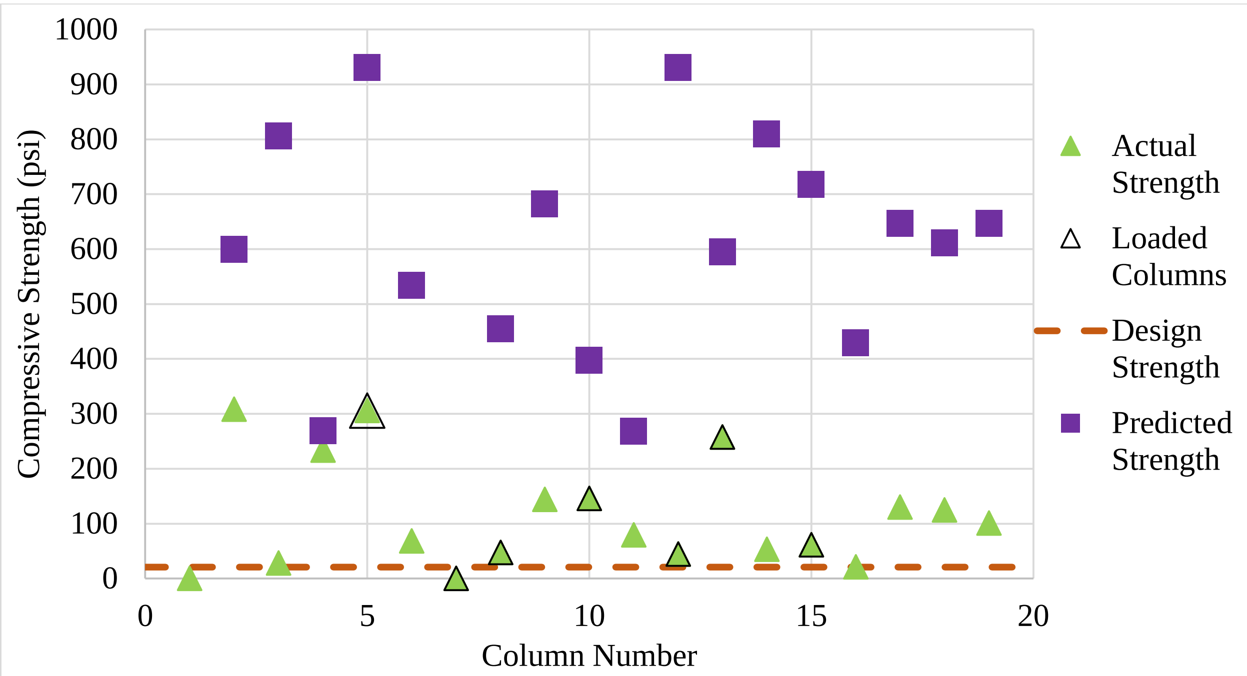

Figure 3.33. Comparison of the predicted column strength versus the actual column compressive strength.

CHAPTER 4: FIELD EVALUATIONS

This chapter provides an overview of full scale soil mixing programs that were either previous performed or conducted concurrent to this thesis. These include: SR 33, Jewfish Creek, Marco Island Executive Airport, US331 over Choctawhatchee Bay, and LPV111. A few unnamed case studies provided by Hayward Baker for additional data are included as well.

4.1 State Road 33, Polk City

State Road 33 on the northern outskirts of Polk City, Florida runs just across the southeast edge of the Green Swamp in Polk County. For over 70 years, a 1000ft section of the road has experienced persistent settlement and has required constant attention from the Florida Department of Transportation's district maintenance office. Figure 4.1 shows an aerial as well as the north-looking and south-looking views of this section of road.

In Figures 4.2 and 4.3 a 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 inches of asphalt used in correcting the surface subsidence, underlain by 5 to 6ft of sand and 72 feet of organic material. The boring was terminated without finding the bottom of the organic layer due to MOT concerns, but conclusively identified the cause, organic soil settlement.

As part of this thesis study, field survey measurements were performed along SR 33 just north of Polk City in the denoted corridor. 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 4.4). Figure 4.5 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.

Figure 4.1 Arial view of a 1000ft section of SR 33 north of Polk City, FL that has been continuously repaired to combat subsidence. Denoted region represents worst area.

Figure 4.2. Visible distress along SR 33 approaching subsidence zone from the south.

Figure 4.3. Visible subsidence along SR 33 approaching from the north.

In cooperation with the FDOT District 1 geotechnical group, eleven cone penetration tests (CPT) were performed along SR 33 on November 20 and 21, 2012. The soundings were done at the survey locations reported earlier. Figure 4.6 shows the seventh of eleven CPT soundings; all CPT data are shown in Appendix C. From this data a soil profile along the roadway was created (Figure 4.7). 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 4.8).

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.

Coincidentally, or not, the location and thickness of the organic material (shown as tan/brown) corresponds directly to the top of roadway surface depression shown in Figure 4.8.

Figure 4.4. Survey measurement location IDs along SR 33 just north of Polk City.

Figure 4.5. Initial survey measurements along SR 33 just north of Polk City.

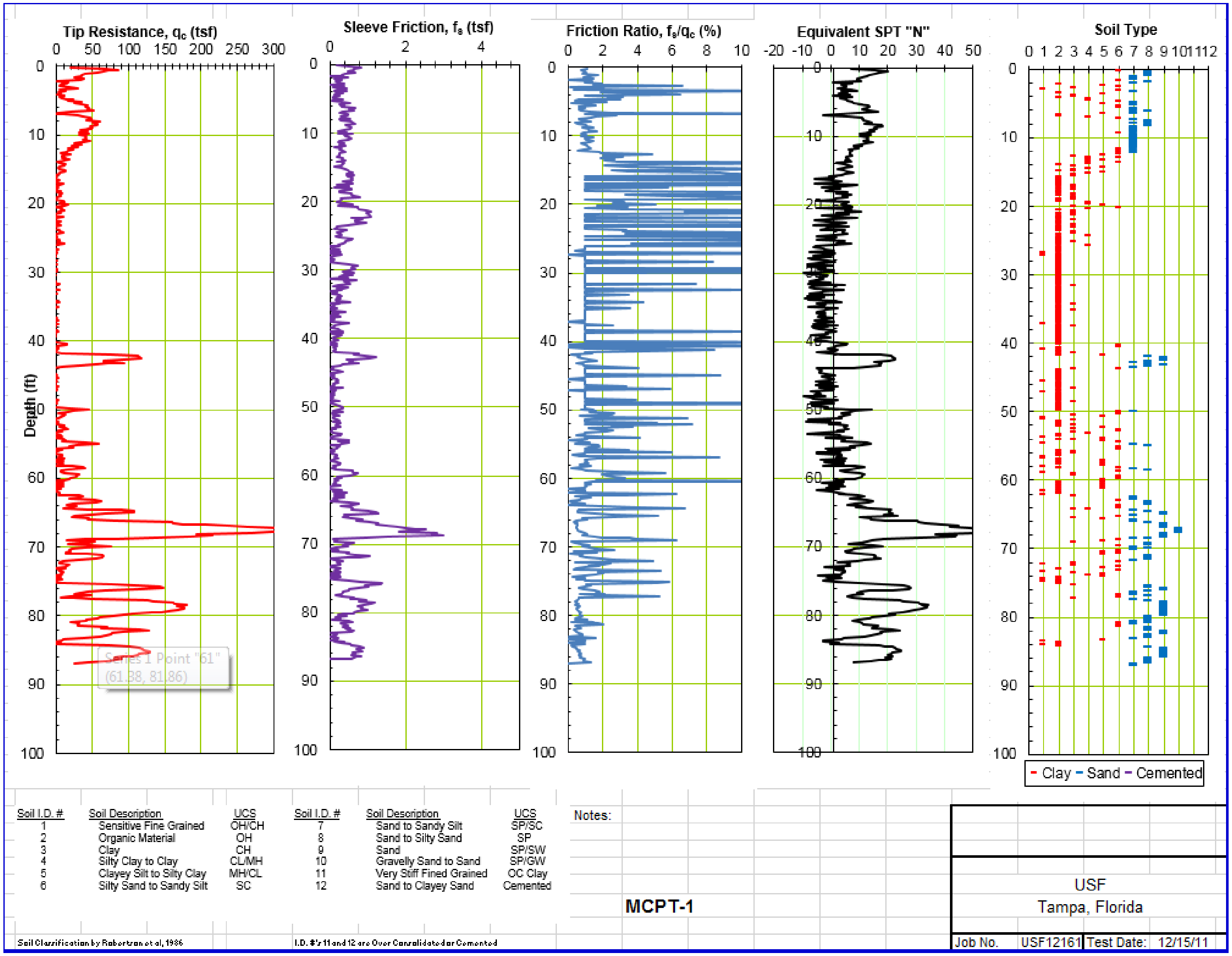

Figure 4.6. CPT 1 along SR33.

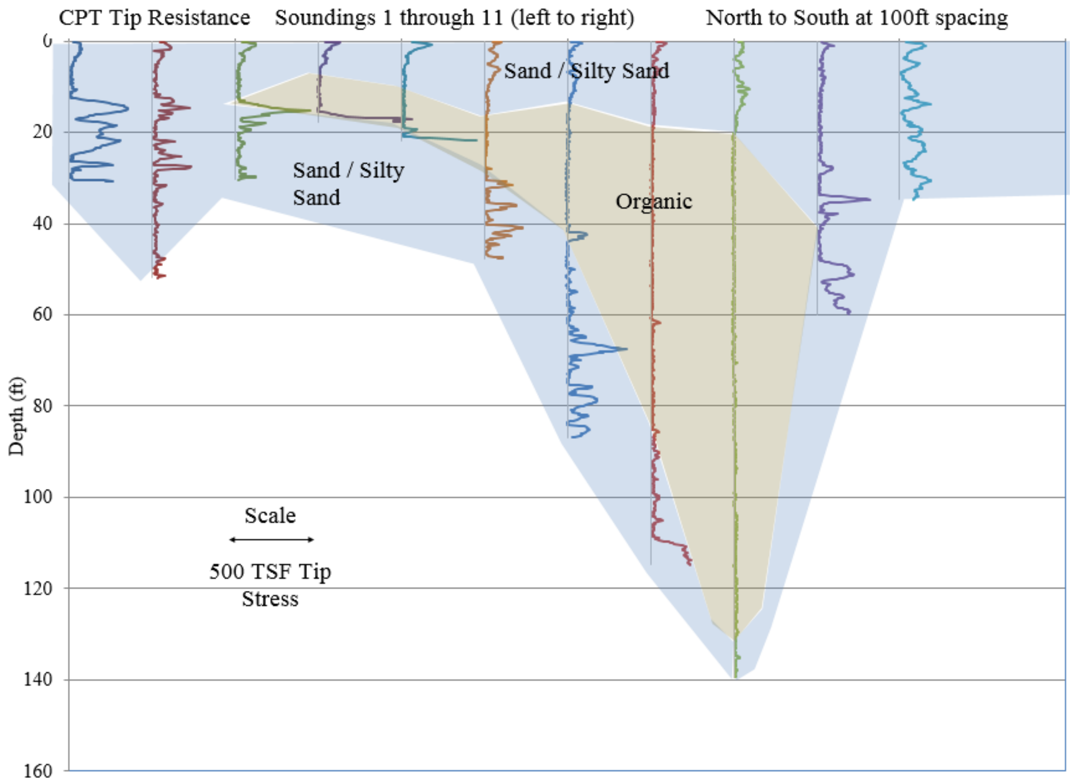

Figure 4.7. Soil profile created from individual CPT soundings along SR33 corridor in Polk City.

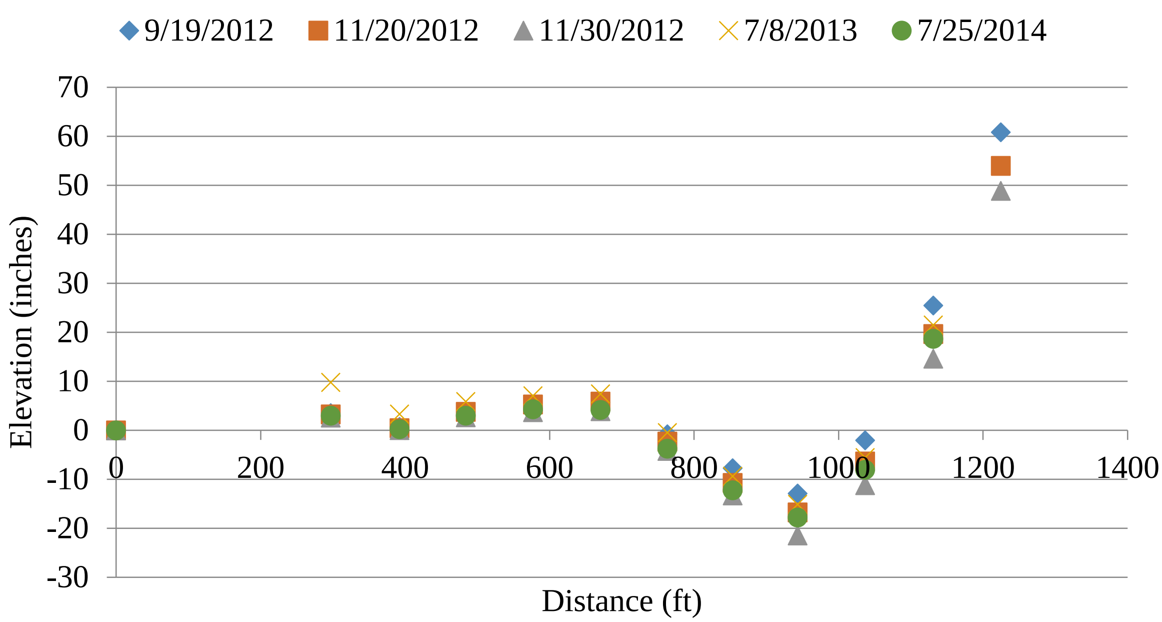

Figure 4.8. Survey data from SR 33 north of Polk City (SB Roadway)

4.2 Jewfish Creek US-1

Use of dry soil mixing (DSM) in the form of mass stabilization was successfully used in 2006 to stabilize a new 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 15ft 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 program with trial binder mix ratios to establish the mix design which resulted in 200-300PCY (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 (26in x ¾in) 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 4.9). Although other test methods were also used, KPS testing was the primary source of day to day quality assurance (1 test every 2500ft^2). In its entirety, the soil mixing program took 13 months to treat 360,000 cubic yards of organic soil.

Figure 4.9. 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 in height over a 2 week period to a height of 25ft. Settlement of the mixed soil slowed to a stop about 2 weeks after the full load had been applied. Figures 4.10 and 4.11 show the treatment process and surcharge test results, respectively.

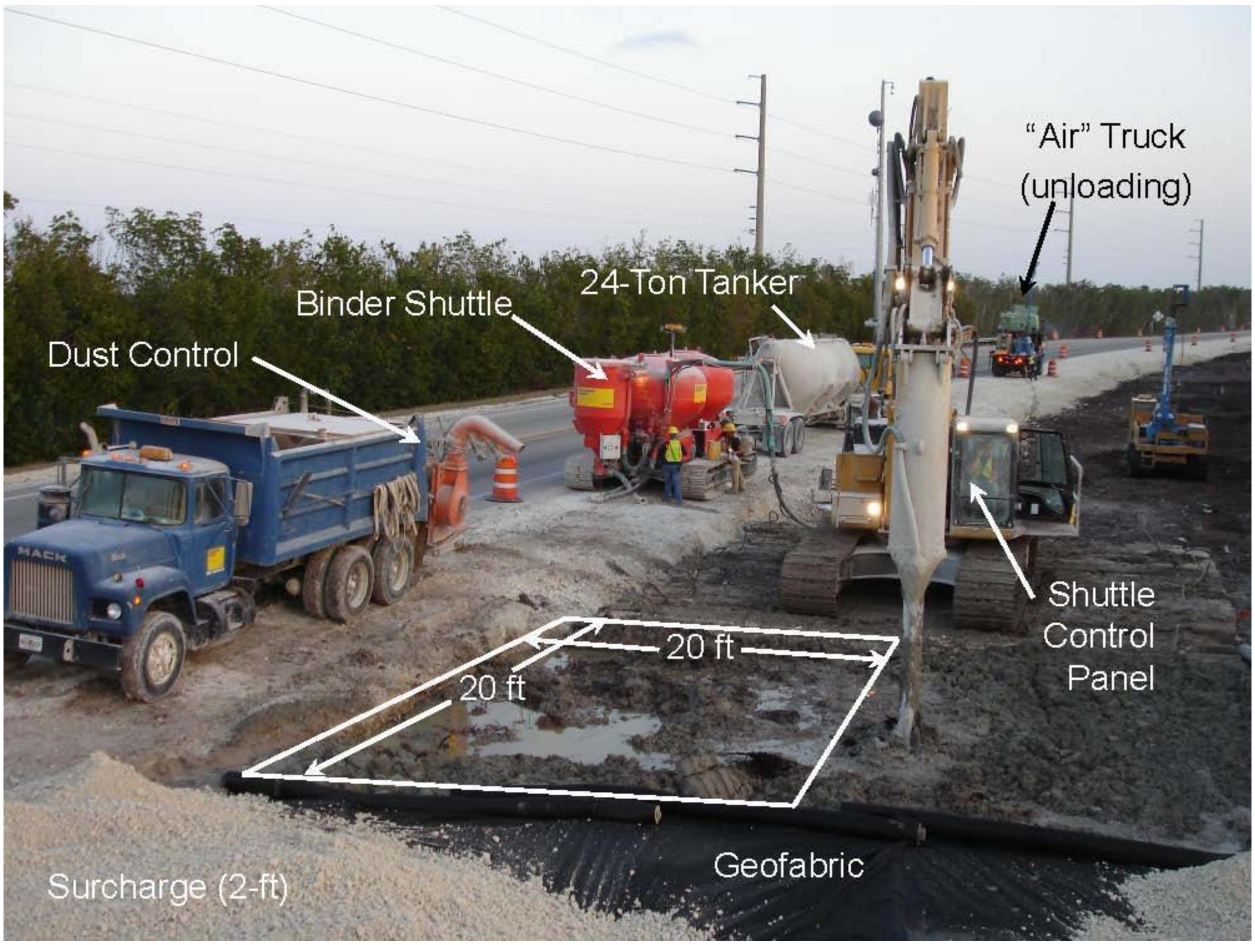

Figure 4.10. Mass stabilization equipment used along US-1 (Garbin and Mann, 2012).

The soil stabilization for this project was completed several years before the onset of this study, so a continuation of the settlement monitoring was initiated. In this vein and 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. These points closely coincided with those previously used by FDOT. Table 4.1 summarizes the historical survey points along the roadway.

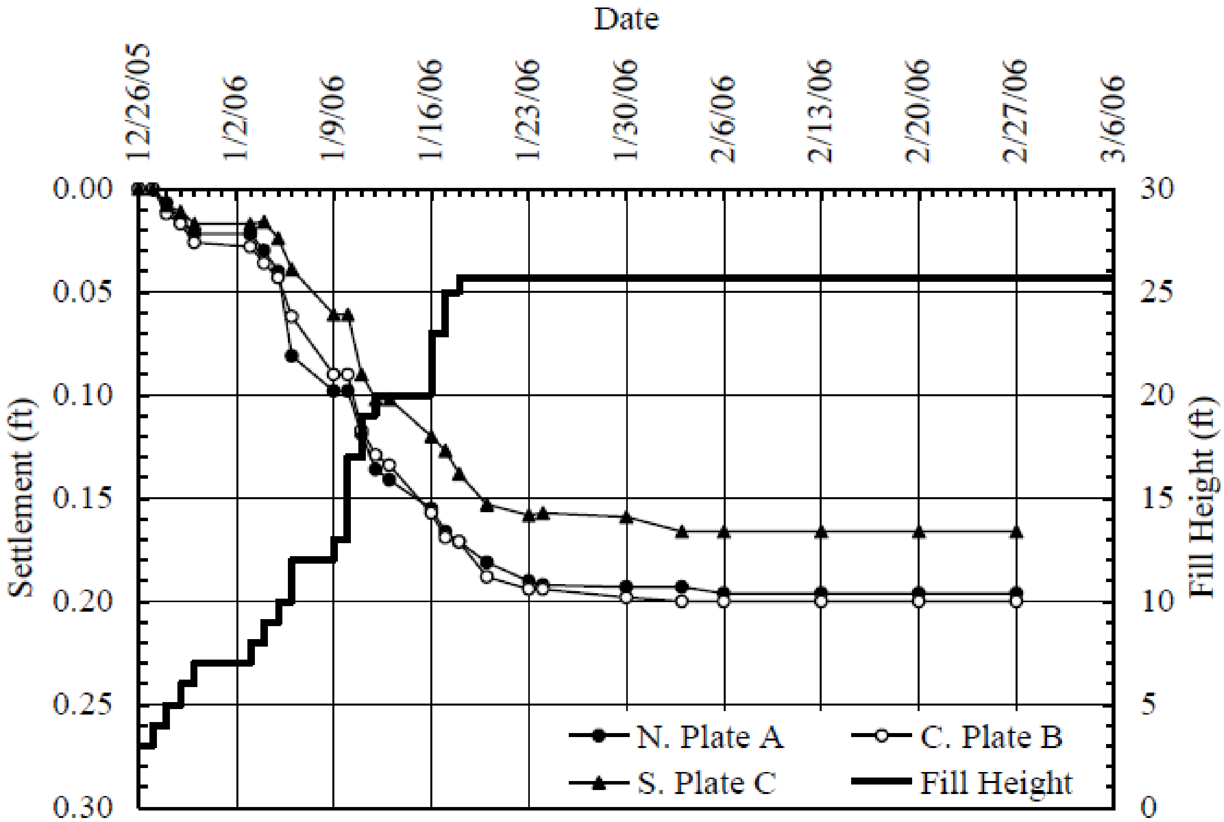

Figure 4.11. Long-term monitoring of mass stabilized soil along US-1 (Garbin and Mann, 2012).

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 4.12. However, one reading near STA 1343-1344 may have been experiencing continued settlement.

Table 4.1. 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.

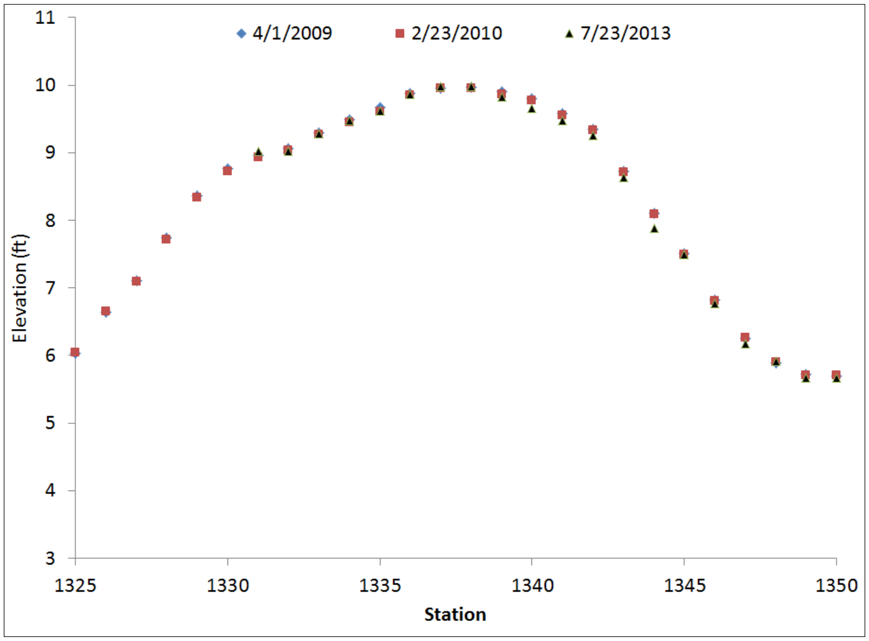

Figure 4.12. Assumed benchmark correction for US-1 at Jewfish Creek (high side of SB roadway).

4.3 Marco Island Executive Airport

Based on the success of DSM at the Jewfish Creek project, the same contractor chose to use it for the Marco Island Taxiway Project. This program stabilized the soil beneath the new taxiway and apron area at the existing Marco Island Executive Airport.

The subsurface profile consisted of 2 to 3 feet of loose sandy fill underlain by layers of highly organic peat extending to depths of 18 feet. Beneath the peat was a loose sandy soil with traces of silt. Due to the proximity to the coast, groundwater depths ranged between 0.5 and 4ft. 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, a 28 day design shear strength of 15psi (2160psf) 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/or 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 3ft) and 275 pcy in areas where the peat layer extended to depths of 18 feet (lower average pH).

The chosen treatment pattern consisted of side-by-side 5ft by 20ft 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 15psi or greater after 2 to 3 days in areas where the peat layer reached depths of 3 to 6 feet.

Given the greater than expected strengths, cement was reduced to 125pcy in all areas. However, it was later found that the KPS rod was bent enough to register deceivingly high resistance readings. Subsequent testing showed 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 target w/c ratio and desired strengths.

Similar to the Jewfish Key post construction monitoring program, 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 4.13 shows the survey points along the taxiway. Tables 4.2 and 4.3 summarize the survey points along the taxiway. Figure 4.14 shows a graph of the top of ground profile for the various surveys performed.

Figure 4.13. Locations of survey points for Marco Island Executive Airport taxiway.

Table 4.2. 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 4.3. 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

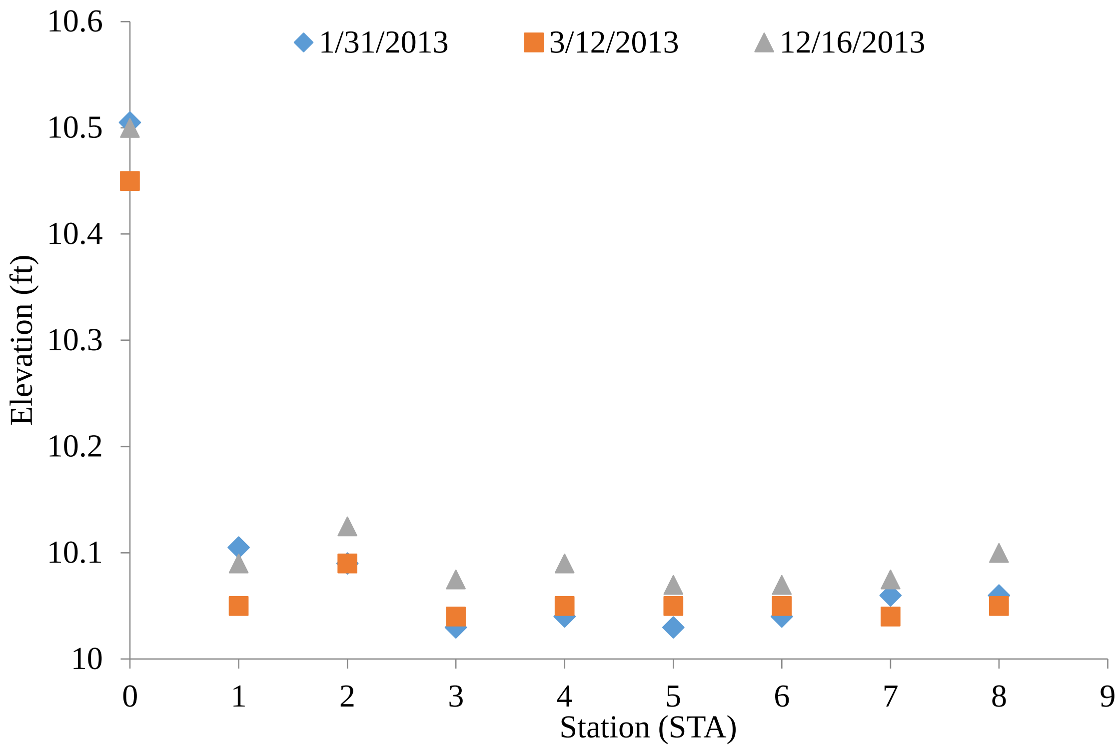

Figure 4.14. Survey data for Marco Island Executive Airport taxiway

In all the survey cases, no appreciable movement was detected over the three-year period of the study. However, the review of the project highlighted the importance of quality assurance and internal review of the test methods.

4.4 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 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. More recently, a widening project of the entire corridor added another parallel bridge and a comprehensive ground stabilization program involving deep and shallow soil mixing was undertaken. Soils supporting the causeway fill varied including sands, silts, clays, and organic deposits. Figure 4.15 shows a concept section view of the soil treatment program which shows a 10ft thick mass treatment shallow transfer platform over deep soil mixed columns to a minimum depth of 45ft.

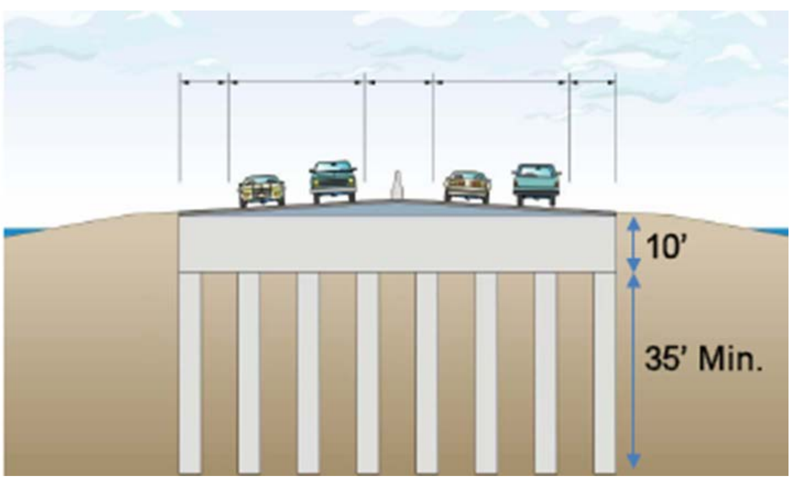

Figure 4.15. Combination of deep and shallow soil mixing used to stabilize causeway.

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 thesis, the project was approximately halfway 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 4.16 and 4.17). Although the magnitude of additional load was minimal in most areas, treatment to a depth of 45ft (elev. -40) encompassed all potentially weak soils.

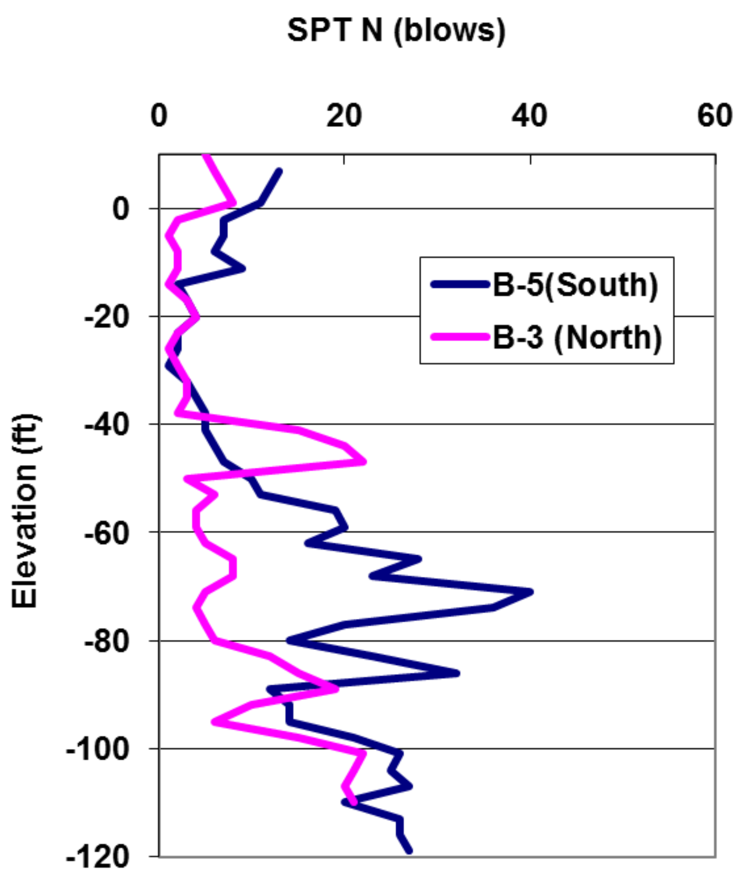

Figure 4.16. Soil strength profile from SPT blow counts at SR 331 soil mixing site.

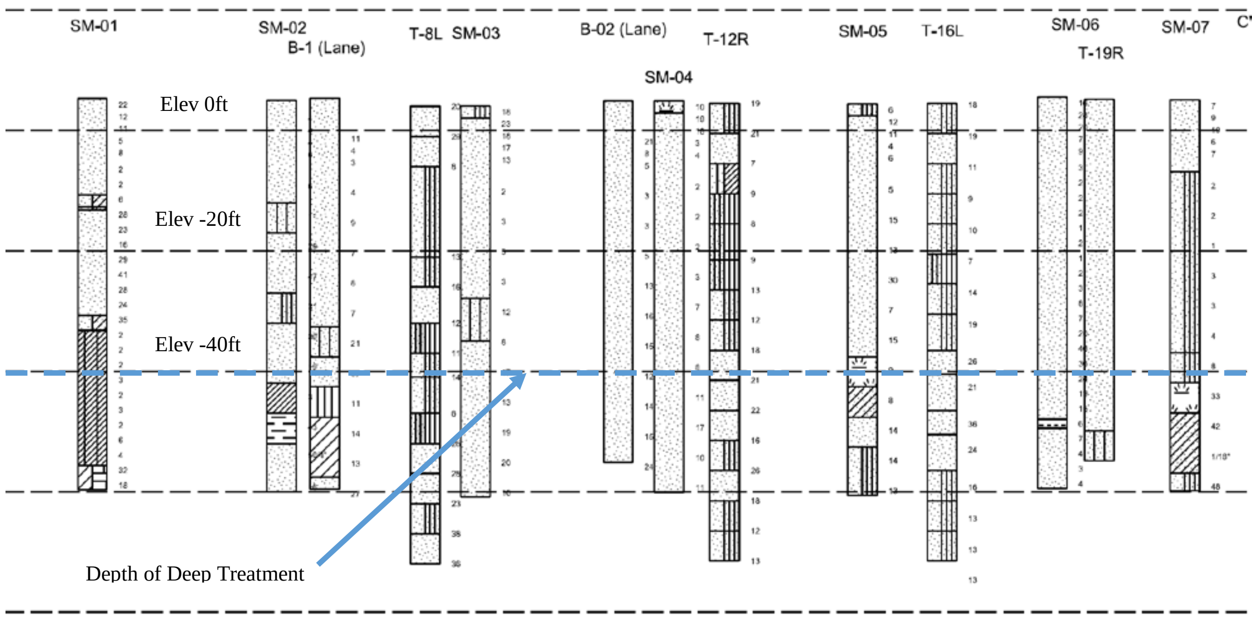

Figure 4.17. Sample of exploratory borings taken along southern portion of causeway.

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 4.4 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 4.4 CF values of 100, 200, and 250, respectively.

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, a 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 as a means of quality assurance. Figure 4.18 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.

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 4.19 to 4.22 show the instrumentation scheme as well as the plan and elevation views of the test section.

Figure 4.18. Soil mixing spoils around twin auger soil mixed demonstration elements.

Table 4.4. Bench scale test matrix for US331 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

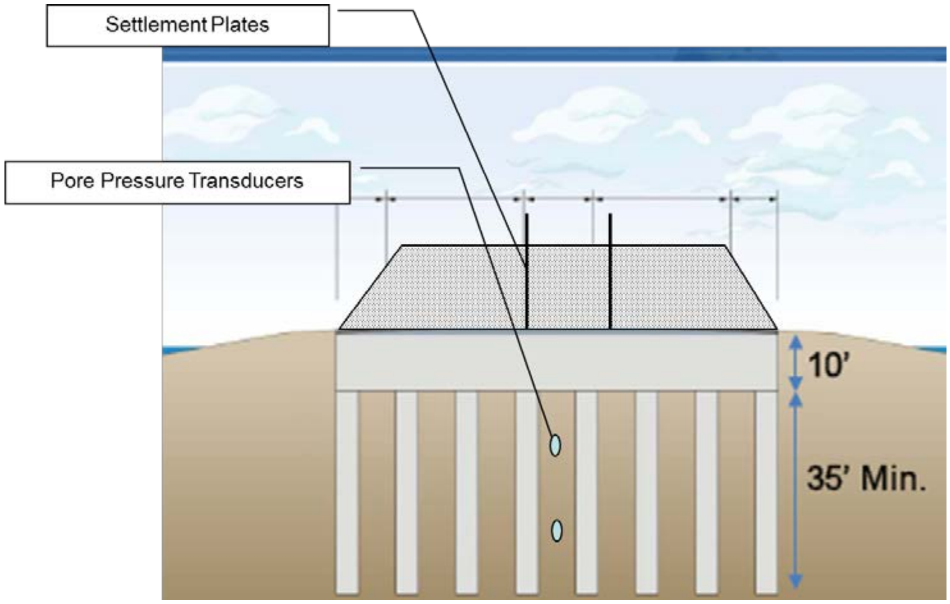

Figure 4.19. Instrumented surcharge program.

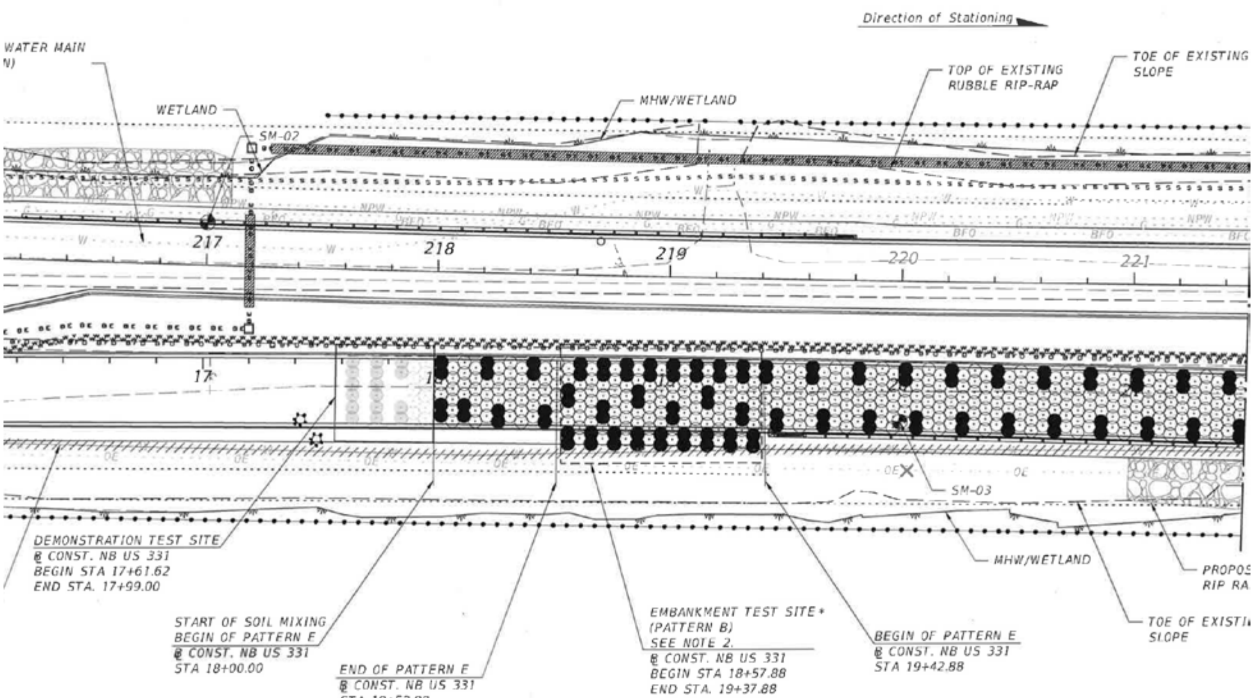

Figure 4.20. Plan view of south causeway treatment layout and test section (FGE, 2014).

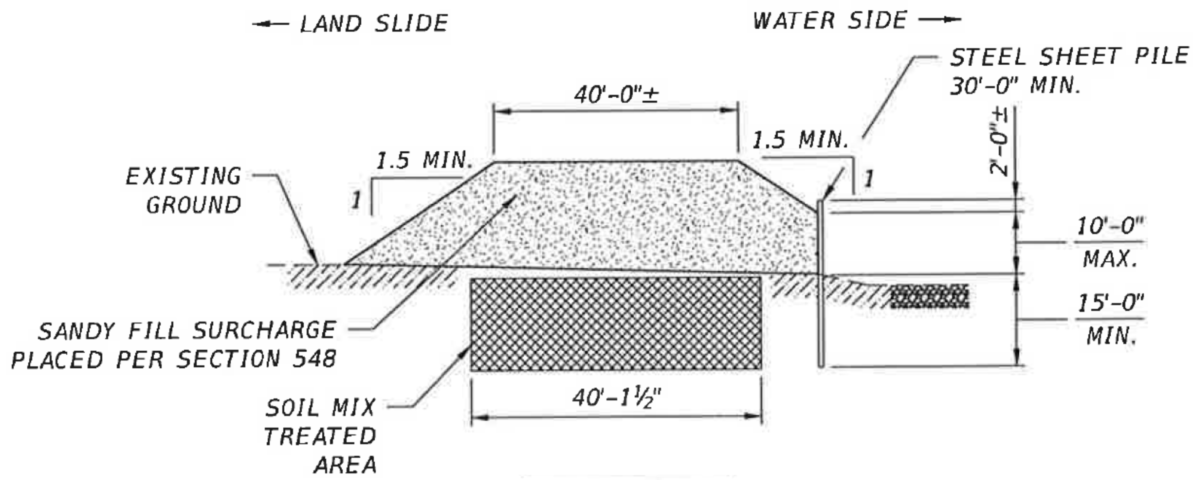

Figure 4.21. Surcharge test section showing sheet pile containment (FGE, 2014).

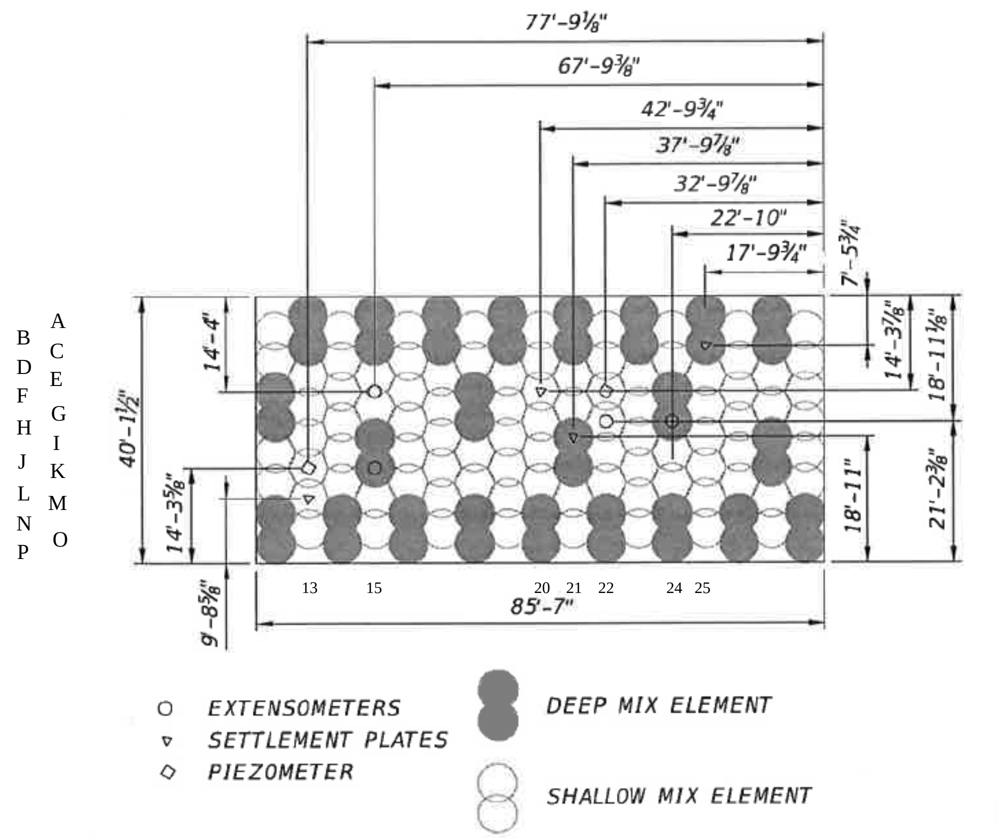

Figure 4.22. Plan view of surcharge test area (FGE, 2014).

As noted in Figure 4.21, 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 4.22). Instrumentation was either installed in shallow element or deep elements. Deep elements shown with dark fill in plan views in Figure 4.22. The instrumentation naming took on the name of the element in which it was installed (Table 4.5).

Table 4.5. 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 4.23 shows the temperature traces for almost a year after soil mixing; sensors were installed after at least 28 days had elapsed.

Due to the cement factor (10-17lbs/cu-ft; 270-459pcy) 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 68°F. 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.

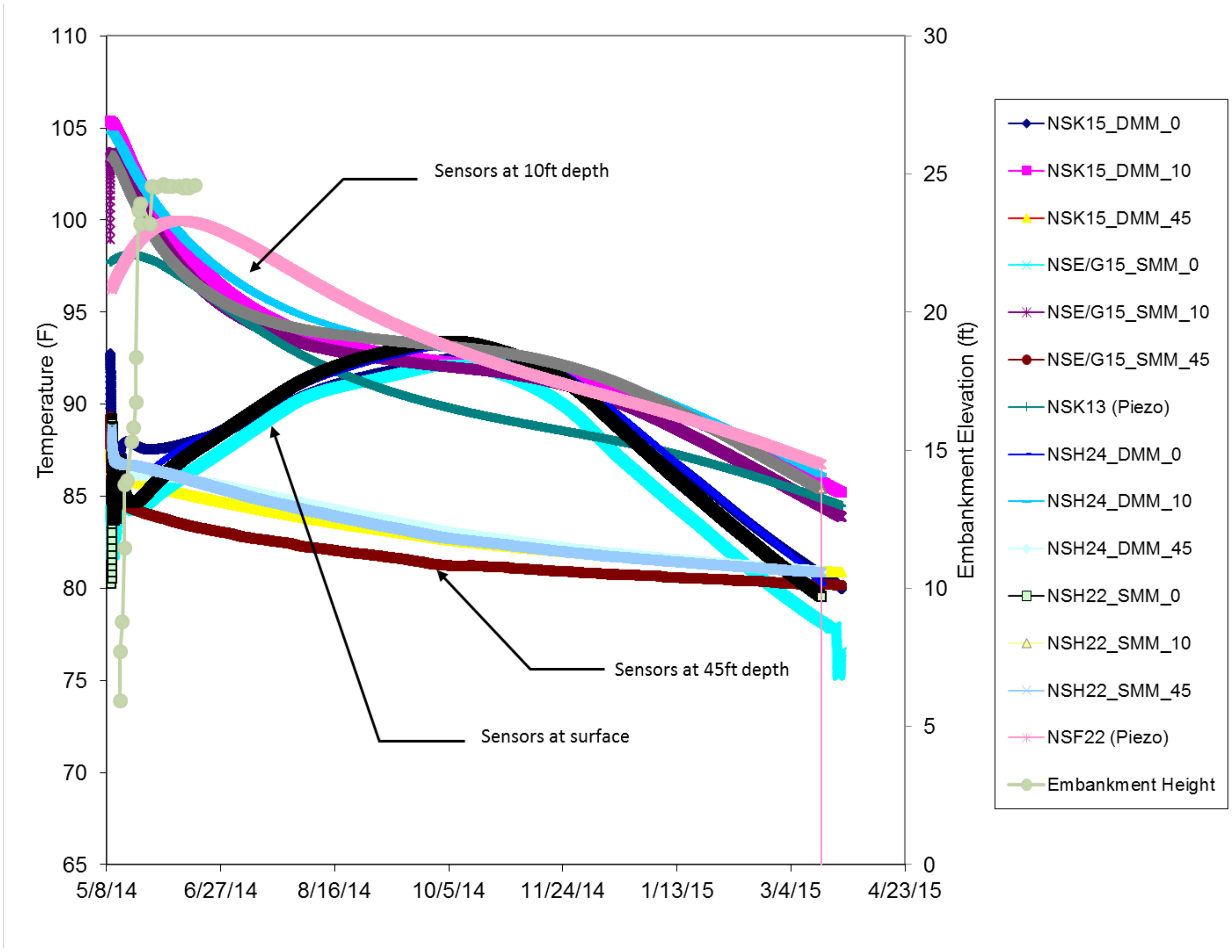

Figure 4.23. Temperature within the soil mix treatment zones (natural soil temperature is 68°F).

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 4.24 shows the fully loaded surcharge and Figure 4.25 shows each of the individual measurements along with the combined overall compression summing each of the three sensors. The surcharge was left in place for 11 months.

Figure 4.24. Surcharge / embankment load fully in place on test section (approx. 19ft).

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 15°F 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 settlement induced movement, thermally induced compression or merely calculated movement from temperature corrections. As expected with soil mixing programs (also shown in the Jewfish Creek load test program, Figure 4.11), settlement was immediate and directly the result of increased surcharge height. Appreciable, long-term settlement was not observed.

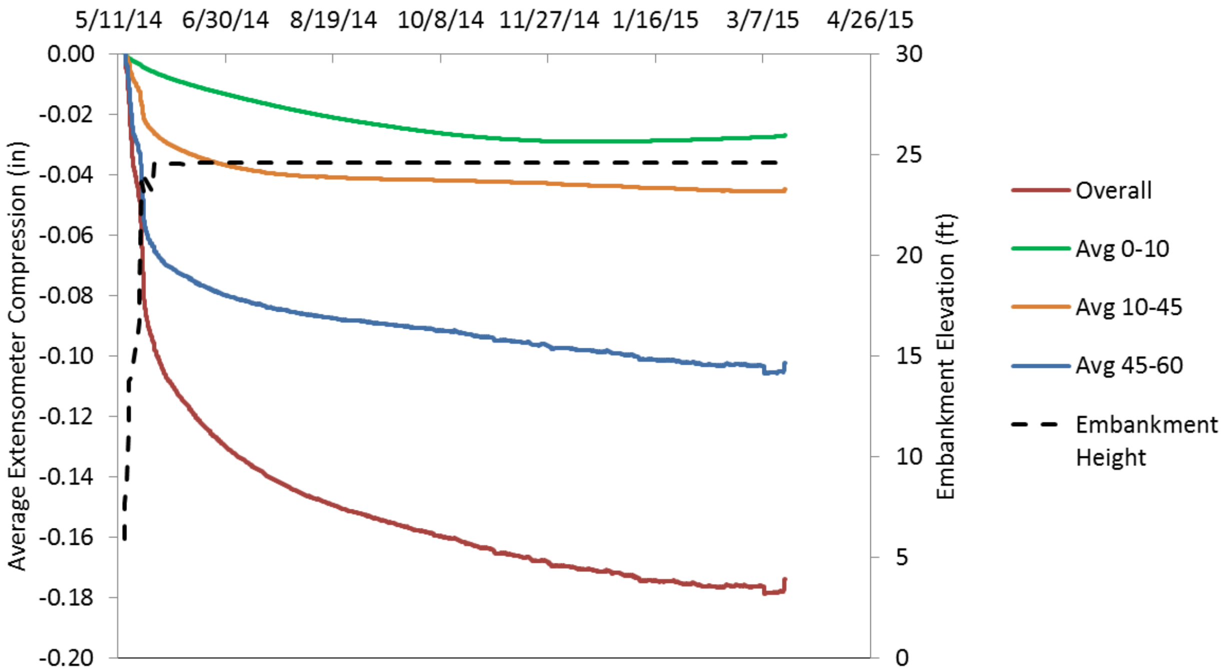

Figure 4.25. Settlement measured from 19ft surcharge loading.

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 4.26 shows the results of the plate load tests.

The on board computer systems of the soil mixing systems used at the US331 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 unconfined compression tests. For this project, quality control and assurance protocols required a minimum amount of cement (CF > threshold), a minimum amount of mixing energy denoted by number of blade rotations (BRN > threshold), a minimum compression strength of cored and wet grab specimens, and a minimum frequency of sample testing to not fall below 2% after the first 200 elements were installed (4% early-on up to 200 elements). Figure 4.27 shows an example field log demonstrating the installation monitoring system used by the contractor.

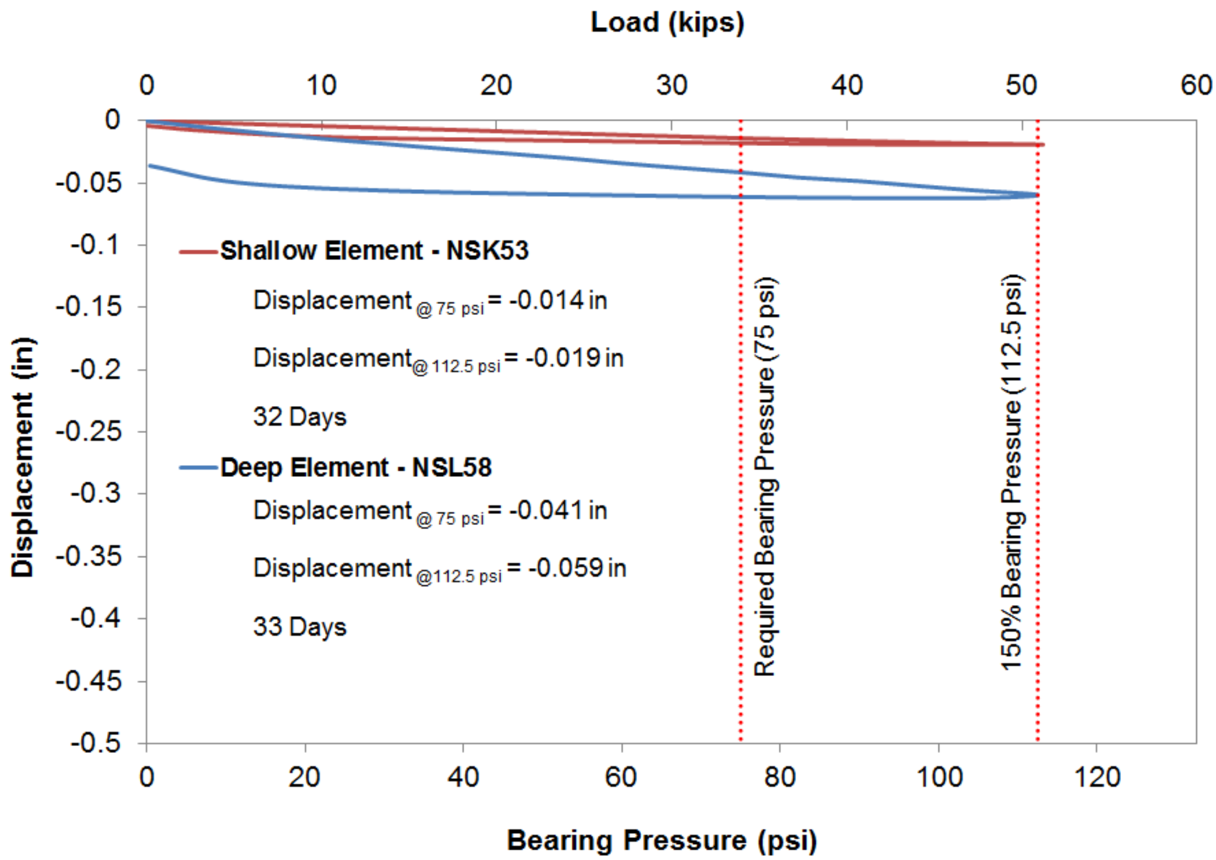

Figure 4.26. Plate load test results (FGE, 2014).

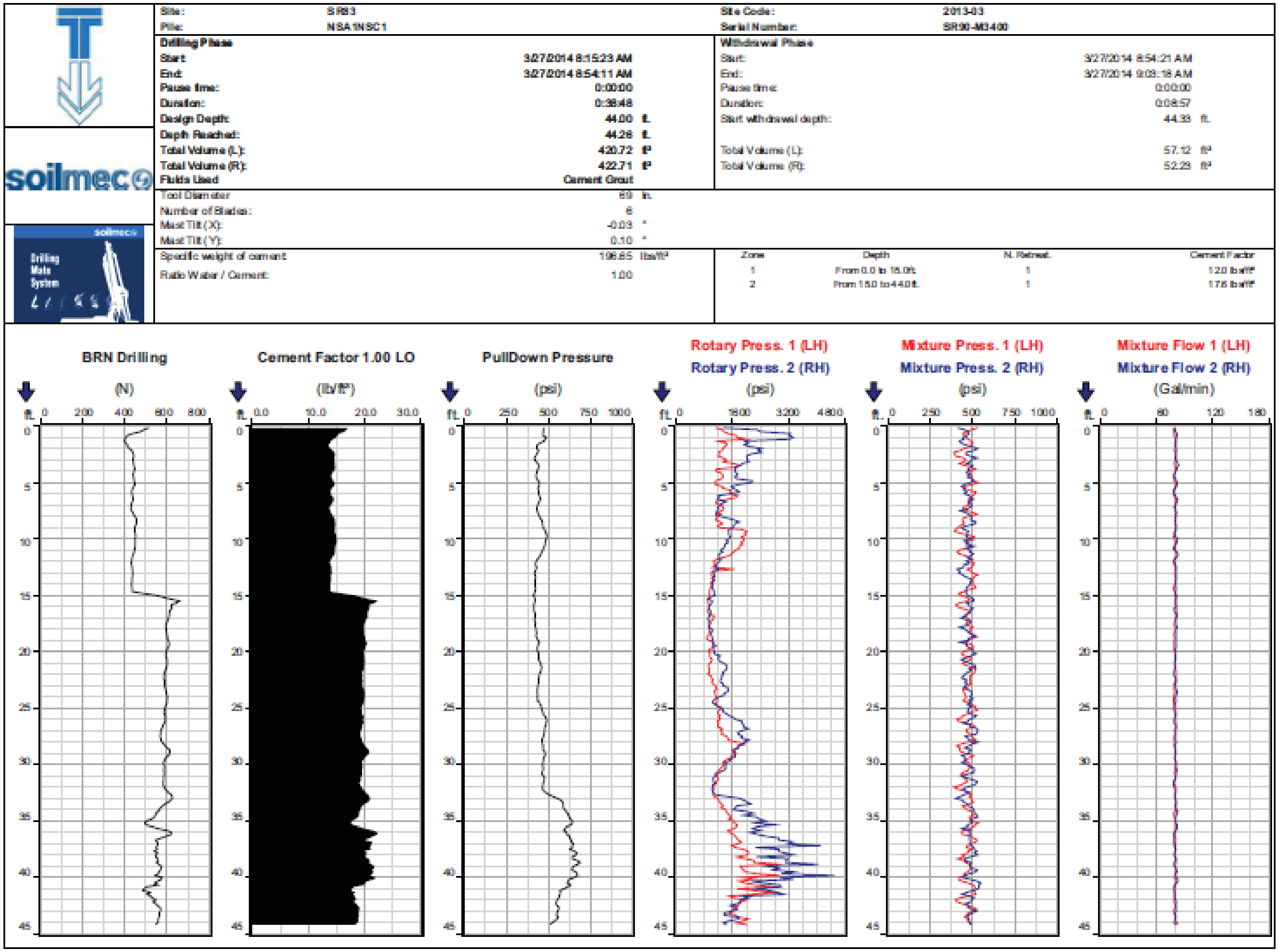

Figure 4.27. Automated measurements taken by on-board quality control system (FGE, 2014).

4.5 LPV111[6]

A part of the New Orleans East Back Levee, the LPV111 project entailed rebuilding the 5.28 mile (8.5km) long levee which was greatly damaged from Hurricane Katrina in 2005; the foundation of which consists mainly of soft organic clay. To remediate these soil conditions, a deep mixing method (denoted as DMM) was chosen due to project constraints involving time, environment, and real estate. Laboratory testing and field testing were conducted prior to the onset of construction to ensure the correct amount of binder was used to reach the desired strength.

This region in southeast Louisiana generally has soil properties described as marine deposits with interbedded and inter-fingered clays, silts, sands, and usually organic layers. This is because of how the area was formed by the Mississippi River deltaic process. The soils found under the LPV111 levee were fill, soft clay, marsh/peat deposits, fat clay, and Pleistocene (Table 4.6). This combination of soils presented two challenges: widely varied geotechnical properties and poor mixing conditions.

Laboratory testing was performed on selected soil samples for the following properties: water content, Atterberg limits, particle size, unit weight, organic content, specific gravity, pH, sodium content and sulphate content. Water content, plasticity, clay content, organic content, and pH are plotted for the three different sections 11B, 12A, and 12B in Figures 4.28 to 4.30.

From these soil properties, the moisture content versus organic content was plotted to show the correlation. All three sections showed a similar curve, only slightly variant from each other (Figure 4.31). In general, high moisture content (above 100) immediately implies organic soil.

[6]Section 4.5 references “Soil mixing in highly organic materials: the experience of LPV111, New Orleans, Louisiana (USA).

Table 4.6. LPV 111 soil profile (Leoni, 2012).

Average Elevation Range (ft)	Soil Type	Soil Description
+19.7 to +1.6	Levee Fill	Stiff to hard Fat Clay (CH), Sandy Clay (CL) and very loose to loose Sand Silt (ML), Silty Sand (SM) with very occasional organics and wood
+8.2 to -13.1	Soft Clay	Very soft to hard Fat Clay (CH) and Lean Clay (CL) with occasional sand and silt layers, sand pockets, peat pockets, wood and organics
-3.3 to -18.0	Peat	Organic Clay (OH) to Peat (P) with roots, fibers and occasional wood
-3.3 to -52.5	Fat Clay	Very soft to stiff Fat Clay (CH) with sand seams and occasional sand and silt layers
-46.0 down	Pleistocene	Firm to hard Fat Clay (CH), Lean Clay (CL), and loose to very dense fine sand (SP)

The bench scale testing phase was completed to mainly examine the three variables of binder type, binder amount, and water-to-binder ratio and their effects on compressive strength. To do this, testing was done in four phases. The first three phases used soils from the three sections shown in Figures 4.28 to 4.30, it progressed from west (12B) to east (12A), then slightly north (11B) (Figure 4.32). The fourth phase was smaller and done purely for the investigation of the organic layers.

For phase one, seven batches were prepared testing varying binder contents of a 75% slag 25% cement combination (denoted as 75/25 PBFC), and varying 100% ordinary Portland cement

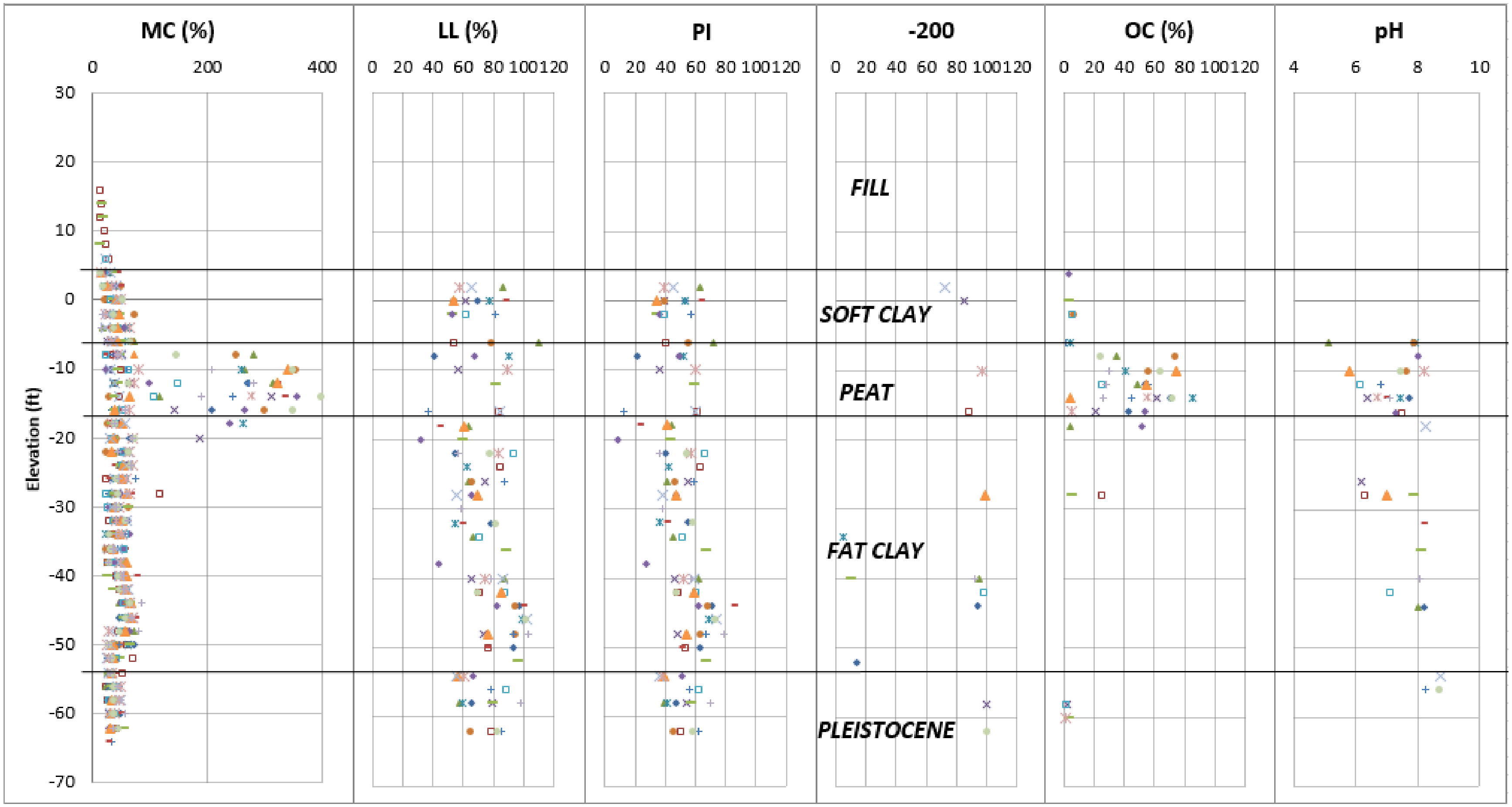

Figure 4.28. Soil properties for section 12B.

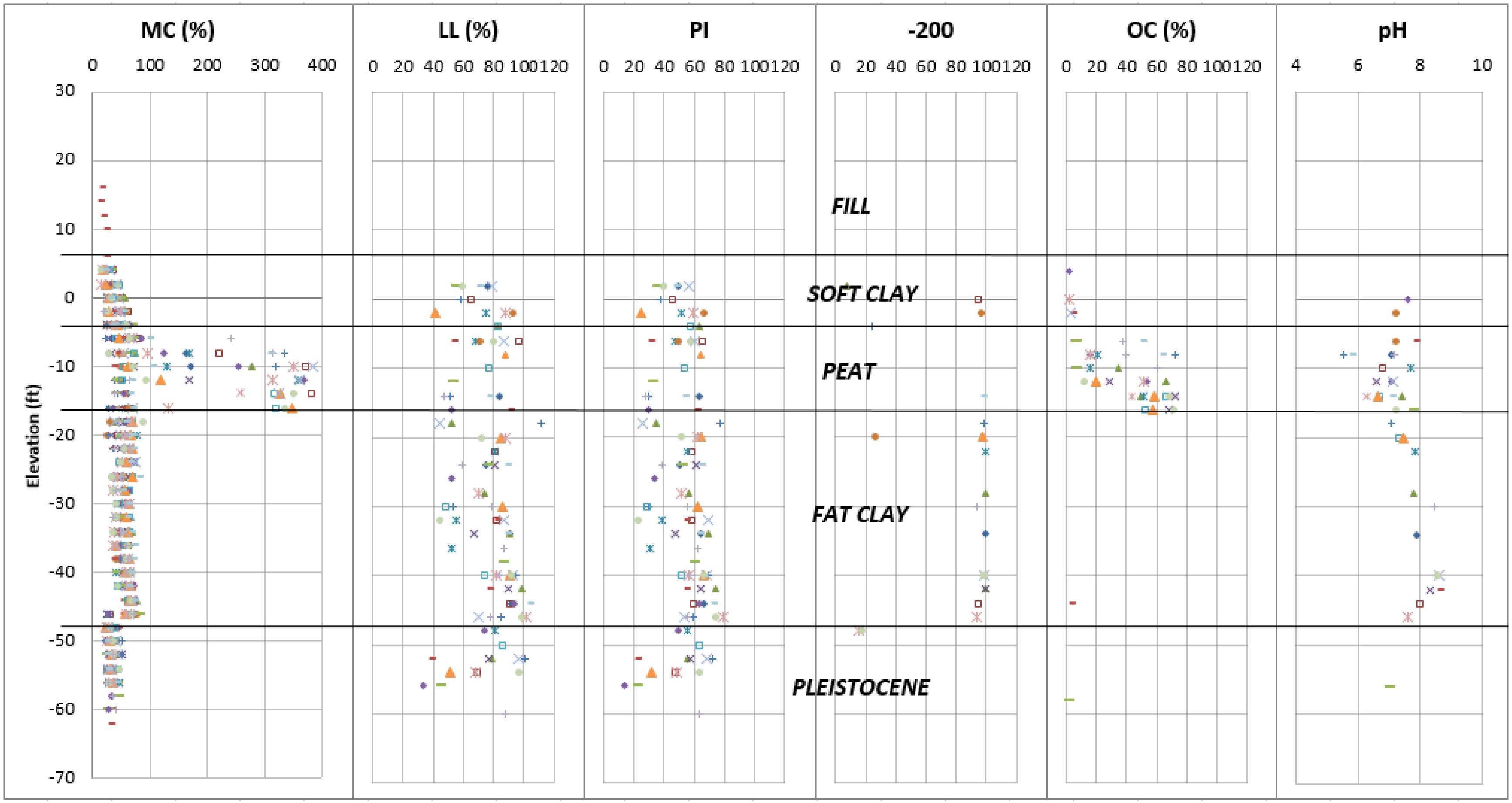

Figure 4.29. Soil properties for section 12A.

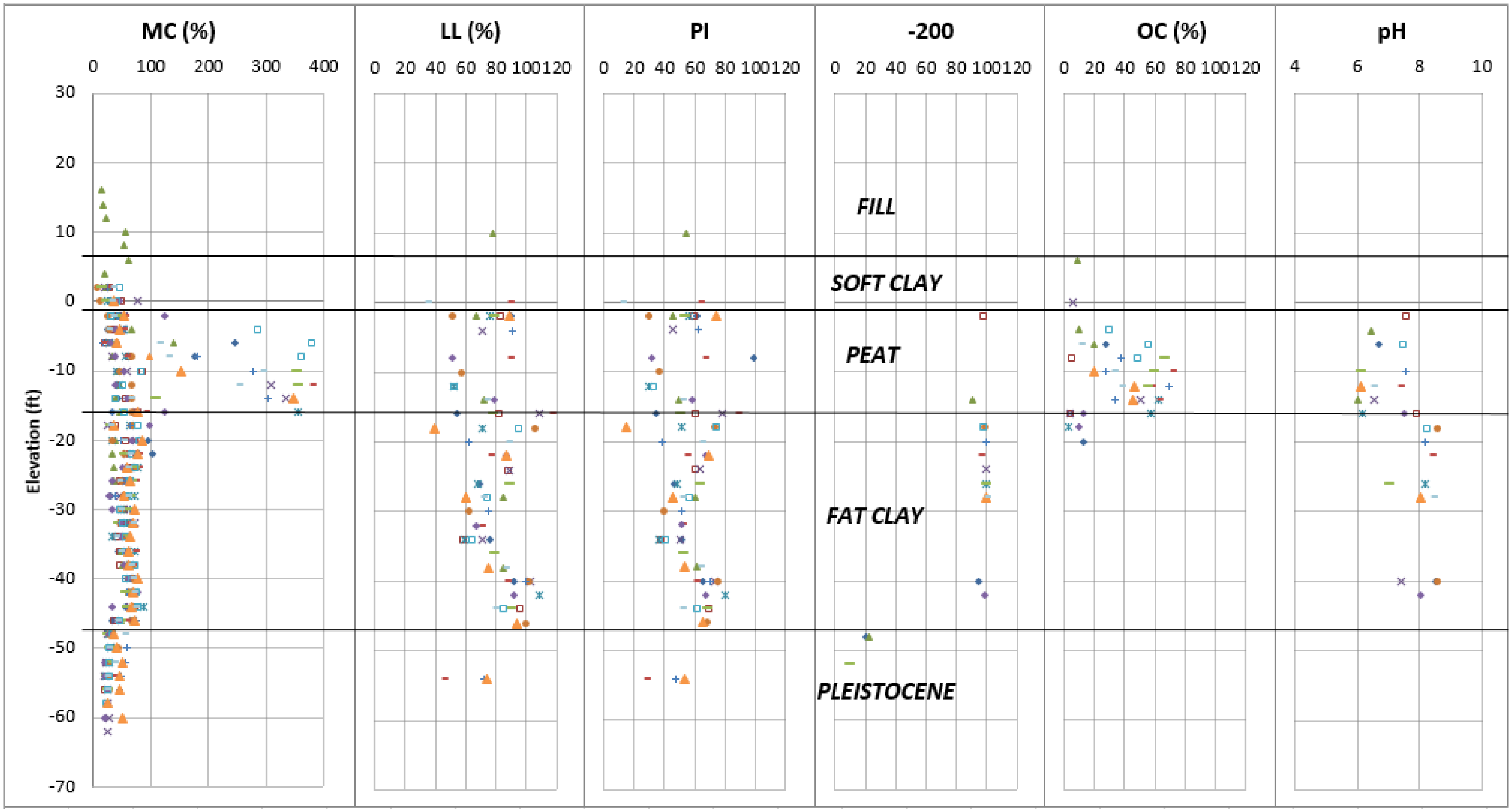

Figure 4.30. Soil properties for section 11B.

contents (OPC). The soil used was taken from the peat layer of 12B and had an average moisture content of 287% before mixing. From this phase several conclusions were drawn: peat is easy to mix, adding water prior to the binder has negative effects on compressive strength, adding slag helped strength gain notably versus pure OPC which did not show the same positive results. It should also be noted that using a minimum binder factor of 590pcy (350kg/m^3) of PBFC and not adding water reached the target strength in the peat.

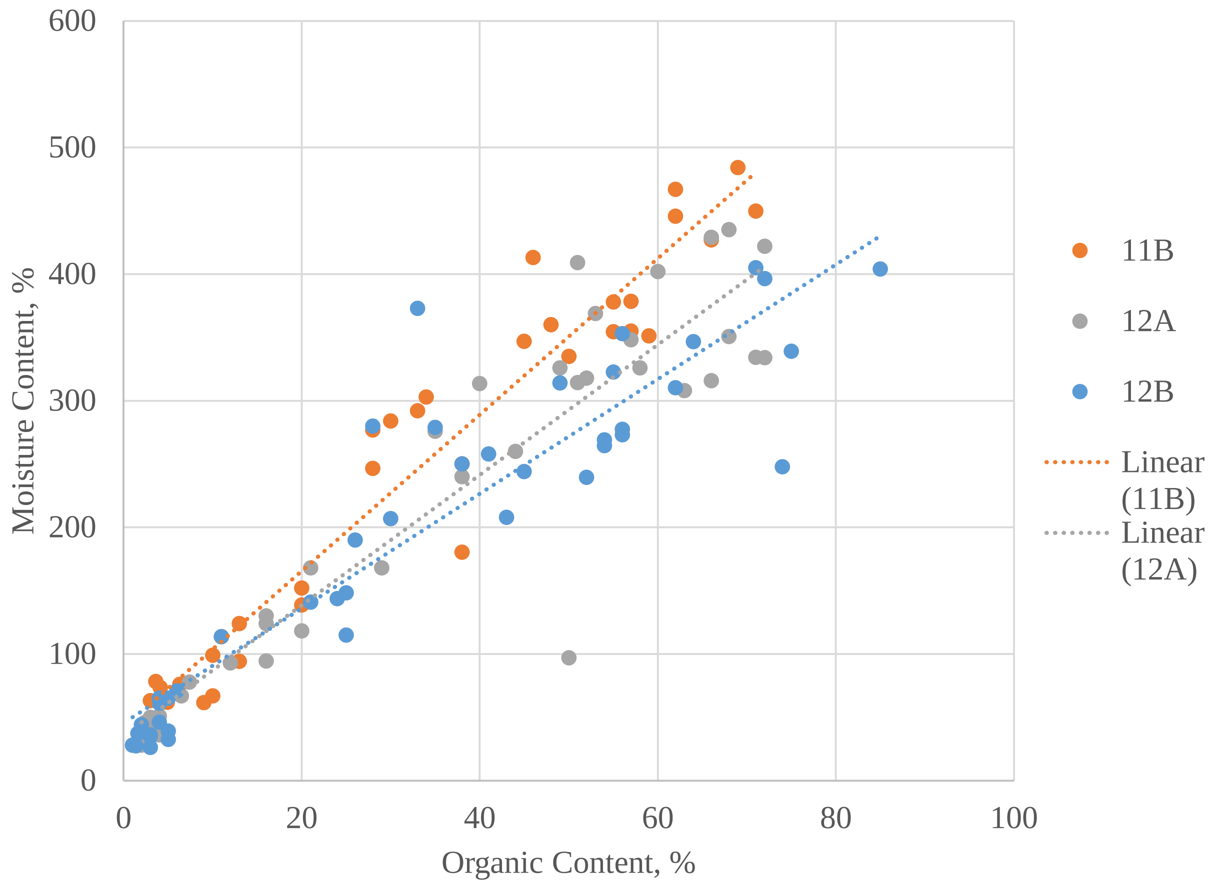

Figure 4.31. Organic content versus moisture content for 11B, 12A, and 12B.

For phase two, four batches were prepared using varying amounts of PBFC. The soil used came from the peat layer of 12A with an average moisture content of 272% before mixing. This phase aimed to improve the thoroughness of the mixes (e.g. a better mixing tool). The conclusions made from this phase were: there was less data scatter seen in phase two than in phase one, the intended strength was met by all mixes, adding fluid to the mixture with the same total water to cement ratio still showed negative effects, and once the binder content reached 590pcy (350kg/m^3) increasing the binder content had little effect on the strength.

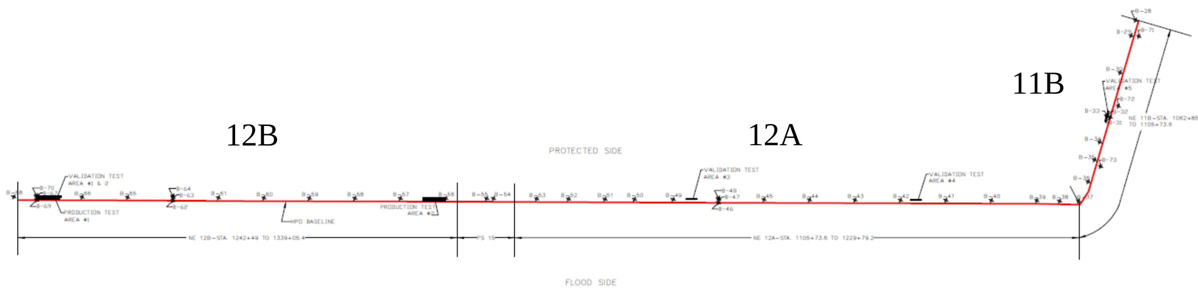

Figure 4.32. LPV 111 plan view, north is pointing up.

Phase three consisted of four batches as well utilizing a high binder content of the PBFC blend. This was because the project schedule required construction to start and thus the application of these conservation parameters was needed. Soil for phase three testing was taken from the peat layer of 11B. The peat for this section had an average moisture content of 320%. Using this amount and blend data scatter was largely reduced. The greatest data consistency was provided by the highest binder factor of the PBFC blend used. The peat was still fairly easy to mix and the target strength was reached from all batches and binder factors.

Phase four was primarily about proportions of slag and cement to see which would be most effective in organic soils. These batches were done with the primary concern of the cost of slag and also its availability. The goal was to find out how much the amount of slag could be reduced if needed. Thus, three batches were created again using soil from the peat layer of 11B. The results showed that a higher PBFC blend was the most efficient in relation to a 50/50 or the even less effective 25/75 blend (25% slag and 75% cement). In this phase the high blend was the only one that attained the needed strength.

Once the bench scale testing was completed, validation tests were conducted at field-scale. This was done to confirm the binder content needed and/or refine it if possible, to determine the best mixing setup, and to determine quality control for DMM production.

Having completed both bench scale tests and validation tests, the data was compared. The target strength for the lab was twice the target strength for the field because it can be difficult to duplicate the mixing efficiency of the lab in the field, however it was found that the validation (field) tests strengths were up to 15% higher than the bench scale tests. The validation tests were able to confirm that the peat is easy to mix, as observed by all phases of the bench scale tests, and that a large amount of binder was still necessary to meet the required criteria.

The bench scale testing program was able to form several conclusions. The first conclusion being that when a binder with a high amount of slag was used it out performed all other binder ratios tested in regards to treating the peat/marsh organic clay deposits. Thus, because slag is being used, which reaches its full strength at 56 days, there was about a 1.25 strength increase from 28 to 56 days. While a high amount of binder was needed, it was determined that increasing binder content only helped strength gain to a point, after which the benefit was diminishing. The fourth conclusion is one that was found in several studies discussed in the literature review in Chapter 2 where adding water before mixing yields adverse effects. When compared to similar batches it is clear that this actually hinders the potential strength. Figure 4.33 shows the data from the bench scale testing. It can be seen here that this soil follows the threshold concept using the 0psi method (Baker, 2015).

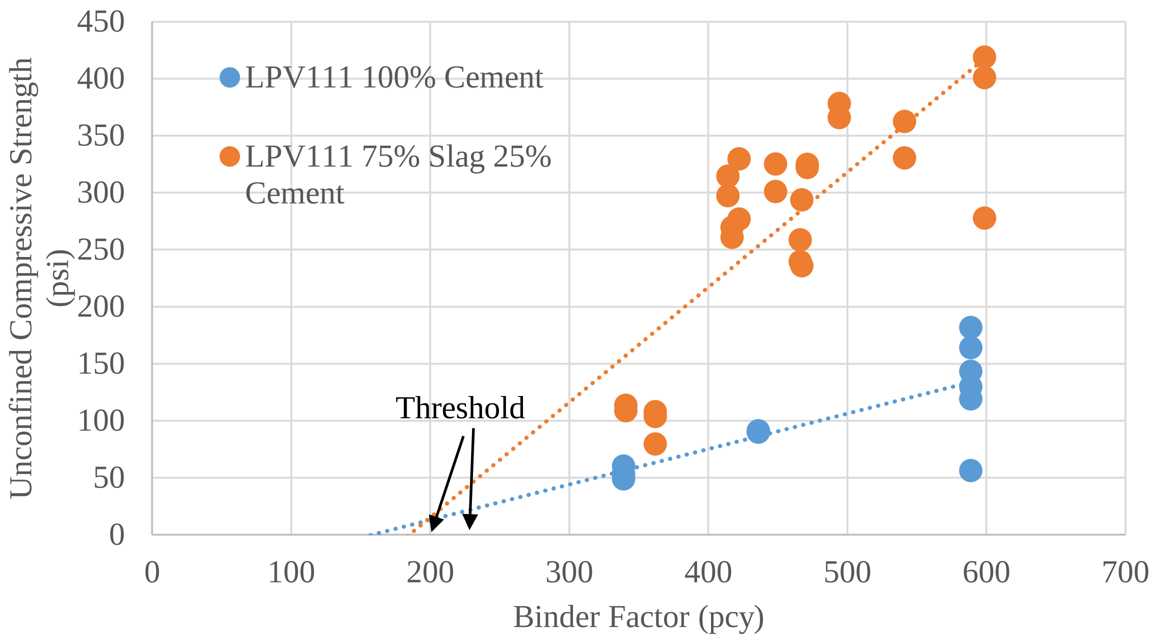

Figure 4.33. Unconfined compressive strength versus binder factor for LPV 111 bench scale testing.

The validation testing showed some positive conclusions as well. As noted earlier the lab strengths required were two times the needed strength in the field, simply because during field mixing it can be difficult to attain the accuracy of a laboratory. However, it was concluded in this study that using well designed mixing procedures, parameters, and equipment configuration can greatly reduce the gap and possibly overturn it (Leoni, 2012).

4.6 Additional Case Studies

While the specific locations of sites and project titles are left out, Hayward Baker, Inc. generously provided laboratory data from a few different field cases of soil mixing.

4.6.1 Case 1

The first case study had a soil profile as described in Table 4.7. The soil layers were mixed together. The average organic content of this soil was 9.7% and the average moisture content was 72.9%. The dry unit weight of the soil was 67.8pcf.

There were four different batches made to test the binder needed to stabilize this soil. Binder factors of 200 pcy, 250 pcy, and 300pcy of 100% cement were tested along with one trial of 250pcy 50/50 slag-cement blend. The water to cement ratios of the mixes ranged from 3.76 to 5.79. The results of these batches are plotted below in Figures 4.34 and 4.35 as a function of early onset strength gain. Each data point represents two specimens. In Figure 4.35 an arrow has been drawn to show that this data follows the 0psi threshold method (Baker, 2015).

Table 4.7. Soil description for field case one.

Depth (ft)	Soil Description
3	Dark grey silt with peat and pockets of grey fine sand, with a trace of weathered limestone
5	Peat with large nodules of light brown silt mixed with root mat
7	A third light brown silt mixed with a trace of roots, a third grey fine sand with silt/peat nodules, and a third light brown silt with peat nodules

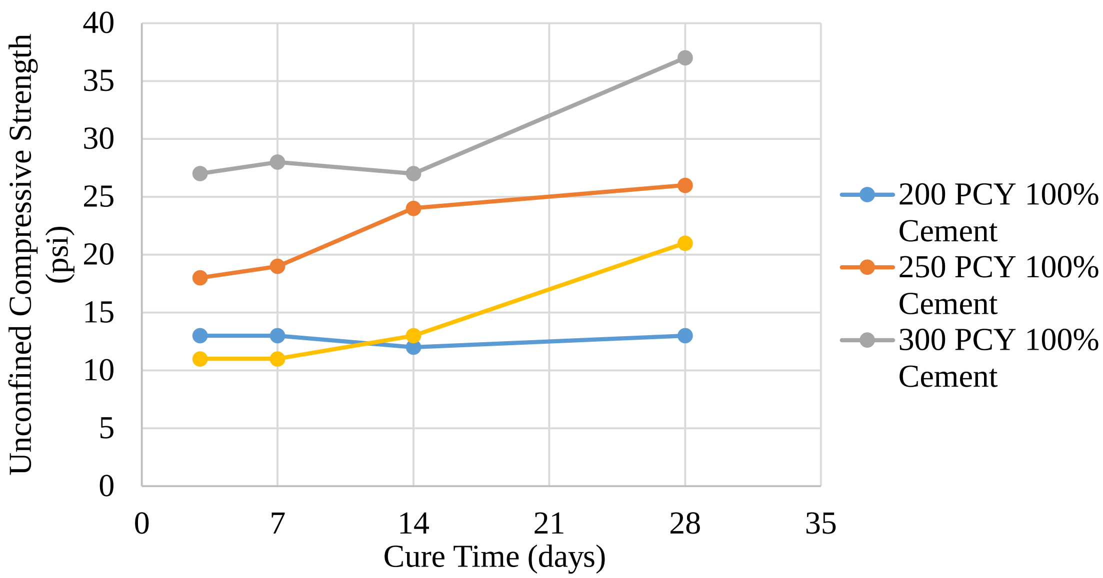

Figure 4.34. Unconfined compressive strength versus cure time comparing all four batches.

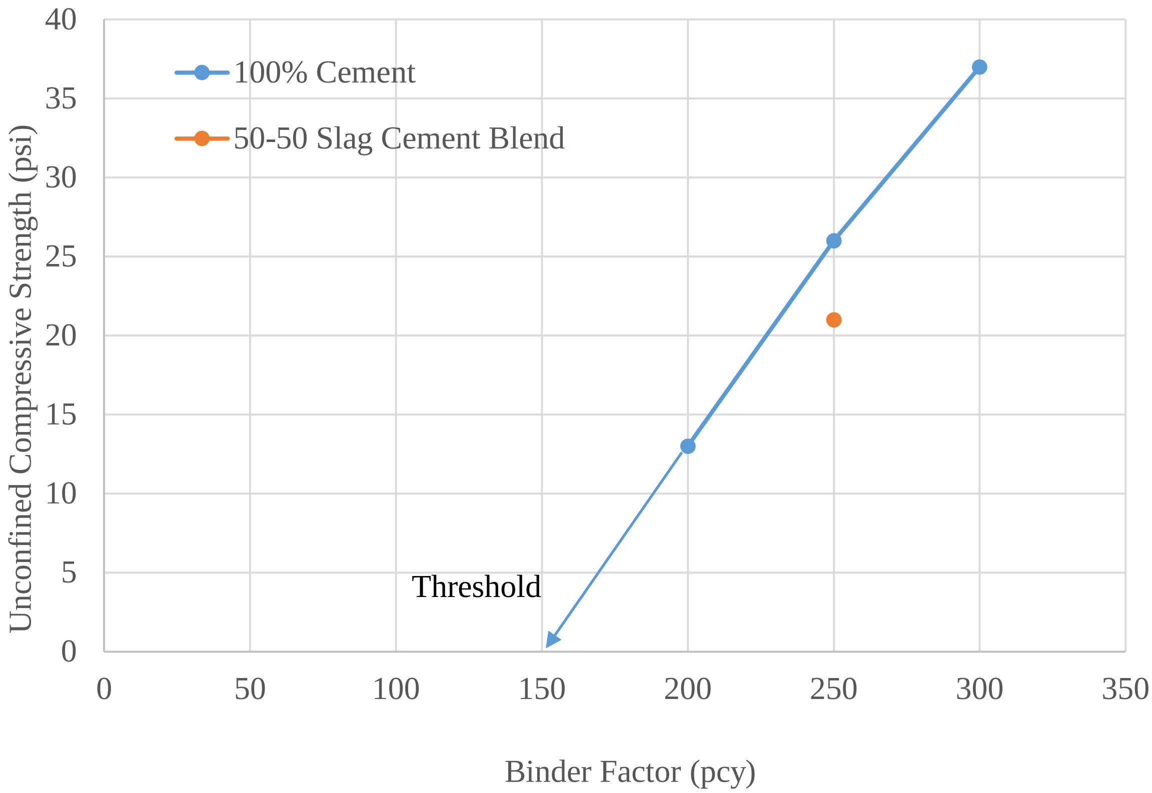

Figure 4.35. Unconfined compressive strength versus binder factor.

4.6.2 Case 2

For the second field study provided, there were different areas tested. The soil description for the varying areas are presented below in Table 4.8. Each section was tested using binder contents of 300 pcy, 375 pcy, and 450 pcy of 100% cement. The soil was again mixed into one sample per section for testing.

For PIT 1, the average organic content was 20.53% and the average moisture content was 74.7%. PIT 3 had an average organic content of 3.18% and average moisture content of 26.9%. PIT 4 had an average organic content of 8.75% and moisture content of 41.9%.

Table 4.8. Soil description of the second field study.

Boring Number	Bucket Number	Depth (ft)	Soil Description
PIT 1	1	0-7	Dark brown silty fine sand with gravel and silt lumps
PIT 1	2	0-7	Dark brown silty fine sand and white fine sand with gravel and silt lumps
PIT 1	3	7-13	Peat with root mat
PIT 1	4	7-13	Peat with root mat
PIT 3	5	0-4	Brown fine sand with crushed shell and gravel
PIT 3	6	0-4	Brown fine sand with crushed shell and gravel
PIT 3	7	4-8	Brown fine sand with crushed shell, gravel, silt and peat with trace of roots
PIT 3	8	4-8	Brown fine sand with crushed shell, gravel, silt and peat with trace of roots
PIT 4	9	0-6	Dark brown silty fine sand with gravel
PIT 4	10	0-6	Dark brown silty fine sand with gravel
PIT 4	11	6-12	Grey/light brown silt with a trace of roots and peat
PIT 4	12	6-12	Grey/light brown silt with a trace of roots and peat

Figure 4.36 plots cement factor versus unconfined compressive strength for all batches. The trends seen correlate quite well with the threshold concept for organic soils (0psi method) that has been previously mentioned in Chapter 2. The blue arrows are pointing to each PIT's threshold value using this method.

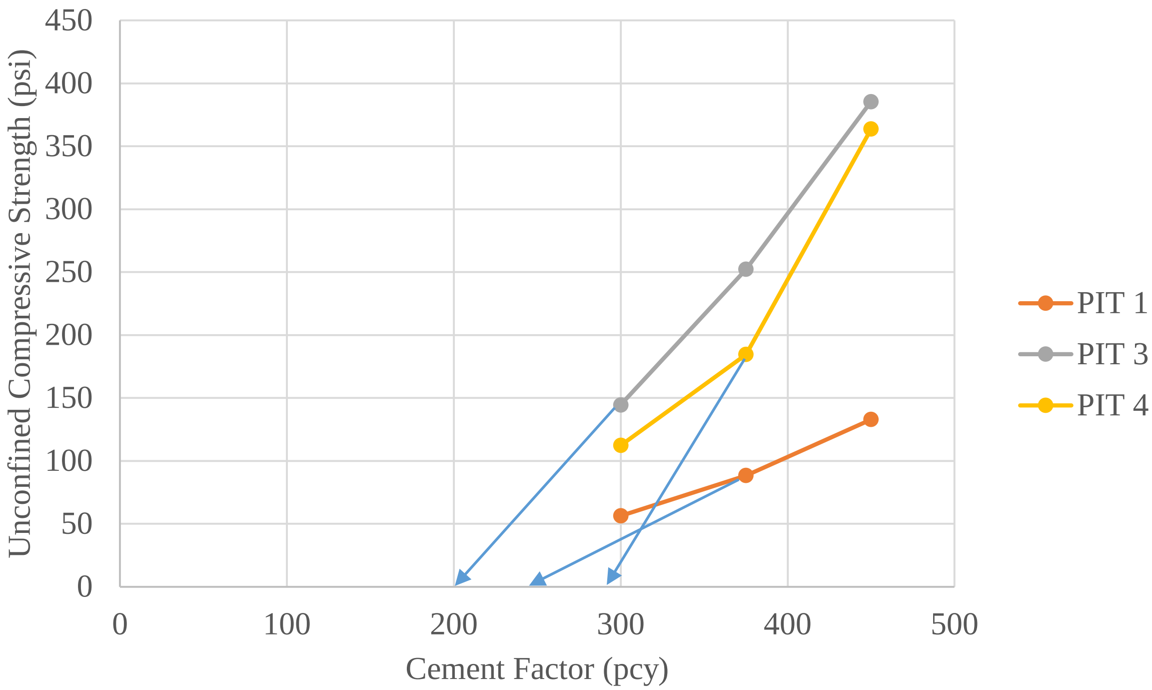

Figure 4.36. Cement factor versus unconfined compressive strength for the three different sections.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

Organic soils encountered in proposed roadway alignments (or under any shallow foundation) are problematic due to the compressible characteristics and if left untreated results in persistent settlement / distress in the pavement. Historically, organic soils have been completely removed and replaced to avoid problems associated with insitu treatment methods. Conversely, inorganic soft soils have been successfully treated in-place by mixing these soils with some form of chemical binder such as cement, slag, or a combination thereof. Application of soil mixing to organic soils is the focus of this thesis.

While mixing stabilization of inorganic soils has evolved both in practice and design, there is no clear guide on what to do if organics are encountered at a job site. Rather, just warnings and hints that a greater amount of binder should be used. As discussed in Chapter 2, the case studies that formed the FHWA design curve for deep soil mixing are mainly based on inorganic soils. These studies contain mostly soft marine clays, with only one organic soil study which had a low organic content (<15%).

The Swedish Deep Stabilization Research Centre made the observation that a binder threshold exists when organic soils are treated, meaning that organic soil needs a minimum amount of binder before strength gain can be realized. The University of South Florida also found this to be true in a laboratory study performed as part of an FDOT funded project. Using 100% cement stabilized specimen data, a proposed threshold curve was introduced. This concept was applied in this thesis to field studies where soil mixing was applied to organic soil deposits.

5.1 Large-Scale Outdoor Laboratory Soil Mixing

While is it possible to achieve positive strength increases in the laboratory, this is not always the case in the field. Shortfalls in strength on the order of 2 to 5 times under laboratory findings have been commonly experienced. Thus, to test whether the laboratory results could be met in the field, a 1/10th scale test bed containing organic soil was fabricated, soil treated and the resulting strength tested. It was partitioned into three sections where both wet and dry soil mixing methods were used and compared to an untreated control section. Each section was loaded to 600psf, the equivalent of a 5ft roadway embankment. The results seen from this test were approximately 1.5in, 2.4in, and 4.25in of displacement for the dry mixed, wet column supported, and control sections, respectively (Figure 5.1).

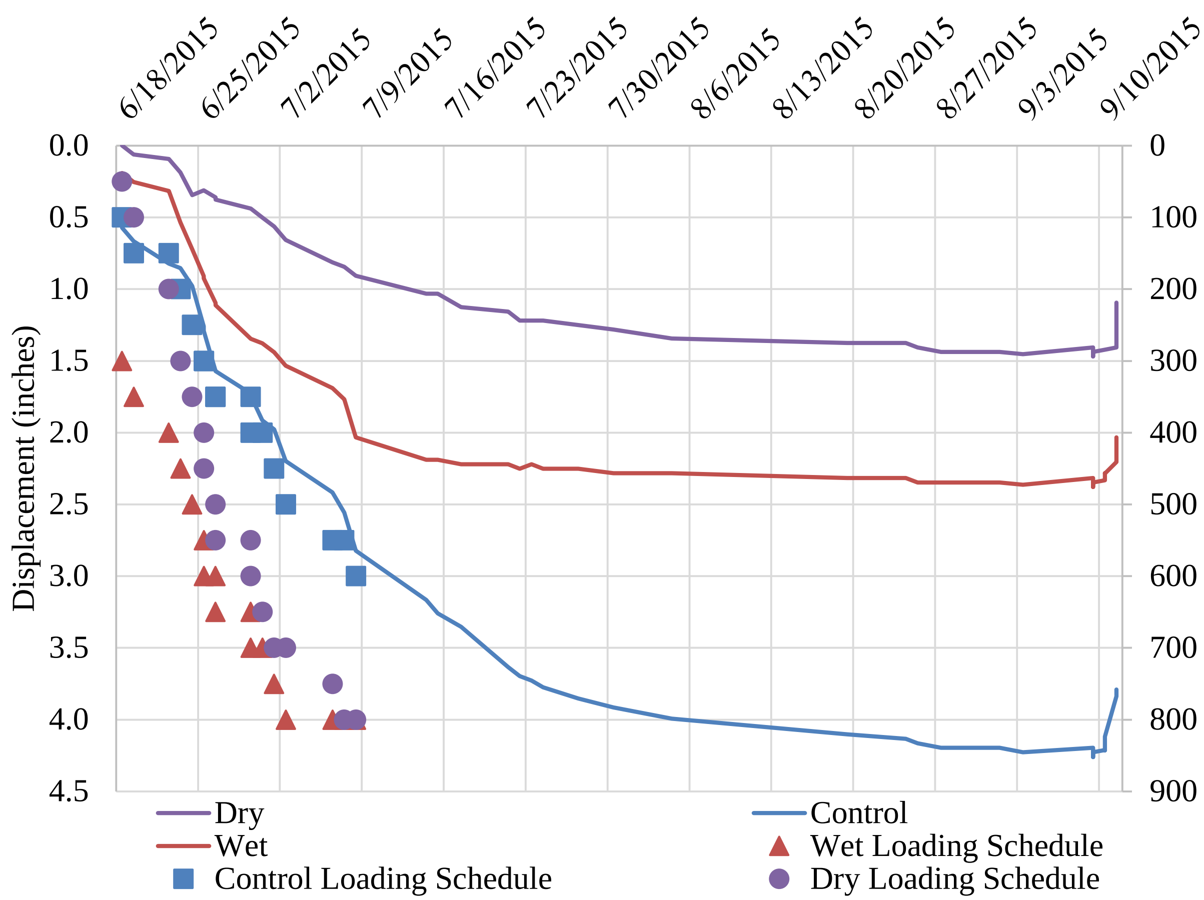

Figure 5.1. Settlement versus time versus loading.

Comparison of laboratory predicted and large-scale column strengths showed variation that exceeded cited ranges of lab/field ratios (Figures 5.2 and 3.33). Two of the nineteen exhumed columns were too weak to get a testable sample and therefore no data is shown. None of the columns showed strengths higher than predicted. The average lab/field ratio was 7 where the values ranged from 1.8 to 31.3, when compared to the average predicted strength of the full column.

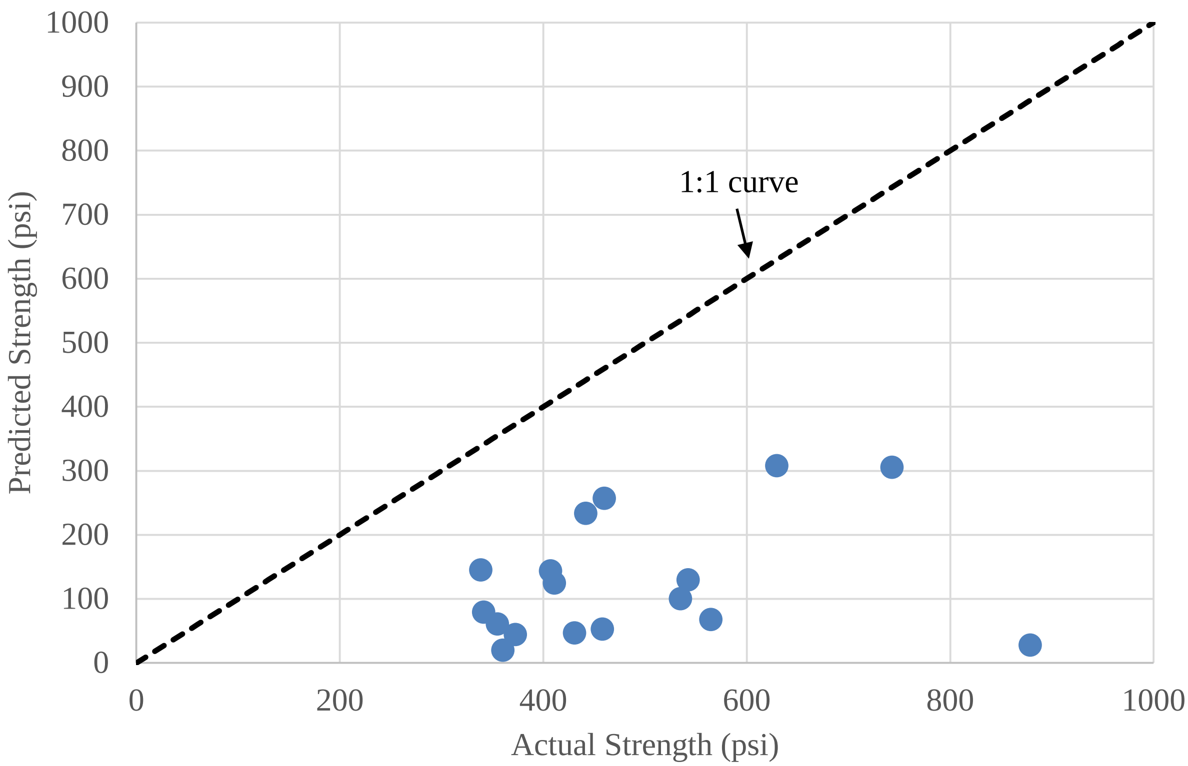

Figure 5.2. Predicted versus actual strength of the wet columns plotted with a 1:1 curve.

The fundamental difference between lab and field conditions lies in accurately documenting the actual mix proportions of the sample. In laboratory conditions, the constituents in the mix cannot escape and must stay within the mixing bowl. In field conditions, the final mix ratio can only be determined/estimated by the amount of grout pumped when the outflow nozzle was located at a given depth. The assumption that all that grout stayed at that elevation or that it was evenly distributed to all soil in the column at that depth cannot be verified. In reality, there tends to be an upward flow of grout to the surface which not only changes the as-built binder content, but also can alter the soil composition with the soil mix column relative to the surrounding soil (or boring log profile).

5.2 Evaluation of Full-Scale Soil Mixing Sites

Concurrent to the bench tests and the 1/10th 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 at previously mixed sites. Two problematic sites were also investigated where continued subsidence of a rural road and bridge over organic and/or soft soil. While not discussed in this thesis, one of the two sites was treated with a mechanical sand column installation method which promoted consolidation drainage and the other was not treated. When comparing the binder injected treatment to mechanical drainage improvements, the binder stabilized soils were not prone to time dependent deformation.

Looking at the unconfined compression test results from one of the projects, US-331, a wide range of strengths resulted from a given soil mix approach, but where the measured strengths exceeded the minimum design strength (excepting a few test columns installed before production began, Figure 5.3). The data represents all demonstration elements with high and low cement factors as well as tested values for blade rotations (mixing effort). The large strength variability (which is not uncommon for soil mixing) can be largely attributed to mixing thoroughness where some samples registered the full strength of pure grout and others more closely aligned with the anticipated soil mixing design strength; soil variation also contributed 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 as discussed above. 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 2 to 5 fold difference cited for lab versus field performance (lab values usually higher).

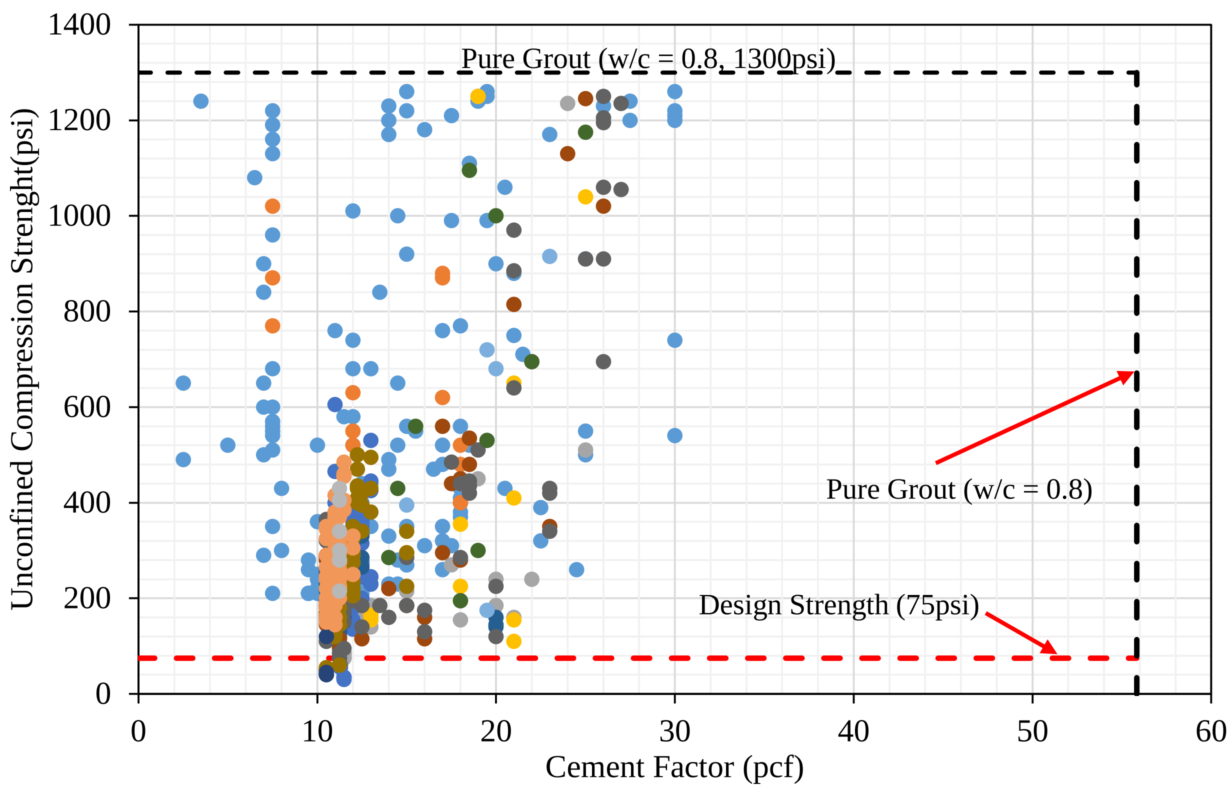

Figure 5.3. Unconfined compression tests from field collected soil mix specimens.

While no soil mixed 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 56pcf and strength from laboratory tests that support the upper limit observed in the field. Figure 5.4 shows the results from unconfined compression tests performed on pure grout with varied w/c ratios. Grout with w/c ratios above 1.0 does not maintain suspension and often results 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.

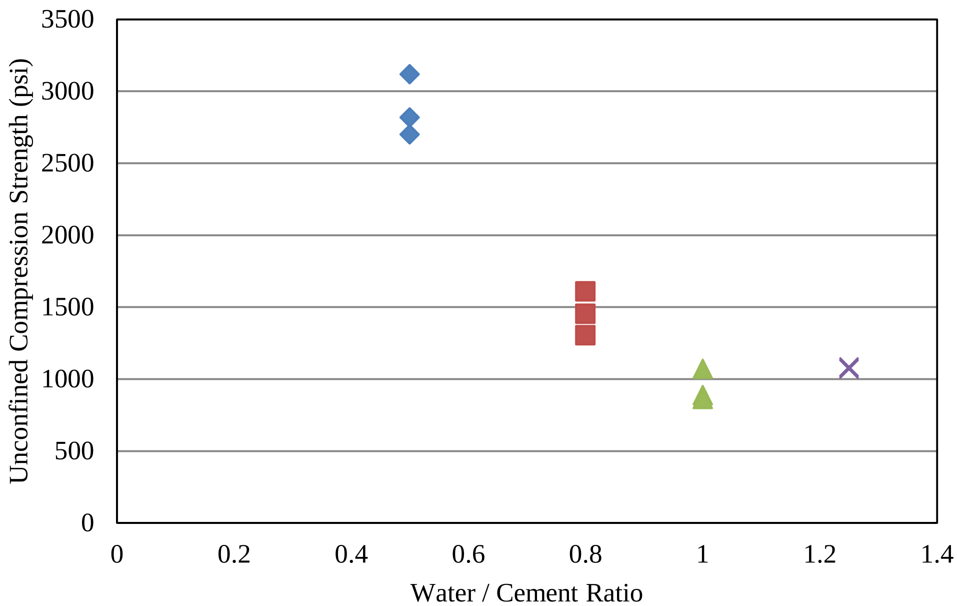

Figure 5.4. Strength of grout at various w/c ratios.

5.3 Application of Threshold Concept

While the US331 data shown above is based on field collected samples and the measured binder content, most field programs begin with a bench scale (lab) study to establish a mix ratio that meets the design requirements. The data presented in Chapter 4 while based on production field studies, were actually laboratory tests where the programs were performed similar to the USF study but on different soils. Recall from Chapter 2, the FHWA design curve was similarly established on the basis of a wide range of laboratory studies of various soils (largely inorganic).

Using the modified FHWA design curve, all data discussed in Chapters 2, 3 and 4 has been plotted in Figure 5.5. While some of the data seems to ideally fit the curve, most data points (again from organic soils) do not achieve the expected strength for their respective w/c ratio.

In Chapter 2, the idea of a threshold concept was discussed, which in essence changes the way the data is viewed in Figure 5.5. Instead of seeing the data as being below the anticipated strength curve, the threshold concept implies the data is to the left of the curve; by defining the amount of unused cement, the effective w/c ratio is increased (moves data to the right). In this section, this approach will be applied. This will be done by first finding what amount of 'throw-away' threshold cement would be required to precisely fit all data directly on the curve (Figure 5.6). Figure 5.7 shows the plot of the calculated threshold versus organic content. Aside from the LPV 111 75% slag 25% cement data, all data seems to provide a general curve. While not exact, it is close to the threshold curve proposed in Chapter 2 (Figure 2.25).

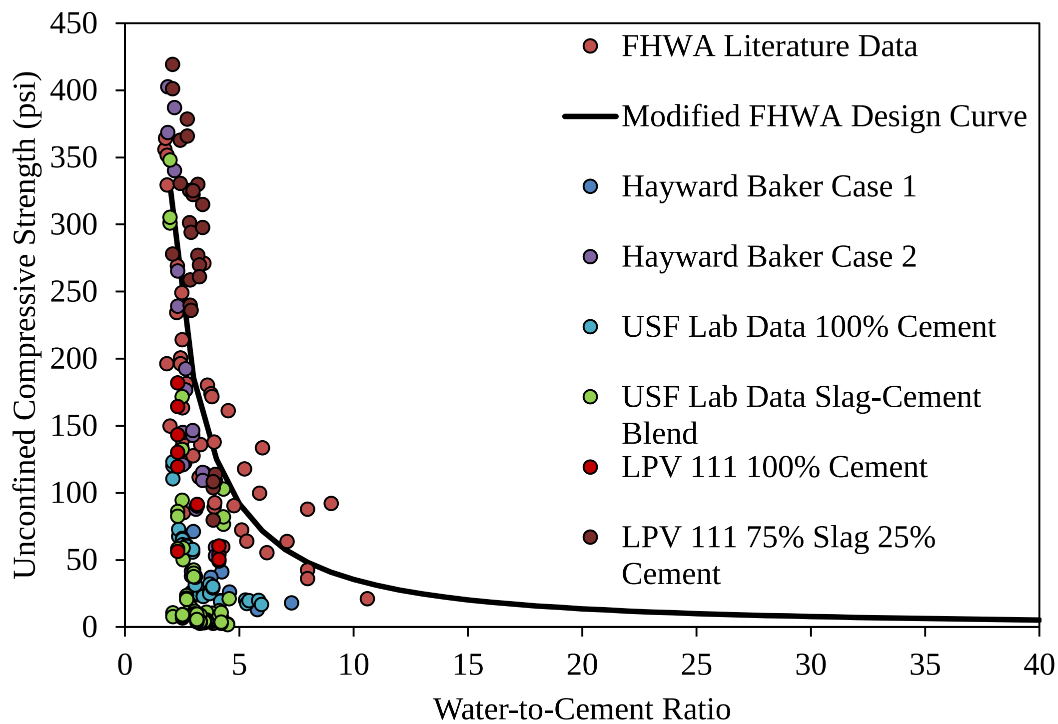

Figure 5.5. Modified FHWA design curve with new case studies data.

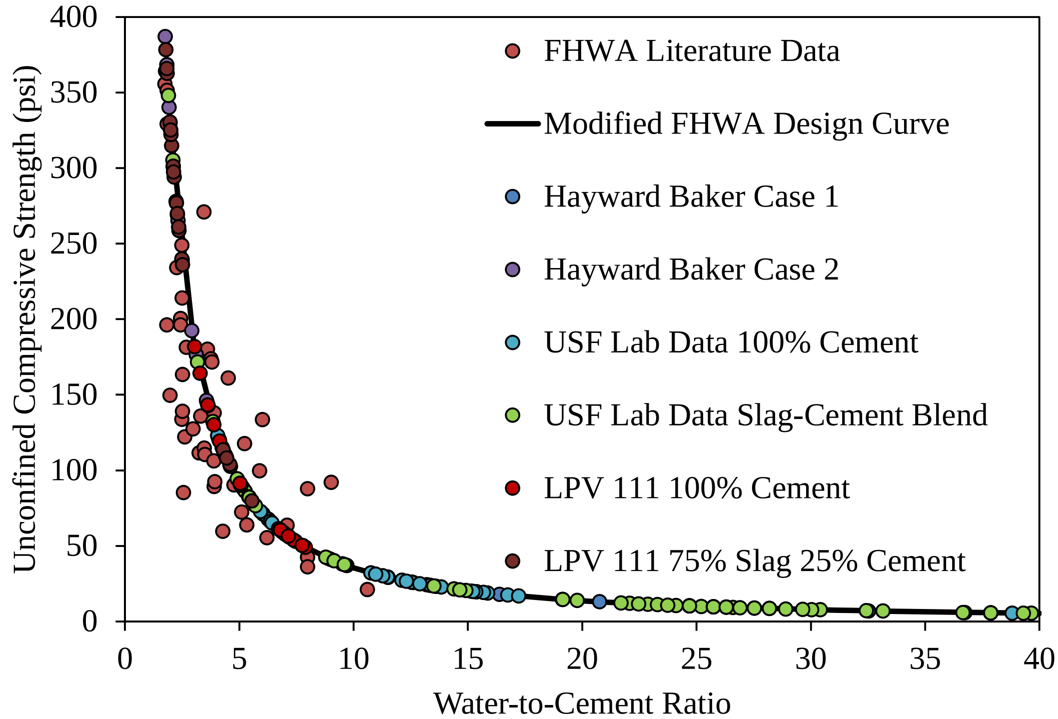

Figure 5.6. All data fitted to the modified FHWA curve.

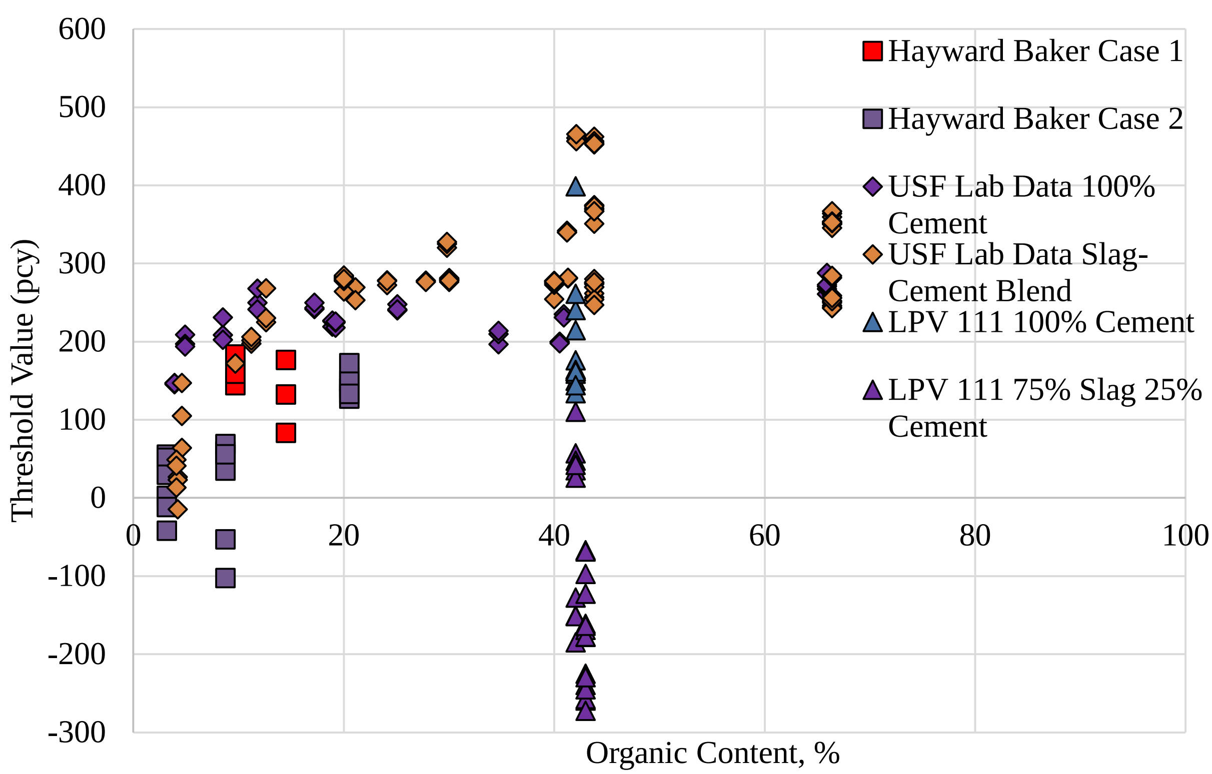

Figure 5.7. Threshold values based on the curve fit in Figure 5.4 versus organic content.

If LPV 111 data using slag is removed, Figure 5.8 is formed, more clearly demonstrating the general curve just mentioned. Thus, for the purpose of this analysis LPV 111 75% slag 25% cement has been excluded. There are a few potential reasons why this data could be a misfit. The first being that the soil type is a clayey organic. From the USF laboratory study of organic soils, it was found that slag was essentially ineffective in terms of stabilizing organic soils above about 20%. This was clearly not the case with LPV 111, where there was incredible success with slag.

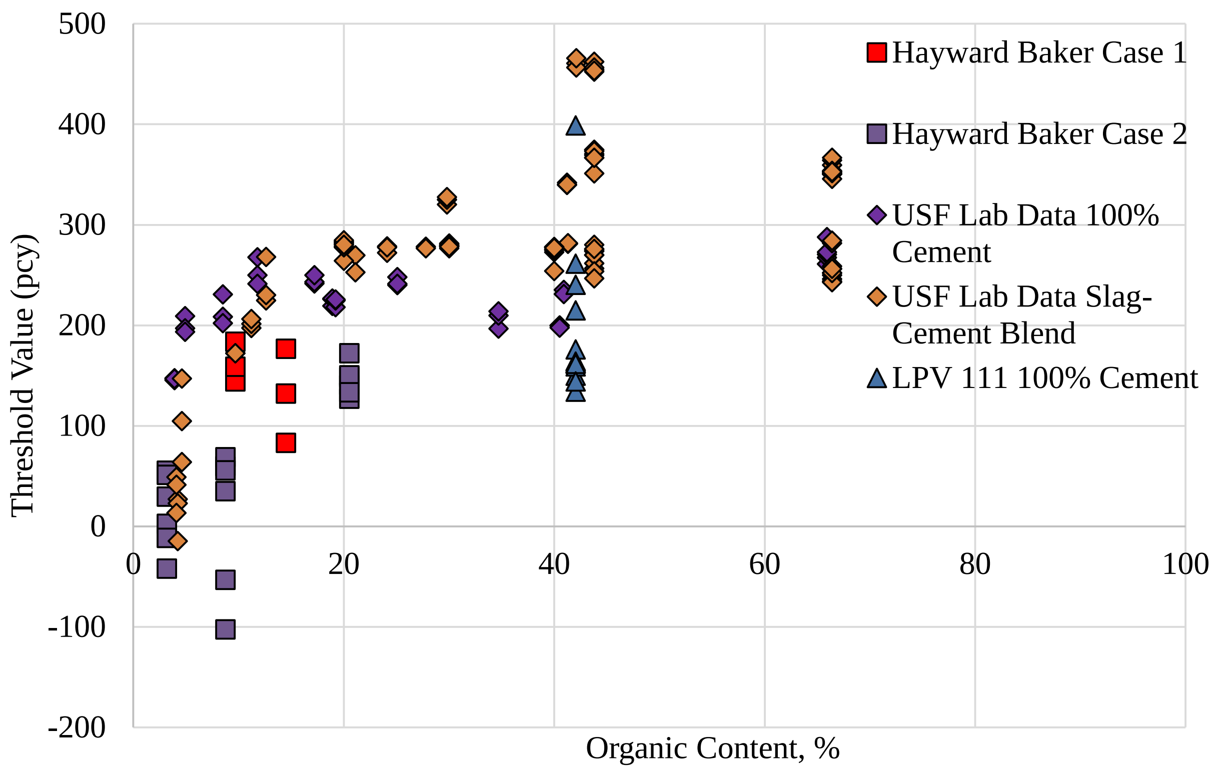

Figure 5.8. Threshold values, excluding LPV 111 75% slag 25% cement, versus organic content.

The main difference between the two is that USF tested a sandy organic soil, whereas LPV 111 was a clayey organic. The other potential issue with this data is that such large amounts of binder were used and strength gain is disproportionally higher for additional amounts of binder at these levels. Figure 5.9 shows only 100% cement binder data (USF, Hayward Baker, and LPV 111) and reveals that when above approximately 300 psi of strength the threshold values become invalid (i.e. higher range of the FHWA curve). Recall, that the original FHWA curve was based on 28 day strengths of pure cement binder mixing programs (not slag). When the points higher than 300psi are disregarded, the data aligns more closely with the general threshold vs OC trend. The main difference is that now there are no negative threshold values that have no physical meaning (Figure 5.10 and 5.11). Negative values may imply the design curve is in accurate in that range.

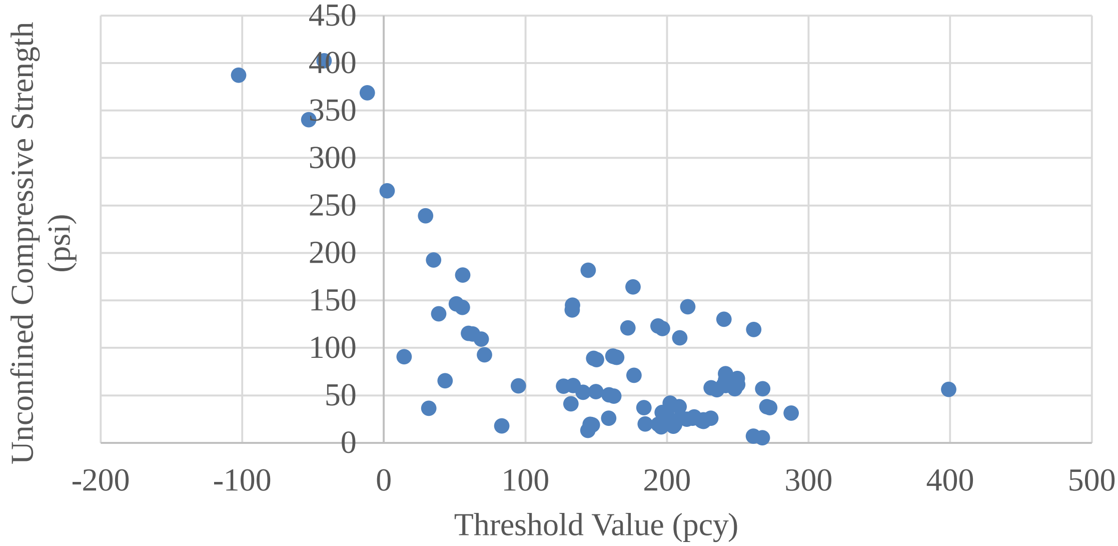

Figure 5.9. Unconfined compressive strength versus threshold values for all 100% cement data.

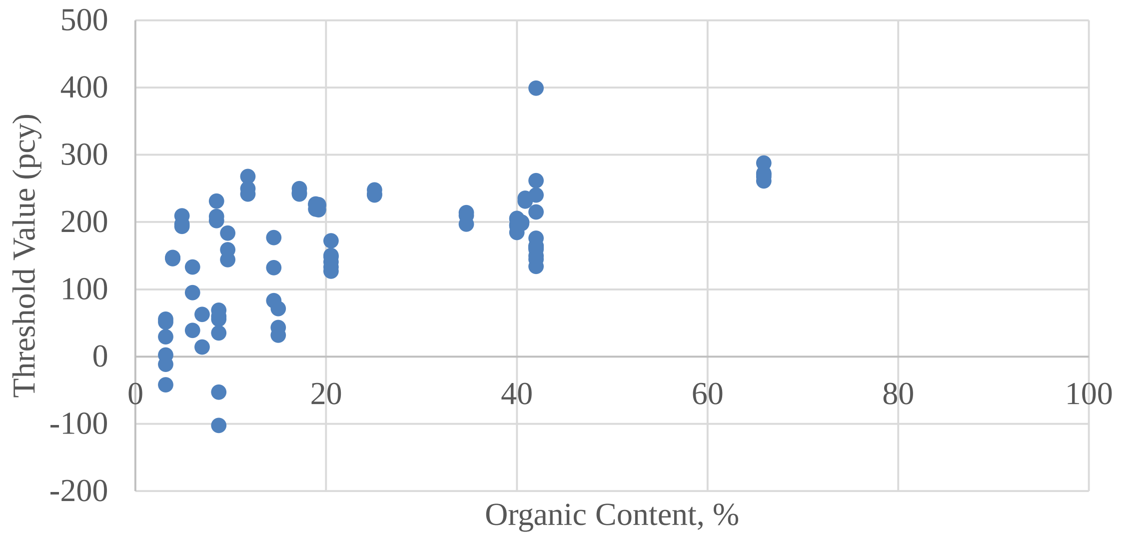

Figure 5.10. Threshold vs. OC before removing higher strength samples.

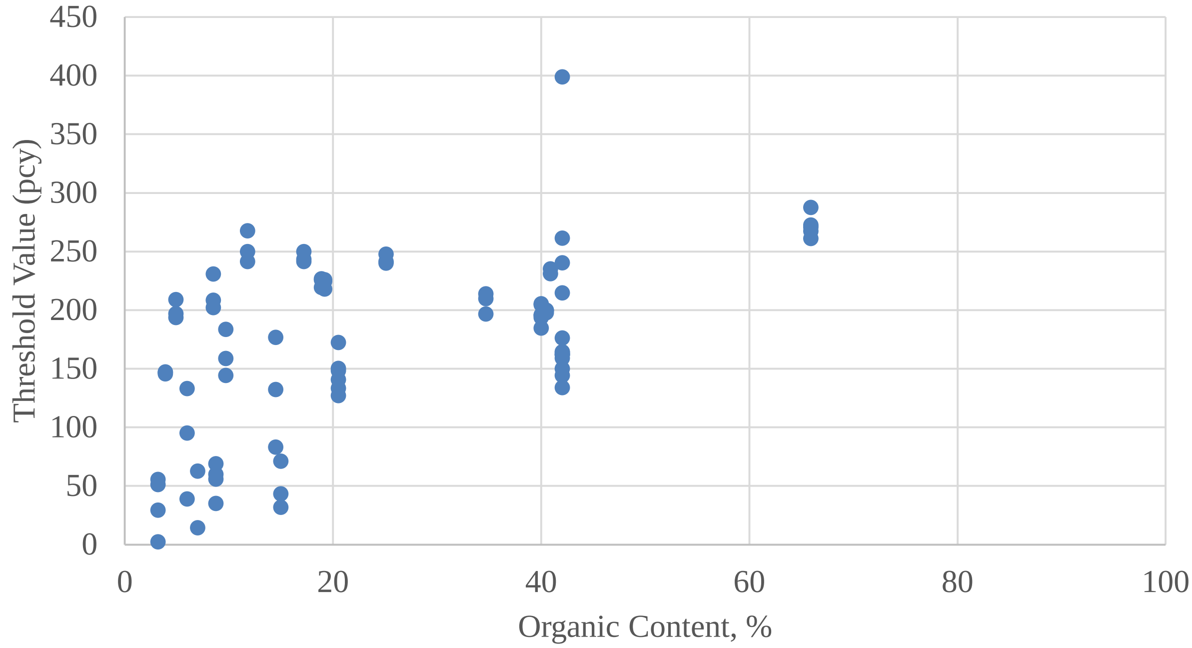

Figure 5.11. Threshold vs OC after removing higher strength samples.

Now, working with the modified set of data, a set of threshold curves was developed. Given that LPV 111 showed that a clayey versus sandy organic when using a slag-cement blend could make such a huge difference, the curves have been separated by the factors of soil type and binder. Cement only sandy organic soil, cement only clayey and sandy organic soil, slag/slag-cement blend only sandy organic soil, and cement, slag, clayey, and sandy soils (all data) threshold curves are presented in Figures 5.12, 5.13, 5.14, and 5.15, respectively, with all data below 300psi .

Finally, all curves have been combined into one graph, Figure 5.16, for comparison purposes. It should be noted that the clayey organic soil data that were omitted due to strength are plotted as single points (extracted from Figure 4.33). These data points were computed as negative thresholds using the curve fit method; the binder factor intercept was used. In general, the biggest difference from a given soil type was from the slag/slag-cement blends in sandy organics when compared to 100% cement. This equates to about a 75pcy difference in the required threshold.

Thus, the conclusion that can be drawn here is the same as before, 100% cement works more efficiently as a stabilizer for sandy organic soils.

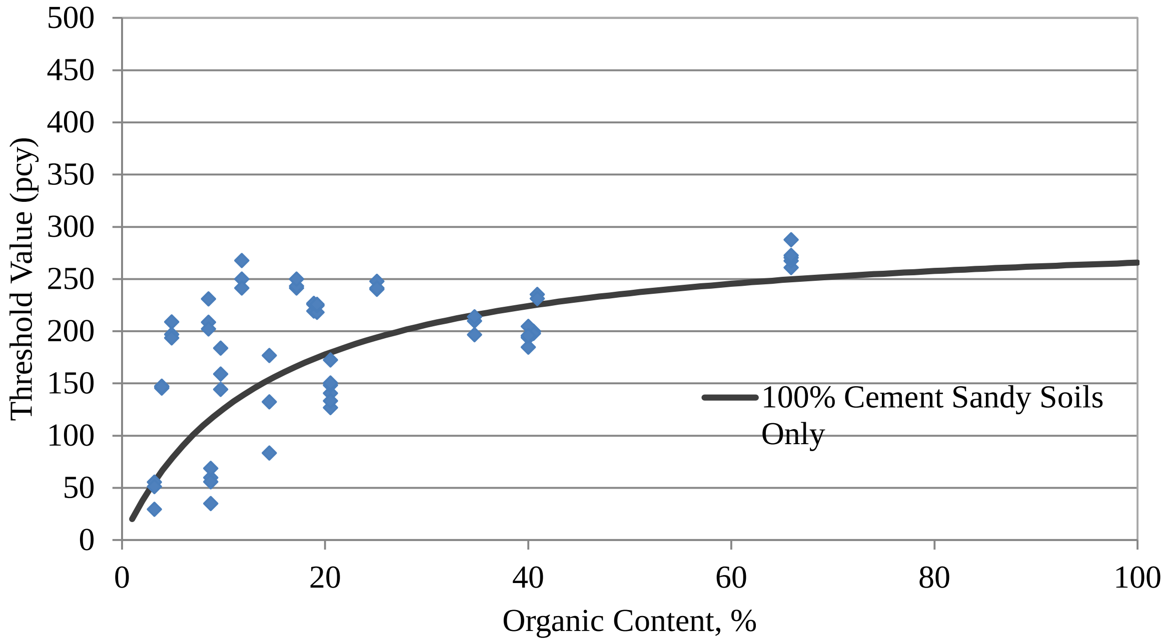

Figure 5.12. Organic content versus threshold for 100% cement sandy soils (both Hayward Baker cases, USF data).

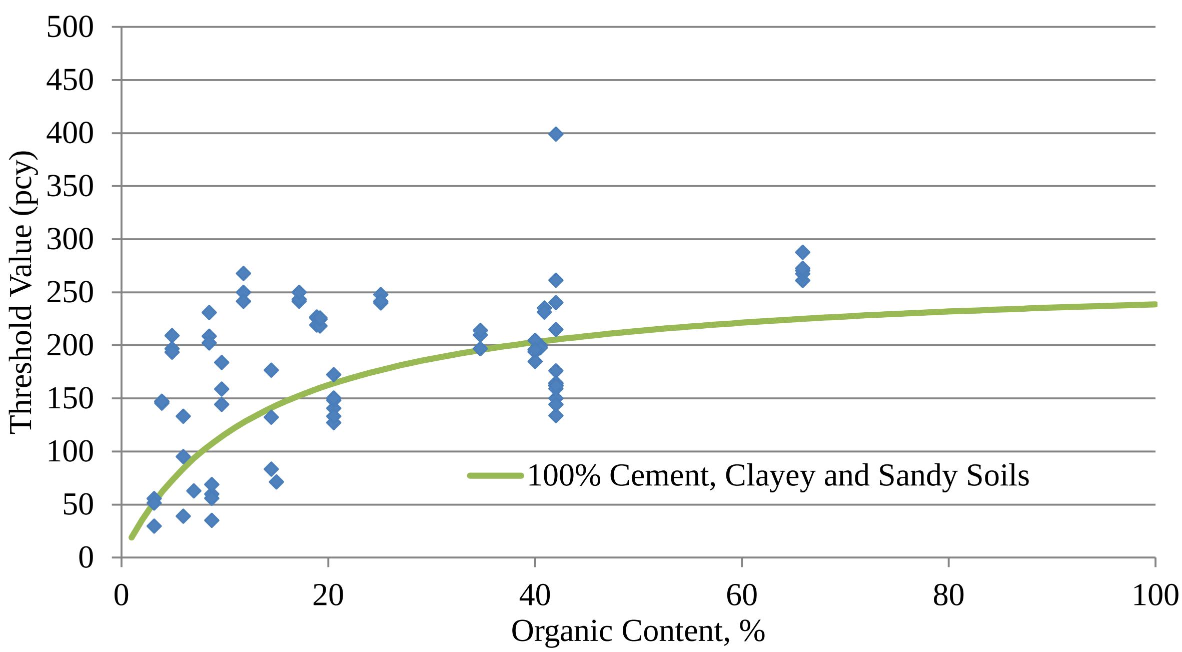

Figure 5.13. Organic content versus threshold for 100% cement sandy and clayey soils (both Hayward Baker cases, USF data, LPV 111).

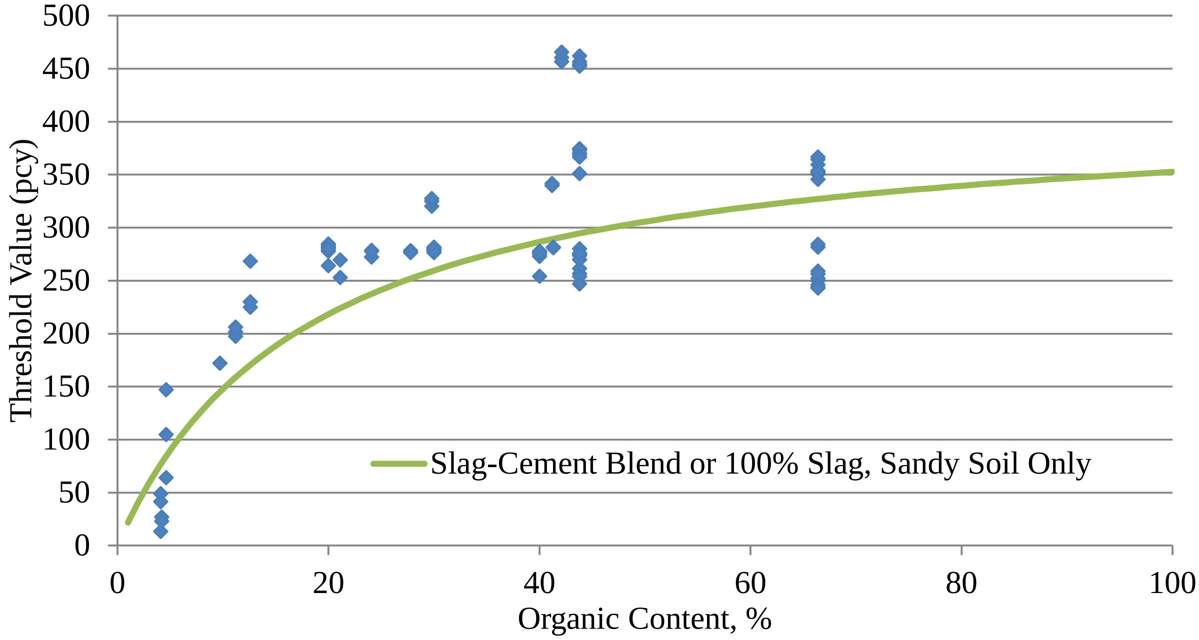

Figure 5.14. Organic content versus threshold for 100% slag/slag-cement blend sandy soils (USF data).

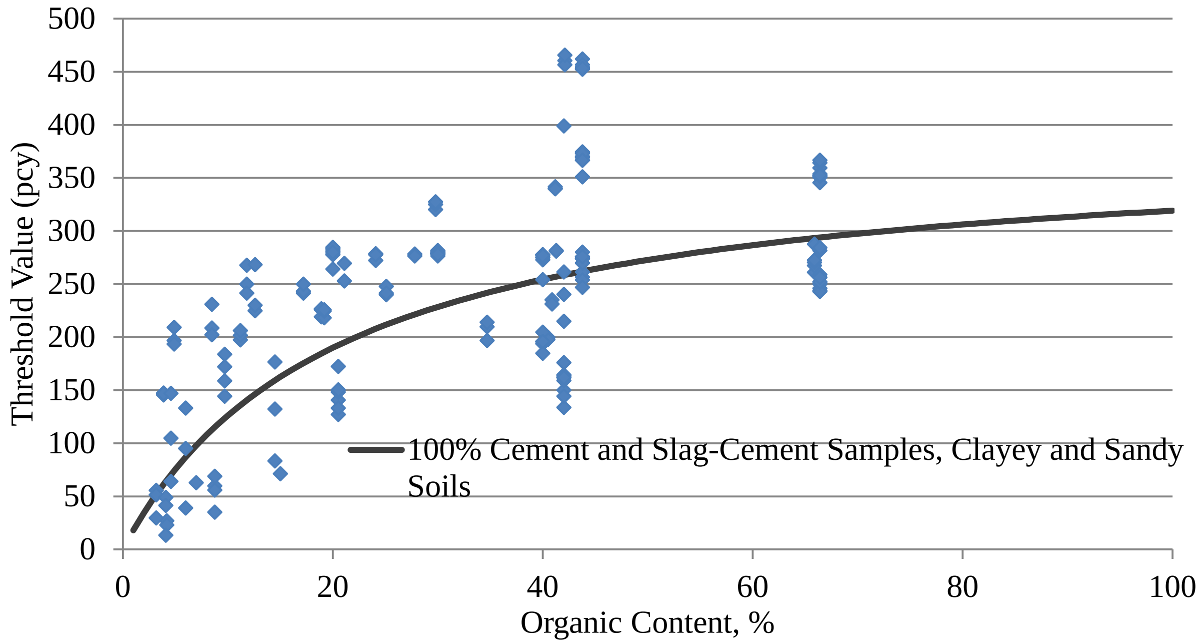

Figure 5.15. Organic content versus threshold for 100% slag/slag-cement blend and 100% cement sandy and clayey soils (all data).

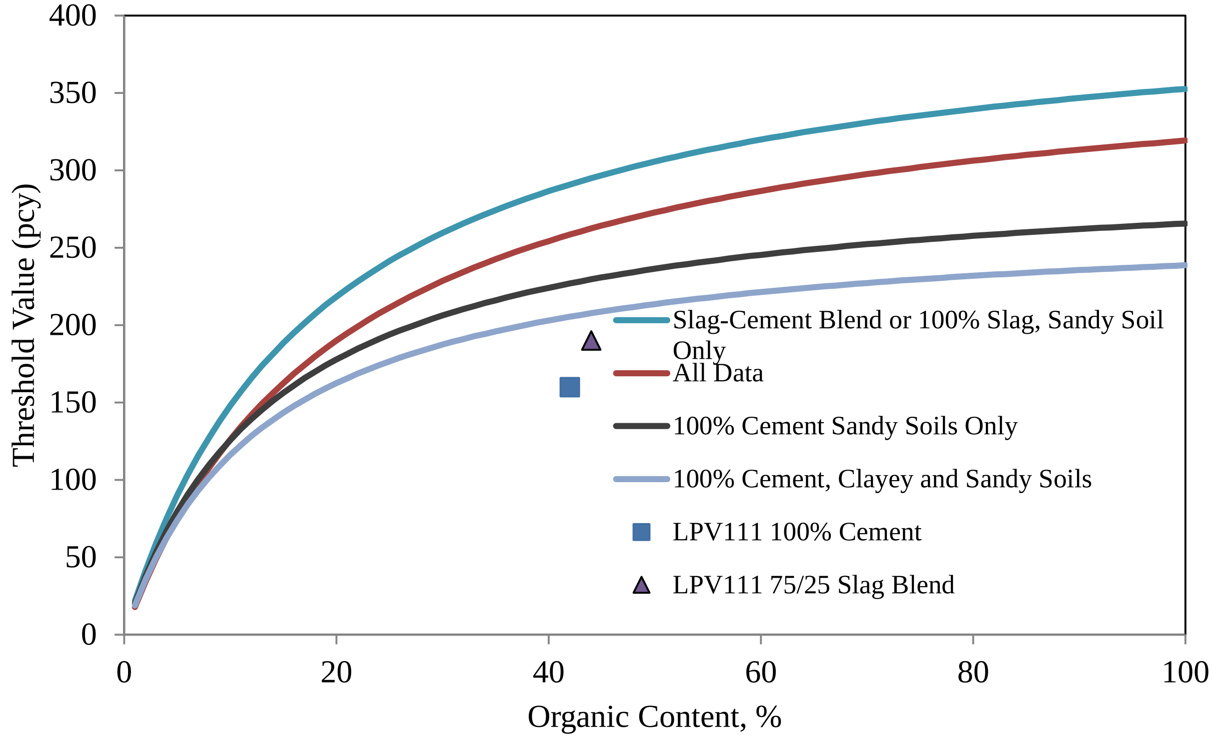

Figure 5.16. Organic content versus threshold for various categories.

Looking at the curve which combined all data in relation to the other threshold curves, it is clear that the slag/slag-cement blend is driving the curve upwards for sandy soil, while the other two are doing the opposite. What is also interesting, is that the curve of 100% cement, sandy and clayey organic soils, seems to be asking for a lower threshold than that of the only sandy organics curve. This would seem to suggest that clayey organic soils might require a lesser amount of binder for stabilization. However, the clay data used to develop this curve was from LPV 111. It is unclear as to how reliable that data is given that the 75% slag 25% cement yielded such strange results and the scatter seen in the bench scale tests was substantial. Substantial scatter suggests that other variables are in play that affected the results. Degree of decomposition, for instance, is not computed separately when performing an organic content test. Rather, burned solid organic material and burned decomposed material can produce the same OC value. However, highly decomposed materials are associated with lower pH.

Now, using the soil and binder type dependent thresholds, the data that was previously predicted poorly (Figure 5.5) using the FHWA curve now more closely agree (Figures 5.17 and 5.18). However, as the design curve was derived for 100% cement usage, Figure 5.18 replots this data without slag samples. The organic soil data now fits the curve as well as the literature data (same degree of observed scatter). Note that all data (including data above 300psi and aside from LPV 111 75% slag, 25% cement) was included for Figures 5.15 and 5.16.

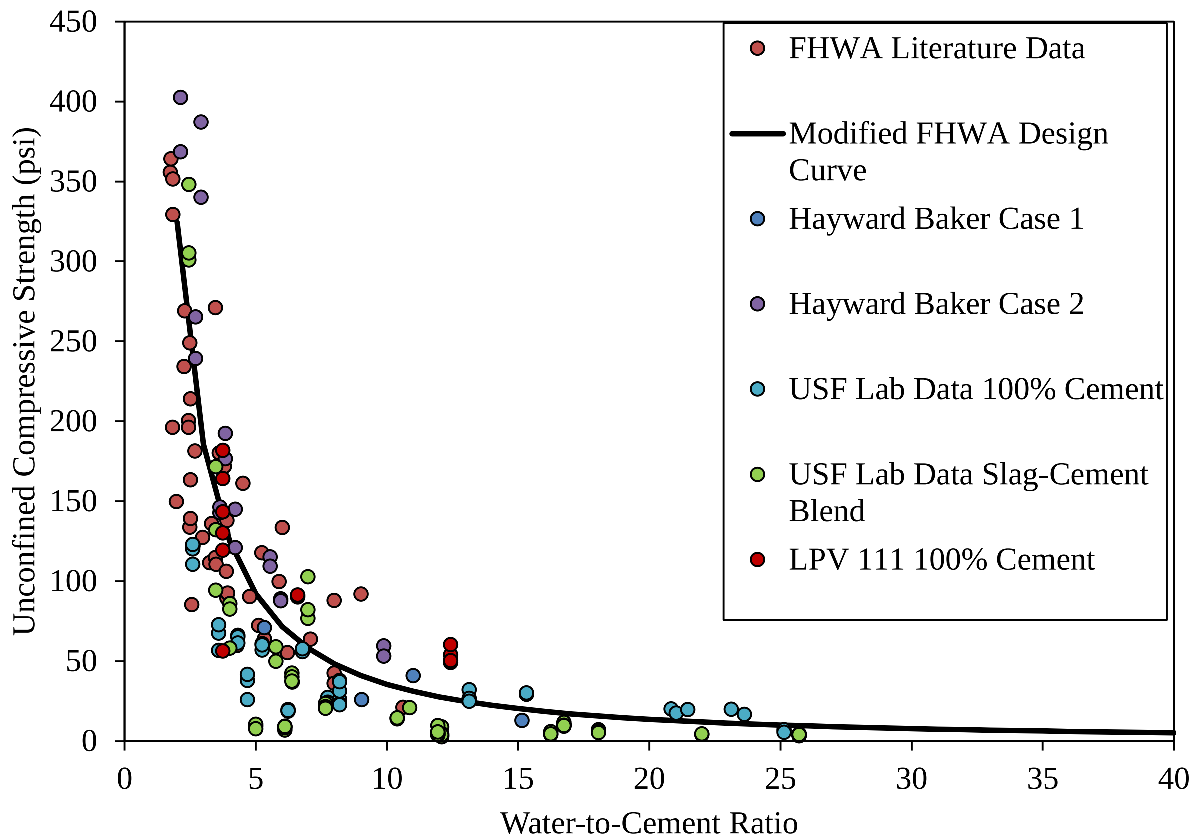

Figure 5.17. Stabilized data using the appropriate threshold curves.

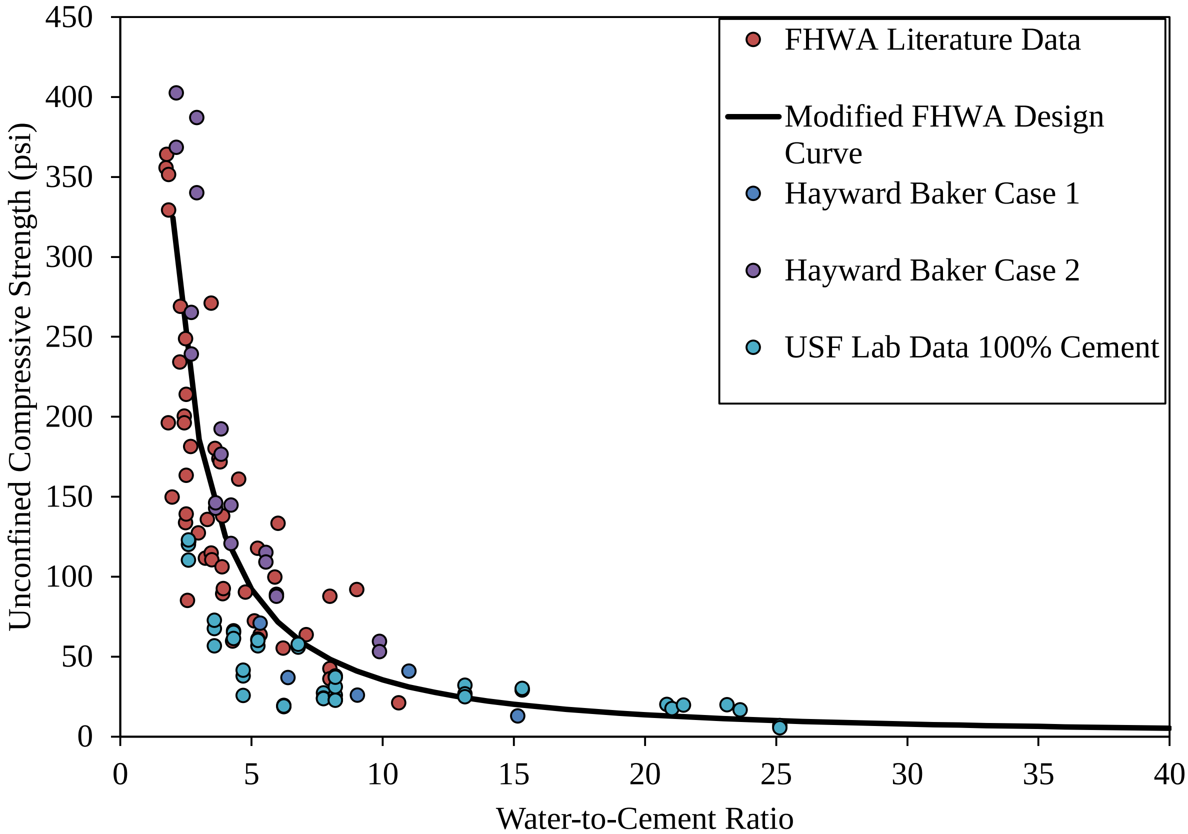

Figure 5.18. Stabilized data using the appropriate threshold curves, excluding USF slag.

5.4 Recommendations for the Future

In future research it would be interesting to see why slag seems to perform much better in organics with clay versus organics with sand. As seen through the USF laboratory research slag performed better or the same as cement below about 20% organics. After this point, slag performed more poorly than cement alone to the point that most specimens could not be tested in unconfined compression. However, as seen in other cases such as with LPV 111 when slag is used in higher organic soils with clay it performs extremely well, much better than cement alone. While in sandy organics some insight was gained into the effect of binder type vs organic content, it would be beneficial to know if such a cut-off exists in clayey organics. This may help to define the conditions in which a slag-cement blend should be used and when only cement should be used. Further research should also be done to refine the threshold curves provided in terms of organic clays as this study did not have enough data to do so. This could perhaps be the next step in better understanding soil mixing performance in organic soils.

5.5 Final Conclusions

This study concluded with a rationale for selecting / predicting the require amount of binder when organic soils are treated with a soil mixing procedure. In this regard, the following conclusions should be considered:

1. There are soil-binder dependent threshold curves for organic soils. This study was able to generate curves for several conditions, however still missing is a wider range of organic contents for clayey organics with a slag-cement blend and clayey organic with 100% cement.
2. Slag blended binders performed better in clayey organics and pure cement better in sandy organics.
3. The scatter in the LPV 111 bench scale data suggests there was something else affecting strength (i.e. degree of decomposition).
4. The threshold computation method that relied on the curve fit to historically cited data was the primary method discussed in this thesis. Using this method, negative thresholds were shown for LPV 111 at higher strengths (top of the FHWA curve). However, using the binder factor intercept method (or 0psi method) to find the threshold, LPV 111 did indicate positive threshold values for both cement and slag-blended specimens (approximately 160-190pcy, respectively). This may indicate the top of the FHWA curve (formed by only a few data points in that region of the curve) may not accurately depict the strength vs w/c ratio for higher strengths.

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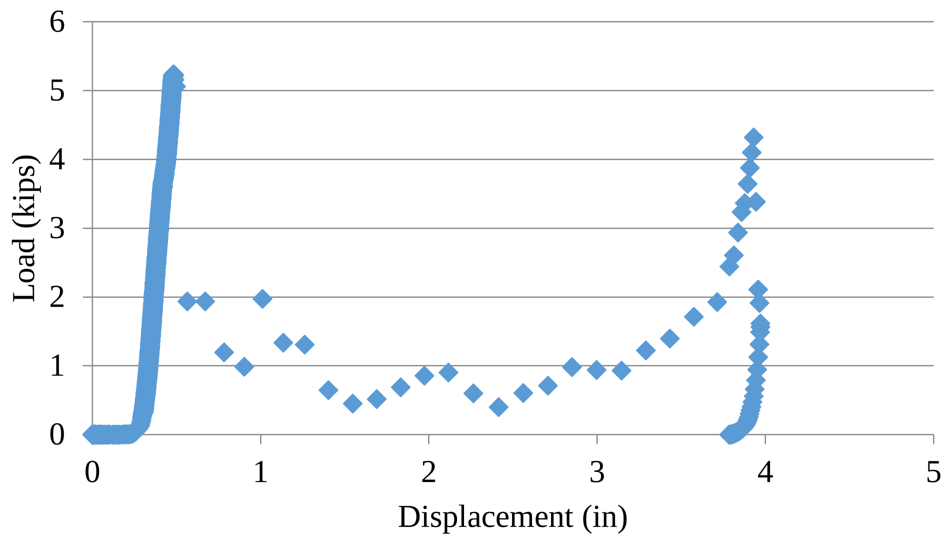
APPENDIX A: WET MIXED COLUMN UNCONFINED COMPRESSION TEST RESULTS



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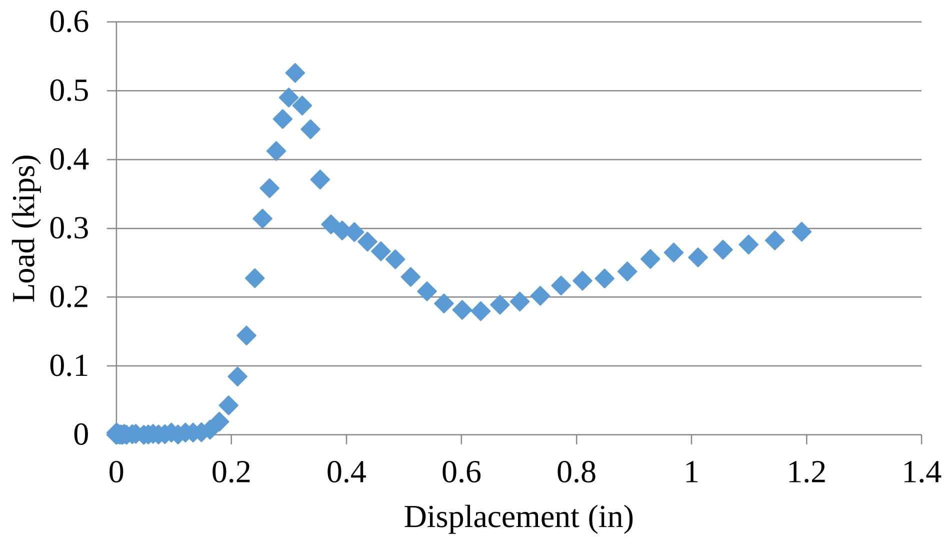

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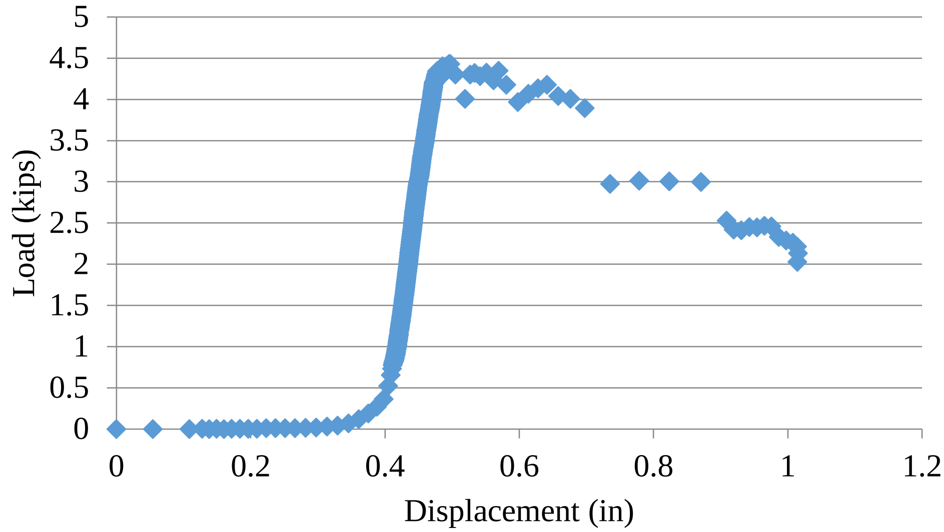

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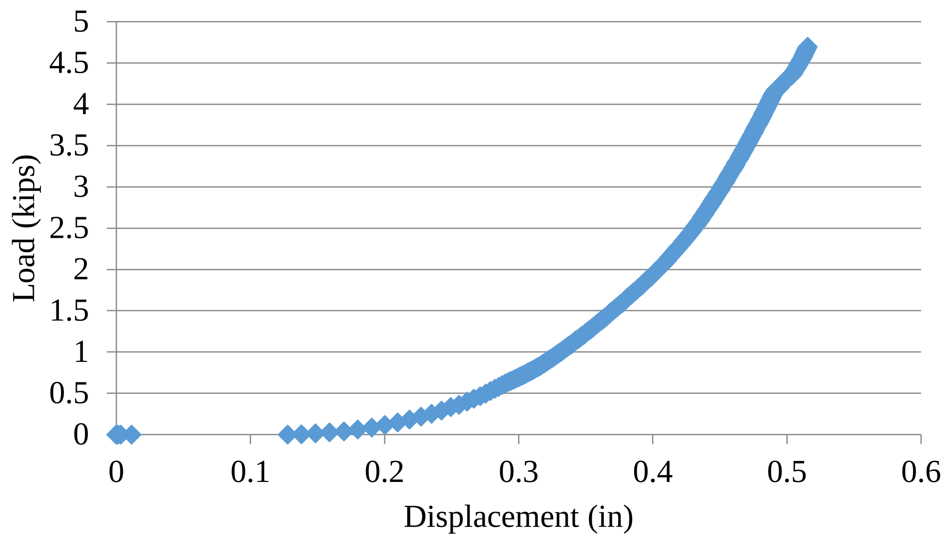

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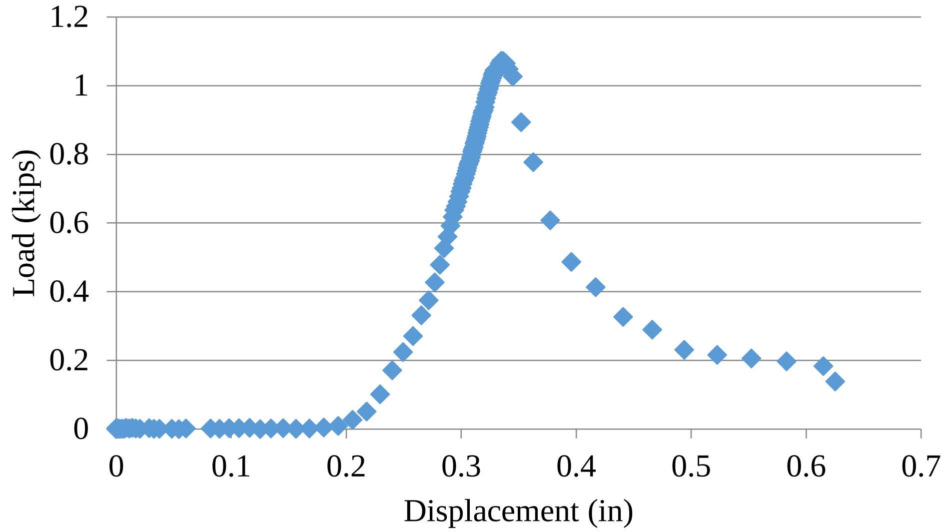

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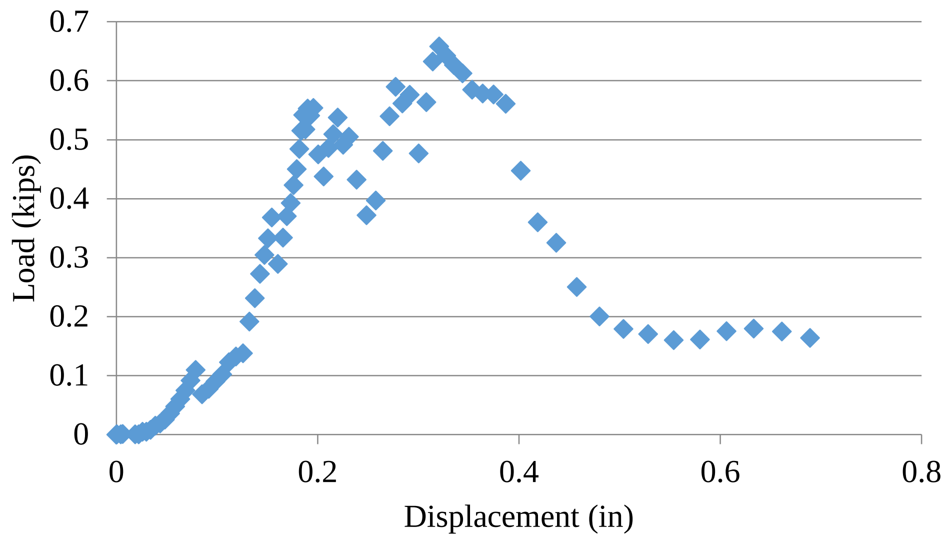

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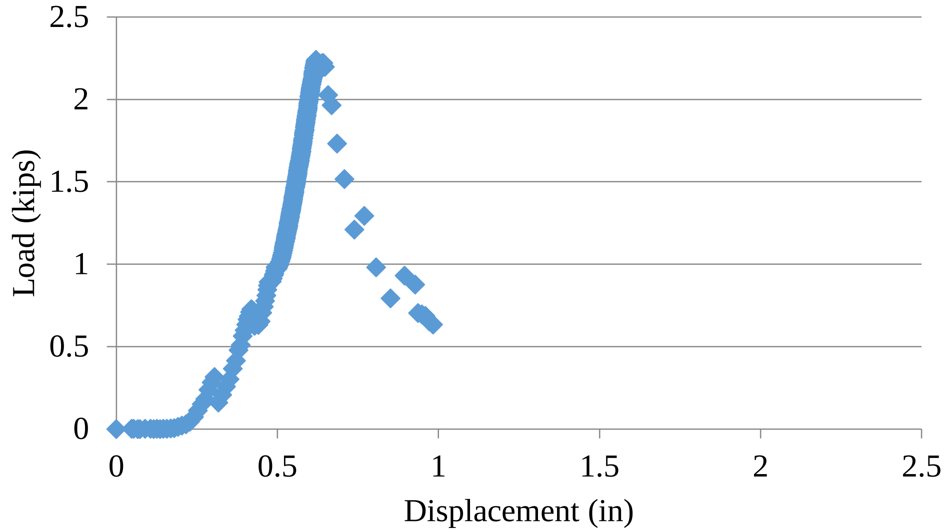

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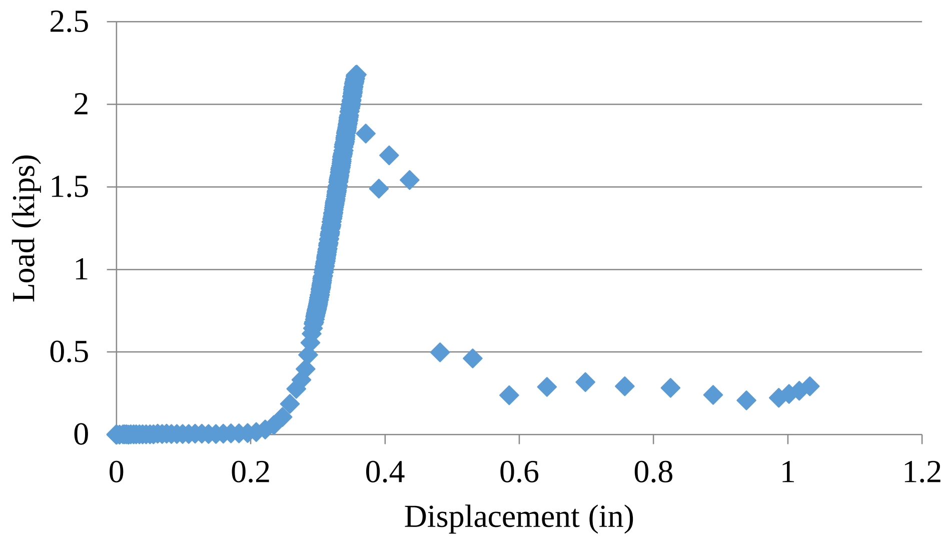

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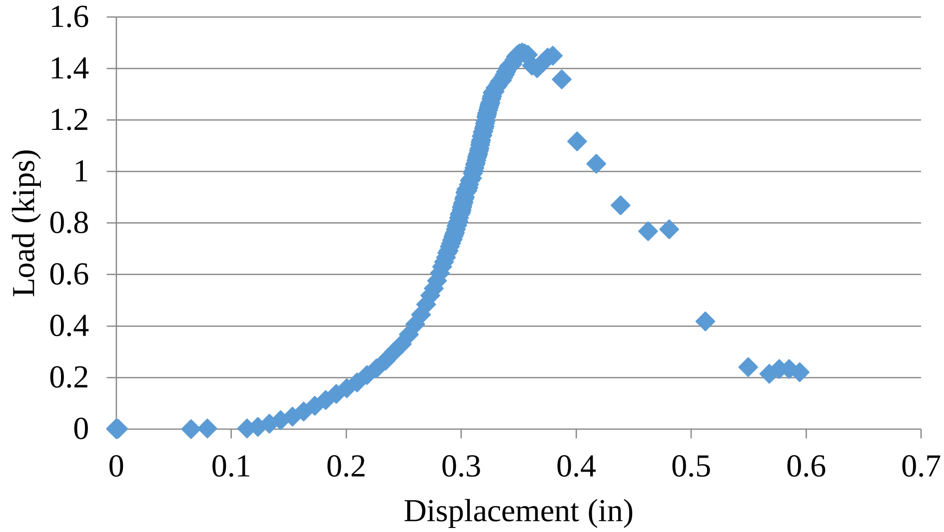

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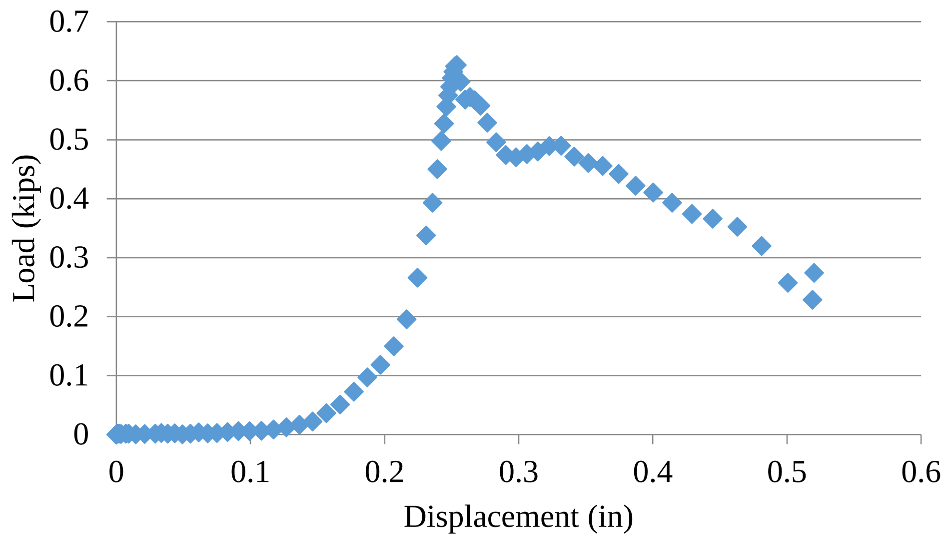

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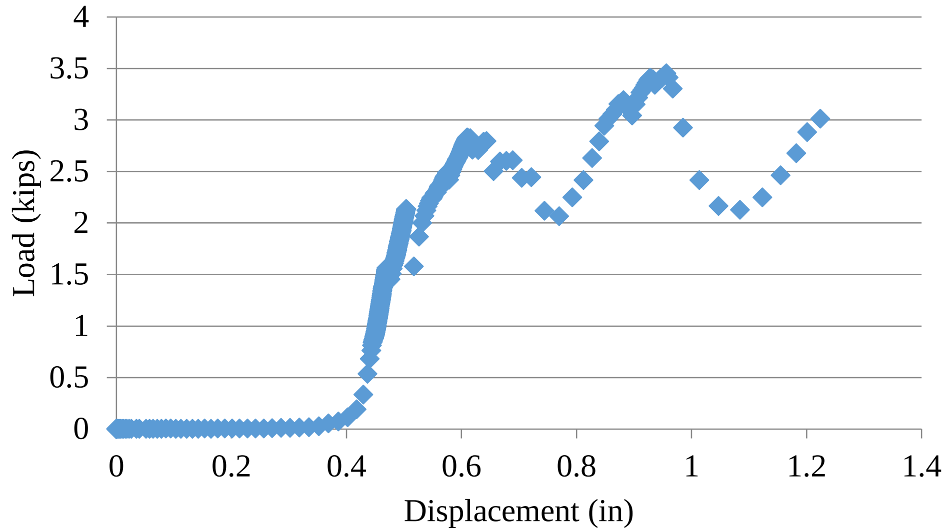

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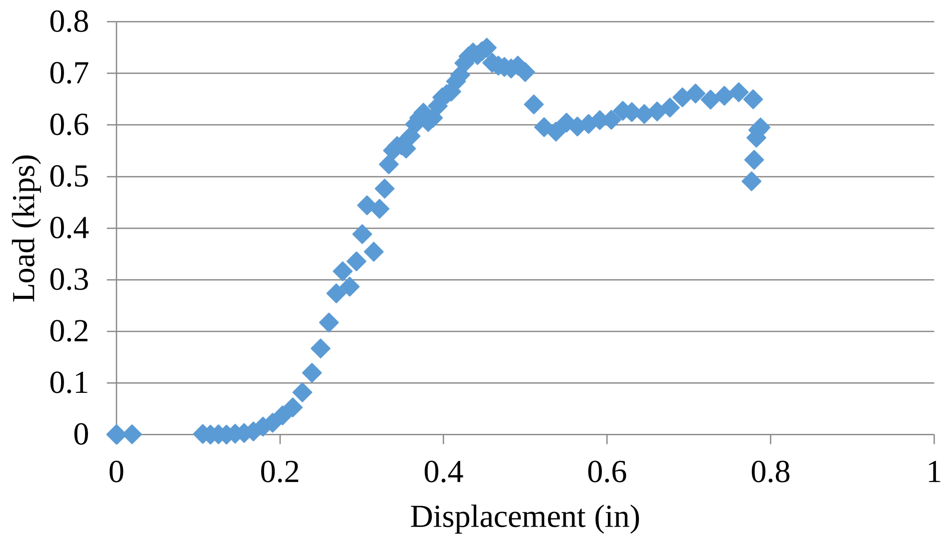

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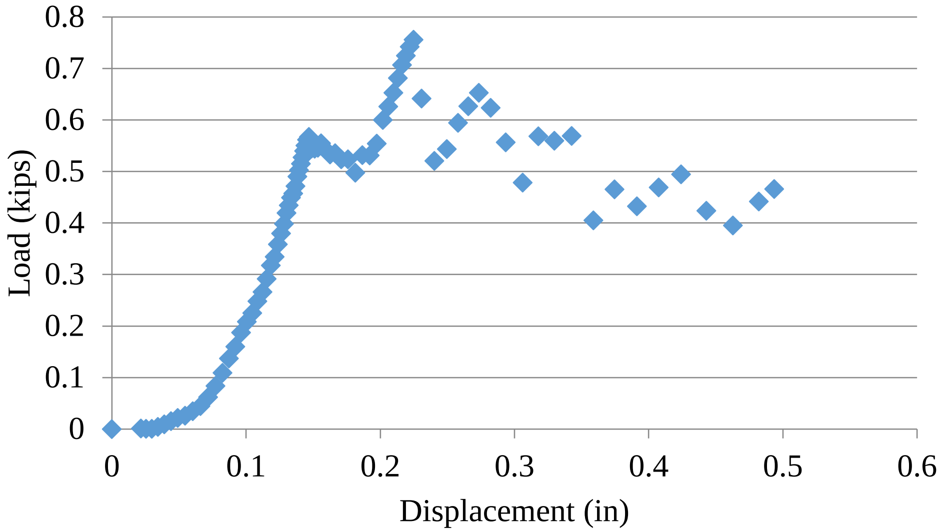

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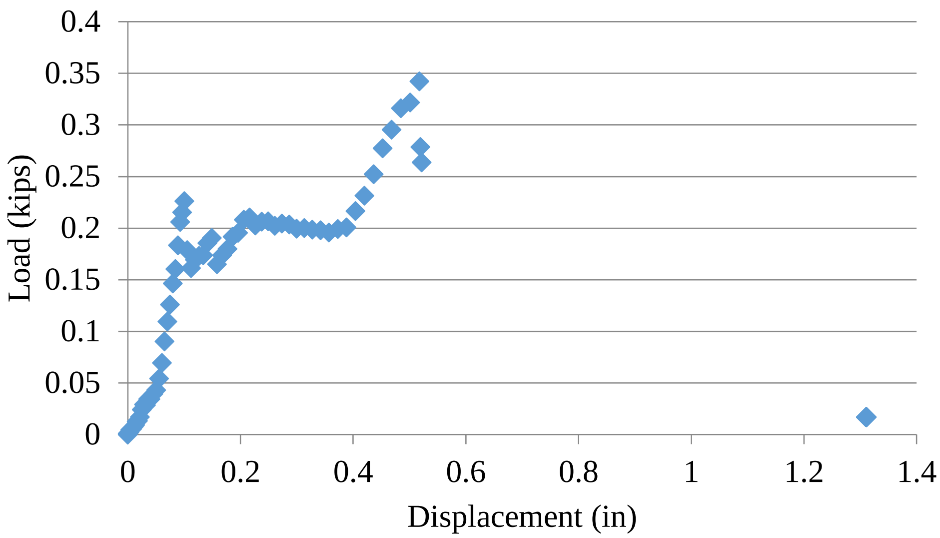

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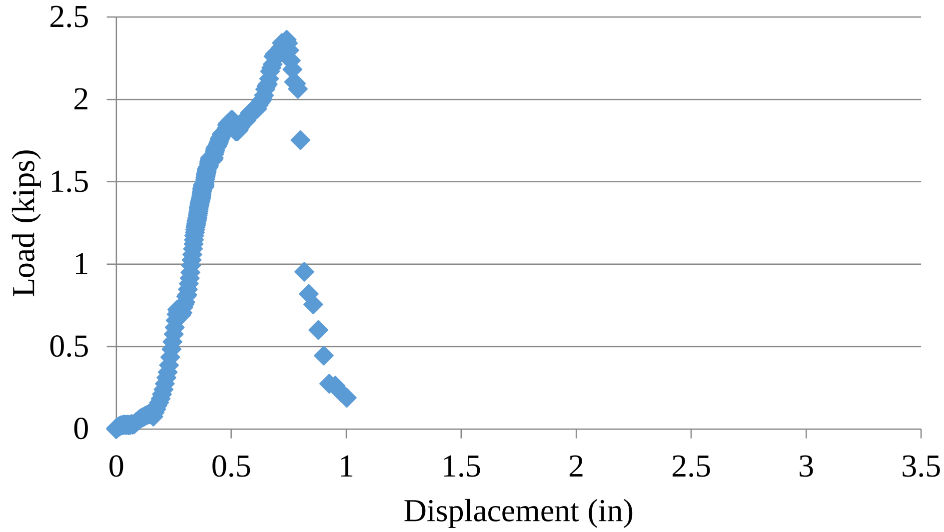

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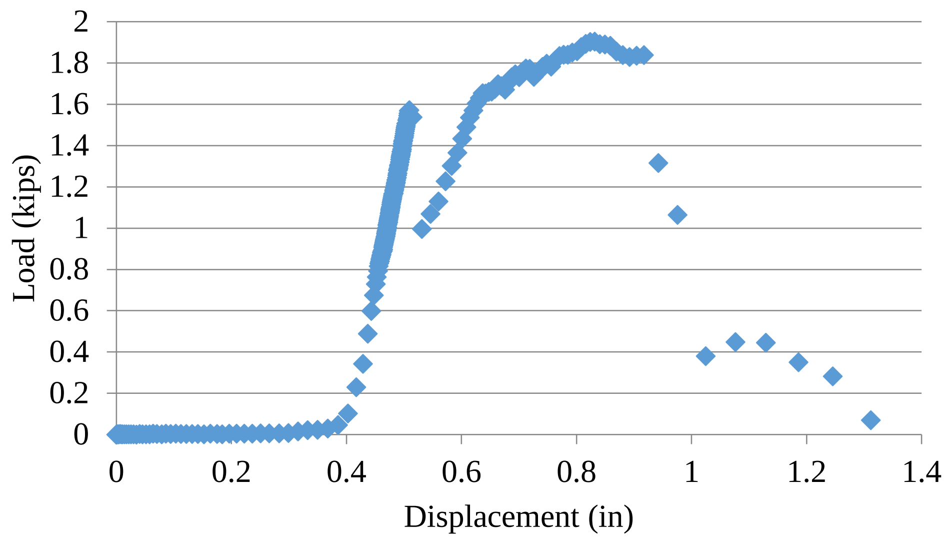

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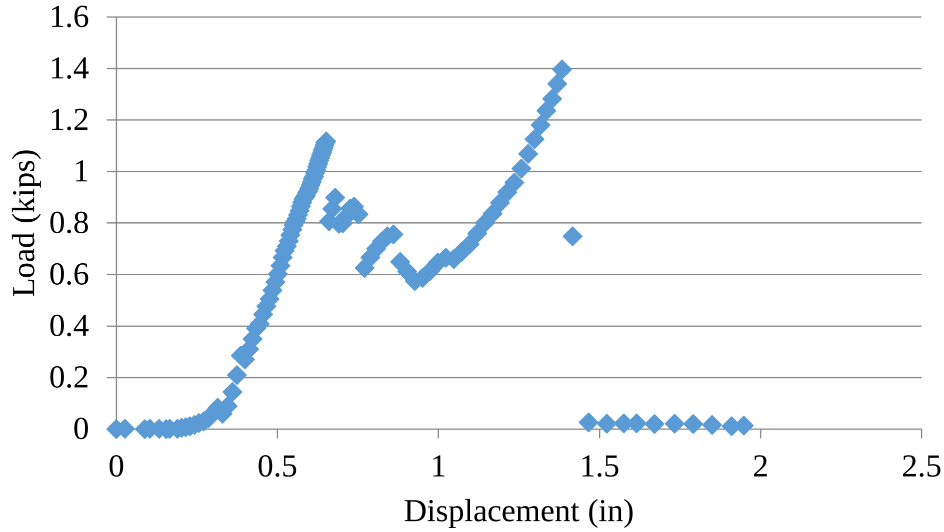


Figure A.17. Column 19

APPENDIX B: SHEAR VANE TEST RESULTS

Table B.1. Shear vane test results.

Batch #	% Slag Replacement	Cylinder #	Cement Factor (pcy)	w/c ratio	% Organic	Strength (psi)
6	100	1	300	4.12	66.4	11.378672
		2	300	4.12	66.4	10.2408048
		3	300	4.12	66.4	11.9476056
		4	300	4.12	66.4	12.2320724
		5	300	4.12	66.4	8.534004
		6	300	4.12	66.4	9.6718712
		7	300	4.12	66.4	9.6718712
		8	300	4.12	66.4	7.822837
		9	300	4.12	66.4	8.534004
7	100	1	400	3.05	66.4	10.2408048
		2	400	3.05	66.4	9.3874044
		3	400	3.05	66.4	10.2408048
		5	400	3.05	66.4	10.2408048
		6	400	3.05	66.4	9.6718712
		7	400	3.05	66.4	11.6631388
		8	400	3.05	66.4	9.956338
		9	400	3.05	66.4	9.6718712
20	100	2	300	3.8	43.8	4.60836216
		3	300	3.8	43.8	4.267002
		4	300	3.8	43.8	4.6937022
		5	300	3.8	43.8	9.3874044
		6	300	3.8	43.8	7.9650704
		7	300	3.8	43.8	5.689336
		8	300	3.8	43.8	5.689336
		9	300	3.8	43.8	10.2408048
21	100	2	300	3.56	40	11.0942052
		3	300	3.56	40	5.4048692
		4	300	3.56	40	4.8359356
		5	300	3.56	40	4.09632192
		6	300	3.56	40	4.09632192
		7	300	3.56	40	4.77904224
		8	300	3.56	40	4.32389536

Table B.1 (continued)

Batch #	% Slag Replacement	Cylinder #	Cement Factor (pcy)	w/c ratio	% Organic	Strength (psi)
22	100	2	300	3.47	30	3.29981488
		3	300	3.47	30	4.5514688
		4	300	3.47	30	3.6980684
		5	300	3.47	30	3.86874848
		6	300	3.47	30	4.03942856
		7	300	3.47	30	3.47049496
		8	300	3.47	30	3.92564184
		9	300	3.47	30	4.15321528
23	100	1	300	3.29	20	8.8184708
		2	300	3.29	20	3.6980684
		4	300	3.29	20	3.35670824
		5	300	3.29	20	4.38078872
		6	300	3.29	20	3.07224144
		7	300	3.29	20	2.73088128
		8	300	3.29	20	4.60836216
		9	300	3.29	20	3.92564184
52	50	2	300	4.21	43.8	2.78777464
		3	300	4.21	43.8	10.8097384
		6	300	4.21	43.8	3.64117504
53	50	2	400	3.15	43.8	3.92564184
		3	400	3.15	43.8	9.6718712
		4	400	3.15	43.8	4.8359356
		5	400	3.15	43.8	5.1204024
		6	400	3.15	43.8	5.689336
		7	400	3.15	43.8	3.9825352
		8	400	3.15	43.8	4.267002
		9	400	3.15	43.8	5.689336
54	50	3	500	2.51	43.8	9.3874044
		5	500	2.51	43.8	8.8184708
		6	500	2.51	43.8	6.8272032
		8	500	2.51	43.8	8.2495372
		9	500	2.51	43.8	9.1029376

APPENDIX C: CPT GRAPHS FOR SR 33

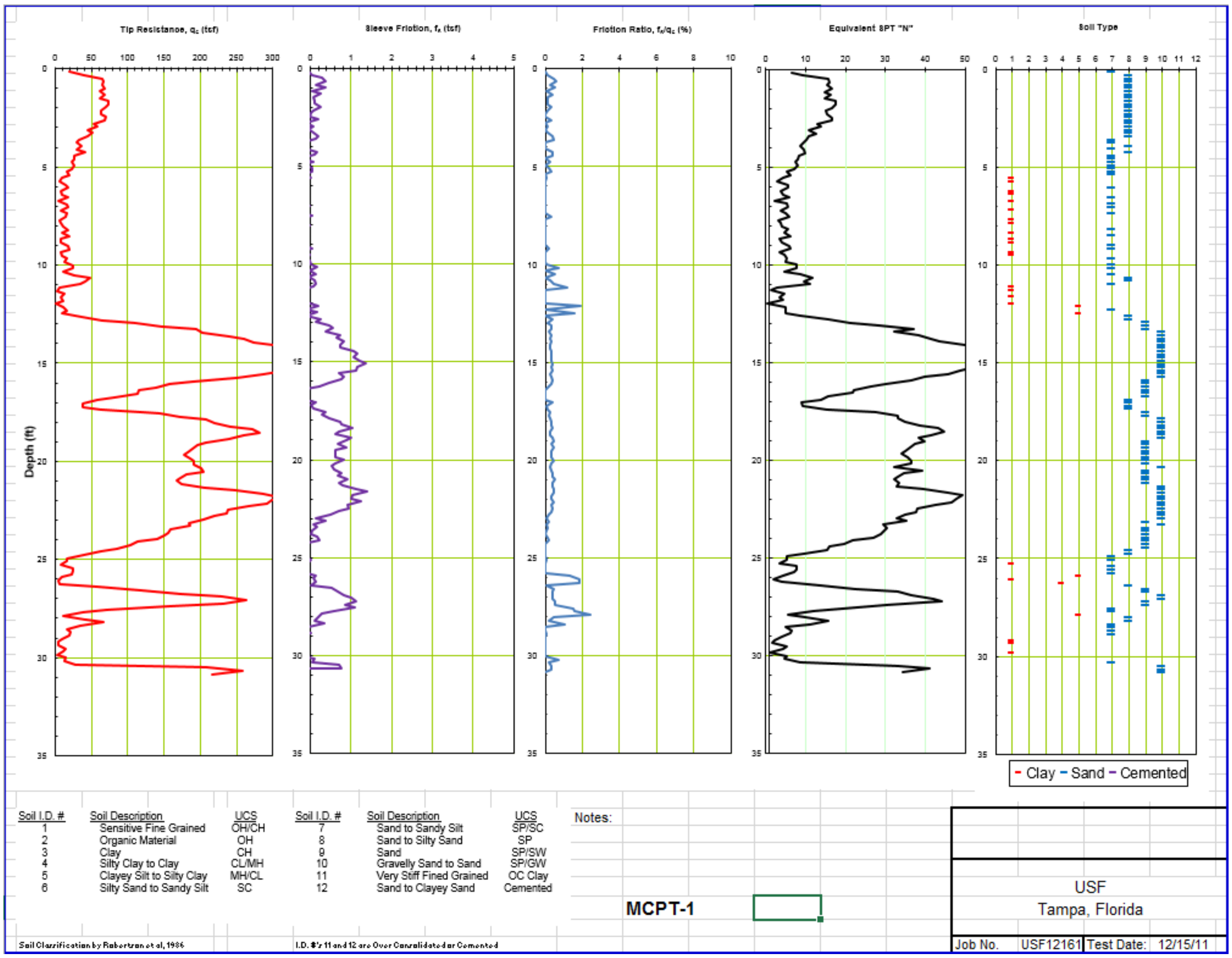


Figure C.1. SR 33 CPT graph 1

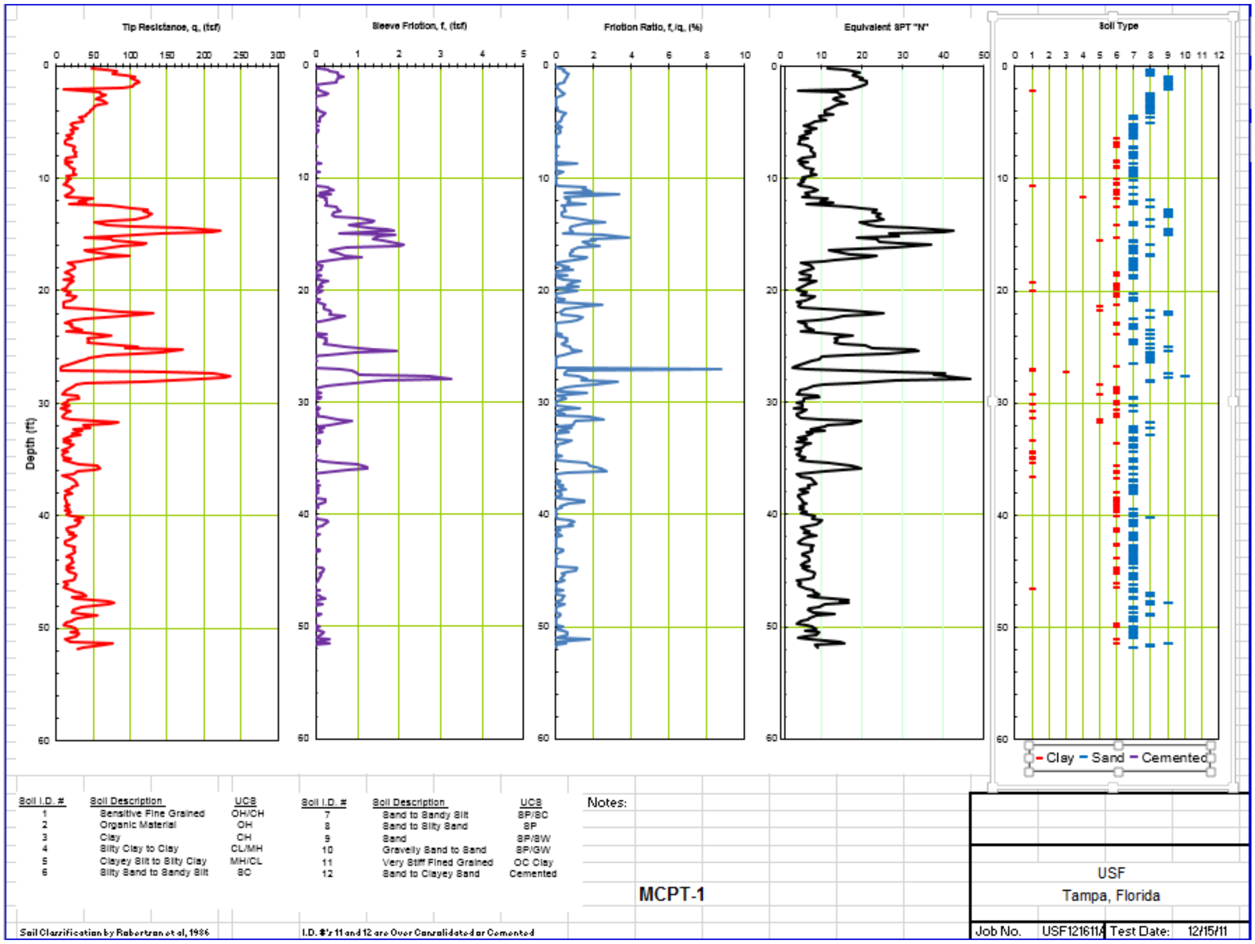


Figure C.2. SR 33 CPT graph 2

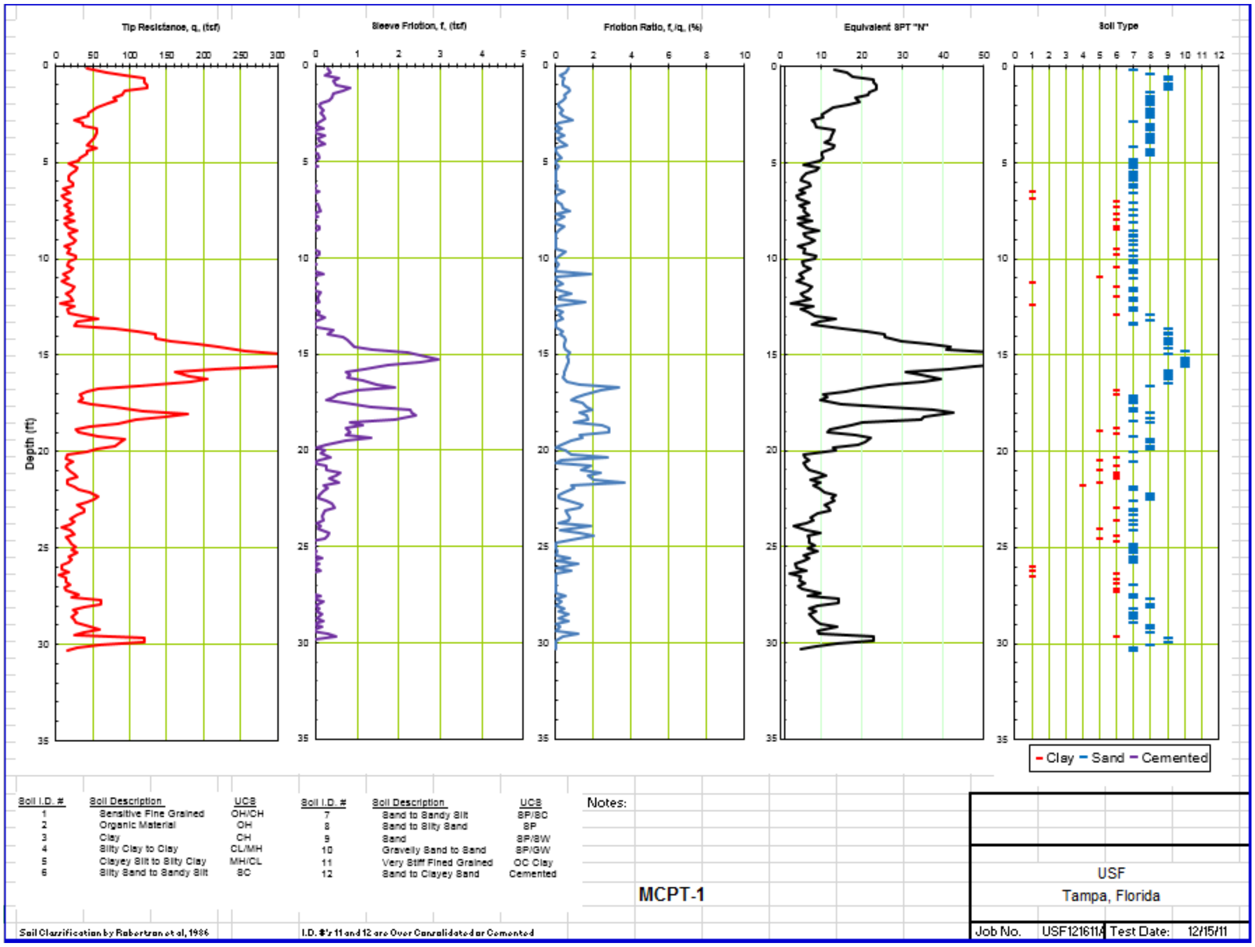
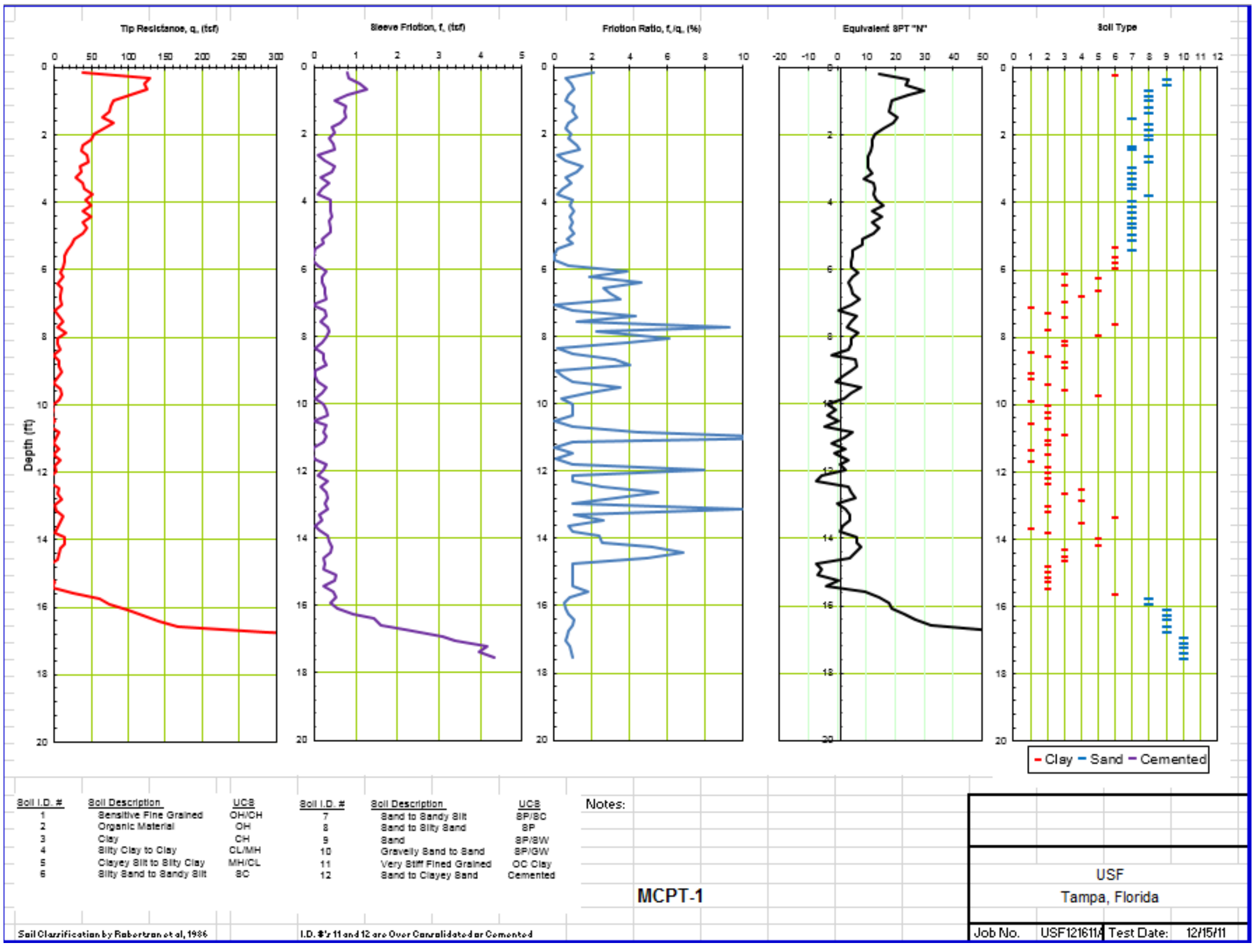


Figure C.3. SR 33 CPT graph 3



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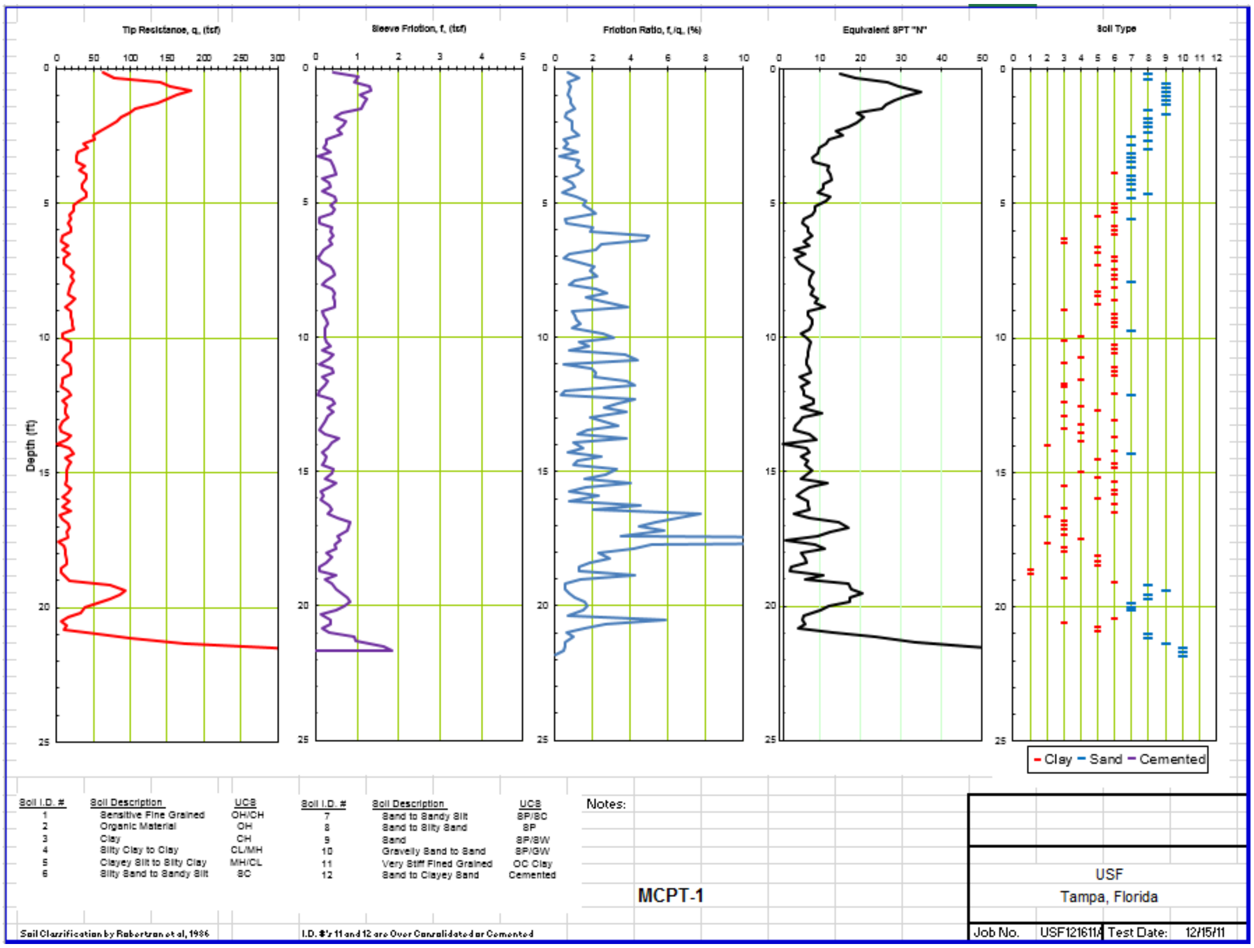

Figure C.5. SR 33 CPT graph 5

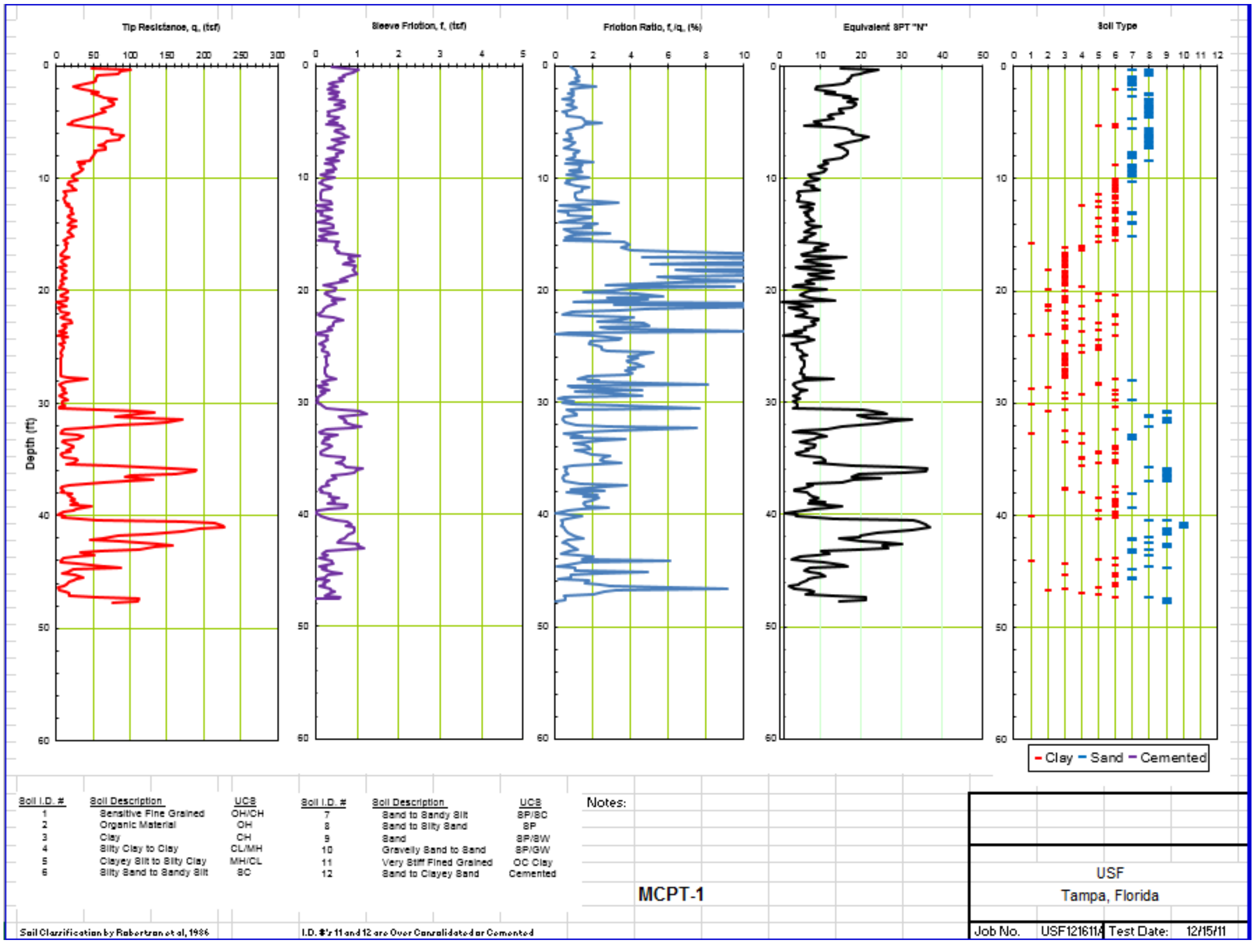

Figure C.6. SR 33 CPT graph 6

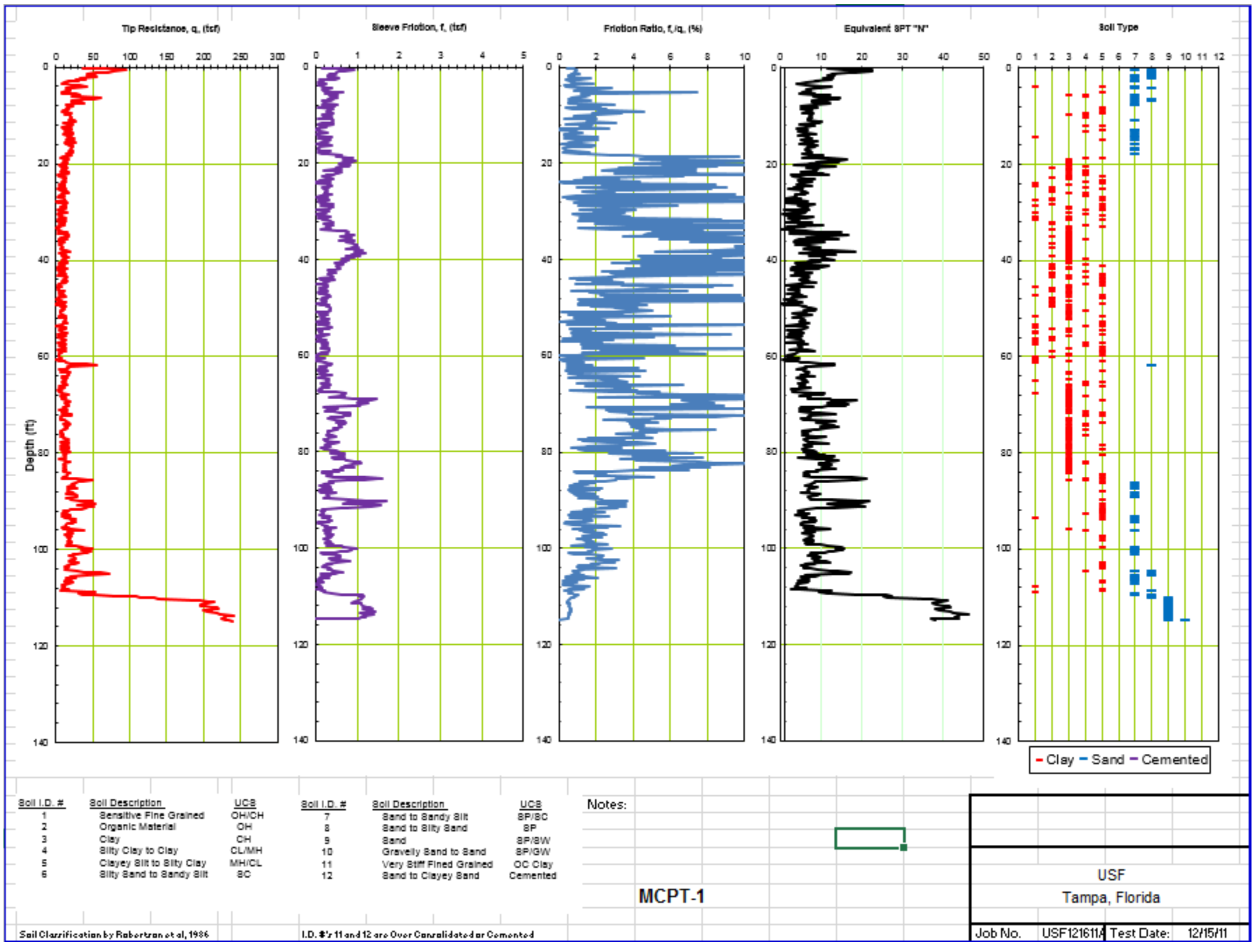

Figure C.7. SR 33 CPT graph 8

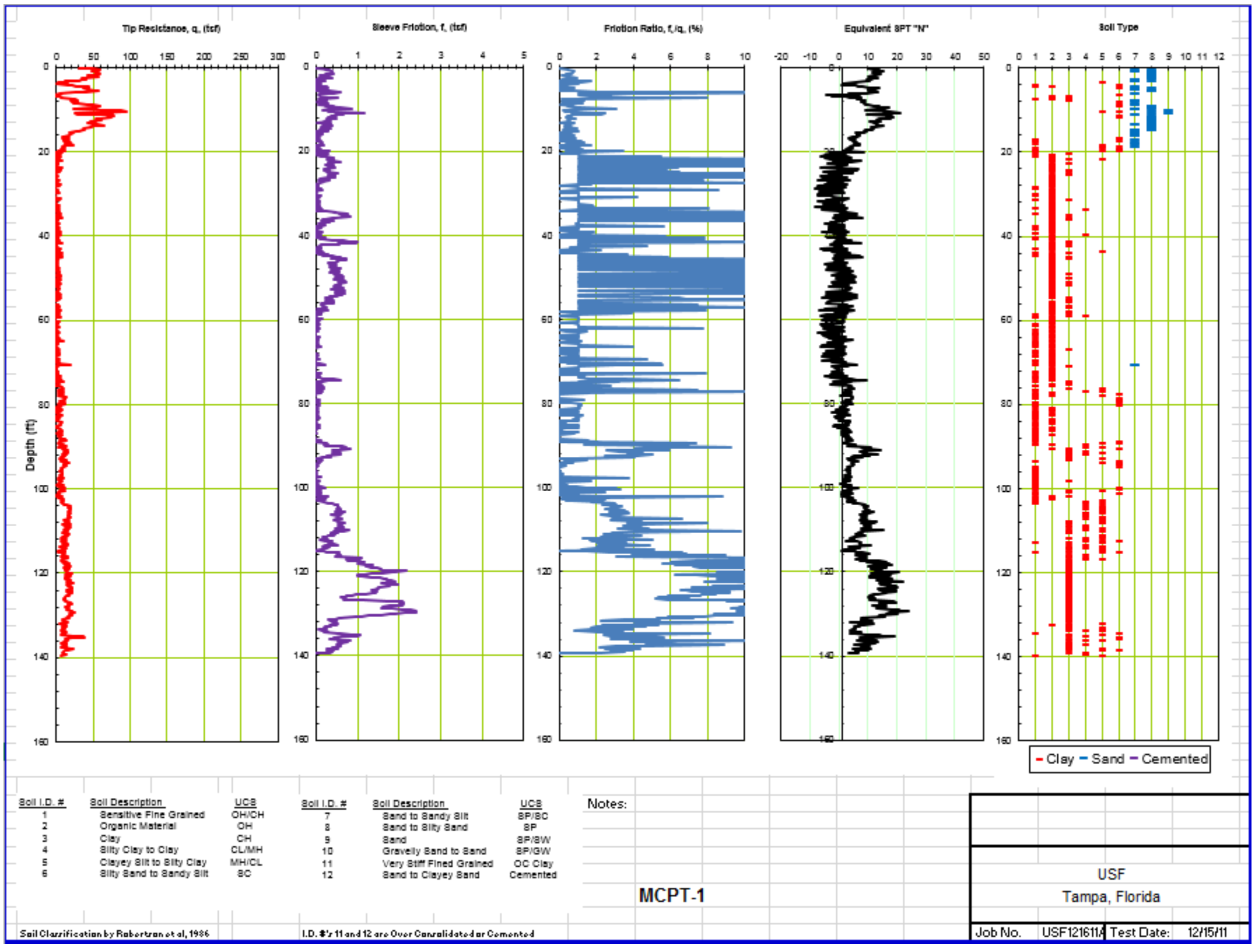

Figure C.8. SR 33 CPT graph 9

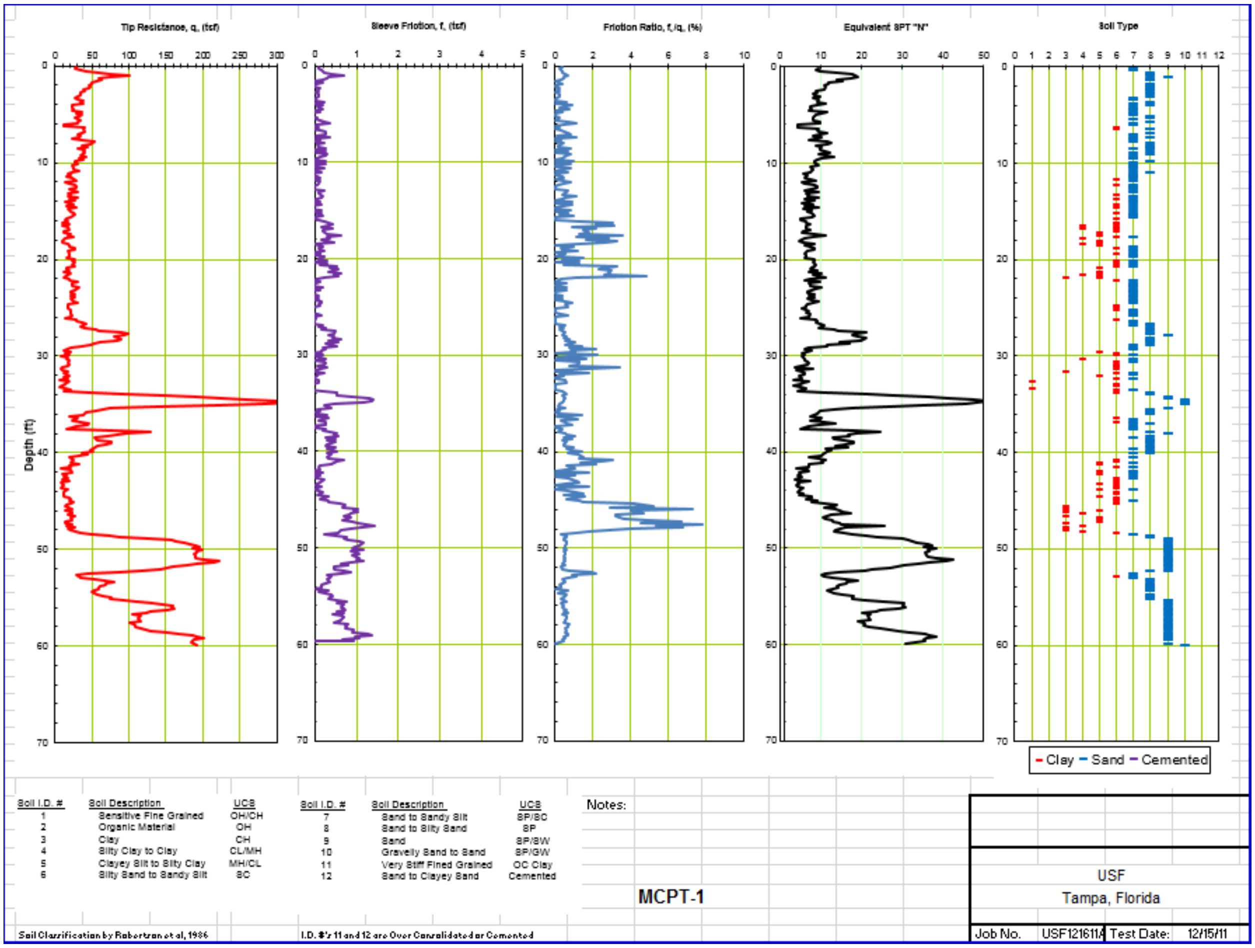

Figure C.9. SR 33 CPT graph 10

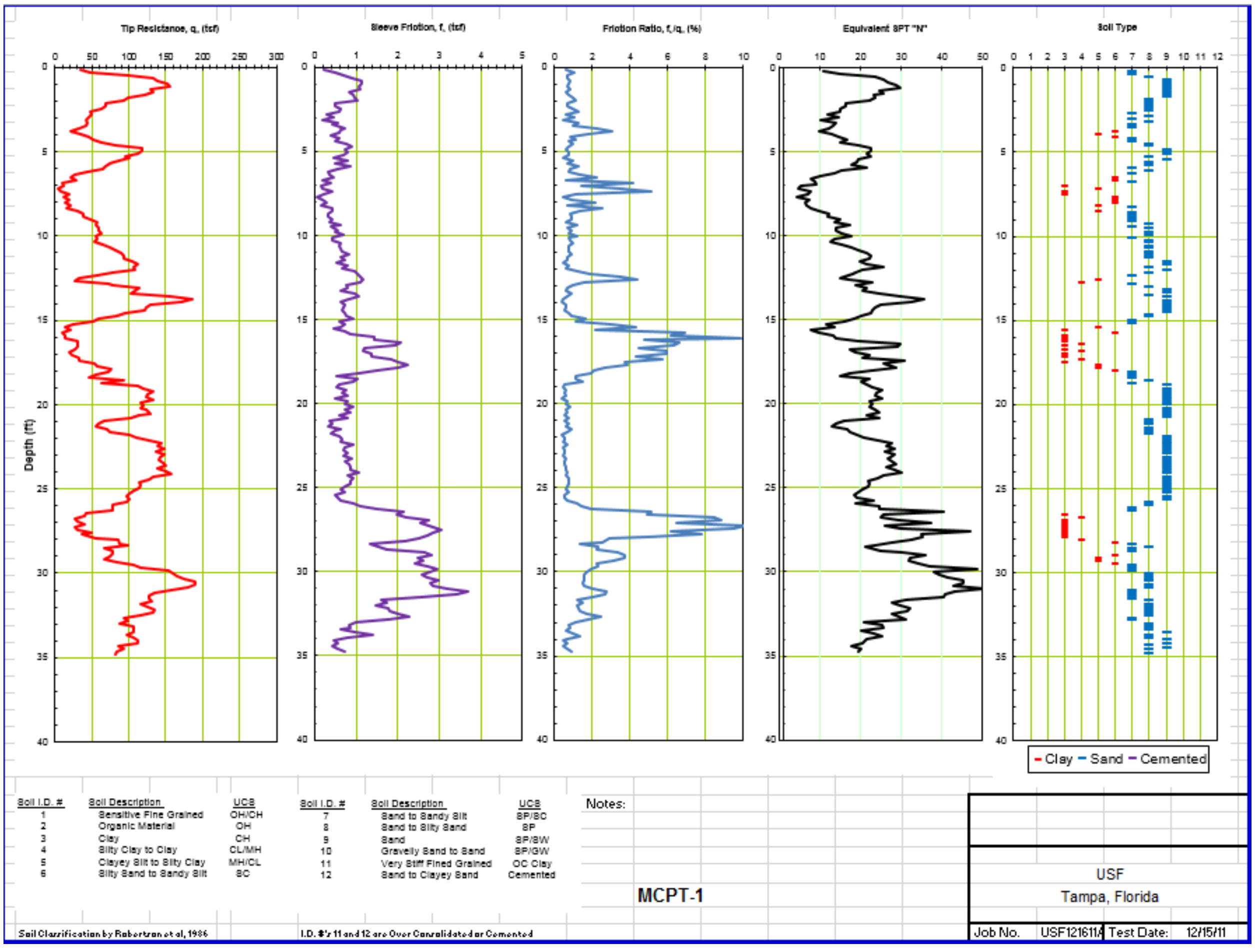

Figure C.10. SR 33 CPT graph 11