

THESIS

EFFECTIVENESS OF POLYMER FOR MITIGATION OF EXPANSIVE SOILS

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## ABSTRACT

### EFFECTIVENESS OF POLYMER FOR MITIGATION OF EXPANSIVE SOILS

The objective of this study was to determine the effectiveness of commercially available polymer treatment as a mitigation technique for expansive soils in transportation applications. Four commercially available polymers were used in this research. A survey of state departments of transportation within the mountain-plains region (Colorado, Montana, North Dakota, South Dakota, Utah, Wyoming) was conducted to define the state-of-the-practice in expansive soil mitigation. A literature review on expansive soil treatments, with a focus on polymer mitigation, was also performed to establish the state-of-the-art in expansive soil mitigation. The soil tested was composed of expansive soil from Fort Collins, Colorado, that classified as low swelling, amended with 15% (high swelling) sodium bentonite. Fifteen percent bentonite was selected to meet the Federal Highway Administration (FHWA) classification for highly expansive soil. Treated and untreated soils were classified, and tested for swelling, strength, and hydraulic conductivity. Four commercially available polymers were tested; lime and fly ash, two common techniques used in treatment of expansive soils, were tested for comparison.

Preliminary swell tests were performed on four commercially available polymers, P1, P2, P3, and P4, to analyze the relative effectiveness of the polymers. P4 was selected for this study based on the high effectiveness of P4 from the swell test results. P4 reduced expansive soil swelling and increased strength, but was less effective than lime or fly ash. Based on reduced swelling, and increased strength, lime was the most effective treatment for stabilizing and strengthening the expansive soil tested. Swell test data do not support use of P4 (or P1, P2, P3) over traditional treatments for swell mitigation of the expansive soil tested in this study. However, lime and fly ash treatments resulted in multiple orders-of-magnitude increases in hydraulic conductivity, while P4 did not. Since water ingress is required for soil swelling, future testing that

couples the effects of hydraulic conductivity and swelling is recommended. In addition, testing of other commercially available polymers, and additional soils (such as sulfate rich soils) is recommended.

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## LIST OF SYMBOLS

<i>NFC</i>	Nelson Farm Clay	<i>w%</i>	water content
FHWA	Federal Highway Administration	<i>V</i>	specimen volume
<i>CEC</i>	cation exchange capacity	<i>Ws</i>	weight of solids in the swell test specimen
<i>LL</i>	liquid limit	<i>hs</i>	height of solids in the swell test specimen
<i>N</i>	number of blows in LL test	<i>dh<sub>1</sub></i>	compression prior to wetting in the swell test
<i>PI</i>	plasticity index	<i>h<sub>1</sub></i>	specimen height prior to wetting in the swell test
<i>SL</i>	shrinkage limit	<i>dh<sub>2</sub></i>	swell/collapse caused by wetting in the swell test
<i>W<sub>%opt</sub></i>	optimum water content	<i>d<sub>i</sub></i>	diameter of Influent pipet
<i>γ<sub>d max</sub></i>	maximum dry density	<i>d<sub>o</sub></i>	diameter of effluent pipet
<i>S%</i>	swell potential	<i>CF<sub>i</sub></i>	correction factor, influent pipet
<i>p<sub>s</sub></i>	swell pressure	<i>CF<sub>o</sub></i>	correction factor, influent pipet
<i>qu</i>	unconfined compressive strength	<i>Pc</i>	cell pressure applied – k test
<i>Su</i>	undrained shear strength	<i>Ph</i>	head backpressure applied – k test
<i>k</i>	hydraulic conductivity	<i>Pt</i>	tail backpressure applied
<i>d</i>	cell diameter	<i>σ'</i>	average effective stress – k test
<i>h</i>	cell height	<i>i</i>	hydraulic gradient

## CHAPTER 1: INTRODUCTION

### 1.1 Research Needs

Expansive soils pose problems to transportation infrastructure throughout much of the Northern Great Plains region of the western United States (Nelson & Miller 1992). Pavements are particularly susceptible to damage from the shrink-swell behavior of expansive soils due the combination of low ground pressures and large surface areas. The pervasiveness of expansive soils in the northern-mountain-plains region is illustrated in Fig. 1.1. When transportation infrastructure cannot be routed to avoid expansive soils, subgrade treatments are often used to mitigate damaging shrink-swell behavior. Traditional subgrade soil treatments are based on methods and technologies primarily developed and refined in the 1950s to 1980s (Petry & Little 2002), and do not incorporate state-of-the-art (i.e., nontraditional) expansive soil stabilizers (for example, refer to the practices described in the Colorado Department of Transportation 2015 Pavement Design Manual, CDOT 2015).

Stabilizers used to mitigate shrink-swell behavior of expansive soils can be divided into two categories, traditional stabilizers (lime, portland cement, and fly ash) and nontraditional stabilizers (organic compounds, salts, potassium, *polymers*, etc.) (Kolay et al. 2016). Shrink-swell reductions with traditional stabilizers are mechanistically based on calcium exchange (swell reducing) and pozzolonic (cementing) reactions. Nontraditional stabilizers, rely on alternative methods for stabilization. For example, potassium-based stabilizers rely on the penetration of potassium ions into the inter-clay-platelet galleries of high swelling smectite clay to form (relatively) lower swelling illite clays.

Use of traditional stabilizers for expansive soil mitigation in transportation earthworks is relatively straightforward, but requires careful design of a soil-specific treatment program, and rigorous quality assurance during implementation. The design program will determine the optimum combination of additive (percent by mass), soil compaction, and soil moisture content to

attain required engineering properties. This program will then be implemented in the field by pulverizing the native expansive soil to a prescribed maximum clod size and to a prescribed depth, in-place mixing of a prescribed mass-percent of additive in dry or slurry form, 24 h – 7 d in-place curing if using lime, and soil compaction to a prescribed range of densities (Little et al. 2000, Petry & Little 2002). The effectiveness of traditional stabilizers has been shown to decrease as soil activity increases, becoming less effective for highly expansive soils (with a plasticity index  $\geq 50$ ; Petry and Little 2002). Traditional soil stabilizers are also ineffective in clayey soils in contact with carbon dioxide leading to carbonation, and in soils containing sulfate salts or with potentially soluble sulfates in response to changes in pH or redox conditions (Petry & Little 2002).

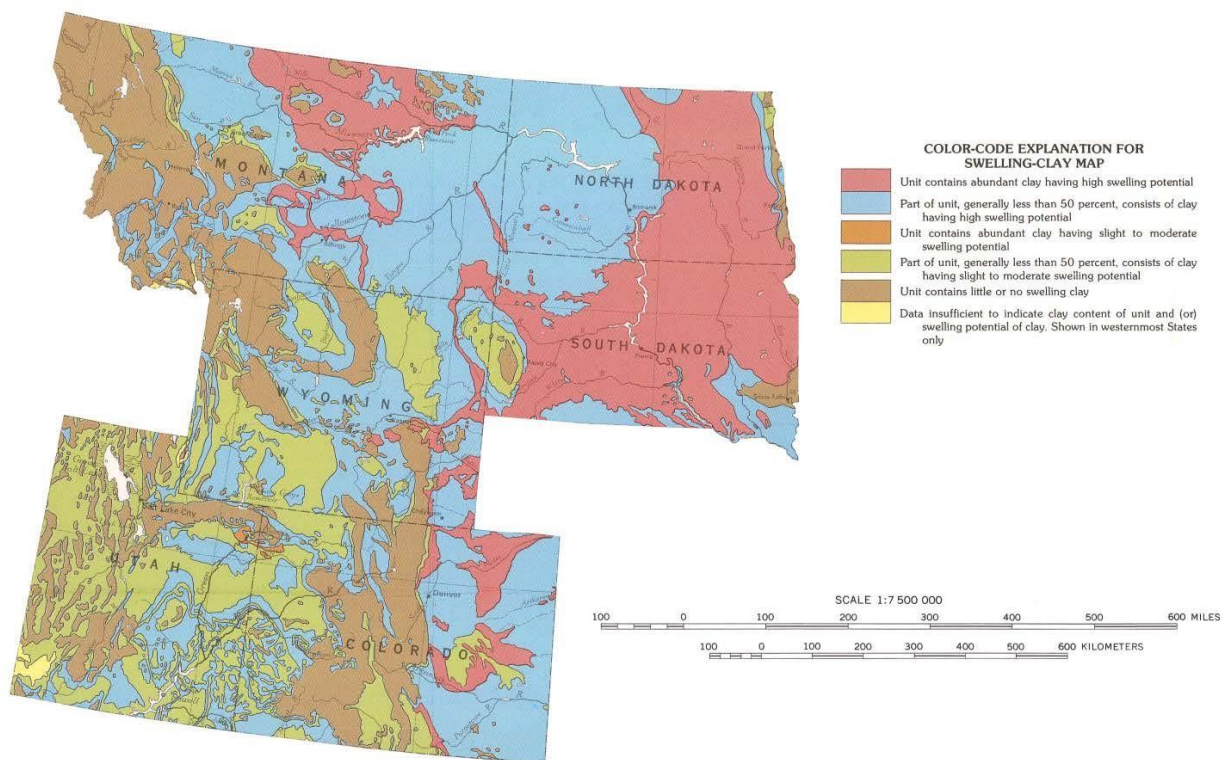


Figure 1.1 Map of expansive soils in the northern-mountain-plains region of the continental U.S. (adapted from Olive et al. 1989).

Numerous nontraditional stabilizers have been previously proffered, and some of these stabilizers have been demonstrated to be effective for specific soil-additive combinations (Petry

& Little 2002). The current state-of-the-art in nontraditional stabilizers is polymer based additives; these polymer stabilizers are commercially available polymers abbreviated as P1, P2, P3, and P4. The name and composition of these polymers are proprietary. Polymer based stabilizers are touted in the manufactures literature to be “effective and green”, “sustainable”, and “cost effective” alternatives to existing soil stabilization technologies. However, adoption of these stabilizers is hindered by both a lack of un-biased information on their effectiveness, and a lack of understanding of the mechanisms by which these materials function (Petry & Little 2002). Understanding mechanisms is fundamental to predicting soil conditions were a specific additive might be effective, and to forecasting long-term behavior in real-world conditions.

Polymer-based stabilization of expansive soils, including stabilization of high swelling sodium montmorillonite, has been demonstrated (e.g., Inyang et al. 2007, Mohammed & Vipulanandan 2016, Azzam 2014, Mousavi et al. 2014). Additionally, extensive scientific literature is available examining the mechanisms of clay-polymer interactions, including mechanisms that reduce or eliminate shrink-swell behavior (e.g., Azzam 2014). However, independent demonstration of the effectiveness (or ineffectiveness) of commercially available polymer stabilizers for expansive soils, and identification of the specific mechanisms through which these stabilizers function, is minimal to nonexistent.

## **1.2 Research Objectives**

The primary objective of this study was to identify and assess the effectiveness of expansive soil mitigation for transportation earthworks by commercially available polymer amendment.

## **1.3 Research Methods**

Research efforts needed to complete this study included i) literature review, ii) material collection, iii) laboratory testing, and iv) data analysis.

### 1.3.1 Literature Review

- Literature review of traditional expansive soil mitigation technologies used in the Mountains Plains region.
- Literature review of commercially available polymer stabilization technologies.

### 1.3.2 Material Identification and Procurement

- Identification and procurement of natural expansive soil encountered in the northern-mountain-plains region.
- Identification and procurement of commercially available polymer-based expansive soil stabilizers.

### 1.3.3 Laboratory Testing

- Laboratory testing of untreated expansive soil, refer to Table 1.1.
- Laboratory testing of traditionally treated expansive soil, refer to Table 1.1.
- Laboratory testing of polymer-treated expansive soil, refer to Table 1.1.

Table 1.1 Laboratory testing program.

Tests \ treatment methods	Units	Test Method	Test objective(s)
Liquid limit, $LL$	[%]	ASTM D4318	Description of expansive soils, assessment of treatment effects on soil properties, and evaluation on polymer-treatment mechanisms
Plastic limit, $PL$	[%]	ASTM D4318	
Plasticity index, $PI$	[%]	ASTM D4318	
Water content	[%]	ASTM D2216	
Optimum water content, $W_{\%opt}$	[%]	ASTM D698	
Maximum dry density, $\gamma_{d\ max}$	[lb/ft <sup>3</sup> ]	ASTM D698	
Swell potential, $S_{\%}$	[%]	ASTM D4546	Assessment of soil swelling potential
Swell pressure, $P_s$	[kPa]	ASTM D4546	
Hydraulic conductivity, $k$	[m/s]	ASTM D5084	Assessment of change of soil permeability from amendment
Soaked unconfined compressive (UCS) strength, $q_u$	[kPa]	ASTM D5102	Assessment of strength gained from amendment

Note: Testing was for untreated, traditionally-treated, and polymerically-treated soil.

The laboratory testing program (Table 1.1) is designed to (1) classify the soil tested, and (2) measure and compare the swelling of the untreated, traditionally treated, and polymer treated soil to assess the impacts of swell mitigation techniques.

#### **1.3.4 Data Analysis**

- Analysis of results from laboratory testing.
- Comparison of potentially reductions in swelling and permeability values, and potentially increases in strength values of traditionally-treated and polymer-treated expansive soil relative to untreated soil.

#### **1.4 Thesis Structure**

This thesis comprises four chapters: Chapter 1 introduced the need for research on the use of commercially available polymers for expansive soil stabilization; Chapter 2, titled “EXPANSIVE SOIL MITIGATION FOR TRANSPORTATION EARTHWORK—STATE-OF-THE-PRACTICE REVIEW,” includes a summary of the traditional mitigation techniques for expansive soils in the US and the northern-mountain-plains region based on a survey conducted with the departments of transportation in the northern-mountain-plains region; Chapter 3, titled “EXPANSIVE SOIL MITIGATION FOR TRANSPORTATION EARTHWORK—STATE-OF-THE-ART REVIEW,” provides background on the use of polymers to treat expansive soils, as well as background on the use of lime and fly ash; Chapter 4, titled “COMPARATIVE ASSESSMENT OF EXPANSIVE SOIL STABILIZATION BY COMMERCIALY AVAILABLE POLYMERS,” presents laboratory testing performed to assess the relative effectiveness of polymers versus two selected traditional stabilizers (fly ash and lime). Finally, Appendices A through M provide details on literature review, testing equipment, procedures and materials, and all supplemental data for the experimental work in Chapter 4.

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## **CHAPTER 2: EXPANSIVE SOIL MITIGATION FOR TRANSPORTATION EARTHWORK — STATE-OF-THE-PRACTICE REVIEW**

### **2.1 Introduction**

The objective of this chapter is to inform engineers and contractors engaged in expansive soil stabilization in transportation earthworks the existing options and practices used in the Mountain Plains Region (Colorado, Montana, North Dakota, South Dakota, Utah, and Wyoming). In this chapter, a literature review summary containing the most commonly practiced stabilizing techniques in the US is presented. A survey was conducted with the mentioned State's Department of Transportation, regarding expansive soils, identification methods, and mitigation techniques and treatments. The filled-out survey and the conclusion derived from the survey is presented in this chapter.

### **2.2 Identification of Expansive Soils**

The two main factors that trigger swelling of soils are unloading and addition of water (Mitchel & Soga 2005). Soil swelling depends on the mineralogical composition and particle size distribution (Yazdandoust & Yasrobi 2010). Clays, particularly those containing significant quantities of smectites, have high swelling potential when hydrated. Swelling and contraction of clayey soils with moisture variation is a phenomenon that causes problems to infrastructures across the globe (Inyang et al. 2007). For example, historically, more than 50% of soil related damage to structures and infrastructure has been due to soil expansion (John & Holtz 1973). Thus, knowledge of soil swelling potential and methods to reduce swelling are important for preventing such undesirable outcomes. Swelling of expansive soils mostly occurs at the upper soil layers where the soil is affected by moisture variations, therefore, knowledge of the active zone depth is important in expansive soils. The active zone depth is defined as the depth where the expansive soil is affected by moisture variations that trigger swelling (Petry & Little 2002).

Direct and indirect laboratory tests can be conducted to identify expansive soils. Direct tests involve measurement of soil swell whereas indirect tests, such as Atterberg limits, involve measuring a corollary property and are the most commonly used identifiers of expansive soils.

The following methods are commonly used to identify expansive soils:

- Swell potential (%) and swell pressure (kPa) are direct methods to measure expansivity of soils. One-dimensional swell tests (ASTM D4546 - Standard Test Methods for One-Dimensional Swell or Collapse of Soils) are used to measure swell potential and swell pressure, which is more time consuming than measuring Atterberg limits. A soil having a swell potential of less than 0.5% is considered as low expansive (Federal Highway Administration report number FHWA-RD-77-94);
- Liquid limit (*LL*) defines the water content where a soil transitions from a plastic to a liquid, and is a measure of the ability of a soil to hold water (ASTM D4318 - Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils). The greater the *LL*, the higher the affinity to water, and the greater the correlated potential degree of expansivity. A soil having a *LL* of less than 50% is considered to be low swelling (FHWA-RD-77-94);
- Plastic limit (*PL*) defines the water content where a soil transitions from a semi-solid to a plastic (ASTM D4318 - Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils). Plastic limit values reduce the plasticity index (*PI*), which is also used as an indicator of stability against swelling. A soil with a *PI* of less than 25% is considered to be low swelling (FHWA-RD-77-94). Moreover, a *PI* of 10% or less is considered as an indicator of a stable (i.e., non-expansive) soil (American Coal Ash Association 2003).

### 2.3 Classification of Expansive Soils

A summary of expansive soils identification and classification metrics from FHWA-RD-77-94, which classifies potential swell based on *LL* and *PI* is provided in Table 2.1. Soil swell potential is categorized as low, marginal, or high.

Table 2.1 Classification of expansive soils from FHWA-RD-77-94

LL, %	PI, %	Potential Swell, %	Potential Swell Classification
> 60	> 35	> 1.5	High
50-60	25-35	0.5-1.5	Marginal
< 50	< 25	< 0.5	Low

Rao (2006) notes that the United States Bureau of Reclamation uses the classification system presented in Table 2.2, based on the work of Holtz and Gibbs (1956). This classification system relies on colloid percent to define the active portions of the soil, coupled with *PI*, and shrinkage limit (*SL*; the water content where a soil transitions from a solid to a semi solid):

Table 2.2 Classification of expansive soils from United States Bureau of Reclamation

Colloid Content, %	PI, %	SL, %	Total Volume Change, %	Degree of Expansion
< 15	< 18	< 10	< 10	Low
13-23	15-28	10-20	10-20	Medium
20-31	25-41	20-30	20-30	High
> 28	> 35	> 30	> 30	Very High

### 2.4 Expansive Soils Treatment Methods

This section includes a summary of the mitigation techniques and treatments commonly used in the United States to mitigate expansive soils. Mitigation methods have been categorized into physical and chemical treatments. The pros and cons of each method also are listed. This summary is based on the work of Petry and Little (2002) and Nelson and Miller (1997). Supplemental references are included as relevant.

### **2.4.1 Physical Treatments**

Permanent mitigation of expansive soils can be achieved by removing the upper layer of the expansive soils within the active zone depth (where the expansive soil is affected by moisture variations). This ideal solution, in many cases, is costly and may not be practical. Physical treatments, in general, involve application of external factors to treat expansive soils rather than internally changing expansive soils to non-expansive. In this section, the most commonly practiced physical treatments are presented.

#### **2.4.1.1 Removal and Replacement**

This method involves removing a specified depth of expansive soil and replacing with a non-expansive (stable) soil. The excavation depth is decided based on the active zone depth, which removes the depth where the expansive soil is most detrimental. Ardani (1992) states that the backfill soil should be silts or low-permeable-non-expansive clays. However, the to-be-replaced soil should not be granular because this simply shifts the problem, due to high rates of permeability. Ardani (1992) states that the Colorado Department of Transportation (CDOT) relies upon *PI* to determine depth of excavation.

#### Advantages:

1. Non-expansive fill material can be compacted to a higher density, which leads to higher strength.
2. Does not require specialized equipment.
3. Requires no soil additives.

#### Disadvantages:

1. Requires available fill material.
2. The high volume of excavation can be costly.
3. Even after excavation, the underlying and adjacent expansive soils often must be protected by horizontal or vertical membranes.

#### **2.4.1.2 Remolding and Compaction**

Remolding and compaction is defined as excavating the expansive soils to a prescribed depth, determined based on the active zone depth, and re-compacting the same soil to a desired density. Remolding and compacting a soil at a moisture content higher than optimum and with a lower density than the maximum dry density results in less swelling (Dubose 1955; Petry & Little 2002).

##### Advantages:

1. Economical for soils with low swelling tendency, high dry density, and low initial water content.

##### Disadvantages:

1. If the active zone depth of the expansive soil is too deep, a drainage system, to minimize access of water to the underlying unmolded expansive soil, is often necessary.
2. Careful control of density and water content are required.
3. This method minimizes, but does not prevent, swelling/collapse.

#### **2.4.1.3 Surcharge Loading**

Swelling can be prevented by applying a pressure to the soil that is greater than the swelling pressure (Ardani 1992).

##### Advantages:

1. This method is good when the expansive soil has low tendency to swell or the overlying structure is heavy.

##### Disadvantages:

1. Determining the active zone is needed to evaluate the maximum potential swell pressure.
2. This method is only applicable for low to moderate expansive clays (Petry & Little 2002).

3. This method is not effective for lightly loaded structures, such as highways, because the loads from these structures are not sufficient to exceed the swell pressure of most expansive soils.

#### **2.4.1.4 Pre-wetting**

Pre-wetting involves ponding of water on the expansive soil to induce initial swelling. This reduces the soil's tendency to swell in the future. This method is more effective for desiccated clays in dry and hot seasons. To facilitate water percolation, sand drains can be drilled in the soil vertically to lessen the time required to provide sufficient water.

##### Advantages:

1. Pre-wetting can be highly effective for desiccated soils.
2. Pre-wetting can be the most economical expansive soil treatment method if done well (Ardani 1992).

##### Disadvantages:

1. A long period of time is required to increase the water content of the expansive soil effectively, up to two years.
2. If not combined with an additive, such as lime, the soil may not be workable and meet strength requirements.
3. Protection of the surface from evaporation is required to prevent shrinkage. Polyethylene sheets can be used after water is injected to the desired depth of the expansive soil to retain moisture (Petry & Little 2002).
4. Uncertainty in specifying a reasonable time of ponding and determining the active zone diminishes the usefulness of this method (Ardani 1992).

#### **2.4.1.5 Horizontal Barriers**

Moisture barriers are horizontally placed to prevent moisture migration to the expansive soils (McDonald 1973). These barriers are typically polymeric geomembranes. Horizontal barriers are typically constructed around buildings, or used in highway shoulders. These barriers remove the source of soil swelling (i.e., addition of moisture).

##### Advantages:

1. Horizontal moisture barriers are effective in preventing moisture intrusion into a specific area.
2. Horizontal barriers do not require extensive excavation or reworking of existing soils.

##### Disadvantages:

1. The length of horizontal barriers must be sufficient to prevent moisture intrusion.
2. Proper techniques are required to attach the barriers to the building foundations.
3. Barriers can be easily damaged during placement, and may be damaged by vegetation.
4. Slopes are required for the barriers to provide proper drainage.
5. The swelling potential of the soil will remain the same after installing horizontal barriers.

#### **2.4.1.6 Vertical Barriers**

Vertical barriers are placed vertically to prevent moisture migration to expansive soils and are typically used in conjunction with horizontal barriers. Vertical barriers can be constructed from asphalt, lean concrete, polyethylene, or by creating capillary barriers using adjacent dissimilar soils. Vertical barriers should be installed to, at least, half of the active zone of expansive soils. These barriers remove the source of soil swelling (i.e., addition of moisture).

##### Advantages:

1. Vertical moisture barriers prevent lateral moisture intrusion into a specific soil volume.

Disadvantages:

1. The backfill materials must be impervious to prevent water accumulation, which can be uneconomical.
2. The swelling potential of the soil will still remain the same after installing vertical barriers.

**2.4.1.7 Membrane Encapsulated Soil Layers (MESL)**

Membrane encapsulated soil layers (MESL) are moisture barrier soil-membranes. MESL are sometimes used with lime and fly ash additives to prevent expansive soils from absorbing moisture (Stark et al. 2000). For highway constructions, MESL is applied over the subgrade and then bent at the ends vertically to a depth of 3 - 4 ft (Falk & Hager 1994) to form both a horizontal and vertical barrier.

Advantages:

1. This method deactivates any moisture migration into the soil.
2. Can be economical for low expansion soils. For high expansion soils, even one puncture in the MESL may allow enough water to reach the soil and cause failure of the MESL with high swelling clays.
3. Can be more effective than soils stabilized with lime in preventing swelling.

Disadvantages:

1. The swelling potential of the soil will remain after installing MESL.
2. Not economical for deep highly expansive soils.
3. The MESL material must be strong enough to withstand potential damages during placement from folds and wrinkles (Falk & Hager 1994).

**2.4.1.8 Asphalt Treatment**

Asphalt treatment involves adding asphalt layers on expansive soils to prevent moisture migration.



Advantages:

1. Easier to fix asphalt failures than failures associated with cement.

Disadvantages:

1. Asphalt must be applied in a continuous manner on subgrades and ditches (in highways) to prevent localized wetting.

**2.4.1.9 Electrochemical Soil Treatment**

Hausmann (1990) defined electro-osmosis as the pulling out of moisture from soils using an electrical potential. Brandon et al. (2009) mention that using electro-osmosis can accelerate flow from the soil when drainage is required or into the soil when applying another stabilizer, such as aluminum. The latter is called electro-kinetic treatment.

Advantages:

1. Dewateres and hardens expansive soil by providing a high concentration of desired exchangeable cations.
2. By placing electrodes into the expansive soil, desired stabilizing solutions can be transferred into the soil.

Disadvantages:

1. Skilled labor is required to apply treatment.
2. This method is typically not economical.

**2.4.1.10 Explosive Treatment for Expansive Shales**

Similar to excavation and re-compaction of the same soil except that instead of using heavy equipment the soil is exploded to restructure the expansive soil layers. Explosives used are typically dynamite or ammonium nitrate & fuel oil mixtures (ANFO).

Advantages:

1. This method can be less costly than removal and replacement.

### Disadvantages:

1. Skilled labor, careful drill procedures, and precise calculation of explosive charges are required (Ardani 1992).

### **2.4.2 Chemical Treatments**

Swelling of expansive soils can be reduced (i.e. soil stabilization can be achieved) by adding materials that i) reduce the affinity of the clay to water, ii) bond the clay particles together (Inyang et al. 2007), and iii) reduce the access of water to the soil. The followings are the commonly used chemical treatments in the US.

#### **2.4.2.1 Lime Treatment**

Quicklime (CaO) treatment relies on the cementitious properties of CaO. Lime inclusion improves the soil strength, compressibility, and swelling. In the late 1960s, lime treatment became, and remains, the most widely used method by Departments of Transportation in the United States (National Lime Association 1991). Lime for soil stabilization is categorized as quicklime and hydrated lime. Lee (1989) suggested that quicklime slurries can be more beneficial than hydrated lime slurries. Calcareous soils with more than 15% calcium carbonate (CaCO<sub>3</sub>) and alkaline soils react well with lime. A lime proportion of 2% - 10% and a mixing depth of 1 - 2 ft can be effective in stabilizing soils.

### Advantages:

1. Lime can increase unconfined compressive strength and produce stable soils that can resist swelling and collapse.
2. Lime can be used dry when enough water is available in the soil, or can be used as a slurry. Before applying lime, leaving the mixed lime for a few days after the final mixing is effective in increasing workability and compaction.

3. Lime can improve pozzolonic reactions and reduce leaching of calcium from expansive soils (McCallister and Petry 1990).

Disadvantages:

1. A curing temperature of over 21 °C is required for up to two weeks to produce proper soil strength.
2. Some components in the lime-treated soils, such as organics, carbon dioxide, iron compounds, and sulfate can slow the pozzolanic reactions of lime leading to strength loss (Mitchell 1986). The source of sulfate in soils could be soil minerals and water. Calcium and aluminum in the treated soil react with soluble sulfates, and produce ettringite, which causes expansion (Majeed et al. 2014).
3. Soil-pH tests are required to determine the percentage of lime needed.
4. The degree of pulverization of quicklime can often be difficult to control. Quicklime needs good pulverization to facilitate the pozzolanic process (Petry & Little 2002).
5. Water, either surface or groundwater, should be prevented from saturating the soil; saturation of lime-treated soils causes the lime to leach reducing treatment effectiveness.
6. Lime has been shown to substantially increase hydraulic conductivity of treated soils (Majeed et al. 2014).

**2.4.2.2 Fly Ash Treatment**

Fly ash is primarily composed of silicon dioxide ( $\text{SiO}_2$ ), aluminum oxide ( $\text{Al}_2\text{O}_3$ ), and calcium oxide ( $\text{CaO}$ ), although the property and composition of fly ash is dependent upon the coal fired. Inclusion of fly ash into an expansive soil decreases plasticity index, hydraulic conductivity, swell potential ( $S\%$ ), and swell pressure ( $P_s$ ) depending on the amount of the fly ash added. Fly ash also increases dry unit weight, shear strength, and resistance to penetration

of the treated soils. Kumar and Sharma (2004) found a 20% fly ash content, which may not be practical, for enhancing the mentioned properties.

Advantages:

1. For silty soils, fly ash can be effective in increasing pozzolanic reactions.
2. Fly ash can also solve problems associated sulfate bearing lime-treated soils (Mccarthy et al. 2009).

Disadvantages:

1. Fly ash often requires combination with lime or other pozzolons.
2. The cost of self-cementitious class C fly ash is relatively higher than the other types of fly ash that require supplemental additives (such as lime or cement).

**2.4.2.3 Cement Treatment**

Similar to lime treatment, cement treatment involves inclusion of an amount of cement (dry or slurry) into the top layer of expansive soils. In the beginning of 1970s, the Portland Cement Association showed that Portland cement could be effective in reducing swelling of soils having low to moderate plasticity. A cement proportion range of 4 - 6% could be effective in stabilizing soils.

Advantages:

1. Generally, cement treatment gives more strength to soils than the other methods.
2. Can be more effective than lime in minimizing shrinkage.
3. Less time is required between applying cement treatment and final mixing for reactions to occur.
4. For soils that are not lime reactive, Portland cement is a good alternative.

Disadvantages:

1. Portland Cement treatment is less effective than lime for clays with high plasticity

2. Cement may cause the stabilized soils to crack more easily
3. More energy is required to produce cement from limestone than lime.

#### **2.4.2.4 Salt Treatment**

Salt treatment includes adding salt solutions, such as  $\text{CaCl}_2$  or  $\text{NaCl}$  into expansive soils to decrease the clay activity. Brandon et al. (2009) states that adding salt changes the ionic composition of the clay and densifies the soil particles, thus leading to greater strength and reduced swelling.

##### Advantages:

1. For soils having high sulfur content, salts ( $\text{CaCl}_2$ ) are good alternatives to lime stabilizers.
2. Salts reduce the freezing and thawing effect on expansive soils, as salts make the soil freeze at a lower temperature (Brandon et al. 2009).
3. Singh and Das (1998) showed that  $\text{NaCl}$  increases unconfined compressive strength and CBR of treated soils.

##### Disadvantages:

1. Only two salt types that can be used economically for soil stabilization are  $\text{NaCl}$  and  $\text{CaCl}_2$ .
2. Salts can leach easily from the stabilized soils, adversely affecting the stability of the soils.
3. Relative humidity as high as 30% should be maintained before salt is applied.

#### **2.4.2.5 Organic Compounds Treatment**

Organic compounds have been tested to stabilize expansive soils but have not been effective or practical for field application (Petry & Little 2002). For example, Tremblay et al. (2002) tested organic compounds with cement to test if mixtures increase the strength of a fine-grained soil. Tremblay et al. (2002) found that no considerable change of strength was seen, however,

the mixture increased soil pH and SO<sub>4</sub> concentration in the soil, indicative of cementing effectiveness. Enzymes are also among the organic treatment methods that have been tested to stabilize expansive soils (Scholen 1992). Tingle and Santoni (2003) found no strength improvement in soils treated with enzymes.

Advantages:

1. Enzymes have large positive charges that causes the negatively charged clay surfaces to neutralize, and have less tendency to react with water, thus, theoretically, making the soil more stable (Scholen 1992).

Disadvantages:

1. Organic compounds have low diffusion rates into expansive soils limiting the effectiveness of the treatments.
2. Many organic compounds are not soluble in water. This insolubility in water may result in decreased reactivity and lower effectiveness.
3. Compared to lime and other treatment technologies, organic compounds have shown inferior stabilizing effectiveness (Petry & Little 2002).

**2.4.2.6 Polymer Treatment**

The exact composition of polymers used in stabilization is generally undisclosed, and only the brand name is provided. Researchers tested various types of polymers for soil stabilization some of which are commercially available (and some of which are not). Two types of polymers are generally available to stabilize soils: natural polymer and synthesized polymer (Brandon et al 2009). Most commercially available polymers are synthesized polymers. Brandon et al. (2009) states that polymer, glues the soil particles together and creates a more stabilized system. Azzam (2014) shows the effect of polymer in creating a nano-composite structure giving the soil more strength and resistance to volume change.

Advantages:

1. Polymers increase unconfined compressive strength, and reduce Atterberg limit, swell potential, swell pressure, and hydraulic conductivity (Azzam 2012).
2. Unlike lime and fly ash, polymers do not require curing, thus, requiring shorter construction times.

Disadvantages:

1. The effectiveness of commercially available polymer treatments is largely unknown.
2. Natural polymers may have leaching and degradation problems.
3. The cost of polymer is generally higher than lime (Brandon et al. 2009).

**2.5 State-of-Practice in Mountain Plains Region**

Colorado State University contacted six States' Department of Transportation (DOT) in the mountains plains region to learn the methods used to mitigate expansive soils. Five of the states responded. Questions were prepared in the form of a survey. The survey summary and the name of the DOT respondents are shown in Table 2.3.

Table 2.3 Survey on expansive soils treatment methods used by five states in the mountains plains region

<b>1. Expansive soils identification: please list the methods your DOT uses to identify expansive soils. How does your DOT decide on the severity of expansive soils based on your identification methods?</b>	
<b>Colorado</b>	We do the swell test, Atterberg limits and gradation analysis. We use AASHTO LRFD Bridge Design Table 10.4.6.3-1 to identify potentially expansive soils.
<b>Montana</b>	Primarily we would use lab testing (swell tests from Shelby tube samples). We might use other methods such as soil property index correlations, or even just visual observation. Severity is based up on the lab results, pavement ride data, or discussion with maintenance personnel or personal experience on the "real world feel" of the issue while driving the roadway.
<b>South Dakota</b>	Geographic location, laboratory testing, knowledge and experience, past performance.
<b>Utah</b>	We identify the expansive soils by observation of damaged pavements. Severity determined by Atterberg limits and soil swell tests.
<b>Wyoming</b>	Swell tests, classification, observations of roadway conditions, and maintenance records.

<b>2. The techniques used to mitigate expansive soils: please list any remedial measures that your DOT has ever taken to eliminate or mitigate swelling problems:</b>	
<b>Colorado</b>	Removal and replacement, remolding and compaction, surcharge loading, pre-wetting, horizontal barriers, lime treatment, fly ash treatment
<b>Montana</b>	Removal and replacement, remolding and compaction
<b>South Dakota</b>	Removal and replacement, Remolding and compaction, Lime treatment, Fly ash Treatment
<b>Utah</b>	Removal and replacement, lime treatment
<b>Wyoming</b>	Removal and replacement, pre-wetting, membrane encapsulated soil layers (MESL), lime treatment



<b>3. Please list the mitigating techniques that have worked best, and why.</b>	
<b>Colorado</b>	For shallow and lightweight structures, such as retaining walls, we recommend a deep foundation system such as piles or caissons to support the structure and prevent differential settlement. Our bridge abutments and piers are also founded on deep foundation system to prevent uplift and settlement. If feasible and not cost prohibitive, removal and replacement of expansive material is also effective. For pavement, lime and fly ash treatment are effective techniques provided that the treated soil does not contain sulfate. Remolding and compaction are common mitigation techniques in pavement construction.
<b>Montana</b>	Removal and replacement has worked, but we haven't really tried too many different techniques.
<b>South Dakota</b>	Removal and replacement - only used for isolated specific design requirements to do associated expense. Remolding and compaction - standard undercut practice utilized for all associated grading projects. Lime treatment - used extensively 1970s on during interstate construction. Results quite variable. Fly ash treatment - used on several projects to mitigate specific fault/heave traces with good results.
<b>Utah</b>	We haven't done much. Besides some excavation and replacement, we replaced an extensive section of rigid concrete pavement with flexible asphalt pavement and incorporated positive surface drainage.
<b>Wyoming</b>	Membrane.

**4. Please list the techniques that have NOT worked, or that your DOT no longer uses, and why.**

<b>Colorado</b>	Partial removal of the expansive material and replacement with granular material does not solve the heave problem. Bath tub situation is created with this mitigation technique and the heave problem persists.
<b>Montana</b>	N/A
<b>South Dakota</b>	Limited use of lime treatment due to previous results.
<b>Utah</b>	Deep dynamic compaction (DDC). The soil was thought to be collapsible when it was actually expansive. DDC did not work at all.
<b>Wyoming</b>	Disruption of shale layers with explosives.

**5. Please provide the names or links of the document guidelines that your DOT uses in dealing with expansive soils.**

<b>Colorado</b>	CDOT Materials Manual and AASHTO LRFD Bridge Design Manual.
<b>Montana</b>	We generally follow FHWA guidelines such as the soil stabilization manual or textbooks on expansive soils. We may also research journal articles.
<b>South Dakota</b>	N/A
<b>Utah</b>	None
<b>Wyoming</b>	FHWA/WY-94/05

**6. Does your DOT use polymer as a stabilization technique? If yes, please explain why? What is the manufacturer/company that provides the Polymer for your DOT?**

<b>Colorado</b>	I am not aware of.
<b>Montana</b>	not yet
<b>South Dakota</b>	no
<b>Utah</b>	no
<b>Wyoming</b>	no

**7. Please provide any additional comments/suggestions.**

<b>Colorado</b>	Draining surface water away from structures using flexible and rigid membranes is a CDOT standard.
<b>Montana</b>	N/A
<b>South Dakota</b>	N/A
<b>Utah</b>	N/A
<b>Wyoming</b>	N/A

**Responders**

<b>State</b>	<b>Name</b>	<b>Position</b>
<b>Colorado</b>	Ilyess Ksouri	Geotechnical Engineer
<b>Montana</b>	Jeff Jackson	Geotechnical Engineer
<b>South Dakota</b>	Kevin Griese	Geotechnical Engineer
<b>Utah</b>	Matt Rink	Senior Design Engineer
<b>Wyoming</b>	Jim Coffin	Chief Engineering Geologist

The following conclusions can be derived from this survey:

1. The DOTs use one or more of the following methods to identify expansive soils; i) Atterberg limits tests, ii) swell tests, iii) visual inspection, iv) spatial and geographic distribution, and v) post-construction and maintenance records. In addition, Federal Highway Administration guidelines are used by Montana and Wyoming DOTs for expansive soils identification.
2. The two common physical treatments used by the DOTs are “removal and replacement” and “remolding and compaction”. Whereas, the two commonly used chemical treatments are lime and fly ash. In addition, MESL is highly recommended by Wyoming DOT.
3. Colorado DOT does not recommend partial removal and replacement because of persisting swelling (the expansive soil should be excavated to the active zone depth). Colorado and South Dakota DOTs report ineffectiveness of lime, a traditional stabilizer, in sulfate-bearing lime treated soils, and post-construction (longevity) issues of lime. Wyoming DOT reports the ineffectiveness of explosive treatment for expansive shales.
4. Polymer is not used for mitigation of expansive soils by any of the DOTs.

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## **CHAPTER 3: EXPANSIVE SOIL MITIGATION FOR TRANSPORTATION EARTHWORK— STATE-OF-THE-ART REVIEW**

### **3.1 Introduction**

The objective of this chapter is to provide a concise summary of the state-of-the art in expansive soil mitigation by fly ash, lime, and polymers. Research shows that use of fly ash and lime in stabilizing expansive soils has been very successful (Kumar & Sharma 2004; Buhler & Cerato 2007). However, there are certain conditions where lime and fly ash lose effectiveness, especially with soils containing sulfur, gypsum, and sulfate salts. The main motivation for researchers to develop nontraditional stabilizers, such as polymers, has been curing time, regional cost, availability, and the potential environmental impacts of the traditional stabilizers such as cement (due to CO<sub>2</sub> generation during production), lime (due to CO<sub>2</sub> generation during production, and potential leaching to waters), and fly ash (due to potential leaching of constituent metals to the environment; Sahviv 1987; Gu & Doner 1993; Liu et al. 2009; Kolay et al. 2016).

### **3.2 Traditional Stabilizers; Fly Ash and Lime**

#### **3.2.1 Fly Ash Stabilization**

Fly ash is a silt-size residual material generated from burning coal. The properties and composition of fly ash are dependent on the composition of the burned coal. Fly ash is primarily composed of silicon dioxide (SiO<sub>2</sub>), aluminum oxide (Al<sub>2</sub>O<sub>3</sub>), and calcium oxide (CaO). Inclusion of fly ash into expansive soils decreases liquid limit (*LL*), plasticity index (*PI*), hydraulic conductivity (*k*), swell potential (*S*%), and swell pressure (*P<sub>s</sub>*), with the decrease in each properties dependent on the quantity and composition of fly ash added (Kumar & Sharma 2004). Fly ash also increases dry unit weight ( $\gamma_d$ ), unconfined compressive strength (*q<sub>u</sub>*), and resistance to penetration of the treated soils ( Sanglerat 1972; Cokca 2001; Kumar & Sharma 2004). Class C, Class F, and Class N fly ashes are the most common types of fly ash (ASTM C618). Kumar and Sharma (2004) showed that a 20% (by mass) fly ash addition, in many cases, is most effective

for expansive soil stabilization. However, 20% fly ash may not be practical for many applications (American Coal Ash Association 2003).

Per ASTM C618, Class C fly ash is typically produced from burning lignite or subbituminous coal. Class C fly ash is relatively more pozzolanic than Class F because a higher CaO content. ASTM C618 defines the chemical and physical characteristics of Class C fly ash. Class C fly ash has self-cementitious properties and does not require additional additives for pozzolonic reactions. Cokca et al. (2009) state that both high calcium and low calcium class C fly ashes can be used for expansive soils stabilization.

Class F fly ash is typically produced from burning anthracite or bituminous coal (ASTM C618). ASTM C618 defines the chemical and physical characteristics of Class F fly ash. Class F fly ash is relatively less cementitious than Class C fly ash because Class F fly ash contains a lower amount of CaO, and therefore requires additional additives such as lime or cement to initiate pozzolonic reactions.

### ***3.2.2 Lime Stabilization***

Lime is generally an effective stabilizer for expansive soils due to cementitious reactions. Quicklime and hydrated lime are both used for soil stabilization. High-calcium lime is recommended over low-calcium lime for stabilizing expansive soils because higher amount of calcium increases the cementitious properties of lime. Lime inclusion improves soil strength, compressibility, and swelling resistances (National Lime Association 2006).

Quicklime is composed of calcium oxide (CaO). Quicklime is produced by burning limestone (calcium carbonate, CaCO<sub>3</sub>) in high temperature kilns, and functions best if pulverized, which maximizes surface area for reactions. ASTM D6276 (Standard Test Method for using pH to Estimate the Soil-Lime Proportion Requirement for Soil Stabilization) is commonly used to determine optimum lime content for soil stabilization. Hydrated lime is composed of calcium hydroxide (Ca[OH]<sub>2</sub>). Hydrated lime is produced from quicklime's reaction with water. Quicklime

is generally recommended over hydrated lime for expansive soils stabilization because quicklime is more reactive than hydrated lime.

### **3.2.3 Stabilizing Mechanisms of Calcium Oxide (CaO) In Fly Ash and Lime**

Calcium oxide is the main soil stabilizing component of lime and fly ash. Lime is composed of CaO, and Class C fly ash may contain a high amount of CaO. Tuncer and Basma (1991) noted that CaO stabilizes expansive soils because of i) cation exchange, ii) flocculation and agglomeration, iii) carbonation reactions, and iv) pozzolanic reactions. The effectiveness of CaO as a stabilizer is due to the followings (Mutaz et al. 2011).

- I. Increases pozzolanic reactions, which lead to long-term strength gain.
- II. Increases resilient modulus, which is defined by Lee et al. (1997) as the ratio of repeated maximum axial deviator stress to recoverable axial deformation.
- III. Increases workability.
- IV. Can have long-term (but not permanent) effectiveness, and can withstand severe seasonal and environmental conditions.
- V. Reduces swelling by exchange of calcium ions with the clay minerals.
- VI. When lime is added into an expansive clay, [OH] increases, which leads to an increase in pH. The increase in pH favors dissolution of alumina and silica within the clay. Dissolved alumina and silica then react with calcium ions, and produce calcium-silicate-hydrate (C-S-H) and calcium-aluminate-hydrate (C-A-H), the two cementing compounds of pozzolanic reactions. Pozzolanic reactions facilitate workability of the clay and cause the clay to be less plastic, less compressible, less expansive, have higher strength, and reduce potential leaching.

### **3.3 Polymer Stabilization of Expansive Soils**

Researchers have tested various types of polymers for soil stabilization, some of which are commercially available. Polymers, in general, have been shown to mitigate expansive soil via increase of  $qu$ , and reduction of  $LL$ ,  $PI$ ,  $S\%$ ,  $P_s$ , and  $k$  (Azzam 2012). The composition of polymers used in stabilization is often not disclosed, and only the brand name is often provided. However, vinyl acetate acrylic based copolymers are the main component of many commercially available polymers (refer to Chapter 3).

#### **3.3.1 Polymer Types for Soil Stabilization**

The following sections list and describe recommended polymers based on academic research, and commercially available polymer brands.

##### **3.3.1.1 General Polymers**

The followings include recommended polymers by a number of researchers.

1. Sodium carboxymethyl cellulose (Inyang et al. 2007)
2. Polyacrylamide (Inyang et al. 2007)
3. Polyethylene oxide polymers (Inyang et al. 2007).
4. Urea formaldehyde (Yazdandoust & Yasrobi 2010)
5. Melamine formaldehyde (Yazdandoust & Yasrobi 2010)
6. Expanded polystyrene (Illuri & Nataatmadja 2007)
7. Polypropylene homopolymer (Azzam 2012)
8. Propylene (Azzam 2014)
9. Furan (Mirzababaei et al. 2009)
10. Poly(methyl methacrylate) (Mirzababaei et al. 2009)
11. Poly(vinyl acetate) (Mirzababaei et al. 2009)
12. Lignosulfonate (natural powder polymers) (Brandon et al. 2009)
13. Hydroxyl- aluminum (synthetic gel polymer) (Bryn et al. 1984)



### **3.3.1.2 Commercial Polymers**

The following list includes names of a number of commercial polymers.

1. EnviroTAC® (vinyl acetate acrylic based liquid copolymers)
2. SoilTAC® (vinyl acetate acrylic based liquid copolymers)
3. Road Packer Plus Polymer (cationic-exchanger liquid polymers) (Mousavi et al. 2014)
4. BaseBind® EnviroTech (lignosulfonate powder polymers) (Brandon et al. 2009)
5. Wacker Etonis® 930 (powder polymers) (Kavak et al. 2010)
6. Aggrebind® (liquid polymers)
7. GRT9000® GRT (liquid polymers)
8. Rovene® 6126 MPC (styrene acrylic liquid polymers)
9. SealMaster® (liquid polymers )
10. T-PRO® 500 (latex liquid polymers)
11. Soil-Sement® (liquid polymers)
12. DeltaGreen® PM10-50 (liquid polymers)
13. DirtGlue® Industrial Polymer (liquid polymers)
14. SoilTech Mk. III (liquid polymers)

### **3.3.2 Expansion Mitigation Results by Polymers**

In general, polymers increase mechanical properties of the treated soils leading to higher elastic modulus, lower gas permeability, and higher strength (Azzam 2012). Specific results are described subsequently for non-commercial polymers studied by academic researchers, and commercial polymers.

#### **3.3.2.1 Non-commercial Polymers**

##### **3.3.2.1.1 Sodium Carboxymethyl Cellulose, Polyacrylamide, and Polyethylene Oxide Polymers**

Inyang et al. (2007) studied the effectiveness of aqueous solutions of three polymers (anionic sodium carboxymethyl cellulose, CMC, cationic polyacrylamide, PAM, and neutral

polyethylene oxide, PEO) as soil stabilizing agents for Na-montmorillonite. Inyang et al. (2007) concluded that cationic PAM polymer results in up to four times lower volumetric swelling ratio levels than that of water depending on the aqueous concentration ratio (0.5 to 4 g/L concentrations were tested). Inyang et al. (2007) showed that anionic CMC and neutral PEO effects on reducing swelling were insignificant.

#### 3.3.2.1.2 Urea Formaldehyde and Melamine Formaldehyde

Yazdandoust and Yasrobi (2010) studied the effect of repeated wetting and drying on expansive clays rich in Na-montmorillonite treated with urea formaldehyde and melamine formaldehyde polymers. The polymers reduced swelling and shrinkage through repeated wetting and drying cycles. Yazdandoust and Yasrobi (2010) reported that polymers were generally more effective than lime against cyclic wetting and drying because lime lost effectiveness after a few cycles of wetting and drying.

#### 3.3.2.1.3 Expanded Polystyrene

Expanded polystyrene (EPS) is a cellular solid polymer that has a unit mass of 13-20 kg/m<sup>3</sup>. Illuri and Nataatmadja (2007) studied varying-size crushed waste EPS particles for expansive soil stabilization. Illuri and Nataatmadja (2007) concluded that EPS inclusion into expansive soils reduces  $S\%$ ,  $P_s$ , volumetric shrinkage (TXDOT's Tex-101-E), and crack intensity of expansive soils.

#### 3.3.2.1.4 Polypropylene Homopolymer

Azzam (2012) studied polypropylene homopolymer for expansive soil stabilization, and concluded that polypropylene homopolymer reduces  $PI$ ,  $k$ ,  $w_{\%opt}$ ,  $\gamma_{d\ max}$ ,  $S\%$ ,  $P_s$ , and volumetric shrinkage of the expansive soils. Simultaneously, polypropylene homopolymer inclusion increased  $qu$ , bearing capacity, and ductility.

#### 3.3.2.1.5 Propylene

Azzam (2014) showed that propylene polymers act as nano-fillers that can fill microscale pores of expansive soils, and can reduce  $PI$ ,  $k$ , volumetric shrinkage, and compressibility of expansive soils, and increase  $\gamma_{d \max}$  and  $qu$ . Azzam (2014) stated that propylene can effectively be used for road embankments and slope stabilization.

#### 3.3.2.1.6 Furan, poly(methyl methacrylate), and poly(vinyl acetate)

Mirzababaei et al. (2009) studied the swell mitigation potential of water soluble Furan, poly(methyl methacrylate), PMMA, and poly(vinyl acetate), PVA, for expansive soil mitigation. Mirzababaei et al. (2009) found that Furan and PMMA effectively decreased the swell potential (from 22% to 0.4% and 13%, respectively). However, the poly(vinyl acetate) polymer did not reduce the swell potential.

#### 3.3.2.1.7 Lignin or lignosulfonate (natural polymers)

Natural polymers such as lignin or lignosulfonate are known to have a “gluey effect” on expansive soils, which reduces swell tendency. The problem with natural polymers, however, is that natural polymers may leach from the soils with time (Brandon et al. 2009). Natural polymers are also more likely to biodegrade. BaseBind® EnviroTech is a natural commercially available lignosulfonate powder polymer.

#### 3.3.2.1.8 Hydroxyl-aluminum (synthetic polymer)

Bryn et al. (1984) showed that hydroxyl-aluminum could increase  $qu$  of sensitive clays more than quicklime based on laboratory testing. However, in the field, lime could be more effective at increasing shear strength than hydroxyl-aluminum polymer depending on the mixing technique of the hydroxyl-aluminum polymer with the soil (Brandon et al. 2009).

### **3.3.2.2 Commercial Polymers**

#### **3.3.2.2.1 Road Packer Plus Polymer**

Road Packer Plus ® (RPP) is a “liquid cation-exchanger polymer” manufactured in Canada; the composition of RPP is proprietary. Mousavi et al. (2014) showed that the < 0.1% RPP polymers decreased  $LL$ ,  $PI$ ,  $S\%$ , and  $P_s$  of the treated soils, and increased California bearing ratio (CBR) and  $\gamma_{d\ max}$ .

#### **3.3.2.2.2 Wacker Etonis® 930**

Wacker ETONIS® 930 is a powdered dry polymer that can be mixed with soils before water addition. Kavak et al. (2010) studied the effect of 1% Wacker ETONIS® 930 in stabilizing road base material mostly composed of crushed stone, and concluded that ETONIS® 930 reduces maximum and permanent deformations of the base and the subbase of the road. Furthermore, Wacker Etonis® improved the treated soils in the following ways: i) promoted the strength and durability of the soils, ii) decreased  $\gamma_{d\ max}$  and  $w_{\%opt}$  due to Wacker Etonis® polymer's replacement effect, and iii) increased the CBR of the soils.

### **3.3.3 Stabilizing Mechanisms of Polymers**

Polymers, in general, may interact with clay particles in the following ways:

- Ion dipole interaction: clay exchangeable cations adsorb ionic or nonionic polymer molecules.
- Hydrogen bonding: bonding between the hydrogen and oxygen of the clay occurs with the polymer molecules adsorbed.
- Van der Waals forces: relatively weak forces between all molecules, ions, and atoms.

Polymer molecules may adsorb to clay particle surfaces by the previously listed mechanisms and link the particles together (Luckham & Rossi 1999). Generally, copolymers stabilize clay particles in three ways: i) absorbing water, ii) exchanging ions between polymers and the clay, and iii) establishing a medium that hardens the clay (Rafalko 2008).

Based on scanning electron microscope (SEM) image analyses results polypropylene homopolymer has been shown to improve expansive soil skeleton in the following ways:

- Changes the aggregate of montmorillonite platelets into an inhomogeneous distributed aggregate.
- Increases electrical attraction between the treated grain particle surfaces that causes the surfaces to react less with water (Azzam 2012).

Acrylic copolymers units, of which most of the commercial polymers are composed, have positively-charged water-insoluble dipoles that create a strong connection between the polymer units and the clay particles. These heads attract to the negatively charged clay surfaces, and prevent adsorption of water and increasing shear strength and CBR of expansive soils (Scholen 1995).

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## **CHAPTER 4: COMPARATIVE ASSESSMENT OF EXPANSIVE SOIL STABILIZATION BY COMMERCIALY AVAILABLE POLYMERS**

### **4.1 Introduction**

The goal of this research is to analyze the effectiveness of commercially available polymers as soil stabilizers. A highly expansive soil was used for all tests. Lime, fly ash, and four commercially available polymers were tested, as well as control tests on the unamended highly expansive soil. Fly ash and lime treatments were used to compare the effectiveness of soil treated with polymers. This chapter presents the materials and methods used to identify the behavior of a highly expansive soil with and without treatment with commercial polymers, the results of testing, and explores relative treatment effectiveness.

Improvements achieved from a chemical stabilizer should involve decreased swelling and increased strength (to maintain stability), and decreased permeability (to minimize moisture intrusion into underlying untreated layers that may trigger swelling). Therefore, testing included unconfined compressive strength, swelling, and hydraulic conductivity to assess potential soil improvements from polymer treatment. The expansive soil used in this research is a high swelling clay-bentonite mixture that compose 85% of a low expansive clay (LL= 31 and PI=13) and 15% of a highly expansive clay (i.e., natural sodium bentonite; LL= 420 and PI= 381). Four commercially available polymers were evaluated for this study. Lime, polymer, and fly ash with, 3%, 5%, and 15% additions (by mass) were each added to the base soil to assess the relative effectiveness of polymers; the lime and fly ash ratios were selected based on current practices. The results of testing are described, the effectiveness of the commercially available polymers tested to mitigate expansive soils is discussed, and limitations of this study are described.



## 4.2 Background

Swelling and shrinkage of clayey soils with moisture variation is estimated to be the cause of more than 50% of soil related damage (John & Holtz 1973), deteriorating infrastructure around the world (Inyang et al. 2007). The destructing power of expansive soils has motivated engineers to study and apply numerous methods to mitigate the swelling properties of expansive soils. There are two main approaches to stabilize expansive soils, physical and chemical treatments. Physical treatments include removal and replacement, remolding and compaction, pre-wetting, and horizontal and vertical geomembrane barriers. Chemical treatments include application of additives such as lime, fly ash, cement, polymers, salts, and organic compounds. Chemical stabilizers are often preferred because they generate a stable structure, especially after a few days/weeks of curing or reaction. There are two main mitigating categories for chemical stabilizers used in treatment of expansive soils, traditional and nontraditional stabilizers. Traditional stabilizers include fly ash, lime, and cement, all with pozzolanic properties, whereas nontraditional stabilizers include polymers, salts, and enzymes (Kolay et al. 2016).

Despite the effectiveness of traditional stabilizers in many cases, there are certain conditions where nontraditional stabilizers are preferred. Problems associated with traditional stabilizers, such as lime and fly ash, include, i) limited effectiveness with very active soils, i.e., soils with a plasticity index ( $PI$ ) greater than 50; ii) limited effectiveness with expansive soils containing carbon dioxide (which leads to carbonation), sulfate salts, gypsum, or sulfur (which leads to ettringite formation that leads to swelling); and iii) environmental concerns with stabilizer production and placement. Advantages of nontraditional soil stabilizers, such as polymers, include curing time, cost (potentially), availability, and reduced environmental impacts associated with production (Petry & Little 2002).

Polymers based stabilization has been tested as a potential technology to solve problems associated with traditional stabilizers (Illuri & Nataatmadja 2007; Inyang et al. 2007; Brandon et al. 2009; Mirzababaei et al. 2009; Kavak et al. 2010; Yazdandoust & Yasrobi 2010; Azzam 2012;

Azzam 2014; Mousavi et al. 2014). The composition of polymers used in stabilization is generally not disclosed, and only the brand name provided. However, vinyl-acetate acrylic based copolymers are the main component of many commercially available polymers (and is the main ingredient in P1, P2, P3, and P4). Previous researchers have shown that polymers, in general, can be used to increase the unconfined compressive strength, and reduce Atterberg limit, swell potential, swell pressure, and hydraulic conductivity of expansive soils (Azzam 2012).

## **4.3 Materials**

### **4.3.1 Expansive Soil**

The soil used in this study was a low plasticity clay obtained in Fort Collins, Colorado. This soil has previously been named Nelson Farm Clay (NFC). Based on the Unified Soil Classification System (ASTM D2487), NFC classifies as a low plasticity clay or CL. This soil was initially chosen as a representative expansive soil control. However, based on the Atterberg limit tests, NFC fell under the “low” category of the potential swell classification of FHWA-RD-77-94. In addition, swell tests on NFC (discussed in the results section) confirmed the low expansivity of NFC.

To provide a more robust assessment of polymer treatment, NFC was amended with a natural sodium bentonite from Wyoming, USA. The sodium bentonite used in this study is described in detail in Bohnhoff and Shackelford (2014), and classifies as a CH based on the USCS (ASTM D2487). Bohnhoff and Shackelford (2014) report a liquid limit of 420, and a plasticity index of 381 for the sodium bentonite used in this research. Additional characteristics of the sodium bentonite used are provided in Bohnhoff and Shackelford (2014).

Liquid limit (*LL*) tests were used to determine the bentonite addition ratio necessary to elevate the *LL* and swell potential of the soil to meet the FHWA-RD-77-94 classification for a highly expansive soil. Unamended NFC had a *LL* of 30.9. As shown in Table 2.1, a *LL* greater than 60 is required for a high swell classification in FHWA-RD-77-94. Therefore, different bentonite blends were combined with 60% water (by mass), and tested to assess an appropriate

bentonite addition percentage to close the groove in 25 blows (as specified in ASTM D4318). The following bentonite addition levels were tested:

- 10% bentonite addition: ~60% water was added to a 90% NFC and 10% bentonite mixture (by mass). The Resultant mixture behaved like a fluid and had a blow count of 3 to close the groove during the *LL* test,  $n=3$ . Therefore, a higher bentonite content was required.
- 30% bentonite addition: ~60% water was added to a 70% NFC and 30% bentonite mixture (by mass). The resultant mixture was dry, and required more than 150 blows to close the groove during the *LL* test,  $n > 150$ . Therefore, a lower bentonite content was required.
- 15% bentonite addition: ~60% water was added to an 85% NFC and 15% bentonite mixture (by mass). The soil mixture was dry, and required more than 100 blows to close the groove during the *LL* test,  $n > 100$ . The mixture was then wetted to ~80% (by mass). The soil mixture was wet with less than 10 blows required to close the groove during the *LL* test,  $n < 10$ . These results, i.e.  $n > 100$  and  $n < 10$ , set the range of *LL* for the 85% NFC and 15% bentonite mixture to be between 60% and 80% ( $60 < LL < 80$ ). The *LL* of the mixture was determined to be 75.8%, which met (and exceeded) the FHWA-RD-77-94 classification for a highly expansive soil. A 15% bentonite and 85% NFC mixture was used for all tests. This soil will be referred to as “expansive soil” henceforth.

#### **4.3.2 Fly Ash Treated Soil**

Fifteen percent (by mass) of a self-cementitious Class-C fly ash (ASTM C618) was added to the expansive soil to test fly ash treatment. The choice of this 15% fly ash addition was based on American Coal Ash Association (2003) guidance, which states a typical fly ash addition to be 12 – 15%. The upper end of this range was chosen to maximize the effect of fly ash, and provide

a conservative comparison. Sample preparation of fly ash treated soil for each test is discussed in Section 4.4.

#### **4.3.3 Lime Treated Soil**

Three different percentages of quicklime (CaO) were mixed with expansive soil to create lime-treated soil mixtures. ASTM D6276, the “Standard Test Method for Using pH to Estimate the Soil-Lime Proportion Requirement for Soil Stabilization (Withdrawn 2015)”, was initially attempted to determine the optimum lime-addition content. ASTM D6276 relies on pH of varying lime proportions to determine optimum lime content. This method is also known as the “Eades-Grim” test, and requires a minimum pH of 12.4 for a soil-lime mixture to determine the optimum lime content for stabilization. This method was attempted with quicklime and hydrated lime with varying additions, however, no reasonable results were obtained, viz. application of “Eades-Grim” test was not successful. In addition, the pH of lime-water solutions were below 11.8, which does not meet the minimum pH requirement of ASTM D6276 (refer to Appendix I for additional information). Reasons for failure of this test include i) the quality of the tap and deionized water used in the mixtures, and ii) the reactivity of the lime in the mixture solutions. Of note, ASTM D6276 was withdrawn with no replacement in 2015.

Because no successful results were gained using ASTM D6276, a practical quicklime addition of 3.5 – 5% was considered based on the recommendation of Akawwi and Al-Kharabsheh (2002). Three percent quicklime addition (by mass) was chosen by evaluating this recommended range (i.e. 3.5 – 5%) by Akawwi and Al-Kharabsheh (2002) and swell tests performed as part of this study with varying lime addition percentages. The sample preparation of lime treated soil for each test is discussed in Section 4.4.

#### **4.3.4. Commercial Polymers**

Four commercially available polymers were used in this research. The polymer types are not disclosed herein, and henceforth will be denoted as P1, P2, P3, and P4. P1 is a vinyl

copolymer, which is claimed by the manufacturer to be both a “soil stabilizer” and dust controller. P2, P3, and P4 are also vinyl copolymers, and are claimed to be “soil stabilizers.” Section 2.4.2 includes criteria for a soil stabilizer. Swell tests were run to determine which polymer minimized swelling; P4 was chosen as the most stabilizing polymer addition based on swell test results (described in Section 4.5.4.1). Five percent addition of P4 was chosen based on the results of swell tests (Appendix A) and a practical application rate stated on the P4 manufacturer website.

## **4.4 Methods**

### **4.4.1 Atterberg Limits Tests**

ASTM D4318 was followed for the Atterberg limit testing. For *LL* tests, Casagrande liquid limit device was used. No curing or hydration period was provided for the treated or untreated specimens, i.e., the Atterberg limit tests were run immediately after mixing because Atterberg limit values were only used for classification and relative *LL* and *PI* reduction effects rather than analyzing curing and stabilizing effects, which should be achieved by the tests mentioned in sections 4.4.3, 4.4.4, and 4.4.5. The following describes *LL* sample preparation for each soil mixture.

- For untreated NFC, the soil was passed through the #40 sieve, tap water was added, and the *LL* test was run.
- For expansive soil, NFC was passed through the #40 sieve, and was mixed with powdered bentonite at an 85:15 mass ratio (the mixing ratio of expansive soil). The two materials were manually mixed in a clean bowl in a dry condition until no heterogeneity and particle segregation were visually observed in the mixture.
- For fly ash treated soil, expansive soil was mixed with the fly ash at a 100:15 mass ratio. Expansive soil and fly ash were mixed in a clean bowl in a dry condition until no heterogeneity and particle segregation were seen in the mixture.

- For the lime treated soils, quicklime was crushed, passed through the #40 sieve, and then mixed with expansive soil at an addition of three percent (i.e., the ratio of expansive soil to crushed quicklime was 100:3). Expansive soil and lime were mixed in a clean bowl in a dry condition until no heterogeneity and particle segregation was seen in the mixture.
- For P4 treated soil, a mixing ratio of polymer to expansive soil of 100:5 was used (P1, P2, and P3 polymers were not tested because of their relatively low effectiveness compared with P4 from swell tests as mentioned in Section 4.3.4). Unlike fly ash and lime, P4 was a liquid. P4 fluid was removed from the manufacturer's sample bottle using a syringe, mixed with a designated amount of water, and the diluted polymer solution was added to the expansive soil by spraying. Solution was added and simultaneously mixed using a spatula and gloved hands in a clean bowl until a visually homogeneous paste was formed.

For each *PL* test, the rolling technique on a sintered glass plate was used for approximately 20g of each of the soil mixtures mentioned previously. Liquid limit and *PL* are needed for classification based on FHWA-RD-77-94.

#### **4.4.2 Standard Proctor Compaction Test**

ASTM D698 "Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort [12,400 ft-lbf/ft<sup>3</sup> (600 kN-m/m<sup>3</sup>)]" was used in this research. The followings describe sample preparation for each soil mixture:

- NFC was passed through a #4 sieve, and prepared according to ASTM D698.
- For expansive soil, NFC was sieved through a #4 sieve, and mixed with bentonite at an 85:15 ratio.
- Expansive soil was mixed with fly ash, lime, and P4 to create the treated soil mixtures. An identical mixing procedure for the stabilizers with expansive soil as that of the Atterberg limit test was used.

#### 4.4.3 Swell Tests

ASTM D4546 “Standard Test Methods for One-Dimensional Swell or Collapse of Soils” was followed using Method A (for reconstituted specimens) in odometer rings via an inundated condition. Each soil mixture was prepared in the same procedure as that used for standard Proctor compaction tests. One potential advantage of commercially available polymers is that they are advertised to not require a curing time, whereas curing time is required for lime and fly ash. Therefore, the effect of a 24-hour air-dried curing versus 7-day moisture-maintained curing was evaluated for each stabilizer.

The compaction method used for expansive soil and treated mixtures inside the odometer ring is shown in Figs. 4.1 and 4.2. The odometer ring was placed inside a standard-size compaction mold. Soil was added around the ring to prevent movement of the ring inside the mold during compaction (Fig. 4.1). The soil was compacted in two layers as per ASTM D4546. The compaction method used in this study incorporated 10 blows to first layer (within the ring only) and 25 blows to the second layer (across the entire mold) with a standard Proctor hammer (Figs. 4.1 and 4.2.). A steel straight edge was used to strike off extra soil from atop of the odometer ring. Thus, the height of the compacted specimens was the height of the odometer ring. Appendix C includes further details of this compaction method including specimen preparation, repeatability, and applied energy discussion.

After compaction the following procedures were applied for each treated soil:

- The weight of each specimen was measured, and water content was determined from trimmings.
- Soil specimens, still within odometer rings, were wrapped in plastic.
- Specimens were cured for 7 days at a constant temperature of 40 °C. Curing at 40 °C results in accelerated curing (described in ASTM D5102 “Standard Test Methods for Unconfined Compressive Strength of Compacted Soil-Lime Mixtures”, Note 7).

- Swell tests were run using a consolidometer device (Fig. 4.3.) within a consolidation loading assembly (with 10:1 moment arm; Fig. 4.4), and terminated according to ASTM D4546.
- After swell test were completed, the weight of each specimen was measured to assess changes water content. Appendix B presents the relationship between swell test results and the amount of water gained.
- The static pore water pressure was ignored because of a negligible height of water on top of the swell test specimens (i.e. ~1 cm or ~0.1 kPa). Effects of excess pore water pressure were ignored; any excess pore water pressure were assumed to have dissipated by the end of the swell test. The reasons for this assumption involve i) the minimum duration of the swell test that was at least four days, and ii) trends in the swell/collapse deformation of the specimens versus time (see swell tests in Appendix A).



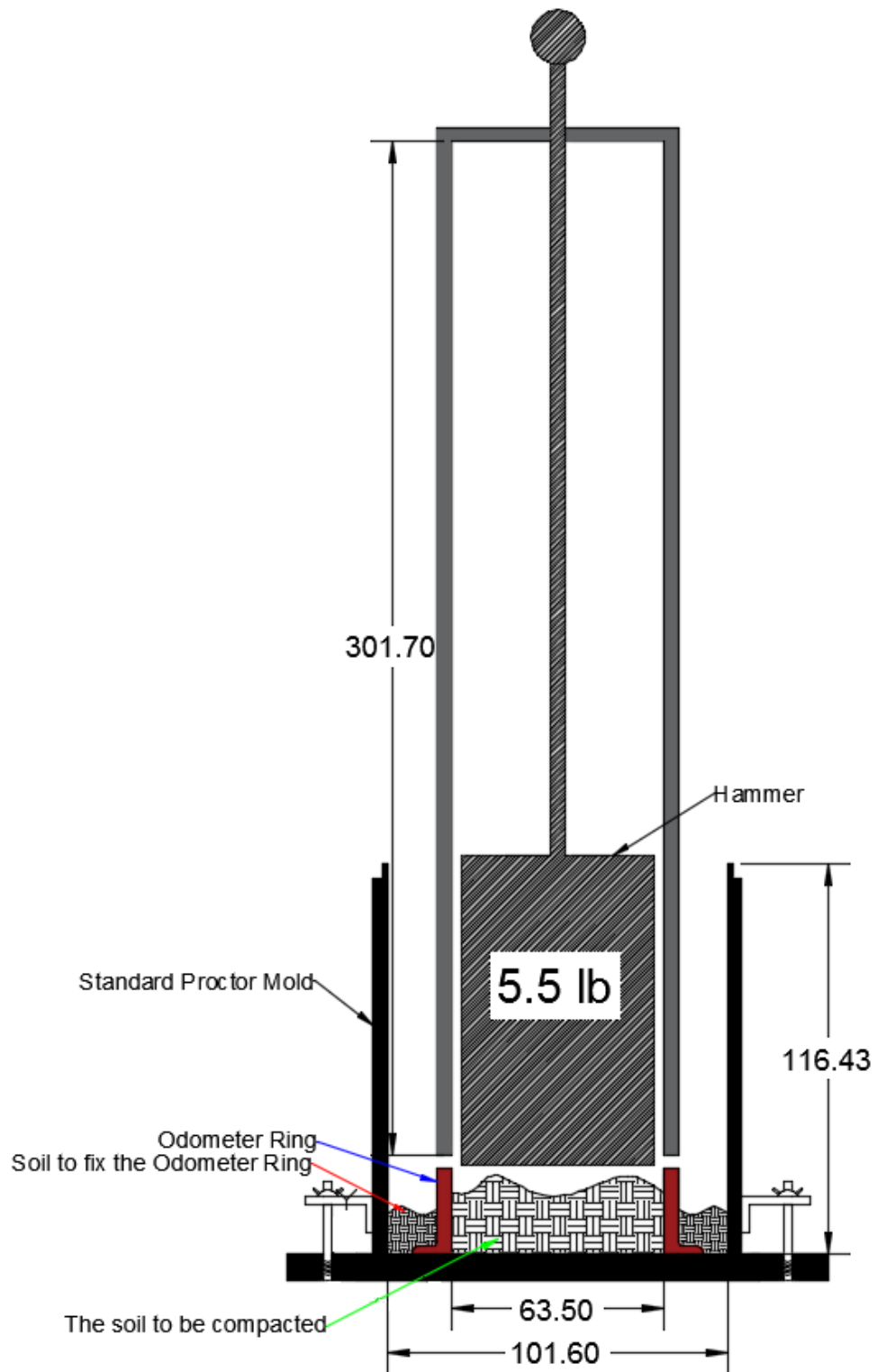


Figure 4.1 Schematic of first layer of compaction – 10 blows by hammer. All units are in mm.

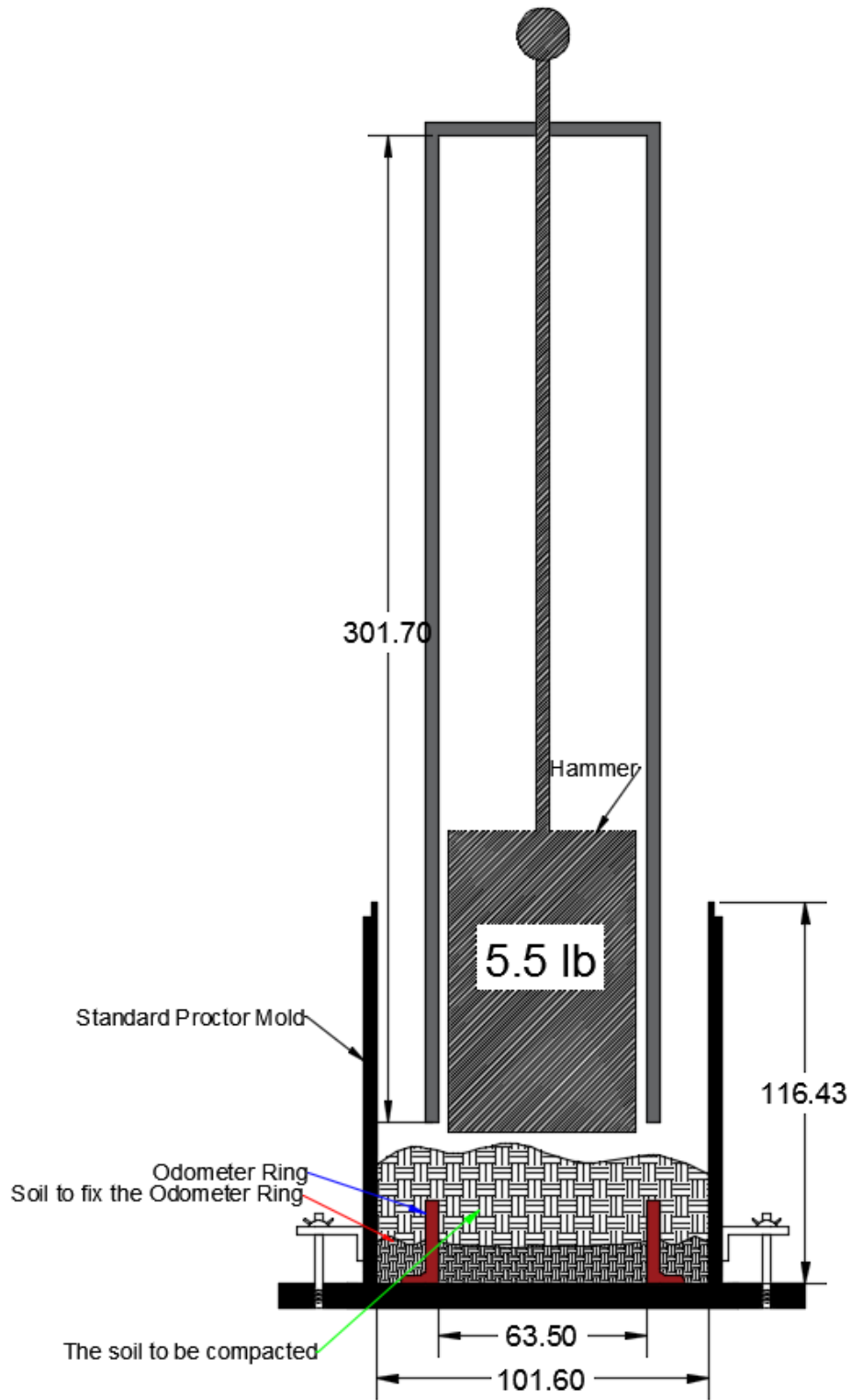


Figure 4.2 Schematic of first layer of compaction – 25 blows by hammer. All units are in mm.

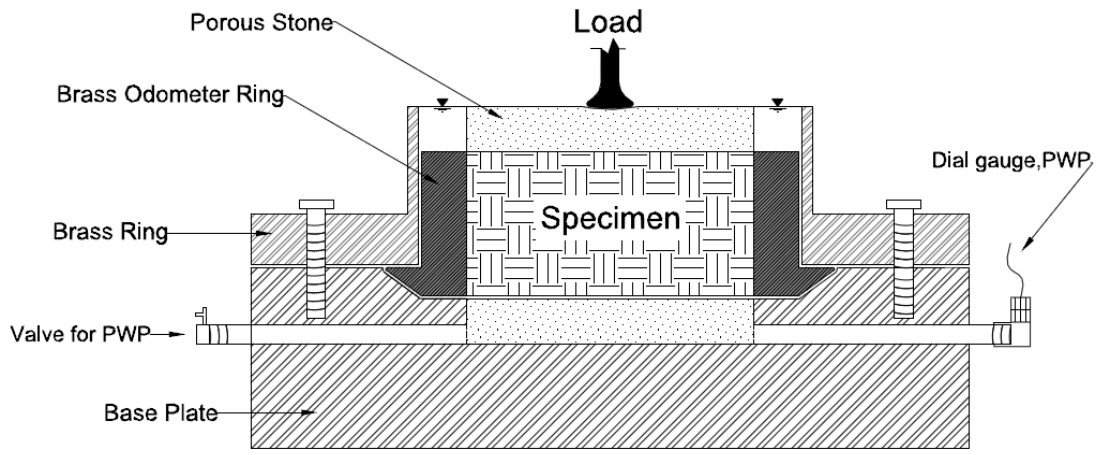


Figure 4.3 A scheme of the consolidometer device. PWP = pore water pressure.

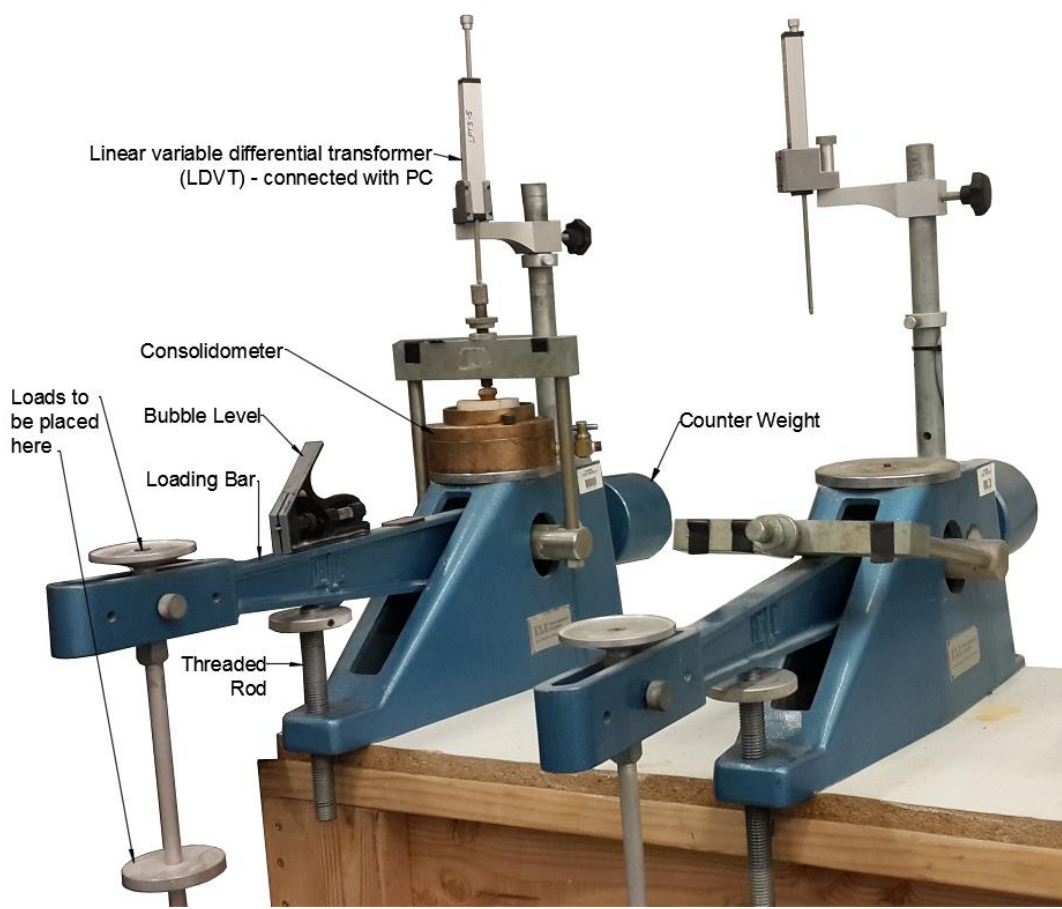


Figure 4.4 Consolidation assembly.

#### **4.4.4 Hydraulic Conductivity Tests**

ASTM D5084 “Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter” was followed using Method C (falling headwater, rising tailwater elevation). A four-inch diameter flexible wall permeameter cell was used for all soil mixtures as shown in Fig. 4.5.

Identical procedures for sample preparation, mixing, and curing were performed as those of the swell tests for all soil mixtures. The specimen height and compaction method described in Tong and Shackelford (2011) were used; a 29.3 mm high compaction mold is used (Fig. 4.6), and compaction is accomplished in a single layer with 19 blows of a standard Proctor compaction hammer.

The hydraulic conductivity ( $k$ ) cell installed to the pressure panels (with a precision of 0.1 psi) is shown in Fig. 4.7. Backpressure saturation and hydraulic gradient were selected per ASTM D5084. ASTM D5084 Section X2.1 recommends minimizing any volume change during the  $k$  test, therefore  $k$  tests were performed such that the applied effective stress was equal to the soil swelling pressure. The  $k$  tests were terminated in accordance with Section 9.5 in ASTM D5084. Appendix E includes corrections applied for the pressure panel pipets measurements to convert the unit on the pipets to millimeters. Tap water was used as the permeant solution.



Figure 4.5 Details of the hydraulic conductivity cell.



Figure 4.6 29.3-mm high hydraulic conductivity molds.

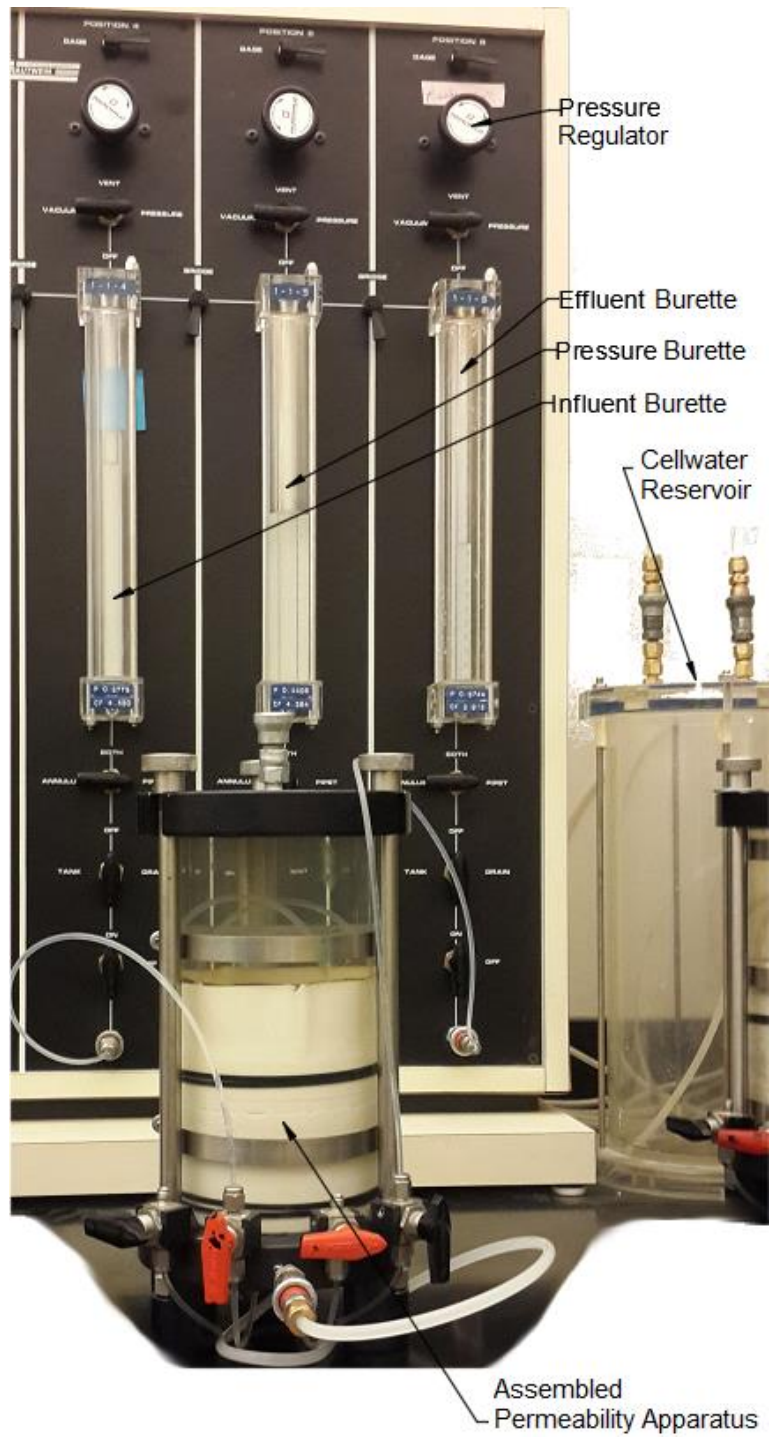


Figure 4.7 Hydraulic conductivity cell and pressure panel used in this study.

#### **4.4.5 Unconfined Compressive Strength Tests**

ASTM D5102 “Standard Test Methods for Unconfined Compressive Strength of Compacted Soil-Lime Mixtures” was followed for all treated and untreated soil mixtures. The same procedures for sample preparation, mixing, and curing were performed as those of the swell and  $k$  test.

As shown in Figure 4.8, the compaction mold used for all soil mixtures was a plastic cylindrical mold with diameter equal to a standard compaction mold (101.60 mm), and height equal to double a standard compaction mold (232.86 mm), in accordance with ASTM D5102, Procedure A, Section 1.3.1. The plastic cylindrical mold was composed of two pieces that could be separated longitudinally, and two hose clamps were used to firmly attach the two pieces together. Soils were compacted in six layers with 25 blows per layer to meet the same compaction effort as that of the standard Proctor compaction test. Plastic wrap attached to the inside part of the separable mold were used to facilitate removing specimens from the mold (Fig. 4.8).

After compaction and removal from the mold, specimens were soaked for 24 hours, based on the guidance of National Lime Association (2006). Soaking was accomplished by placing the specimens on porous stones in a pan of tap water where the water level was kept at the top of the porous stones (Fig. 4.9). For each test, four specimens were prepared; three to be soaked and one unsoaked testing. A vertical load frame was used to test the unconfined compressive strength (UCS) of the soaked and unsoaked specimens. The strain control method of ASTM D5102 (Section 13.2) for brittle specimens was used, where the strain control speed is 2.0% per min (Fig. 4.8). The failure shape of each specimen was recorded for “area correction determination” in accordance with ASTM D5102.

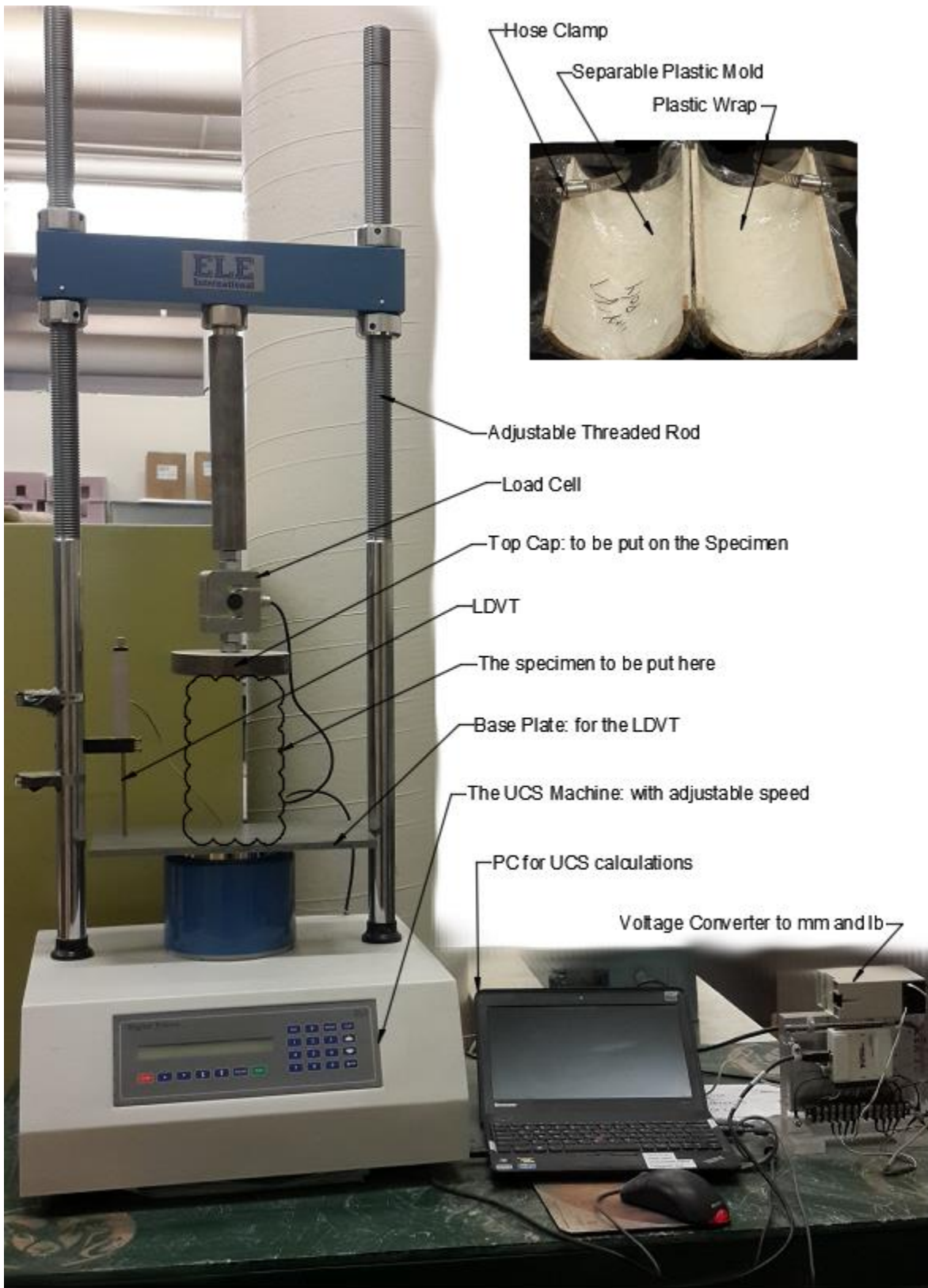


Figure 4.8 The unconfined compressive strength test machine used in this study.





Figure 4.9 Soaking technique used for unconfined compressive strength tests in this study.

## 4.5 Results and Discussion

### 4.5.1 General Comparison Summary

Table 4.1 presents the effects of fly ash, lime, and P4 treatment on expansive soil; the results of each test are described in subsequent sections. Lime treatment was the most effective stabilizing technique for expansive soil based on decreased swelling from swell tests, and increased strength from UCS. Fly ash treatment ranked second, but with much lower effectiveness than lime. Minimal improvement was seen for P4 polymer to mitigate the expansive soil used in this research. However, significantly increased permeability (more than an order of magnitude), which is not desired, was not resulted by P4 as by lime and fly ash.

Table 4.1 Summary of the results of this research

Tests \ treatment methods	Units	expansive soil	P4 polymer (5%) treated soil	Fly ash (15%) treated soil	Lime (3%) treated soil
Liquid limit, $LL$	[%]	75.8	70.3	56.4	50.5
Plastic limit, $PL$	[%]	17.7	19.5	17.6	32.6
Plasticity index, $PI$	[%]	58.1	50.8	38.8	17.9
Optimum water content, $W_{\%opt}$	[%]	18.4	19.4	17.5	20
Maximum dry density, $Y_{d\ max}$	[lb/ft <sup>3</sup> ]	104.9	103.5	103	101
Swell potential, $S_{\%}$	[%]	14.9	4.5	0.5	0
Swell pressure, $p_s$	[kPa]	139	120	33	2
Hydraulic conductivity, $k$	[m/s]	$2.9 \times 10^{-11}$	$7.2 \times 10^{-11}$	$3.1 \times 10^{-8}$	$1.5 \times 10^{-6}$
Soaked unconfined compressive (UCS) strength, $q_u$	[kPa]	0	46	382	1260

Note: P4 was determined to be the most effective polymer type based on swell tests performed in this research. P4 is composed of 29% non-evaporable polymer and 71% evaporable liquid. P4 was determined to be the most dilute (smallest non-evaporable portion) relative to P1, P2, and P3.

### 4.5.2 Swell Test for Determining Optimum Fly Ash Content:

ASTM D4546 defines swell potential as a positive strain that specimens under one kPa exhibit upon inundation. Figure 4.10 includes the swell potential test for addition ratios of 9, 11, 13, 15, and 17% (by mass) used to determine optimum fly ash content. Decreased swell potential

was observed as the fly ash addition percentage increased. However, repeatability of the swell test can be noticed in Figure 4.10. For example, three replicate tests with 15% fly ash content yielded swell potentials of 0.1, 0.5, and 0.7%. The reasons for this variability include the followings:

- The size of the odometer ring is not sufficient to provide a repeatable swell potential value less than 1%.
- At swell potential, any minor inconsistency in specimen mixing, compaction, load application, and leveling may result swell potential fluctuations of less-than 1% range.
- The effect of only a 2% difference between two fly ash additions may not generate a measurable difference in swell potential (for the reasons listed above).

In this study, 15% fly ash was based on reported field application rates rather than the results of swell testing. The choice of 15% fly ash addition was based on American Coal Ash Association (2003) guidance that states a typical fly ash addition to be 12 – 15%. The upper end of this range was chosen to maximize the effect of fly ash and provide a conservative assessment with which to compare fly ash. However, the swell potential for the 15% fly ash content that is used for comparison is 0.5% (the average of three replicate tests) was similar to fly ash proportions as low as 9% (the lowest tested in this study). These data illustrate that the fly ash proportion used for this study was unnecessarily high.

#### **4.5.3 Swell Test for Determining Optimum Lime Content:**

The results of swell potential tests to determine optimum lime content are presented in Figure 4.10. Addition percentages of 2, 3, 4, 5, and 6 were tested. Only 2% lime resulted in a measurable swell (0.1%), all the other lime contents resulted no measurable swell. However, given the variability exhibited in tests to determine optimum fly ash content, all percentages are interpreted to have resulted in near zero swell. In this research, 3% lime addition was used based

on the findings reported by Akawwi & Al-Kharabsheh (2002) and swell potential tests (Figure 4.10), where 3% lime was the lowest lime addition percentage to yield no measurable swell.

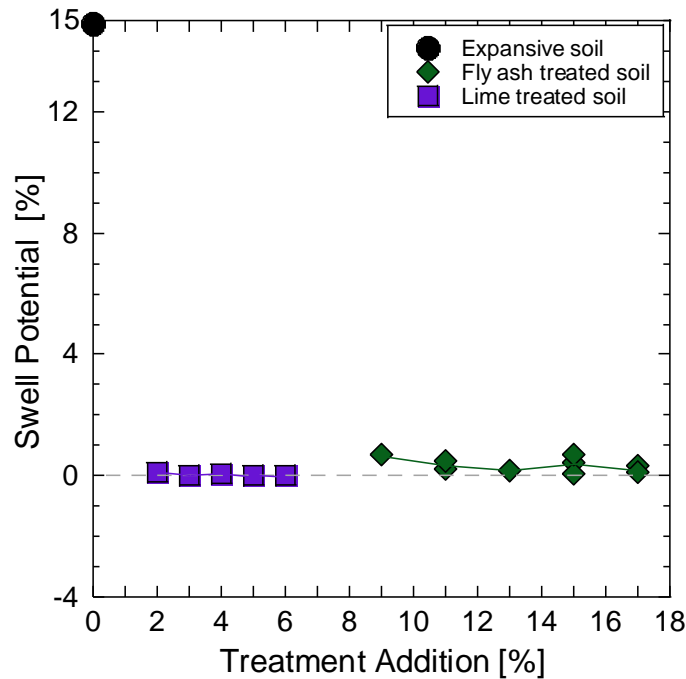


Figure 4.10 The swell potential of different lime proportions

#### 4.5.4 Selecting Polymer Type and Determining Optimum Polymer Content

##### 4.5.4.1 Swell Test for Selecting Polymer Type

Swell potential tests were run with 4.6% polymer addition with P1, P2, P3, and P4 to determine which polymer provided the greatest stabilization of expansive soil with the manufacturers' recommended 24-hr air drying (7 days of curing was avoided to test the actual effect of the polymers with short term curing(air drying)). The polymer addition of 4.6% (~ 5%) was chosen based on a practical polymer addition posted on one of the polymer manufacturer's website. Results for tests air dried for 24 h under ambient conditions showed that expansive soils treated with polymers P1, P2, P3, and P4 swelled 8.7, 8.0, 7.9, and 4.7%, respectively, compared to 3.1% for 15% fly ash treatment, and 0.8% for 3% lime treatment (as expected, fly ash and lime treatments exhibit greater swelling potentials with only 24-hr air drying relative to results for 7-d

curing reported in the previous section (4.5.2 and 4.5.3) and earlier in this chapter). P1, P2, and P3 showed higher swell potentials than P4 (Fig. 4.11). Therefore, P4 with 4.6% percent addition was selected for further study and comparison with 3% lime and 15% fly ash.

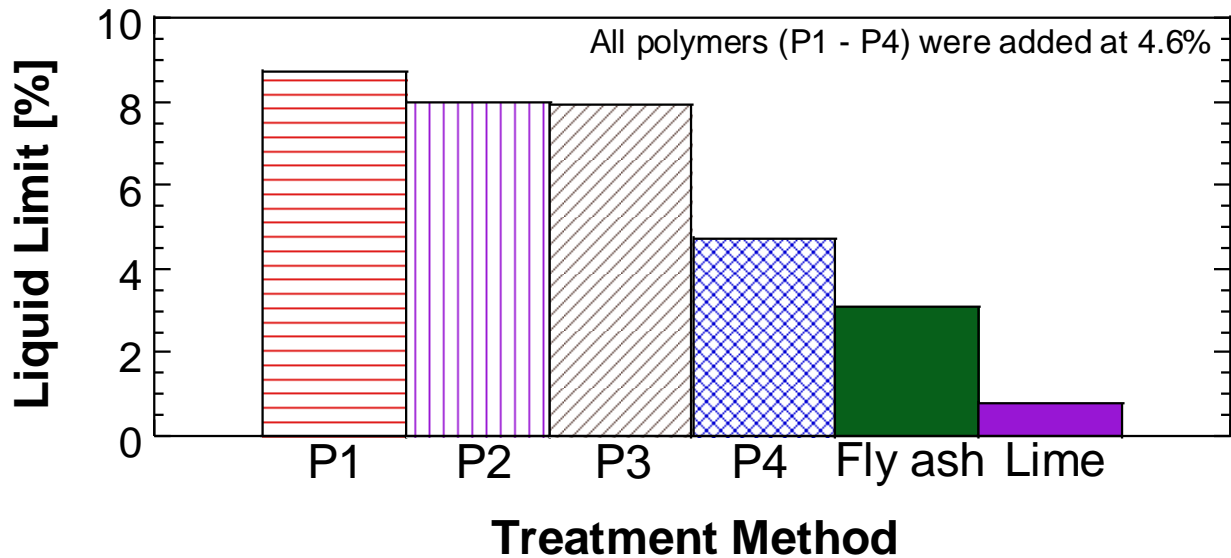


Figure 4.11 Swell potential of expansive soil treated with different polymer types (tests are with 24hr air drying). Data for lime and fly ash are included for comparison.

#### 4.5.4.2 Atterberg Limit Tests Evaluating the Effect of P4 Inclusion

Atterberg limit tests were run for 5% P4 content up to a maximum P4 content of 27%, above which the water content of the treated soil exceeded  $w_{\%opt}$  (i.e., the maximum P4 content provides all the water needed for  $w_{\%opt}$ ). The results of *LL* tests are presented in Table 4.2. A *LL* of the maximum 27% P4 yielded a swelling potential close to the 5% P4, which shows that any P4 addition above (at most) 5% is excessive. In addition, the *PI* value of the maximum 27% P4 is close to 35%, which classifies the soil as “high” according to FHWA-RD-77-94. Therefore, even with this high polymer content, the swelling behavior (as predicted based on Atterberg limits) of expansive soil was not reduced.

Table 4.2 The Atterberg limit test results with varying P4 content

	Fly ash (15%) treated	Lime (3%) treated	P4 (5%) treated	P4 (27%) treated
<i>LL</i> [%]	56.4	50.5	70.3	66.9
<i>PL</i> [%]	17.6	32.6	19.5	32.2
<i>PI</i> [%]	38.8	17.9	50.8	34.7

Note: refer to Table 2.1 for the expansive soil classification used in this research

#### 4.5.5 Atterberg Limit Reductions

The objective of running Atterberg limit tests was to analyze the effect of each stabilizer on *LL* and *PI* of the treated soils, and determine the resulting swell classification based on FHWA-RD-77-94. The *LL* of untreated expansive soil was 75.8%. Fly ash (15%) and lime (3%) reduced the *LL* to 50.2% and 56.5%, respectively, changing the potential swell classification from “high” to “marginal” according to FHWA-RD-77-94 (Table 4.3, Fig. 4.12). However, *LL* of the soil treated with P4 remained above 60%, maintain a “high” swell classification. For *PI*, the same order of effectiveness of the stabilizers was observed (Table 4.3, Fig. 4.12).

Additional curing (7 d, or longer) after adding lime and fly ash into expansive soil may yield still lower *LL* and *PI* values. In contrast, P4 does not require long term curing. The American Coal Ash Association (2003) sets a *PI* of 10% as indicative of a stable soil. Swell tests were run to analyze the stability of the treated soils, and the same order of effectiveness was seen in the swell tests as the Atterberg limit tests (discussed in Section 3.5.7).

Table 4.3 Atterberg limit test results for different treatment methods

	NFC	Expansive soil	P4 (5%) treated	Fly ash (15%) treated	Lime (3%) treated
<i>LL</i> [%]	31.0	75.8	70.1	56.5	50.2
<i>PL</i> [%]	18.1	17.7	19.5	17.6	32.6
<i>PI</i> [%]	12.9	58.0	50.6	38.9	17.6

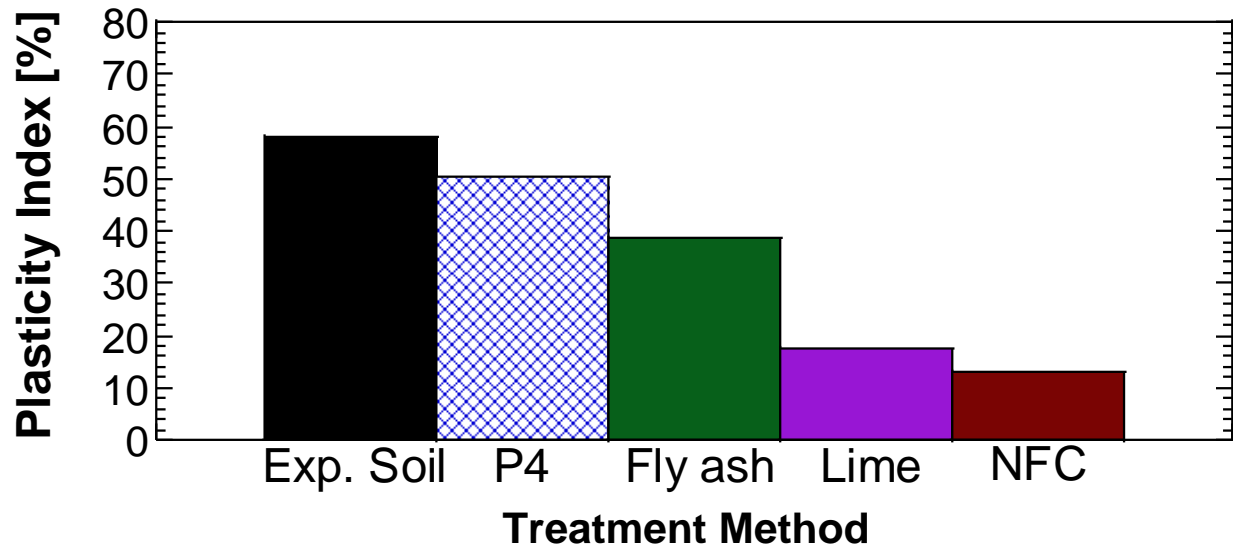
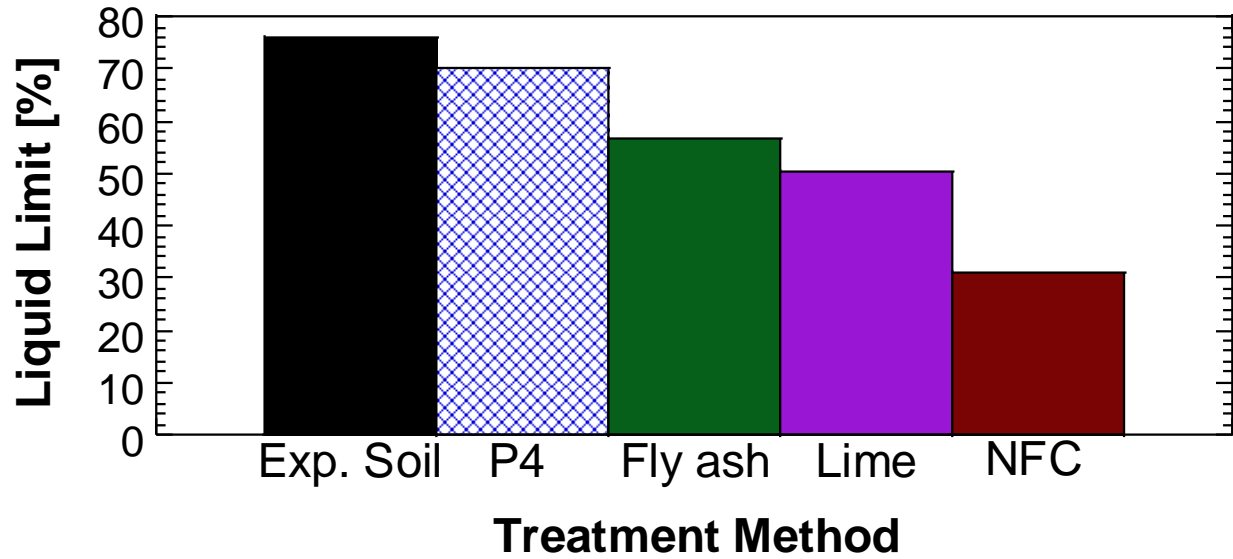


Figure 4.12 Atterberg limit test results for different treatment methods

#### 4.5.6 Compaction

The objective of running standard Proctor compaction tests was to determine  $w_{\%opt}$  and  $\gamma_{d\ max}$ , and compare the effect of each stabilizer on  $w_{\%opt}$  and  $\gamma_{d\ max}$ . No trend is apparent for the effect of the stabilizers on  $w_{\%opt}$  and  $\gamma_{d\ max}$  (Fig. 4.13). The lack of any discernable trend is attributed to i) the dominant effect of the bentonite on compaction, and ii) the lack of reaction time between treatment and compaction (relative to post-treatment behavior). Table 4.4

illustrates that addition of bentonite to NFC dramatically reduced  $\gamma_{dmax}$  from 112.2 lb/ft<sup>3</sup> to 105.0 lb/ft<sup>3</sup> and increased  $w_{%opt}$  from 16.1% to 18.4%. This increase in  $w_{%opt}$  is an indicator hydration and immobilization of water by bentonite, which yields a lower  $\gamma_{dmax}$ .

Table 4.4 Maximum dry unit weight and optimum water content for different treatment methods

	Expansive soil	P4 (5%) treated	Fly Ash (15%) treated	Lime (3%) treated	NFC
$\gamma_{dmax}$ [lb/ft <sup>3</sup> ]	105.0	103.5	103.0	101.1	112.2
$w_{opt}$ [%]	18.4	19.4	17.5	20.0	16.1

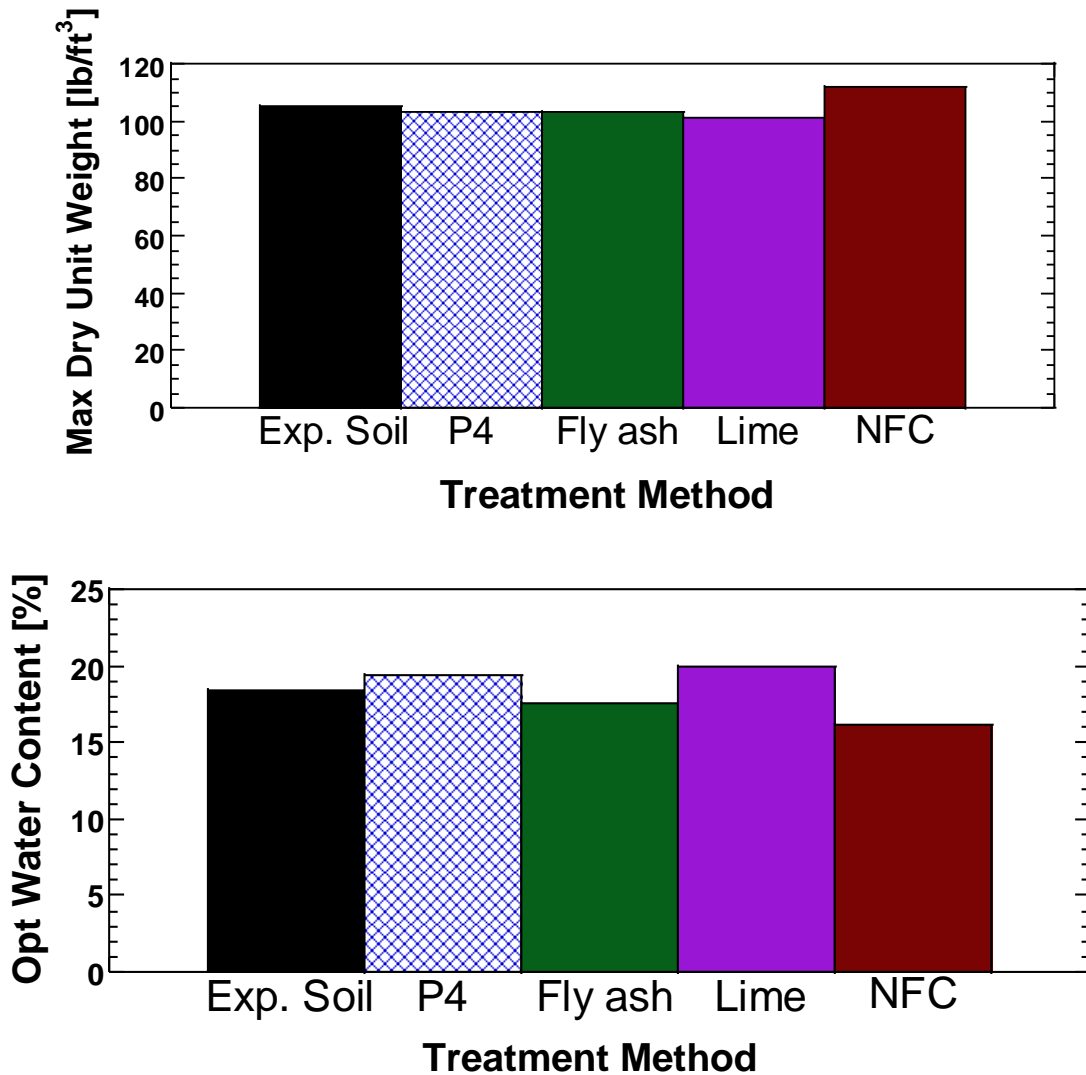


Figure 4.13 Standard Proctor compaction test results for different treatment methods



#### 4.5.7 Swell Potential and Swell Pressure

ASTM D4546 defines swell potential as a positive strain that specimens under 1 kPa undergo. In contrast, swell pressure is defined as the pressure at which the specimen exhibits zero strain upon inundation. Fig. 4.14 presents the results of swell potential and swell pressure tests on treated expansive soil. Lime and fly ash are effective at reducing swelling potential and swelling pressure, and counteracting the effect of the 15% bentonite in expansive soil. For the lime-treated soil specimens, both the swell potential and swell pressure are reduced to values lower than an amended NFC.

Relative to lime and fly ash, P4 does not considerably reduce swell potential and swell pressure. A swell potential reduction from 14.9% to 4.5% and a swell pressure reduction from 139 kPa to 120 kPa show the relative ineffectiveness of P4 at reducing the swelling of expansive soil (Table 4.5).

Table 4.5 Swell potential and swell pressure values for different treatment methods

	Expansive soil	P4 (5%) treated	Fly ash (15%) treated	NFC	Lime (3%) treated
Swell potential [%]	14.9	4.5	0.5	0.3	0.0
Swell pressure [kPa]	139.0	120.0	33.0	8.0	2.0

Note: fly ash, lime, and polymer specimens were cured for seven days in 40 °C prior to testing.

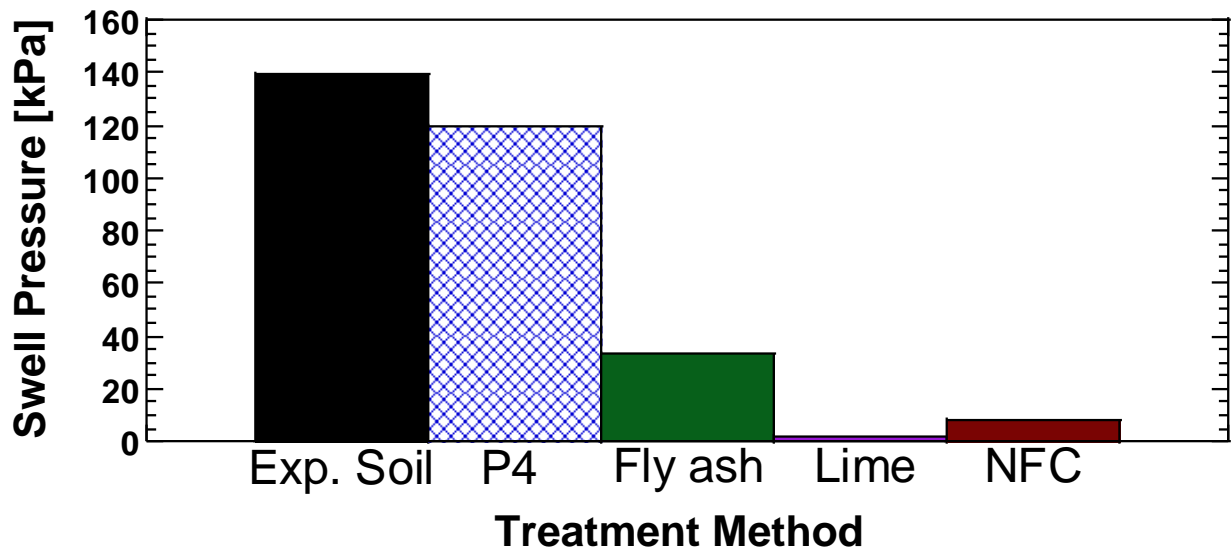
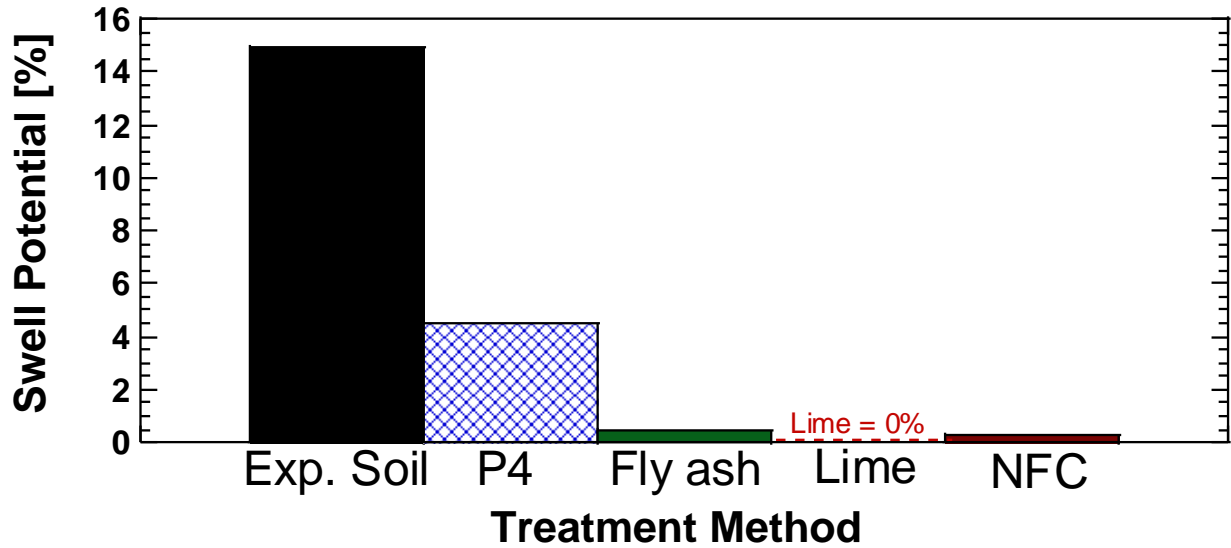


Figure 4.14 The Swell test results for different treatment methods.

The stabilizing effect of each stabilizer in swell tests under these pressures: 1, 10, 50, 100, and 207.3 kPa are shown in Table 4.6. As shown in Fig. 4.15, a reduction in the swelling of expansive soil is apparent as pressure increases. This behavior illustrates that that expansive soil is easily deformable with load applications (typical of loosely compacted soils with a high bentonite content). Lime greatly reduced both swelling and consolidation of the specimens and produced a stable mass within the pressure range tested.

Table 4.6 The swell potential and swell pressure of for different treatment methods

Pressure [kPa]	Swell/Collapse [%]				
	Expansive soil	P4 (5%) treated	Fly ash (15%) treated	NFC	Lime (3%) treated
1	14.9	4.5	0.5	0.3	0
10	5.5	1	0.4	-0.1	-0.3
50	1.5	0.6	-0.4	-0.3	0
100	0.4	0.1	-2.6	-0.4	-0.1
207.3	-0.6	-0.6	-	-	-0.2

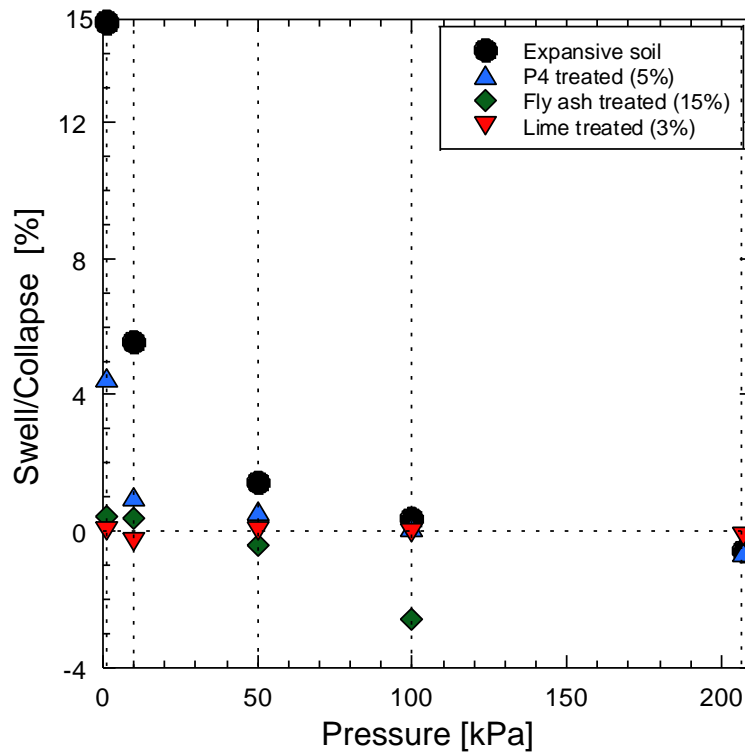


Figure 4.15 Swell test results for all treatment methods.

#### 4.5.8 Hydraulic Conductivity

Table 4.7 includes  $k$  results for expansive soil cured specimens with effective stresses equal to  $P_s$  to minimize volume change from specimen saturation. As shown in Fig. 4.16, higher  $P_s$

correlates to lower  $k$ . Similarly, higher swell potential ( $S\%$ ) correlates to higher  $k$ . Varying applied effective stress does not appear to significantly influence  $k$  (Appendix F).

Lime and fly ash treated soils exhibit high  $k$  potentially due to agglomeration of particles and the effectiveness of preventing swelling, thus allowing flow paths within the soil to remain unsealed. Fly ash treated exhibits a  $k$  ~50-times lower than that of the soil treated with lime, which correlates with  $S\%$  differences of lime and fly ash. P4 results in a low  $k$ , that is close to that of expansive soil and more than 20,000 times lower than that of lime.

Table 4.7 Hydraulic conductivity for expansive soil and each treatment method

	Lime (3%) treated	Fly ash (15%) treated	P4 (5%) treated	Expansive soil
Hydraulic Conductivity [m/s]	$1.5 \times 10^{-6}$	$3.1 \times 10^{-8}$	$7.2 \times 10^{-11}$	$2.9 \times 10^{-11}$
Effective Stress [kPa]	7	33	120	139

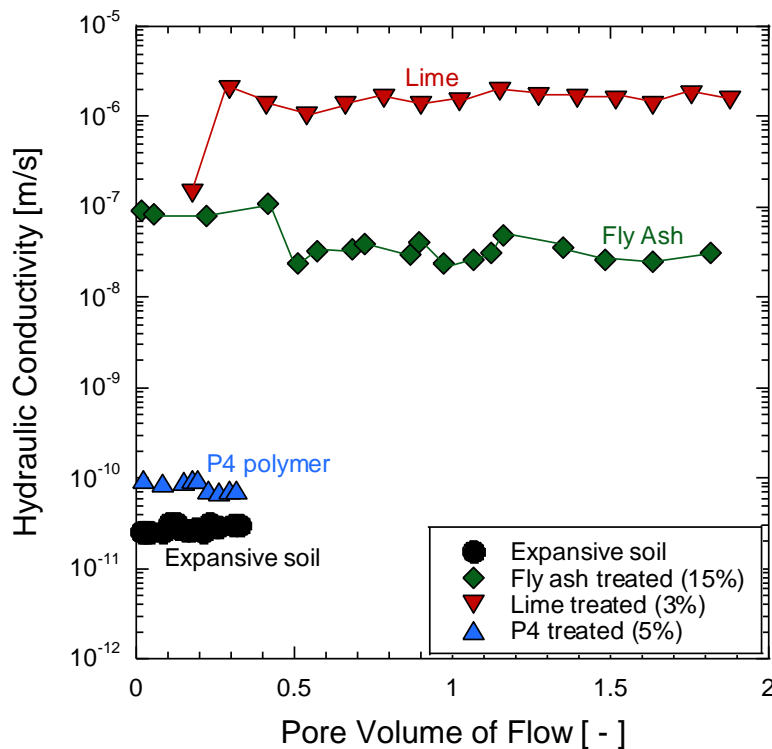


Figure 4.16 Hydraulic conductivity for expansive soil and treatment method.

#### 4.5.9 Unconfined Compressive Strength

Unconfined compressive strength ( $q_u$ ) tests were run to assess the effect of each stabilizer on undrained shear strength of the soils. All specimens were soaked for 24 hours prior to testing. The  $q_u$  of expansive soil and the treated 7-day cured soils are shown in Table 4.8. The lime treated soil exhibited the highest  $q_u$  followed by fly ash treatment (Fig. 4.17), consistent with swell test results. The P4 treated specimens were very weak after the 24-hour soaking, which resulted in a low  $q_u$ . Untreated expansive soil was so weak that a  $q_u$  test could not be run appropriately.

Table 4.8 Unconfined compressive strength for each treatment method as well as untreated expansive soil.

	Lime (3%) treated	Fly ash (15%) treated	P4 (5%) treated	Expansive soil
$q_u$ [kPa]	1260.0	381.7	46.0	0.0

Note:  $q_u$  is unconfined compressive strength

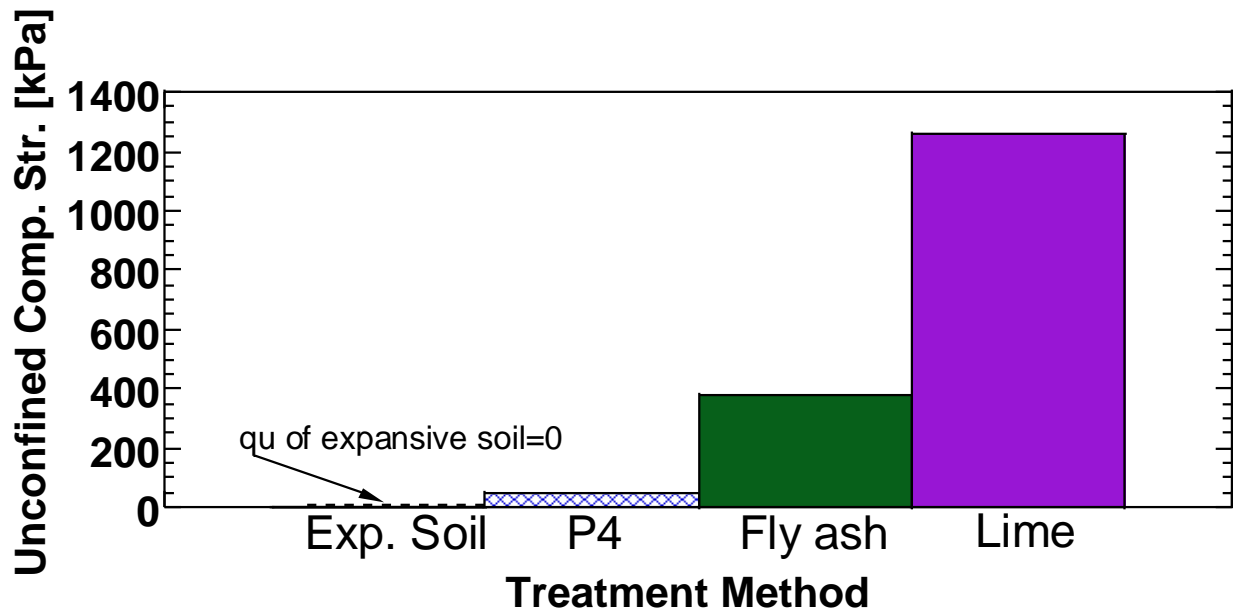


Figure 4.17 Unconfined compressive strength for each treatment method as well as untreated expansive soil.

#### 4.5.10 Effect of Curing Time

One potential advantage of commercially available polymers is that they are advertised to not require a curing time, whereas curing time is required for lime and fly ash. The effect of a 24-h air drying versus 7-d curing was evaluated for each stabilizer. The 24-h air drying was performed by keeping specimens in room temperature for 24 hours without maintaining moisture, whereas, for the 7-d curing, specimens were wrapped with plastic wrap and kept in an oven for 7 days at 40 °C with supplied humidity. Table 4.9 contains the results of swelling potential tests with 7-d curing versus 24-h air drying. As shown in Figure 4.18, 7-d curing greatly influences the swelling potential of fly ash and lime treated expansive soil, such that the swell classification of these soils transitions from “high” to “low” according FHWA-RD-77-94 classification. However, the P4 treated soil shows little improvement from 24 h to 7 d of curing. The reason that polymer treated soil does not improve with 7-d curing is attributed to the quick interaction of polymer with expansive soil (that needs air drying) compared to that of lime and fly ash.

Table 4.9 The swell potential values for assessing seven days of curing vs. 24-h air drying

Air drying / curing	Swell Potential [%]			
	Expansive soil	P4 (5%) treated	Fly ash (15%) treated	Lime (3%) treated
24 h	14.9	4.7	3.1	0.8
7 d	-	4.5	0.5	0.05

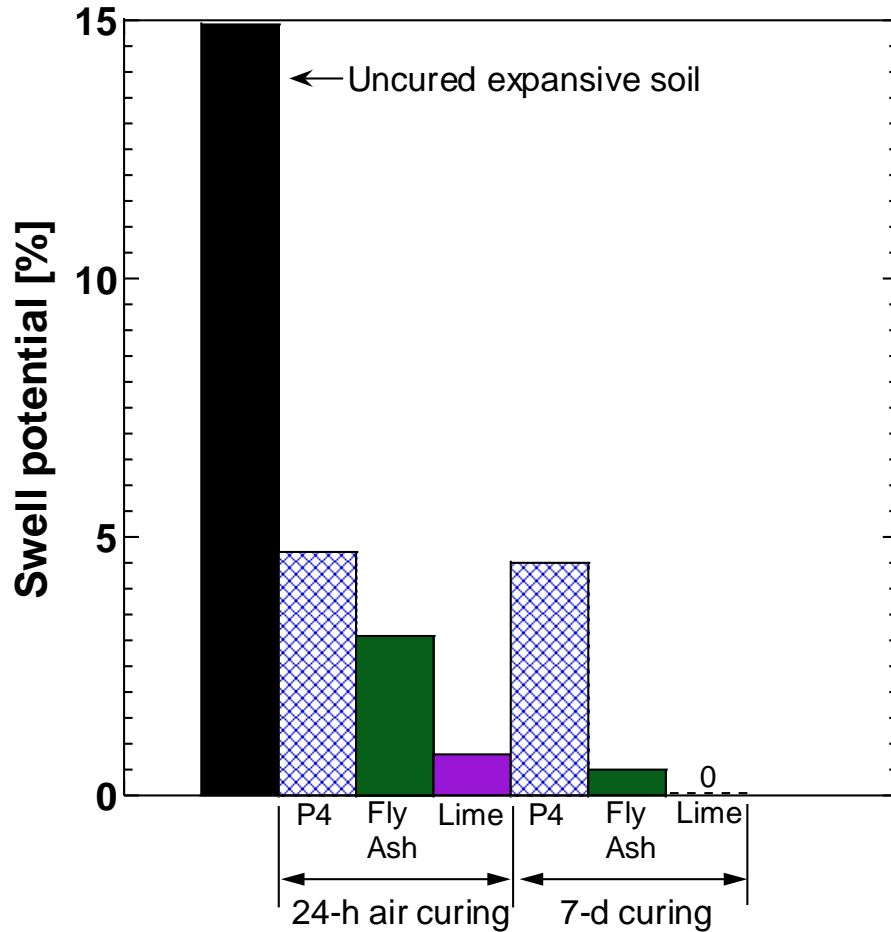


Figure 4.18 Swell potential after 24-h air drying and 7-d of curing.

#### 4.6 Analysis

The objective of this study is to assess if commercially available polymer stabilizers are an effective treatment for a highly active bentonite-amended clay (i.e., expansive soil) relative to traditional stabilizers. Swelling, hydraulic conductivity, and unconfined compressive strength tests were performed to compare the potential effect of polymer and traditional stabilizers. Swell test was performed to compare potential swell-reduction effectiveness of polymer versus fly ash and lime. P4 resulted in reduced swell potential and swell pressure, however, relative to lime and fly ash, the swell reduction capability of P4 was not substantial (refer to Section 4.5.7). Hydraulic conductivity tests were performed to compare the effectiveness of polymer versus fly ash and lime in minimizing moisture intrusion from the treated top layer to underlying untreated layers. P4

resulted in a lower  $k$  compared to fly ash and lime, however the  $k$  was still higher than that of untreated expansive soil, i.e., P4 treatment did not yield a reduction in  $k$  (see Section 4.5.8). The unconfined compressive strength test was performed to compare strength-gaining effectiveness of P4 versus fly ash and lime. Similar to swell test results, P4 resulted in a significantly lower  $q_u$  (i.e., lower improvement in  $q_u$ ) than treatment with lime and fly ash.

Based on the results of swelling and unconfined compressive strength test results P4 polymer is not an effective stabilizer for the expansive soil tested in this study relative to the two traditional stabilizers evaluated (i.e., fly ash and lime). P4 does not provide sufficient internal interactions between the clay particles (to reduce swelling and increase strength) relative to the pozzolanic reactions that result from treatment with lime or fly ash. However, this result is only valid for the highly expansive soil described in this study, and only for the laboratory tests conducted, and may not represent field behavior. Testing a naturally expansive soil in a manner that better represents the hydration-swell process that occurs in the field may yield different results. Polymerically stabilized expansive soil was treated and then compacted in this study. Spraying of polymer on undisturbed soil (effectively creating a physical, not chemical, soil treatment by causing surficial soil layers to function as a moisture barrier) and relying on the unconfined compressive strength of a dry(er) subgrade may better represent field behavior, but cannot be representatively tested with existing standard methods for measuring swelling pressure and swelling potential (i.e., ASTM D4546). Additional representative testing on polymer-treated and lime-treated expansive soil is warranted.

#### **4.7 Conclusions**

The objective of this study was to assess the effectiveness of commercially available polymers in improvement of highly active expansive clays (decreased swelling, decreased permeability, and increased strength) compared to lime and fly ash. A low expansive soil was amended with bentonite to increase activity of the soil and create a highly expansive soil termed



“expansive soil.” Expansive soil was treated with lime, fly ash, and P4 polymer at 3%, 15%, and 5% additions, respectively. All the treated specimens were sealed with plastic and cured for 7 days at 40 °C. A second set of specimens was also tested after 24 hours of air drying at room temperature. Laboratory tests were conducted to establish a comparison of strength, swelling, and permeability of treated expansive soil. The followings can be concluded from this research:

- Lime treatment showed the greatest ability to increase strength and reduce swelling behavior for the soil tested. However, lime treatment also resulted in a large increase in hydraulic conductivity of the stabilized soil relative to the untreated soil; this is a disadvantage of lime treatment. Based on these findings, lime treatment is recommended as a benchmark treatment method with which to compare other polymer treatment technologies.
- Fly ash treatment also yielded increased strength and reduced swelling, although to a lesser extent than with lime, and at a higher addition percentage. Fly ash also resulted in a large increase in hydraulic conductivity relative to the untreated soil.
- P4 polymer was the best performing polymer among the four polymer types based on swell test results discussed this study. However, the stabilizing effect of P4 was poor compared to that of lime and fly ash based on the swelling and unconfined compressive strength tests. P4 did result in an approximately three order of magnitude lower hydraulic conductivity than fly ash treated soil and five order of magnitude lower than lime treated soil (~400 times lower than fly ash and ~20,000 times lower than lime). Because swelling of expansive soils requires addition of water, the lower swelling efficiency reported in this study may not accurately represent field behavior of P4 treated soil relative to lime or fly ash treatment.

These conclusions are applicable to the expansive soil tested in this study [which comprised 85% of a low expansive clay ( $LL= 31$  and  $PI=13$ ) and 15% sodium bentonite ( $LL= 420$

and PI= 381)]. The followings are recommended to further investigate the effectiveness of commercially available polymers for stabilization of expansive soils:

- Testing a naturally expansive soil (without added bentonite).
- Using soil with different particle sizes, rather than just clay, to assess the effectiveness of the stabilizers on a broader range of soils.
- The swell test apparatus should be modified such that a larger sample that can hold larger and more realistic specimens can be tested.
- A method is needed to test polymer treated expansive soil in a manner that is more representative of field conditions. This method should represent the coupled effects of swelling and hydraulic conductivity (i.e., water ingress and resulting swell), and should be field representative for the effect of curing of polymers and traditional stabilizers. This method also should allow within the apparatus small-scale applications of different stabilizers, compactions, and load applications to maximize representation of varying field conditions.
- Longevity of commercially available polymers versus lime or fly ash should be studied.
- The method described above should be applicable with potentially new nontraditional stabilizers other than polymers.

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## APPENDIX A: RESEARCH DETAILS

This appendix includes all supplemental data and graphs for the experimental work reported in Chapter 3. The expansive soil used for testing is termed “expansive soil”, as described in Chapter 3.

### A.1 Specific Gravity Test

Tables A.1 – A.3 report the  $G_s$  of NFC.

Table A.1 First specific gravity test

Specific Gravity ( $G_s$ ) Test #1 for NFC	Weight/ $G_s$
Weight of the flask filled with 500 ml of water ( $w_1$ )=	668.12 g
Weight of the flask filled with 500 ml of water and soil ( $w_2$ )=	746.63 g
Weight of dry soil ( $w_3$ )=	123.86 g
$G_s$ at 22.5°C=	2.7312
$G_s$ at 20°C=	2.73

Table A.2 Second specific gravity test

Specific Gravity Test #2 for NFC	Weight/ $G_s$
Weight of the flask filled with 500 ml of water ( $w_1$ )=	665.82
Weight of the flask filled with 500 ml of water and soil ( $w_2$ )=	699.00
Weight of dry soil ( $w_3$ )=	52.57
$G_s$ at 24.3°C=	2.7112
$G_s$ at 20°C=	2.71

Table A.3 Average specific gravity test

GS (average)=	2.72
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## A.2 Atterberg Limit Tests

### A.2.1 NFC Liquid Limit and Plastic Limit Test Results

#### A.2.1.1 Liquid Limit of NFC

Table A.4 Liquid limit of NFC

Liquid Limit (LL) Test						
Test no.	1	2	3	4	5	6
LL [%]	29.6	30.1	30.1	32.7	32.0	31.6
LL [%] (Average)=	31.0					

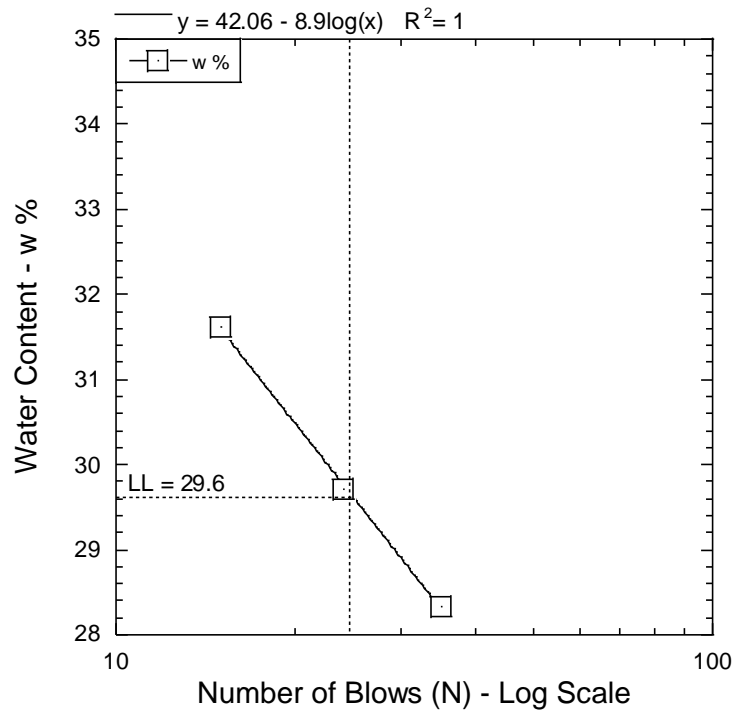


Figure A.1 Liquid Limit test results for NFC – test number one

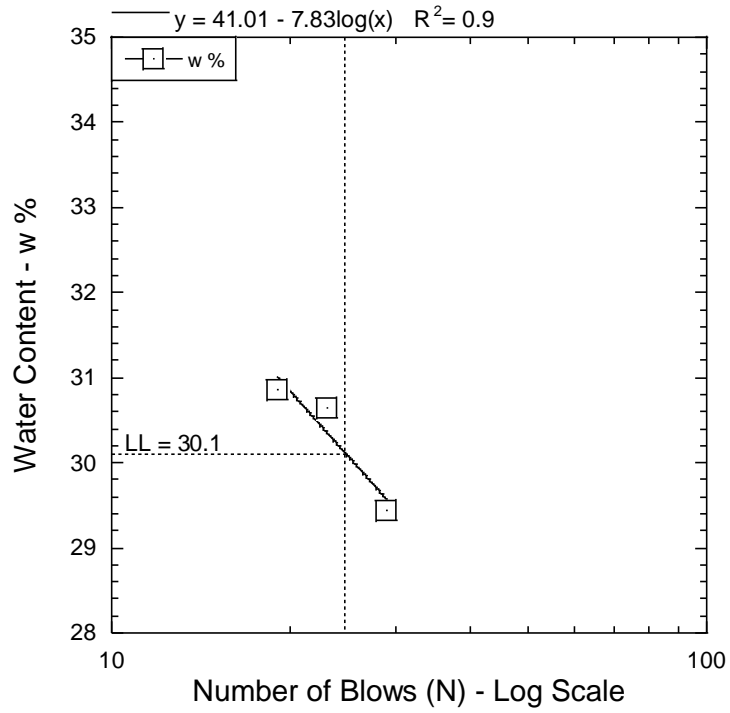


Figure A.2 Liquid limit test results for NFC – test number two

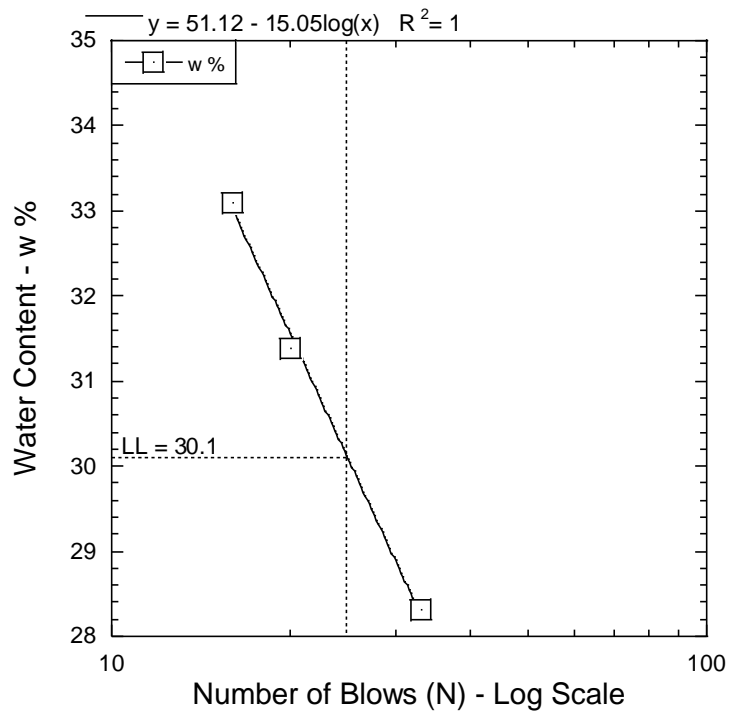


Figure A.3 Liquid Limit test results for NFC – test number three

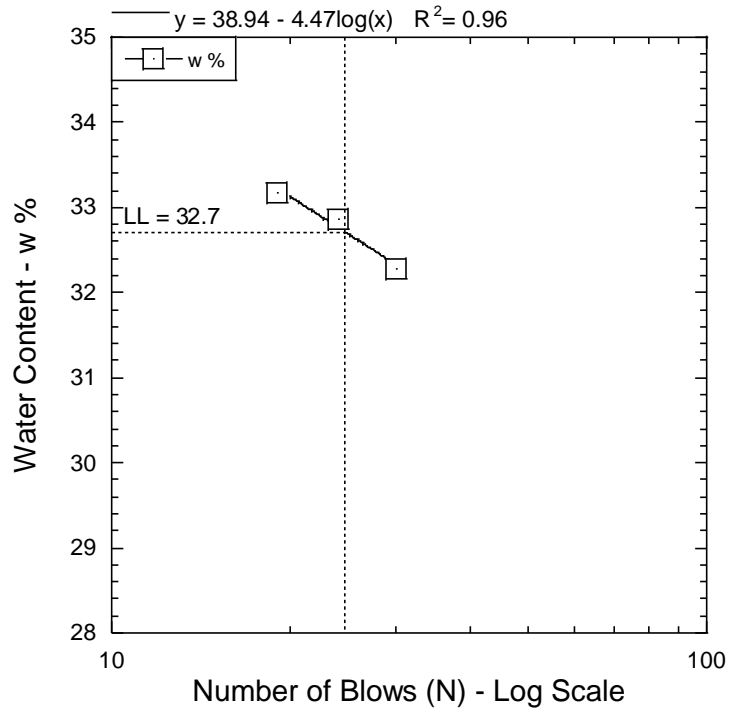


Figure A.4 Liquid limit test results for NFC – test number four

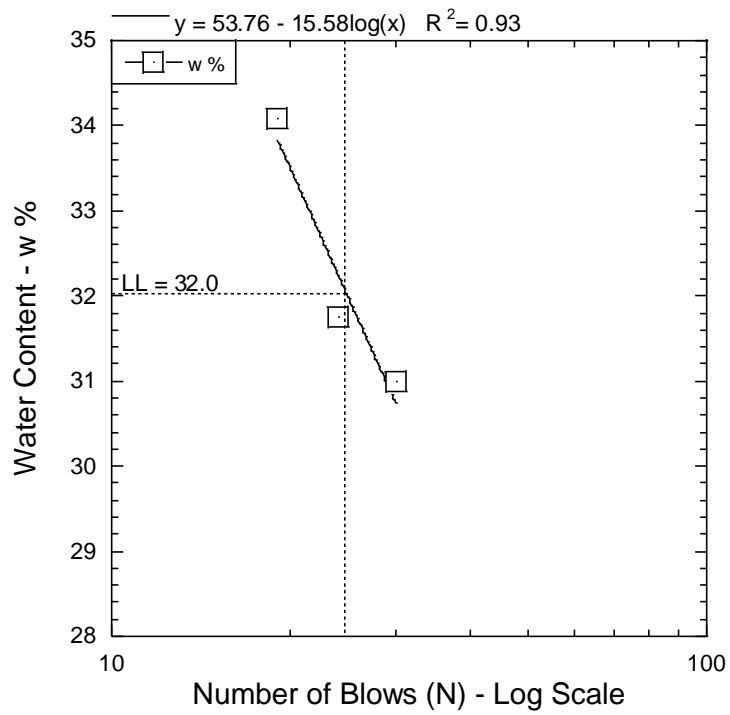


Figure A.5 Liquid limit test results for NFC – test number five

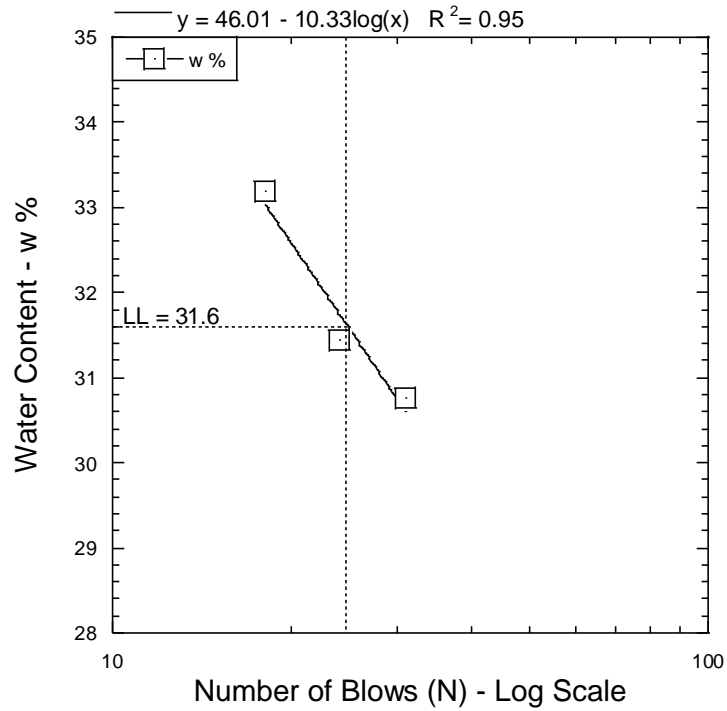


Figure A.6 Liquid limit test results for NFC – test number six

### A.2.1.2 Plastic Limit and Plasticity Index of the NFC

Table A.5 Plastic limit of NFC

Plastic Limit (PL) Test			
Test no.	1	2	3
PL [%]	18.66	17.81	17.79
PL [%] (Average)=	18.1		

Table A.6 Plasticity index (PI) of NFC

LL [%] (Average)	31.0
PL [%] (Average)	18.1
PI [%] =	12.9



**A.2.2 Expansive Soil (85% NFC + 15% Bentonite) Liquid Limit and Plastic Limit Test Results**

**A.2.2.1 Liquid Limit of Expansive Soil**

Table A.7 Liquid limit of expansive soil

Liquid Limit (LL) Test		
Test no.	1	2
LL [%]	76.7	74.8
LL [%] (Average)=	75.8	

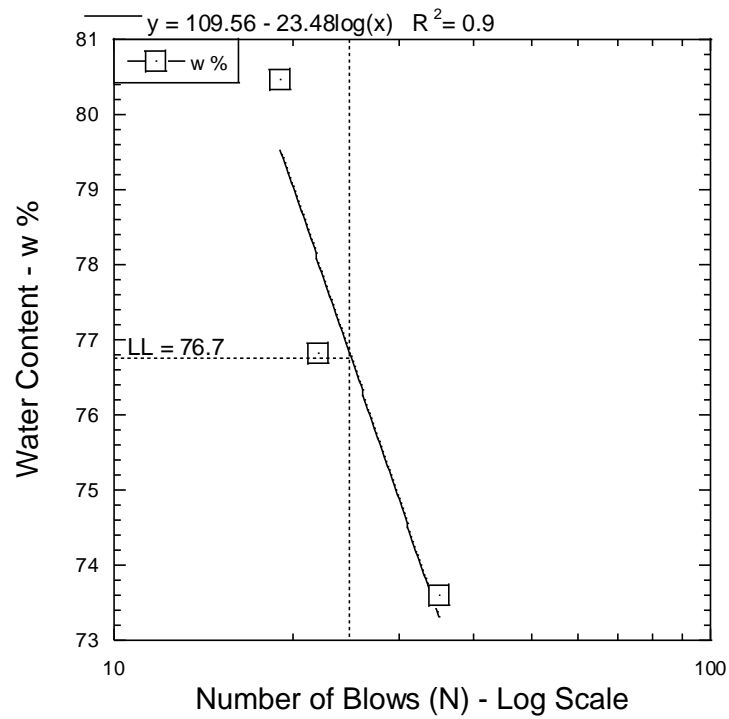


Figure A.7 Liquid limit test results on expansive soil – test number one

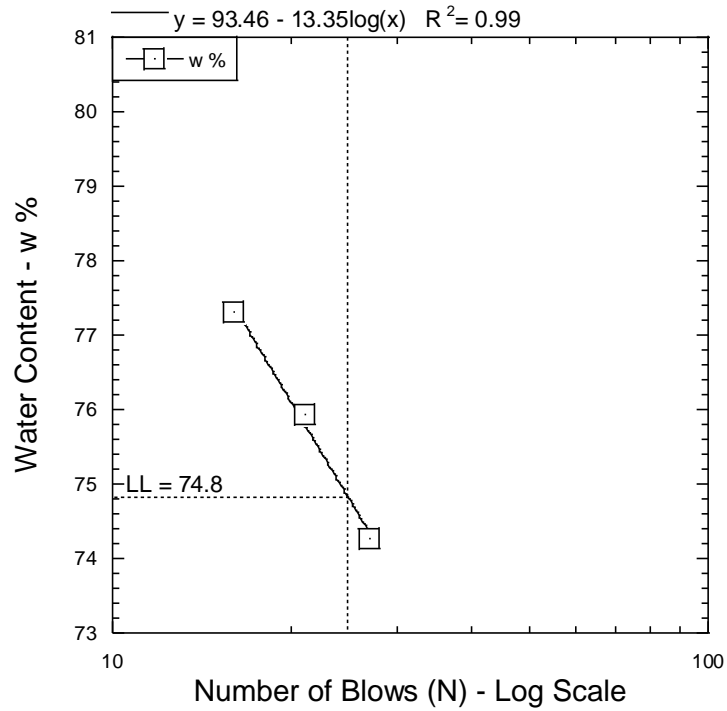


Figure A.8 Liquid limit test results on expansive soil – test number two

#### A.2.2.2 Plastic Limit and Plasticity Index of Expansive Soil

Table A.8 Plastic limit of expansive soil

Plastic Limit (PL) Test			
Test no.	1	2	3
PL [%]	18.86	17.02	17.35
PL [%] (Average)=	17.7		

Table A.9 Plasticity index (PI) of expansive soil

LL [%] (Average)=	75.8
PL [%] (Average)=	17.7
PI [%] =	58.0

**A.2.3 Fly Ash Treated (85% NFC + 15% Bentonite + 15% Fly Ash) LL and PL Tests**

**A.2.3.1 Liquid Limit Test of the Fly Ash Treated**

Table A.10 Liquid limit of fly ash treated soil

Liquid Limit (LL) Test		
Test no.	1	2
LL [%]	56.0	57.0
LL [%] (Average)=	56.5	

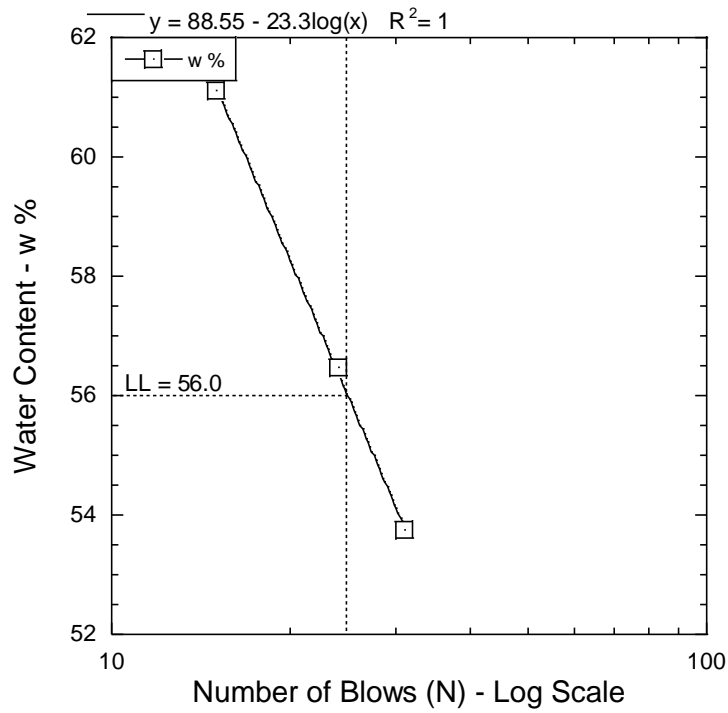


Figure A.9 Liquid limit test results on fly ash treated soil – test number one

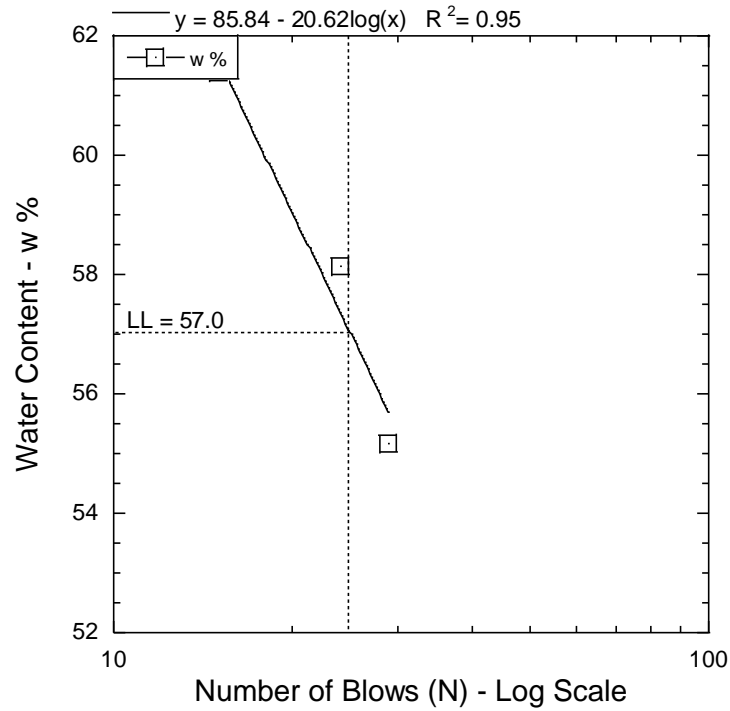


Figure A.10 Liquid limit test results on fly ash treated soil – test number two

### A.2.3.2 Plastic Limit and Plasticity Index of Fly Ash Treated Soil

Table A.11 Plastic limit of fly ash treated soil

Plastic Limit (PL) Test			
Test no.	1	2	3
PL [%]	17.56	17.99	17.21
PL [%] (Average)=	17.6		

Table A.12 Plasticity index (PI) of fly ash treated soil

Plasticity Index (PI) Result	
LL [%] (Average)=	56.5
PL [%] (Average)=	17.6
PI [%] =	38.9

**A.2.4 Lime Treated (85% NFC + 15% Bentonite + 3% Lime) LL and PL Test Results**

**A.2.4.1 Liquid Limit of Lime Treated Soil**

Table A.13 Liquid limit of lime treated soil

Liquid Limit (LL) Test		
Test no.	1	2
LL [%]	50.7	49.7
LL [%] (Average)=	50.2	

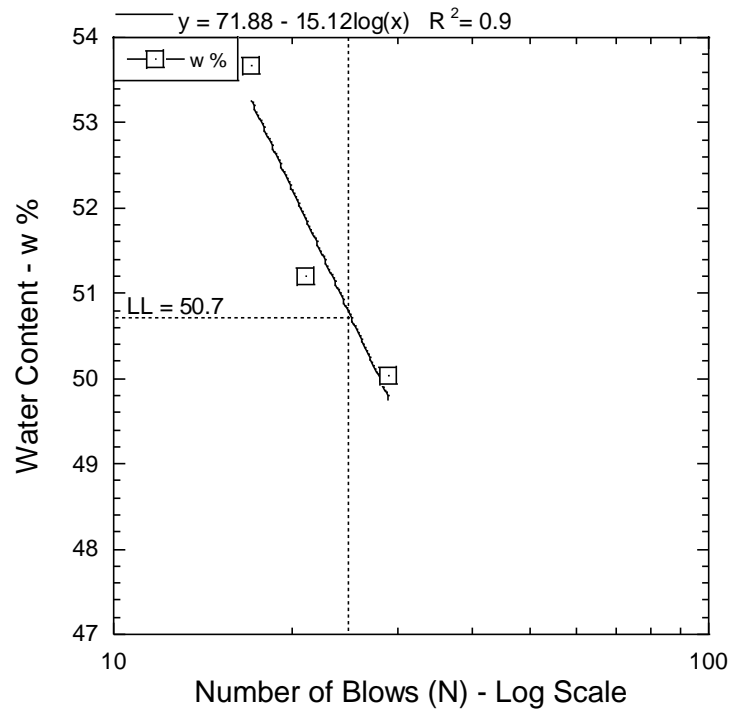


Figure A.11 Liquid limit test results on lime treated soil – test number one

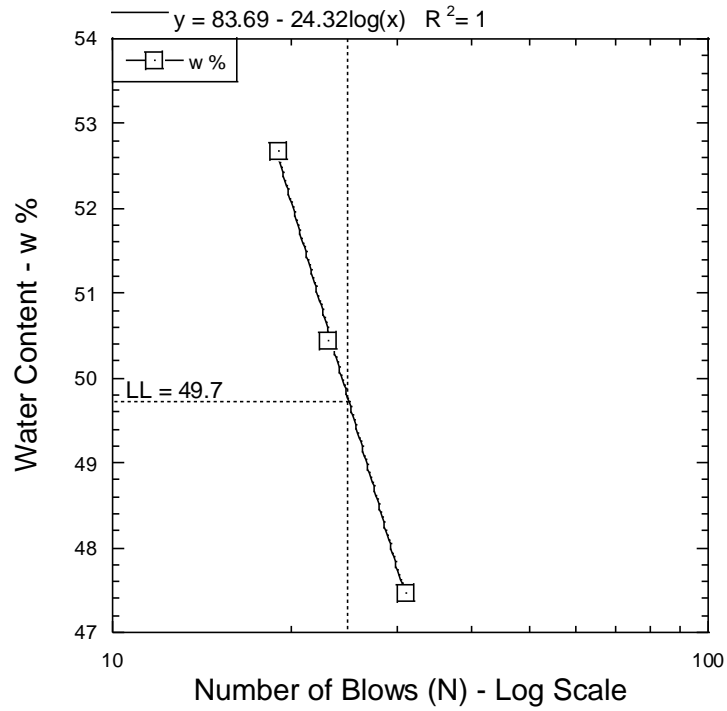


Figure A.12 Liquid limit test result on lime treated soil – test number two

#### A.2.4.2 Plastic Limit and Plasticity Index of Lime Treated Soil

Table A.14 Plastic limit of lime treated soil

Plastic Limit (PL) Test			
Test no.	1	2	3
PL [%]	31.8	32.0	33.8
PL [%] (Average)=	32.6		

Table A.15 Plasticity index (PI) of lime treated soil

LL [%] (Average)=	50.2
PL [%] (Average)=	32.6
PI [%] =	17.6

**A.2.5 P4 Treated (85% NFC + 15% Bentonite + 5% P4) LL and PL Tests Results**

**A.2.5.1 Liquid Limit of P4 Treated Soil**

Table A.16 Liquid limit of P4 treated soil

Liquid Limit (LL) Test		
Test no.	1	2
LL [%]	69.6	70.6
LL [%] (Average)=	70.1	

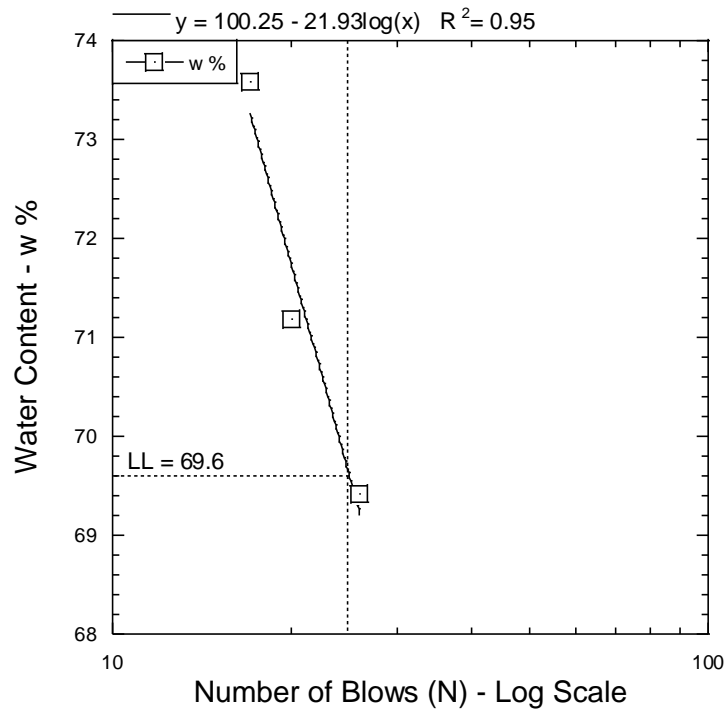


Figure A.13 Liquid limit test results on P4 treated soil – test number one

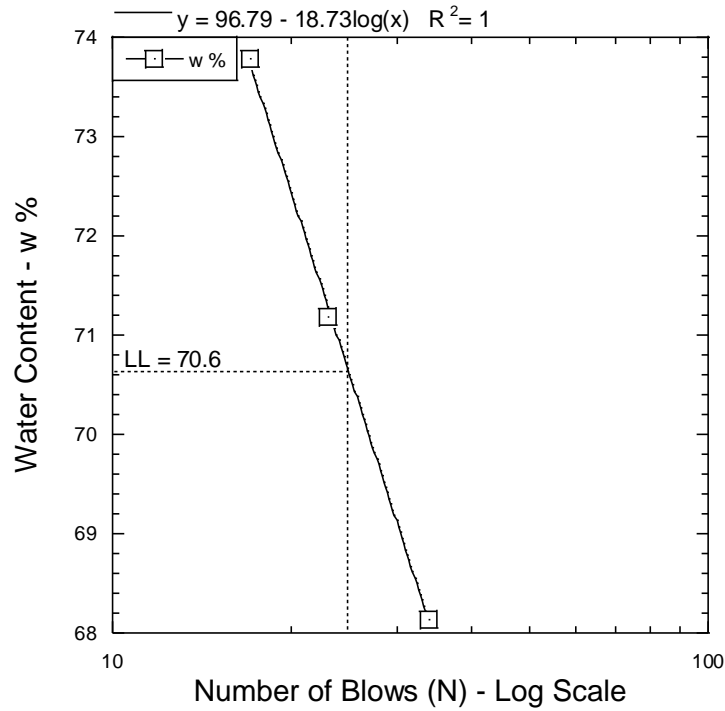


Figure A.14 Liquid limit test results on P4 treated soil – test number two

#### A.2.5.2 Plastic Limit and Plasticity Index of P4 Treated Soil

Table A.17 Plastic limit of P4 treated soil

Plastic Limit (PL) Test			
Test no.	1	2	3
PL [%]	18.2	20.0	20.2
PL [%] (Average)=	19.5		

Table A.18 Plasticity index (PI) of P4 treated soil

LL [%] (Average)=	70.1
PL [%] (Average)=	19.5
PI [%] =	50.6



## A.2.6 Maximum P4 Treated (Expansive soil + 27% P4) LL and PL Test Results

### A.2.6.1 Liquid Limit of Maximum P4 Treated Soil

Table A.19 Liquid limit of P4 treated soil

Liquid Limit (LL) Test	
Test no.	1
LL [%]	66.9
LL [%] (Final Result)=	66.9

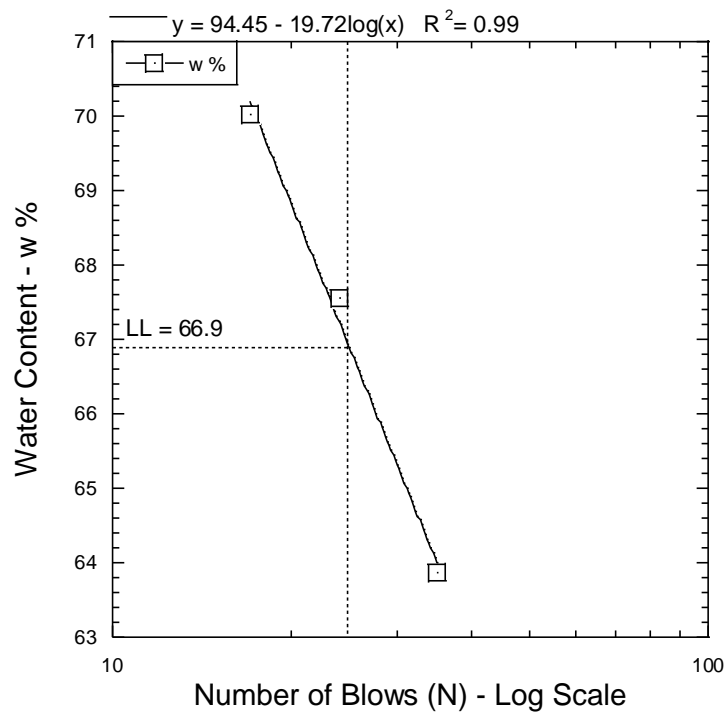


Figure A.15 Liquid limit test results on maximum (27%) P4 treated soil

### A.2.6.2 Plastic Limit and Plasticity Index of Maximum P4 Treated Soil

Table A.20 Plastic limit of 27% P4 treated

Plastic Limit (PL) Test			
Test no.	1	2	3
PL [%]	32.1	32.2	32.4
PL [%] (Average)=	32.2		

Table A.21 Plasticity index (PI) of 27% P4 treated

LL [%] (Average)=	66.9
PL [%] (Average)=	32.2
PI [%] =	34.7

### A.3 Standard Compaction Tests

#### A.3.1 The Standard Compaction Tests on NFC

Table A.22 Standard compaction tests on NFC

Optimum Water Content (w [%]) and Maximum Dry Unit Weight (γ <sub>d</sub> max [lb/ft <sup>3</sup> ])		
Test No.	1	2
w [%]	16.1	16.1
γ <sub>d</sub> max [lb/ft <sup>3</sup> ]	112.9	111.5
Average w [%] =	16.1	
Average γ <sub>d</sub> max [lb/ft <sup>3</sup> ] =	112.2	

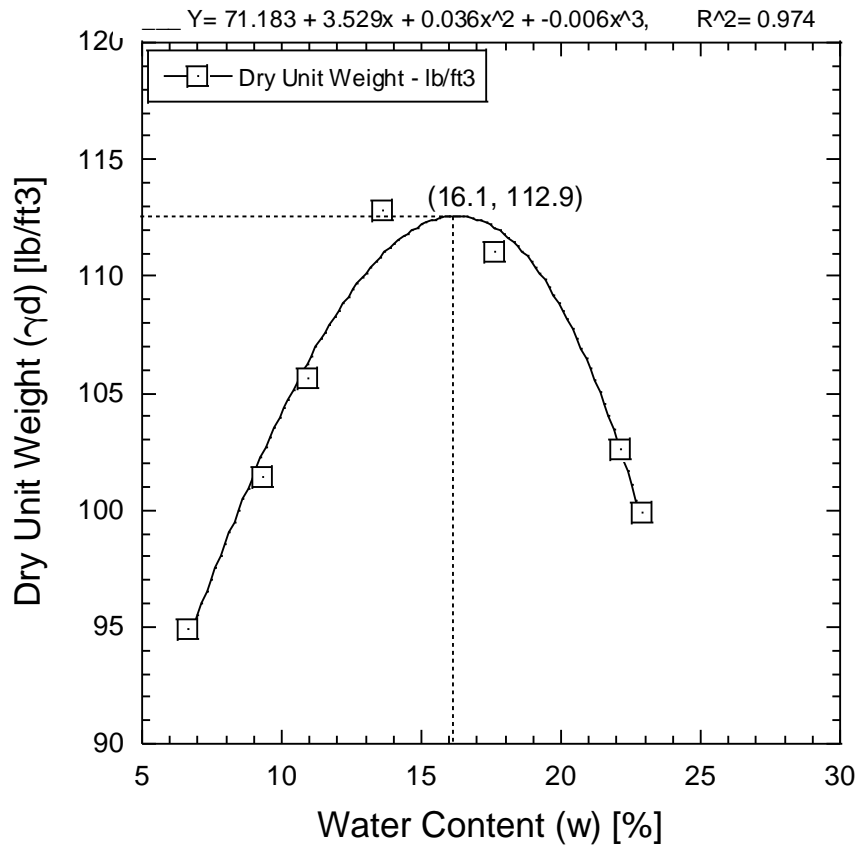


Figure A.16 Standard compaction results on NFC – test number one

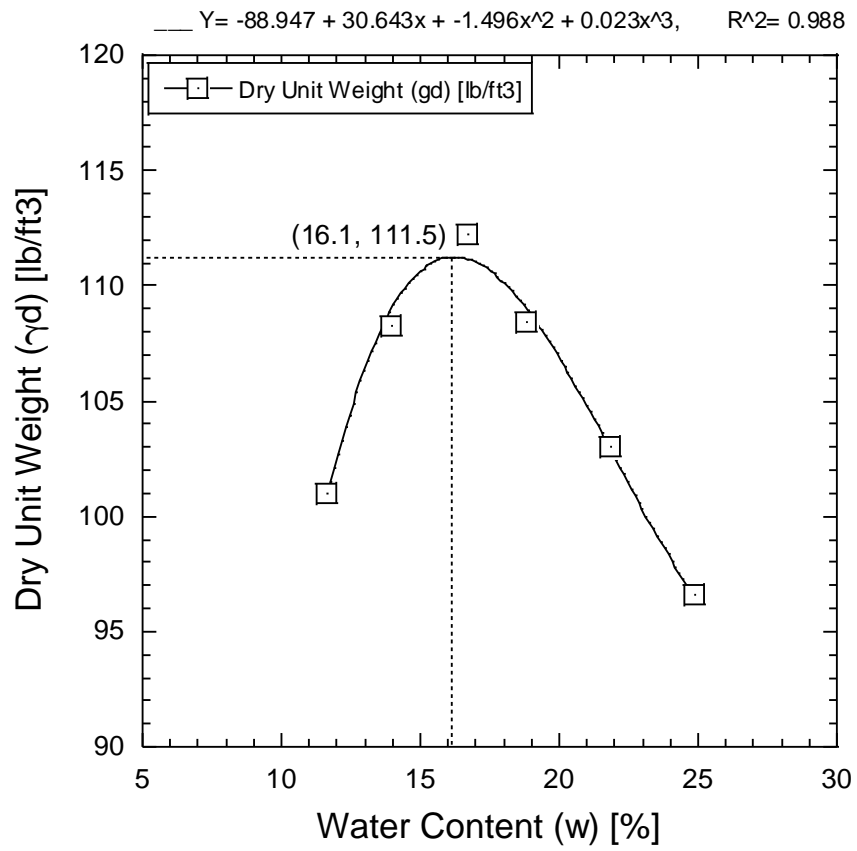


Figure A.17 Standard compaction test results on NFC – test number two

### A.3.2 The Standard Compaction Test on Expansive Soil

Table A.23 Standard compaction tests on expansive soil

Optimum Water Content (w [%]) and Maximum Dry Unit Weight ( $\gamma_{dmax}$ [lb/ft <sup>3</sup> ])		
Test No.	1	
w [%] =	18.4	
$\gamma_{dmax}$ [lb/ft <sup>3</sup> ] =	105.0	

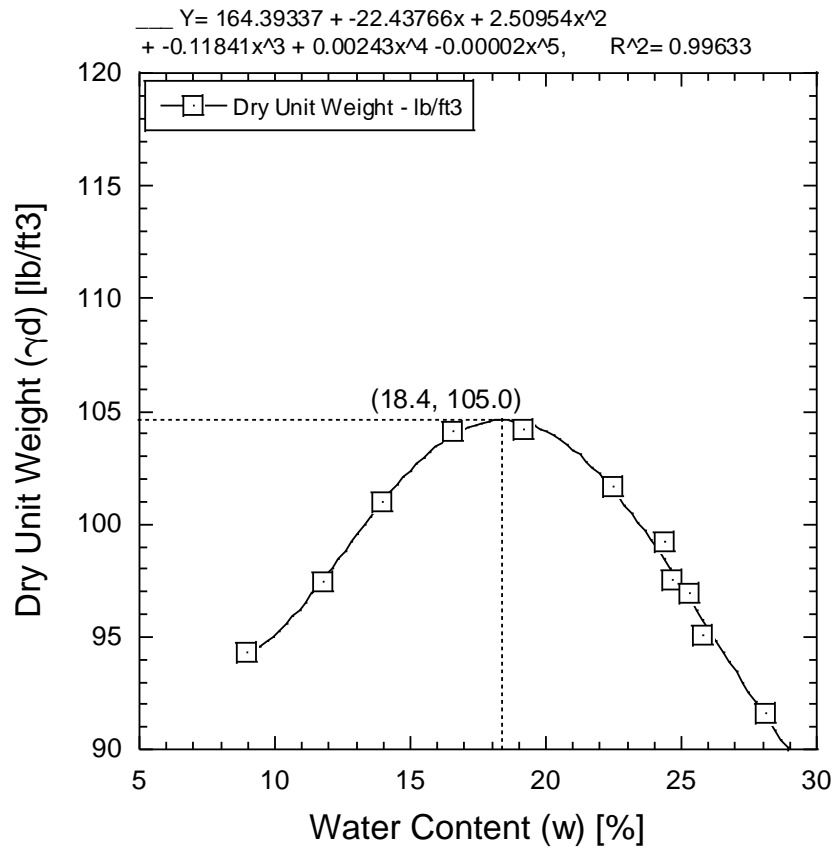


Figure A.18 Standard compaction test results on expansive soil

### A.3.3 Standard Compaction Test on Fly Ash Treated

Table A.24 Standard compaction tests on fly ash treated soil

Optimum Water Content (w [%]) and Maximum Dry Unit Weight ( $\gamma_{dmax}$ [lb/ft <sup>3</sup> ])		
Test No.	1	2
w [%]	17.0	18.0
$\gamma_{dmax}$ [lb/ft <sup>3</sup> ]	103.0	103.0
Average w [%] =	17.5	
Average $\gamma_{dmax}$ [lb/ft <sup>3</sup> ] =	103.0	

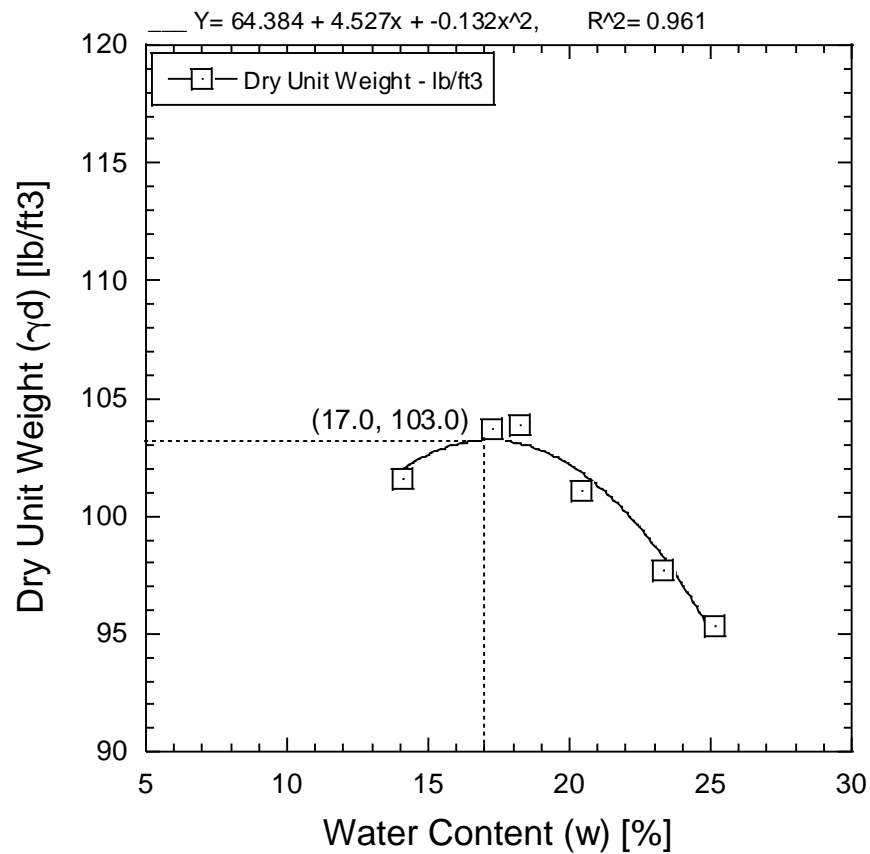


Figure A.19 Standard compaction test results on fly ash treated soil – test number one

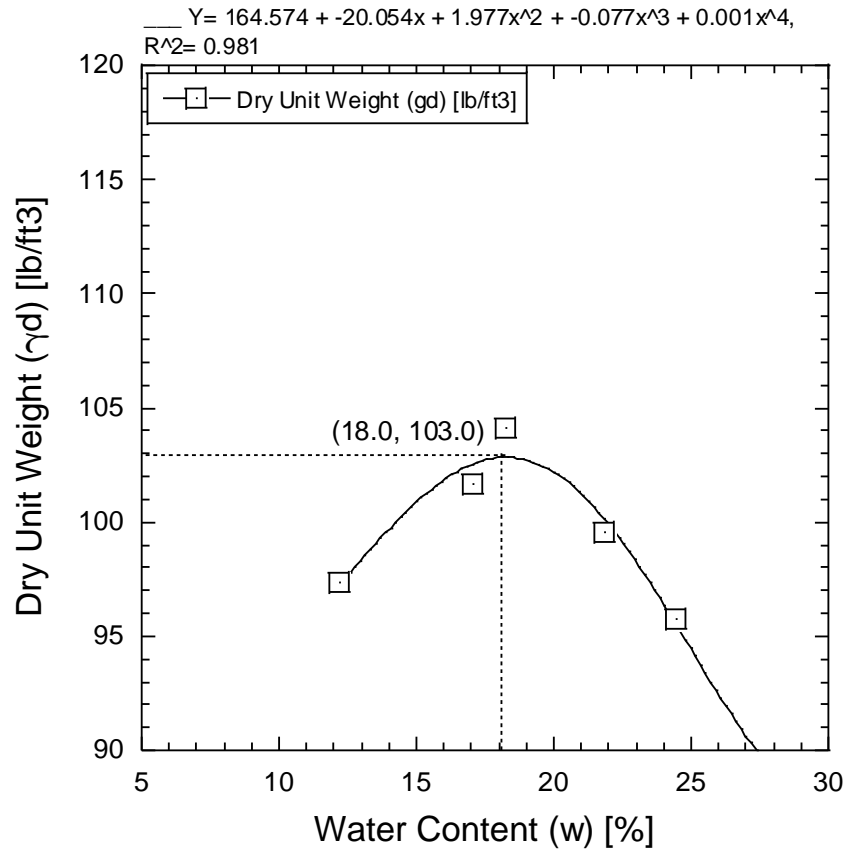


Figure A.20 Standard compaction test results on fly ash treated soil – test number two

### A.3.4 Standard Compaction Test of Lime Treated Soil

Table A.25 Standard compaction tests on lime treated

Optimum Water Content (w [%]) and Maximum Dry Unit Weight ( $\gamma_{dmax}$ [lb/ft <sup>3</sup> ])	
Test No.	1
w [%] =	20.0
$\gamma_{dmax}$ [lb/ft <sup>3</sup> ] =	101.1

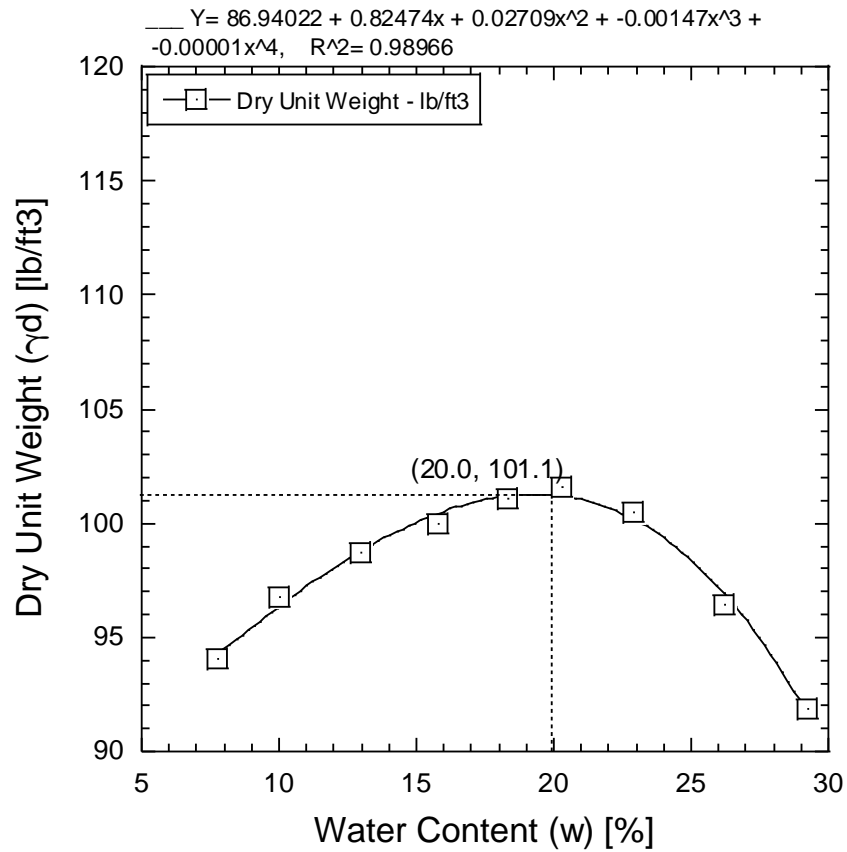


Figure A.21 Standard compaction results on lime treated soil

### A.3.5 Standard Compaction Test on P4 Treated Soil

Table A.26 Standard compaction test on P4 treated soil

Optimum Water Content (w [%]) and Maximum Dry Unit Weight ( $\gamma_{dmax}$ [lb/ft <sup>3</sup> ])	
Test No.	1
w [%] =	19.4
$\gamma_{dmax}$ [lb/ft <sup>3</sup> ] =	103.5

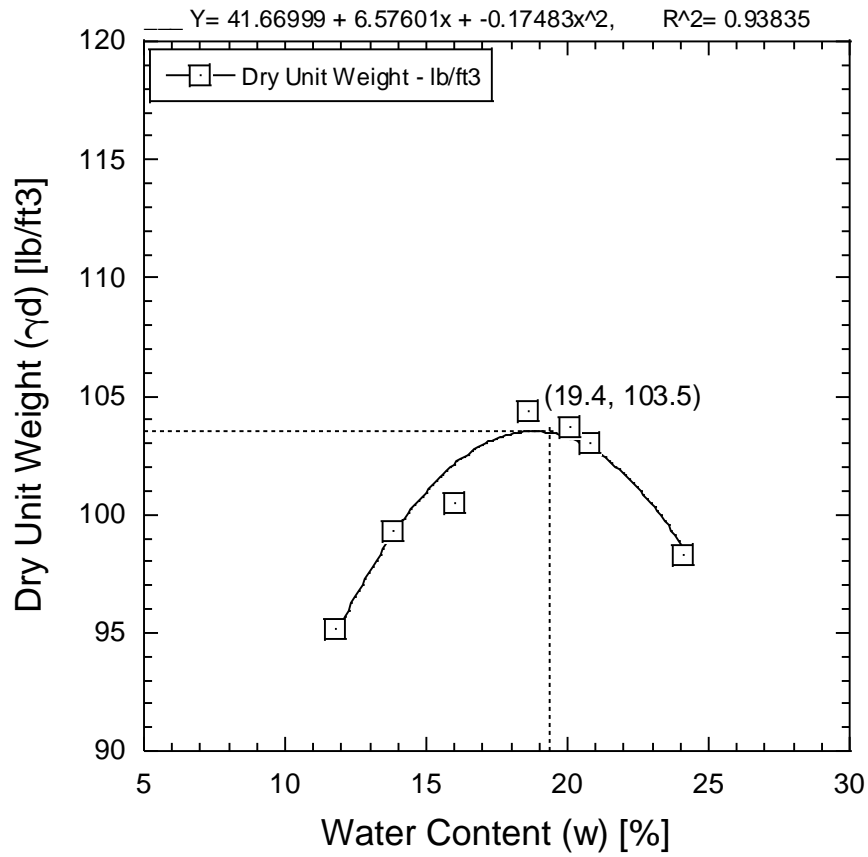


Figure A.22 Standard compaction test results on P4 treated soil



## A.4 Swell Tests

### A.4.1 NFC Swell Test

#### A.4.1.1 Swell Test Under 1 kPa

Table A.27 Swell test under 1 kPa on NFC

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two Inch
Cell height, h=	25.4	mm	One inch
Cell weight=	558.5	g	
Water content, w%=	16.1	%	Optimum Water Content
Soil Total weight, Wt=	147	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	126.6	g	
Height of solids, hs=	14.8	mm	
Compression prior to wetting, dh1=	-0.130	mm	Immediately before wetting
Specimen height prior to wetting, h1=	25.3	mm	Immediately before wetting
Swell/Collapse caused by wetting dh2=	0.085	mm	Linear swell
Swell/Collapse= 100 x dh2/h1=	0.34	%	% swell

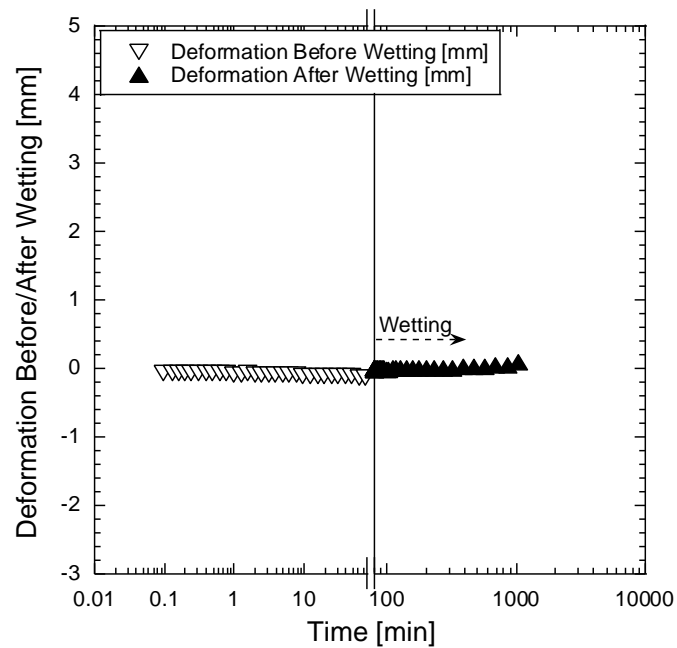


Figure A.23 Swell test under 1 kPa on NFC

**A.4.1.2 Swell Test Under 20 kPa:**

Table A.28 Swell test under 20 kPa on NFC

Swell Test	Values	Units	Comments
Pressure to be applied=	20	kPa	
Load to be placed=	809.45	g	Arm= 10x
Cell diameter, d=	71.1	mm	More than Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	348.5	g	
Water content, w%=	16.1	%	Optimum Water Content
Soil Total weight, Wt=	195.9	g	
Volume of soil, V=	100966.1	mm <sup>3</sup>	
Weight of solids, Ws=	168.7	g	
Height of solids, hs=	19.7	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.974	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	24.5	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	-0.116	mm	Linear Collapse
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	-0.47	%	% Collapse

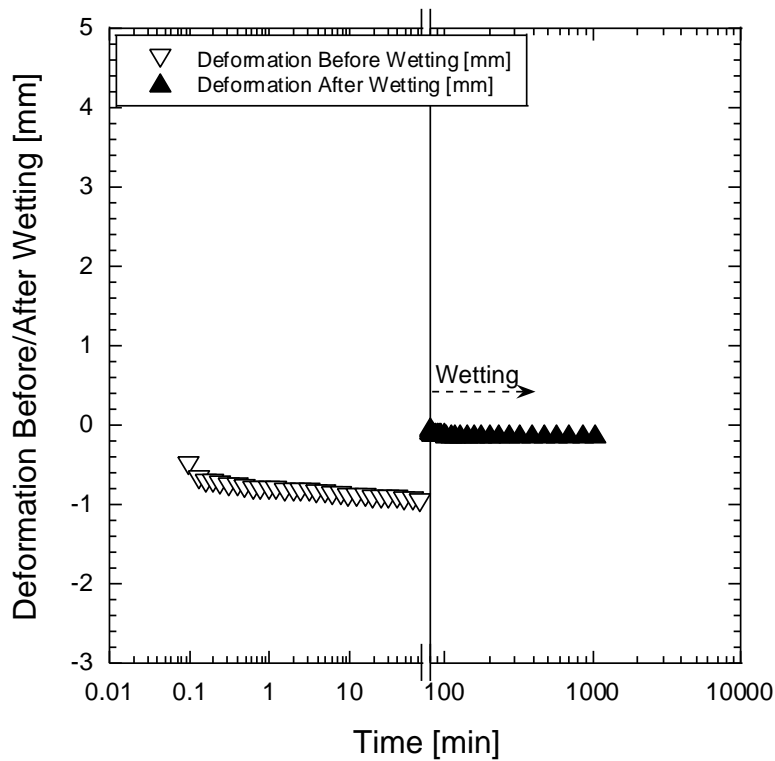


Figure A.24 Swell test under 20 kPa on NFC

**A.4.1.3 Swell Test Under 50 kPa:**

Table A.29 Swell test under 50 kPa on NFC

Swell Test	Values	Units	Comments
Pressure to be applied=	50	kPa	
Load to be placed=	1606.0	g	Arm= 10x
Cell diameter, d=	63.34	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	524	g	
Water content, w%=	16.1	%	Optimum Water Content
Soil Total weight, Wt=	152.3	g	
Volume of soil, V=	79877.4	mm <sup>3</sup>	
Weight of solids, W <sub>s</sub> =	131.2	g	
Height of solids, h <sub>s</sub> =	15.3	mm	
Compression prior to wetting, dh <sub>1</sub> =	-2.164	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	23.2	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	-0.079	mm	Linear Collapse
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	-0.34	%	% Collapse

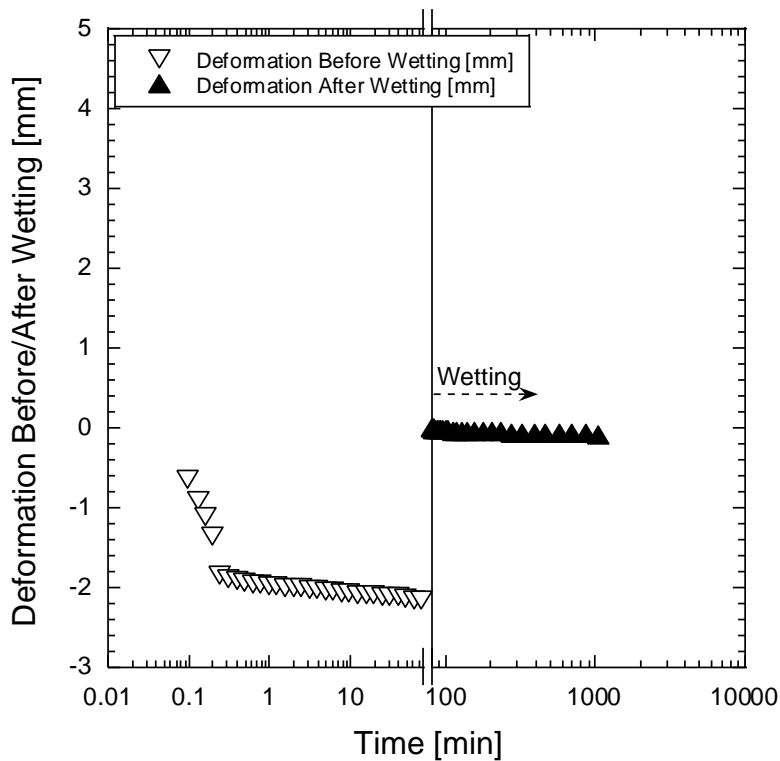


Figure A.25 Swell test under 50 kPa on NFC

#### A.4.1.4 Swell Test Under 100 kpa:

Table A.30 Swell test under 100 kPa on NFC

Swell Test	Values	Units	Comments
Pressure to be applied=	100	kPa	
Load to be placed=	3223.2	g	Arm= 10x
Cell diameter, d=	63.45	mm	Two inches
Cell height, h=	25.3	mm	One inch
Cell weight=	520	g	
Water content, w%=	16.1	%	Optimum Water Content
Soil Total weight, Wt=	160.1	g	
Volume of soil, V=	79965.4	mm <sup>3</sup>	
Weight of solids, Ws=	137.9	g	
Height of solids, hs=	16.1	mm	
Compression prior to wetting, dh <sub>1</sub> =	-2.968	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	22.3	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	-0.093	mm	Linear Collapse
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	-0.42	%	% Collapse

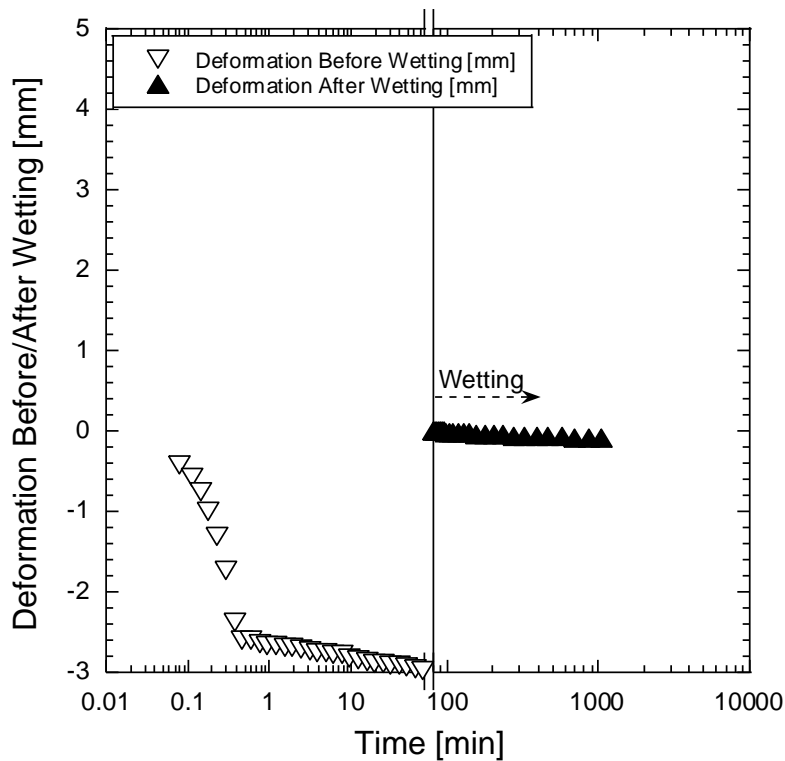


Figure A.26 Swell test under 100 kPa on NFC

**A.4.1.5 Swell Pressure and Swell Potential:**

Table A.31 Swell pressure and swell potential of NFC

Swell Pressure [kPa]	Swell Potential [%]
1	0.3
20	-0.5
50	-0.3
100	-0.4

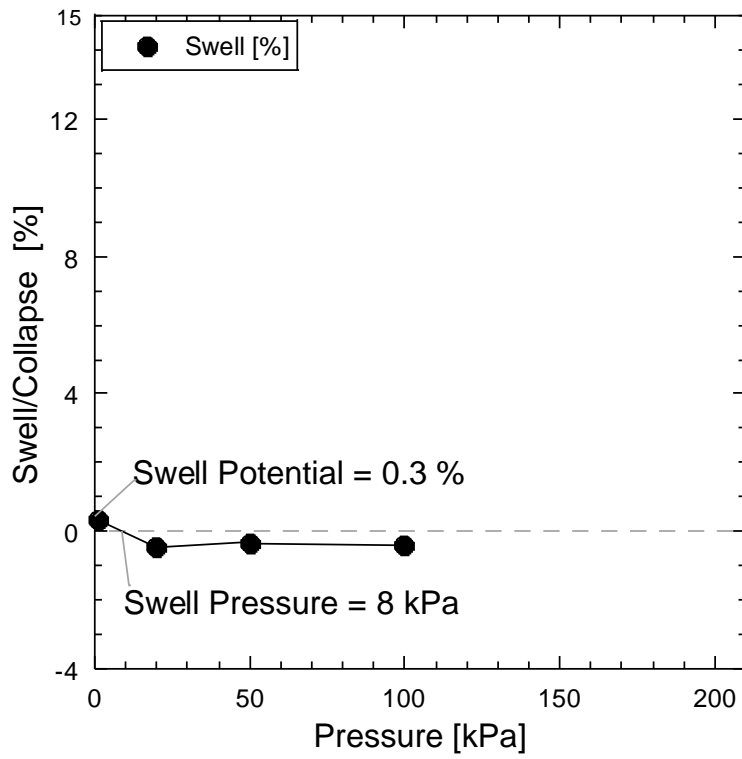


Figure A.27 Swell pressure and swell potential of NFC

## A.4.2 Expansive Soil Swell Test

### A.4.2.1 Swell Test Under 1 kPa

Table A.32 Swell test under 1 kPa on expansive soil

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	30.0	g	Arm= 10x
Cell diameter, d=	61.19	mm	Two inches
Cell height, h=	25.6	mm	One inch
Cell weight=	558.5	g	
Water content, w%=	17.18	%	~Optimum water content
Soil Total weight, Wt=	150	g	
Volume of soil, V=	75370.2	mm <sup>3</sup>	
Weight of solids, Ws=	128.0	g	
Height of solids, hs=	15.0	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.011	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.6	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	3.828	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	14.9	%	% swell

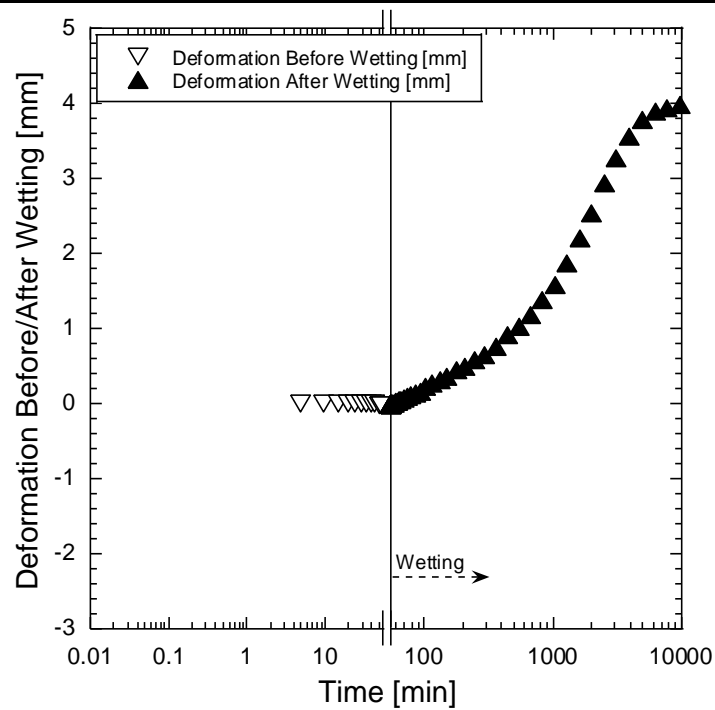


Figure A.28 Swell test under 1 kPa on expansive soil

#### A.4.2.2 Swell Test Under 10 kPa:

Table A.33 Swell test under 10 kPa on expansive soil

Swell Test	Values	Units	Comments
Pressure to be applied=	10	kPa	
Load to be placed=	321.6	g	Arm= 10x
Cell diameter, d=	63.38	mm	Two inches
Cell height, h=	25.3	mm	One inch
Cell weight=	519	g	
Water content, w%=	17.18	%	~Optimum water content
Soil Total weight, Wt=	158	g	
Volume of soil, V=	79820.6	mm <sup>3</sup>	
Weight of solids, Ws=	134.8	g	
Height of solids, hs=	15.8	mm	
Compression prior to wetting, dh1=	-0.169	mm	Immediately before wetting
Specimen height prior to wetting, h1=	25.1	mm	Immediately before wetting
Swell/Collapse caused by wetting dh2=	1.389	mm	Linear swell
Swell/Collapse= 100 x dh2/h1=	5.5	%	% swell

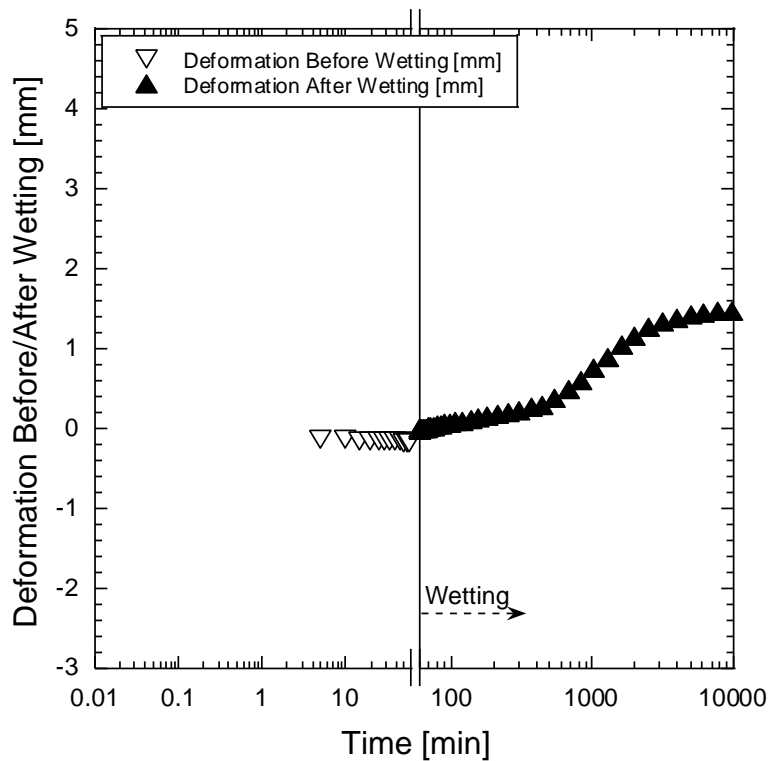


Figure A.29 The swell test under 10 kPa on expansive soil

### A.4.2.3 Swell Test Under 50 kPa:

Table A.34 Swell test under 50 kPa on expansive soil

Swell Test	Values	Units	Comments
Pressure to be applied=	50	kPa	
Load to be placed=	1606.0	g	Arm= 10x
Cell diameter, d=	63.34	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	524	g	
Water content, w%=	17.18	%	~Optimum water content
Soil Total weight, Wt=	152.3	g	
Volume of soil, V=	79877.4	mm <sup>3</sup>	
Weight of solids, Ws=	130.0	g	
Height of solids, hs=	15.2	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.238	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.1	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.367	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	1.5	%	% swell

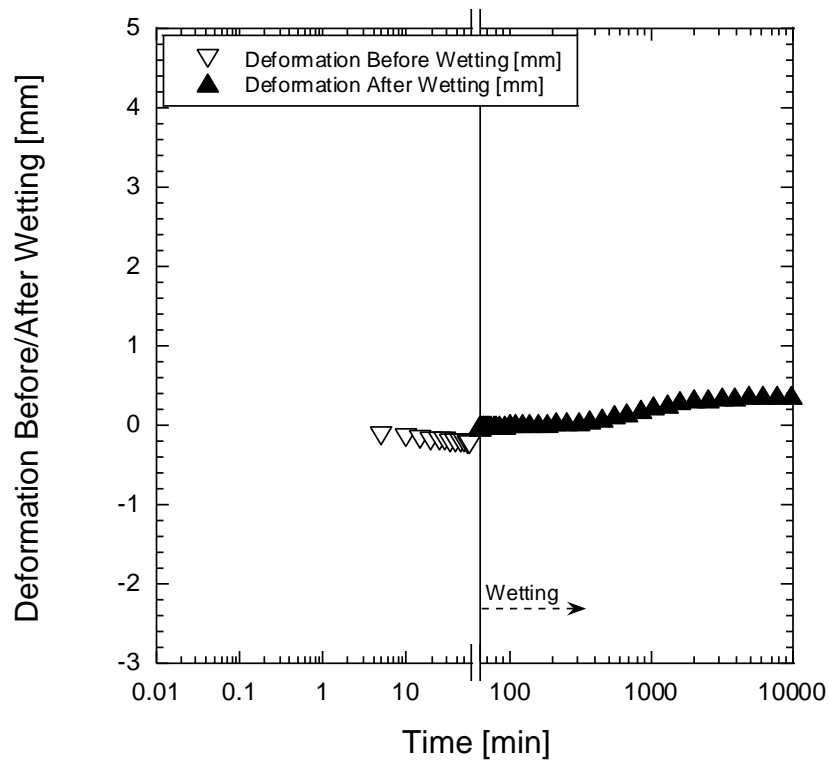


Figure A.30 Swell test under 50 kPa on expansive soil



#### A.4.2.4 Swell Test Under 100 kPa:

Table A.35 Swell test under 100 kPa on expansive soil

Swell Test	Values	Units	Comments
Pressure to be applied=	100	kPa	
Load to be placed=	3223.2	g	Arm= 10x
Cell diameter, d=	63.45	mm	Two inches
Cell height, h=	25.3	mm	One inch
Cell weight=	520	g	
Water content, w%=	17.18	%	~Optimum water content
Soil Total weight, Wt=	160.1	g	
Volume of soil, V=	79965.4	mm <sup>3</sup>	
Weight of solids, Ws=	136.6	g	
Height of solids, hs=	16.0	mm	
Compression prior to wetting, dh <sub>1</sub> =	-1.116	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	24.2	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.095	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.4	%	% swell

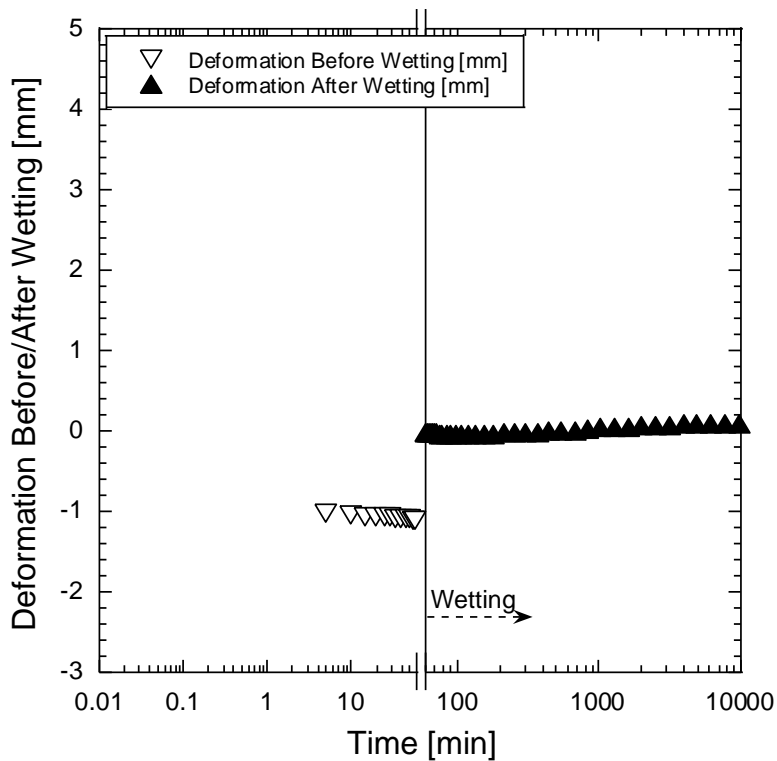


Figure A.31 Swell test under 100 kPa on expansive soil

#### A.4.2.5 Swell Test Under 207.3 kPa:

Table A.36 Swell test under 207.3 kPa on expansive soil

Swell Test	Values	Units	Comments
Pressure to be applied=	207.27	kPa	
Load to be placed=	6657.5	g	Arm= 10x
Cell diameter, d=	63.34	mm	Two inches
Cell height, h=	25.5	mm	One inch
Cell weight=	520	g	
Water content, w%=	18.4	%	~Optimum water content
Soil Total weight, Wt=	161.5	g	
Volume of soil, V=	80192.5	mm <sup>3</sup>	
Weight of solids, Ws=	136.4	g	
Height of solids, hs=	15.9	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.804	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	24.6	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	-0.143	mm	Linear Collapse
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	-0.6	%	% Collapse

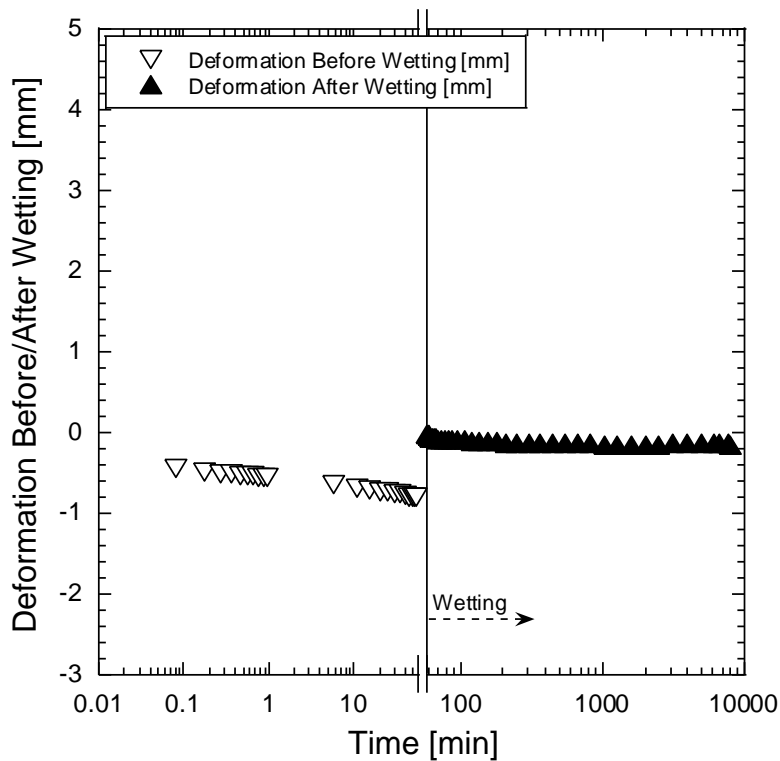


Figure A.32 Swell test under 207.3 kPa on expansive soil

#### A.4.2.6 Swell Pressure and Swell Potential:

Table A.37 Swell pressure and swell potential on expansive soil

Swell Pressure [kPa]	Swell Potential [%]
1	14.9
10	5.5
50	1.5
100	0.4
207.3	-0.6

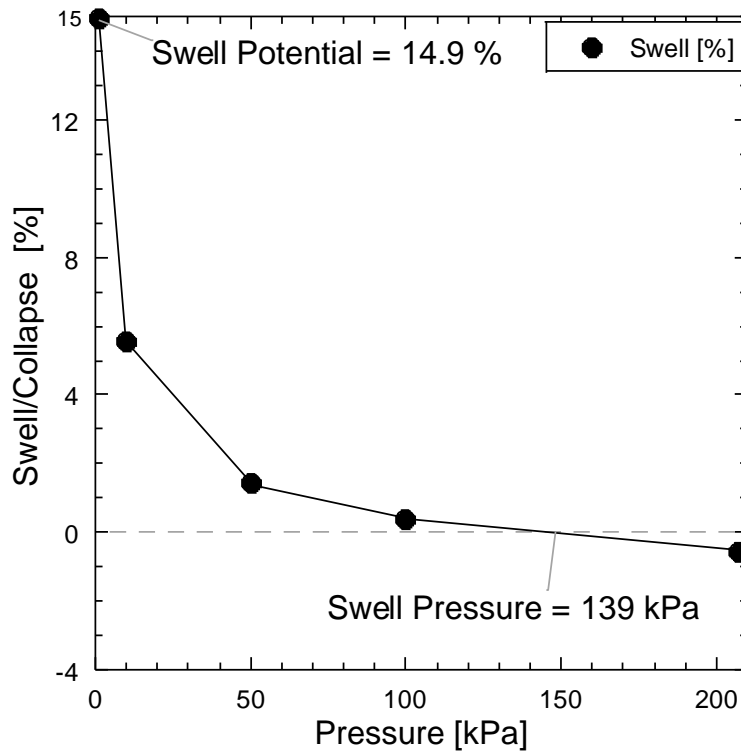


Figure A.33 Swell test under different pressures

### A.4.3 Fly Ash Treated Soil Swell Test (15%)

#### A.4.3.1 Swell Test Under 1 kPa

Table A.38 Swell test under 1 kPa on fly ash treated soil

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	30.0	g	Arm= 10x
Cell diameter, d=	61.19	mm	Two inches
Cell height, h=	25.6	mm	One inch
Cell weight=	558.5	g	
Water content, w%=	17.18	%	~Optimum water content
Soil Total weight, Wt=	152.5	g	
Volume of soil, V=	75370.2	mm <sup>3</sup>	
Weight of solids, Ws=	130.1	g	
Height of solids, hs=	15.2	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.070	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.6	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.115	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.5	%	% swell

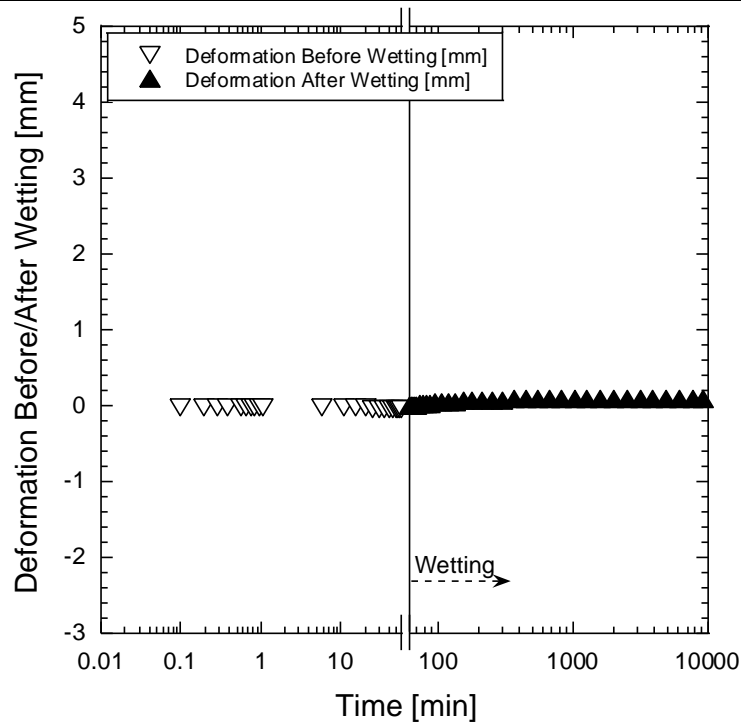


Figure A.34 Swell test under 1 kPa on fly ash treated soil

### A.4.3.2 Swell Test Under 10 kPa:

Table A.39 Swell test under 10 kPa on fly ash treated soil

Swell Test	Values	Units	Comments
Pressure to be applied=	10	kPa	
Load to be placed=	321.6	g	Arm= 10x
Cell diameter, d=	63.38	mm	Two inches
Cell height, h=	25.3	mm	One inch
Cell weight=	519	g	
Water content, w%=	17.18	%	~Optimum water content
Soil Total weight, Wt=	158	g	
Volume of soil, V=	79820.6	mm <sup>3</sup>	
Weight of solids, Ws=	134.8	g	
Height of solids, hs=	15.8	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.037	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.3	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.099	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.4	%	% swell

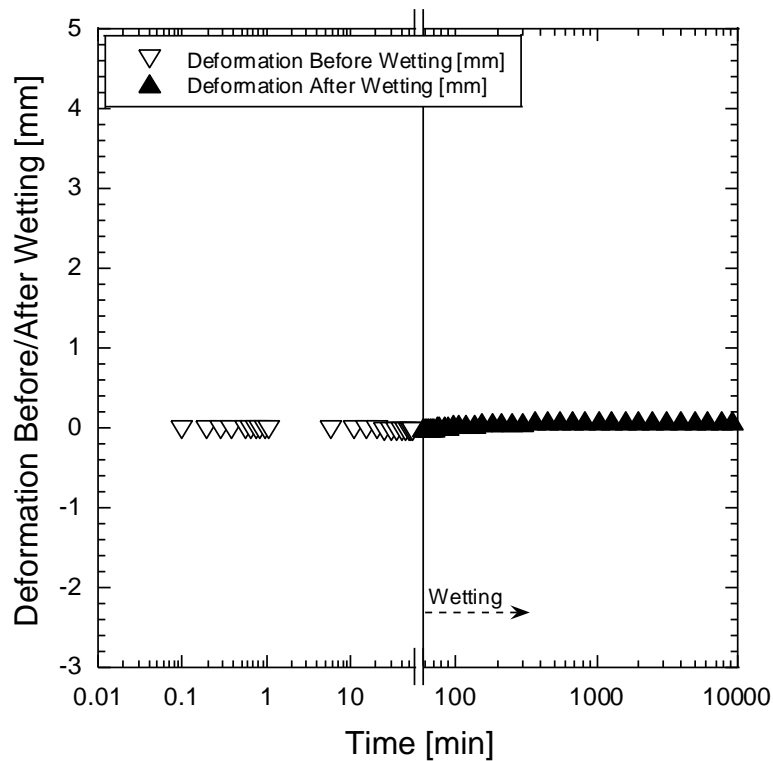


Figure A.35 Swell test under 10 kPa on fly ash treated soil

### A.4.3.3 Swell Test Under 50 kPa

Table A.40 Swell test under 50 kPa on fly ash treated soil

Swell Test	Values	Units	Comments
Pressure to be applied=	50	kPa	
Load to be placed=	1606.0	g	Arm= 10x
Cell diameter, d=	63.34	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	524	g	
Water content, w%=	17.18	%	~Optimum water content
Soil Total weight, Wt=	152.3	g	
Volume of soil, V=	79877.4	mm <sup>3</sup>	
Weight of solids, Ws=	130.0	g	
Height of solids, hs=	15.2	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.110	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.2	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	-0.104	mm	Linear Collapse
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	-0.4	%	% Collapse

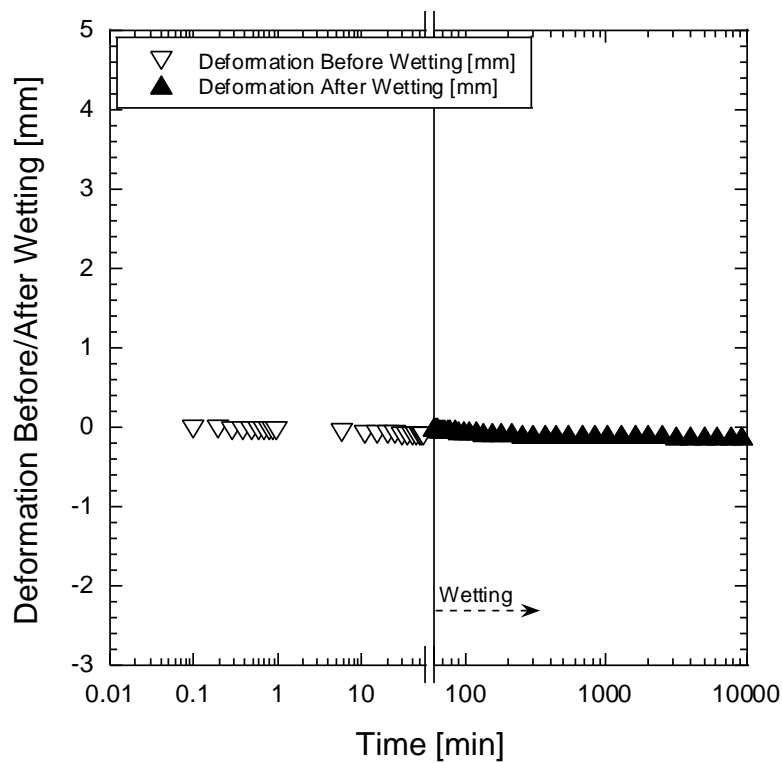


Figure A.36 Swell test under 50 kPa on fly ash treated soil

#### A.4.3.4 Swell Test Under 100 kPa

Table A.41 Swell test under 100 kPa on fly ash treated soil

Swell Test	Values	Units	Comments
Pressure to be applied=	100	kPa	
Load to be placed=	3223.2	g	Arm= 10x
Cell diameter, d=	63.45	mm	Two inches
Cell height, h=	25.3	mm	One inch
Cell weight=	520	g	
Water content, w%=	17.18	%	~Optimum water content
Soil Total weight, Wt=	160.1	g	
Volume of soil, V=	79965.4	mm <sup>3</sup>	
Weight of solids, Ws=	136.6	g	
Height of solids, hs=	16.0	mm	
Compression prior to wetting, dh <sub>1</sub> =	-1.348	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	23.9	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	-0.618	mm	Linear Collapse
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	-2.6	%	% Collapse

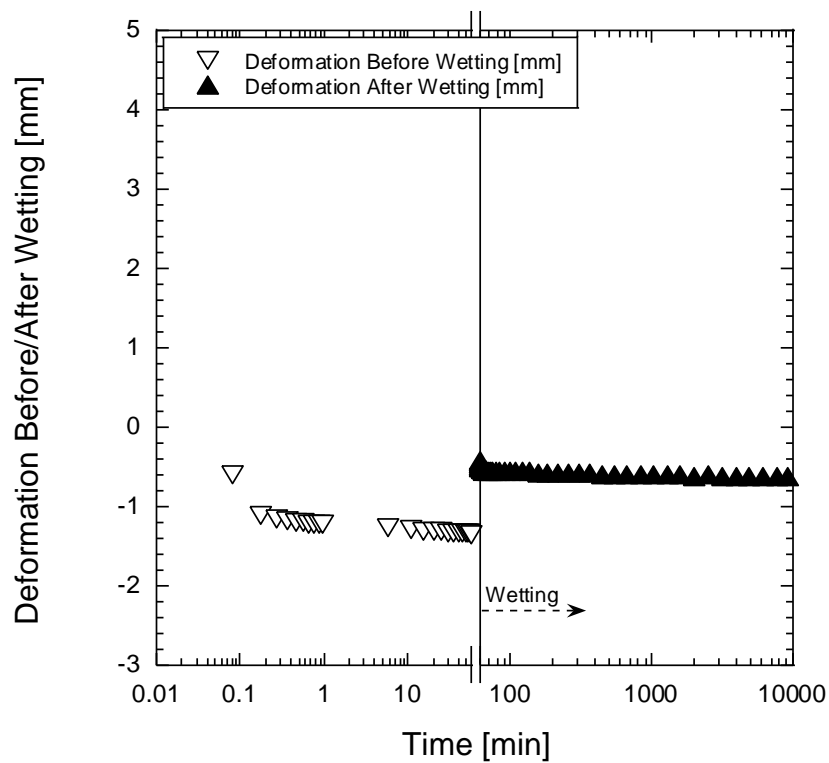


Figure A.37 Swell test under 100 kPa on fly ash treated soil

### A.4.3.5 Swell Pressure and Swell Potential

Table A.42 Swell pressure and swell potential on fly ash treated soil

Swell Pressure [kPa]	Swell Potential [%]
1	0.5
10	0.4
50	-0.4
100	-2.6

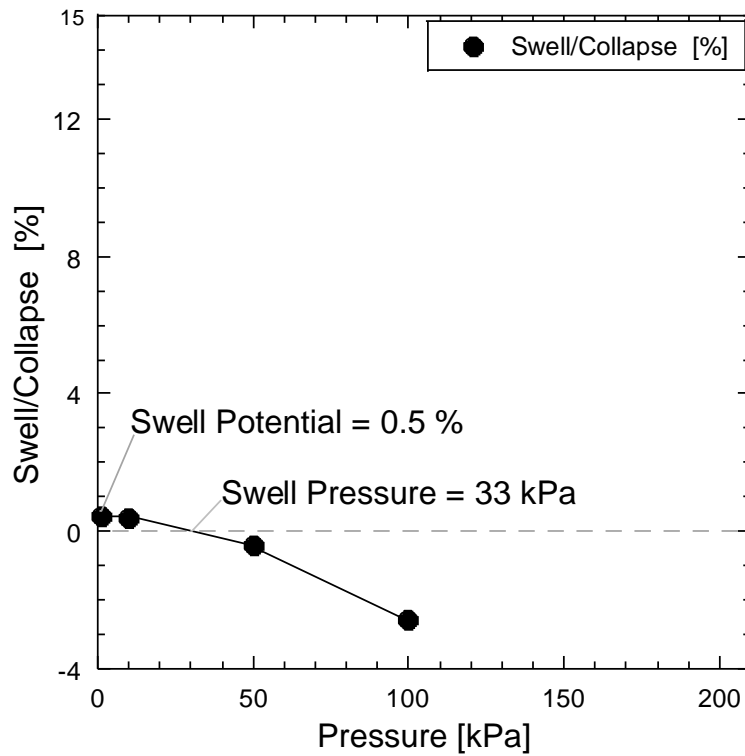


Figure A.38 Swell pressure and swell potential of fly ash treated soil



### A.4.4 Fly Ash Treated Swell Test - Additional Tests

#### A.4.4.1 Swell Test Under 1 kPa

Table A.43 The swell test under 1 kPa on fly ash treated soil – additional tests

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.2	g	Arm= 10x
Cell diameter, d=	63.4	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	558.5	g	
Water content, w%=	17.18	%	~Optimum water content
Soil Total weight, Wt=	152.5	g	
Volume of soil, V=	80186.7	mm <sup>3</sup>	
Weight of solids, Ws=	130.1	g	
Height of solids, hs=	15.2	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.038	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.014	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.1	%	% swell

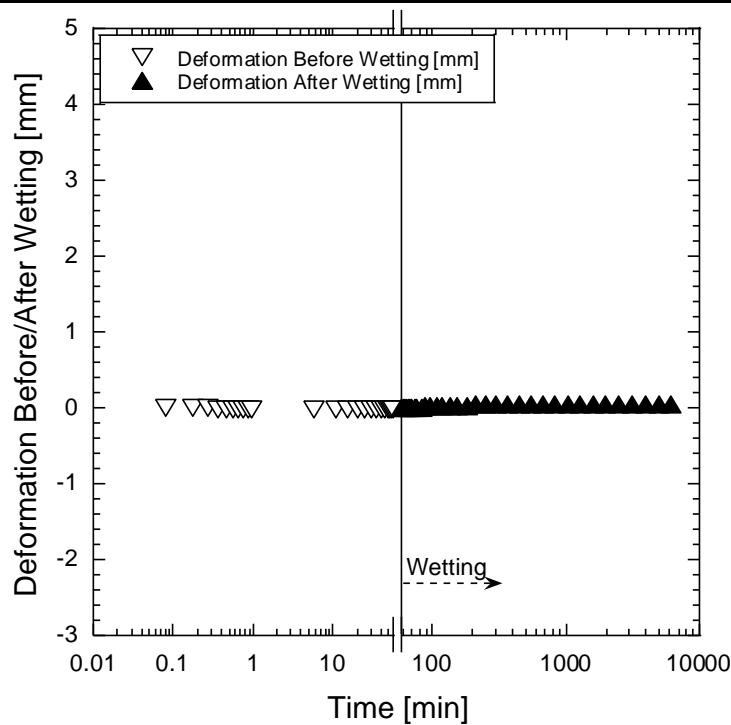


Figure A.39 Swell test under 1 kPa on fly ash treated soil – additional tests

#### A.4.4.2 Swell Test Under 10 kPa

Table A.44 Swell test under 10 kPa on fly ash treated soil – additional tests

Swell Test	Values	Units	Comments
Pressure to be applied=	10	kPa	
Load to be placed=	321.6	g	Arm= 10x
Cell diameter, d=	63.38	mm	Two inches
Cell height, h=	25.3	mm	One inch
Cell weight=	519	g	
Water content, w%=	17.18	%	~Optimum water content
Soil Total weight, Wt=	158	g	
Volume of soil, V=	79820.6	mm <sup>3</sup>	
Weight of solids, W <sub>s</sub> =	134.8	g	
Height of solids, h <sub>s</sub> =	15.8	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.029	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.3	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.048	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.2	%	% swell

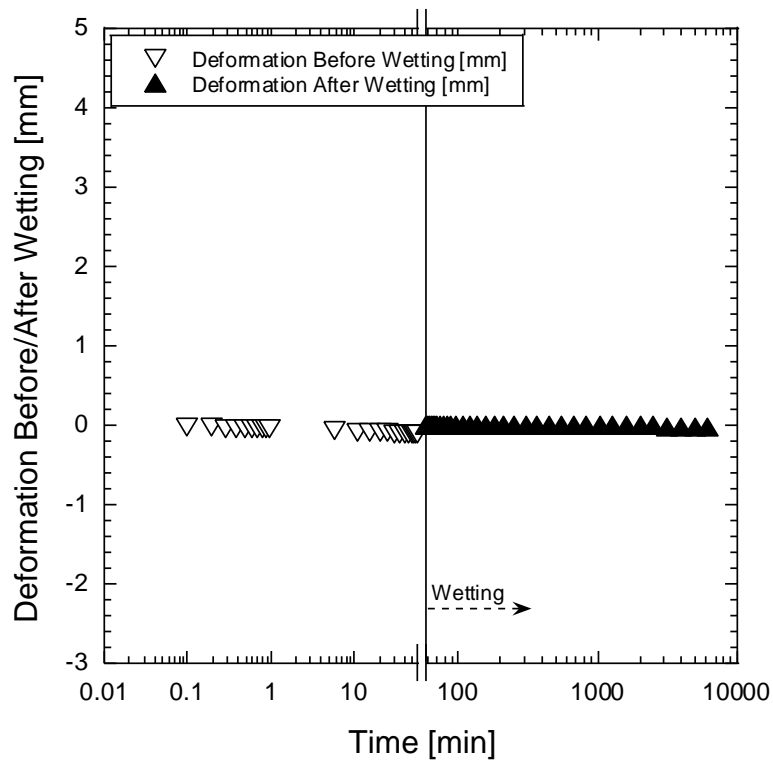


Figure A.40 Swell test under 10 kPa on fly ash treated soil – additional tests

### A.4.4.3 Swell Test Under 50 kPa

Table A.45 Swell test under 50 kPa on fly ash treated soil – additional tests

Swell Test	Values	Units	Comments
Pressure to be applied=	50	kPa	
Load to be placed=	1606.0	g	Arm= 10x
Cell diameter, d=	63.34	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	524	g	
Water content, w%=	17.18	%	~Optimum water content
Soil Total weight, Wt=	152.3	g	
Volume of soil, V=	79877.4	mm <sup>3</sup>	
Weight of solids, Ws=	130.0	g	
Height of solids, hs=	15.2	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.119	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.2	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	-0.016	mm	Linear Collapse
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	-0.1	%	% Collapse

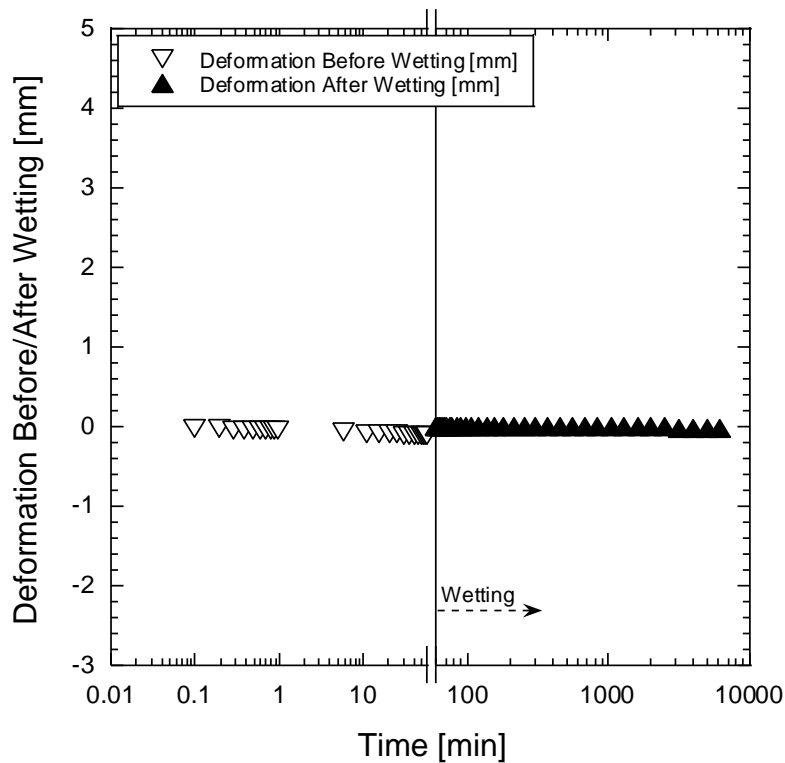


Figure A.41 Swell test under 50 kPa on fly ash treated soil – additional tests

#### A.4.4.4 Swell Test Under 100 kPa

Table A.46 Swell test under 100 kPa on fly ash treated – additional tests

Swell Test	Values	Units	Comments
Pressure to be applied=	100	kPa	
Load to be placed=	3223.2	g	Arm= 10x
Cell diameter, d=	63.45	mm	Two inches
Cell height, h=	25.3	mm	One inch
Cell weight=	520	g	
Water content, w%=	17.18	%	~Optimum water content
Soil Total weight, Wt=	160.1	g	
Volume of soil, V=	79965.4	mm <sup>3</sup>	
Weight of solids, Ws=	136.6	g	
Height of solids, hs=	16.0	mm	
Compression prior to wetting, dh <sub>1</sub> =	-1.228	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	24.1	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	-0.681	mm	Linear Collapse
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	-2.8	%	% Collapse

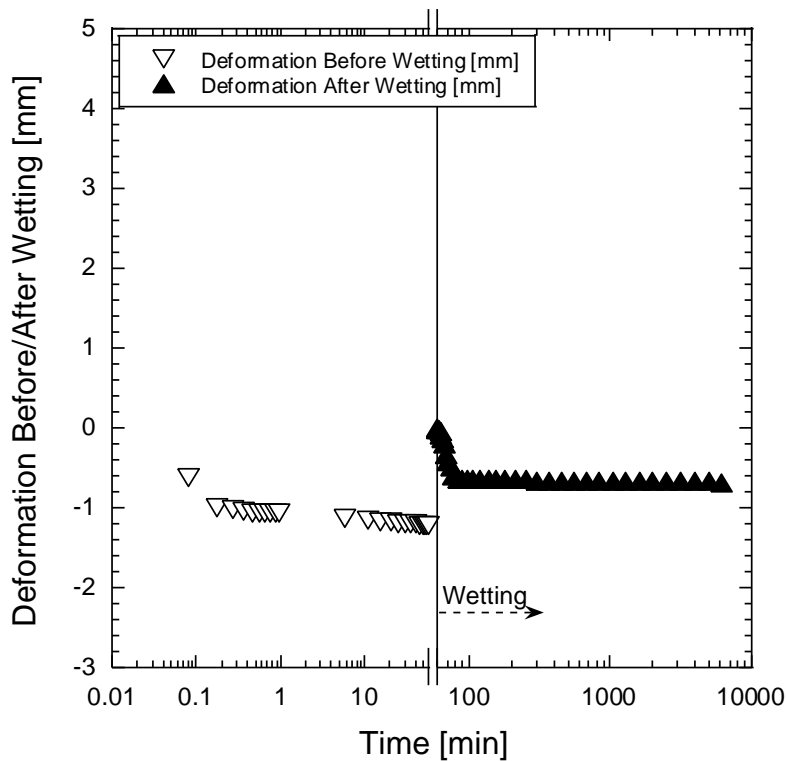


Figure A.42 Swell test under 100 kPa on fly ash treated soil – additional tests

#### A.4.4.5 Swell Pressure and Swell Potential

Table A.47 Swell pressure and swell potential on fly ash treated soil – additional tests

Swell Pressure [kPa]	Swell Potential [%]
1	0.1
10	0.2
50	-0.1
100	-2.8

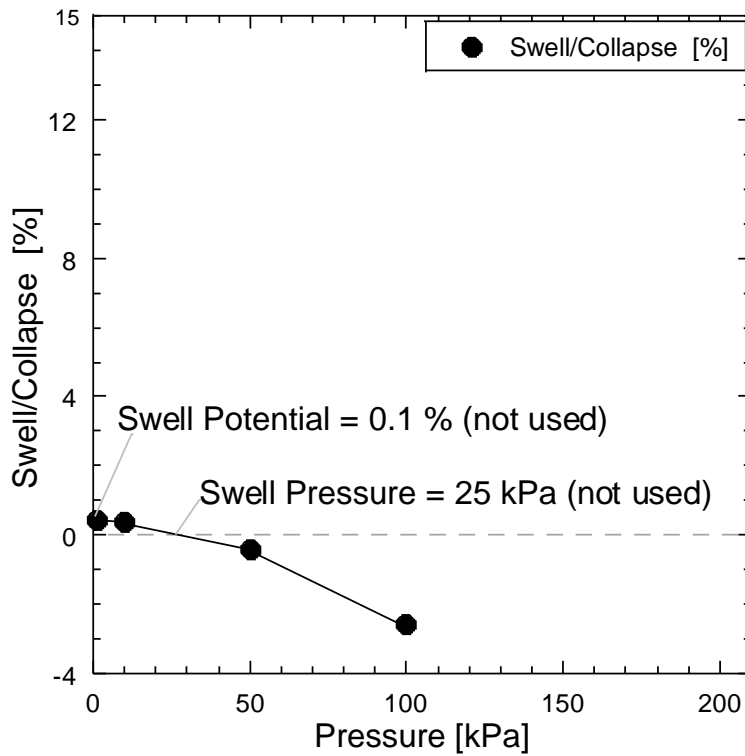


Figure A.43 Swell pressure and swell potential of fly ash treated soil – additional tests

### A.4.5 Swell Tests for Determining Optimum Fly Ash Content

#### A.4.5.1 Fly Ash Content of 9%

Table A.48 Swell test under 1 kPa on soil treated with 9% fly ash

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	521.1	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	162.54	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	138.3	g	
Height of solids, hs=	16.2	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.007	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.183	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.7	%	% swell

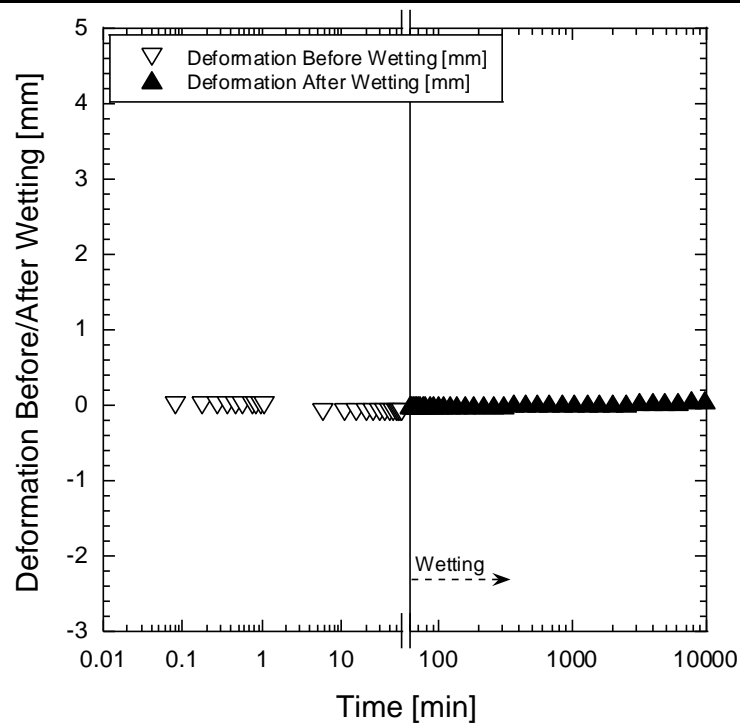


Figure A.44 Swell test under 1 kPa on soil treated with 9% fly ash

### A.4.5.2 Fly Ash Content of 11%

Table A.49 Swell test under 1 kPa on soil treated with 11% fly ash

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	519	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	161.26	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	137.2	g	
Height of solids, hs=	16.0	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.088	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.3	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.061	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.2	%	% swell

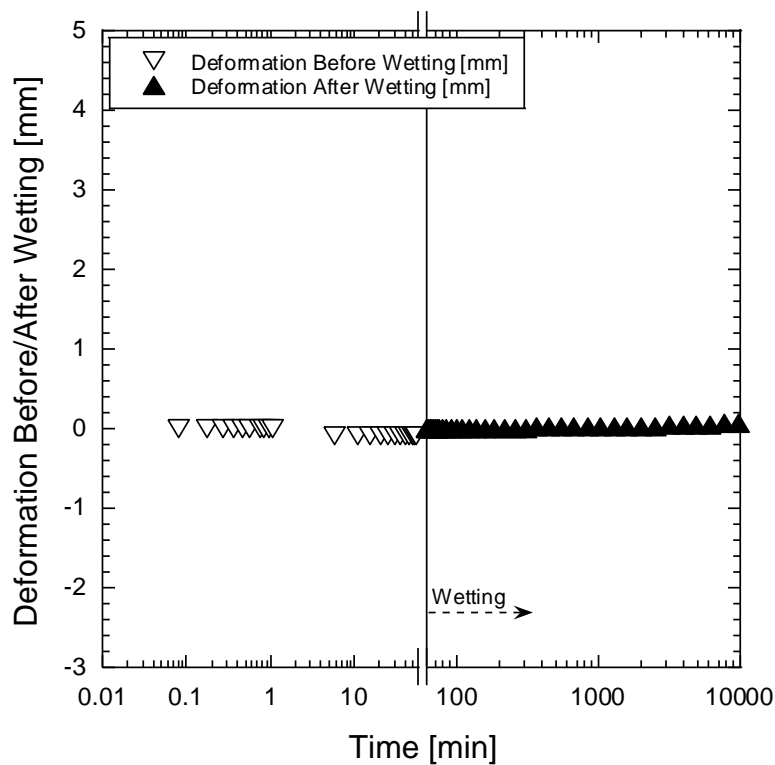


Figure A.45 Swell test under 1 kPa on soil treated with 11% fly ash

### A.4.5.3 Fly Ash Content of 11% - Second Trial

Table A.50 Swell test under 1 kPa on soil treated with 11% fly ash – second trial

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	0	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	164.66	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	140.1	g	
Height of solids, hs=	16.4	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.007	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.127	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.5	%	% swell

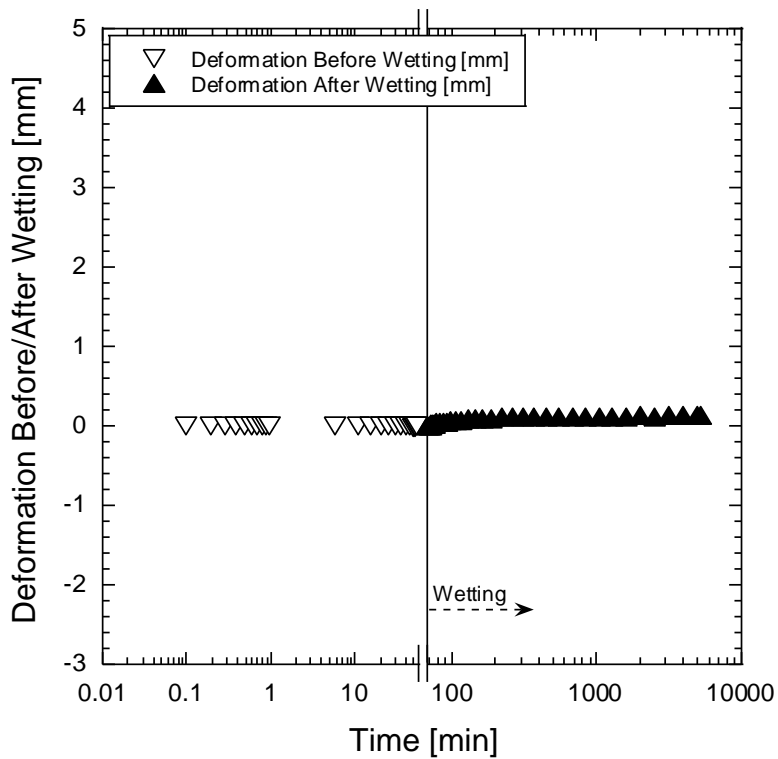


Figure A.46 Swell test under 1 kPa on soil treated with 11% fly ash – second trial



#### A.4.5.4 Fly Ash Content of 13%

Table A.51 Swell test under 1 kPa on soil treated with 13% fly ash

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	523.24	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	162.3	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	138.1	g	
Height of solids, hs=	16.1	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.003	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.040	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.2	%	% swell

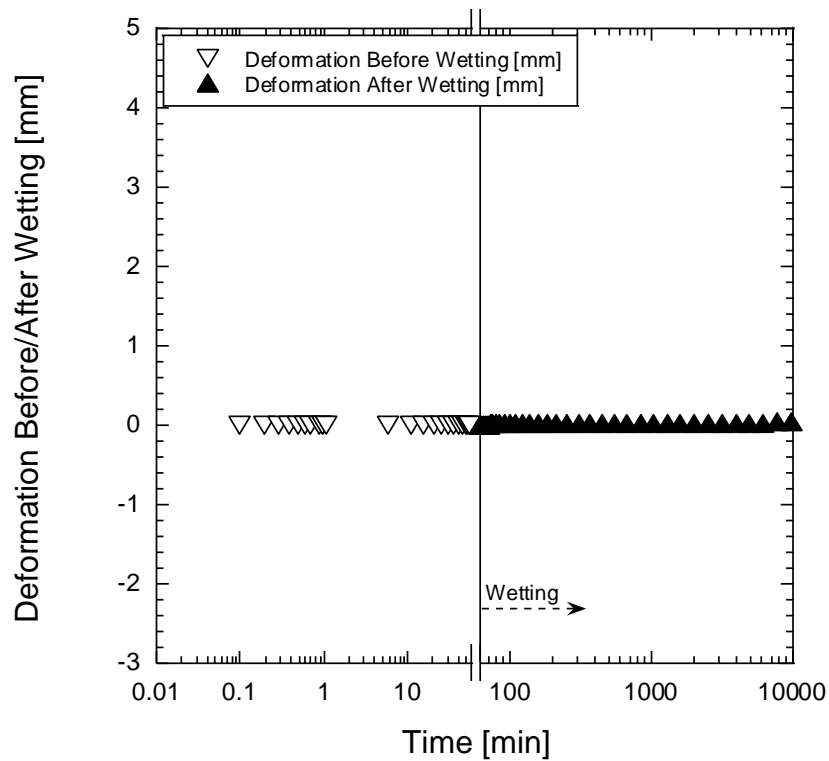


Figure A.47 Swell test under 1 kPa on soil treated with 13% fly ash

#### A.4.5.5 Fly Ash Content of 15% - Second Trial

Table A.52 Swell test under 1 kPa on soil treated with 15% fly ash - second trial

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	519.74	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	160.52	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	136.6	g	
Height of solids, hs=	16.0	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.020	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.172	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.7	%	% swell

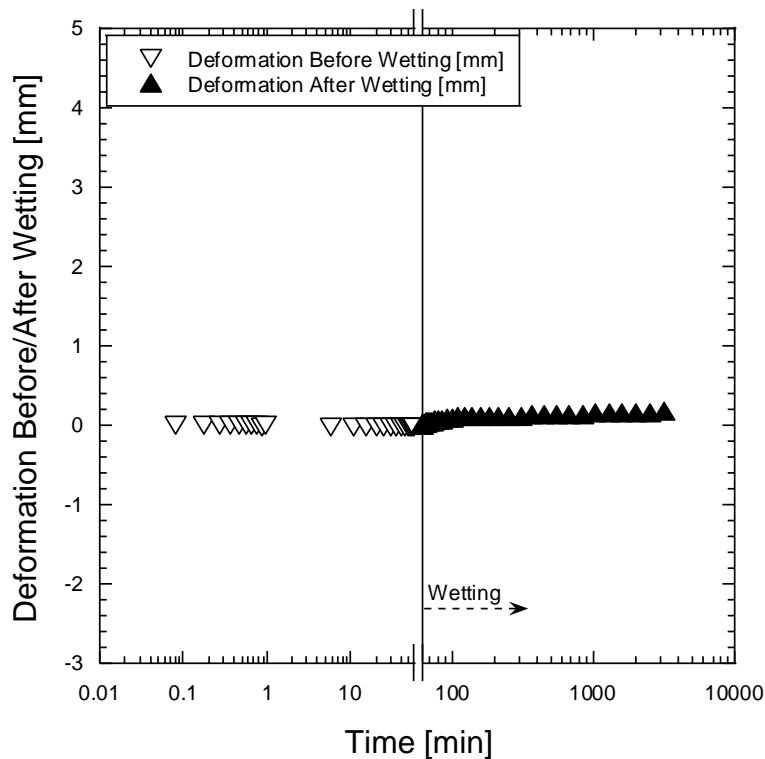


Figure A.48 Swell test under 1 kPa on soil treated with 15% fly ash - second trial

### A.4.5.6 Fly Ash Content of 15% - Third Trial

Table A.53 Swell test under 1 kPa on soil treated with 15% fly ash - third trial

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	519	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	165.08	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	140.5	g	
Height of solids, hs=	16.4	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.013	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.017	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.1	%	% swell

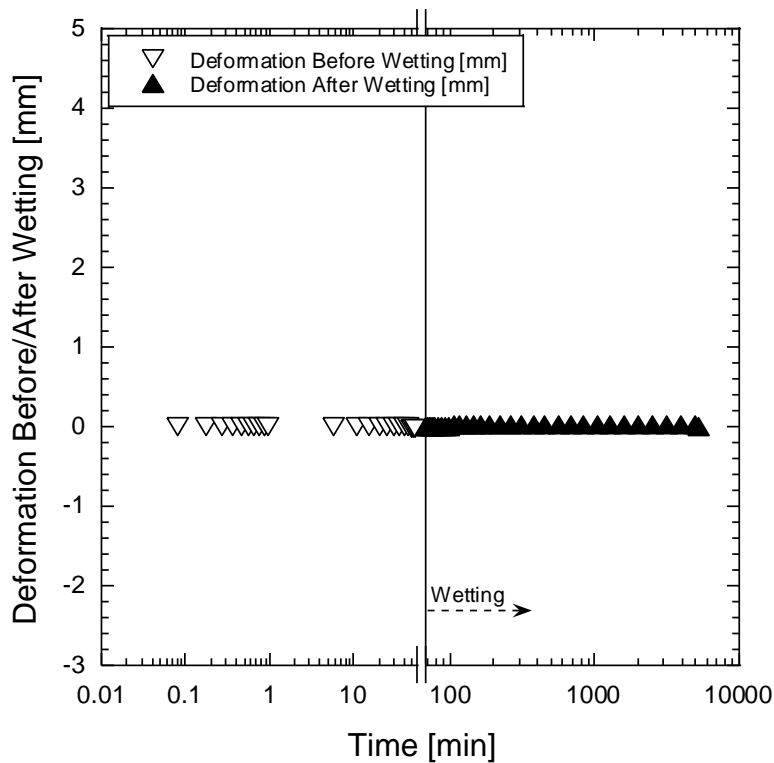


Figure A.49 Swell test under 1 kPa on soil treated with 15% fly ash - third trial

### A.4.5.7 Fly Ash Content of 17% - First Trial

Table A.54 Swell test under 1 kPa on soil treated with 17% fly ash - first trial

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	<u>Leaking problem</u>
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	525.81	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	156.43	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	133.1	g	
Height of solids, hs=	15.6	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.034	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.080	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.3	%	% swell

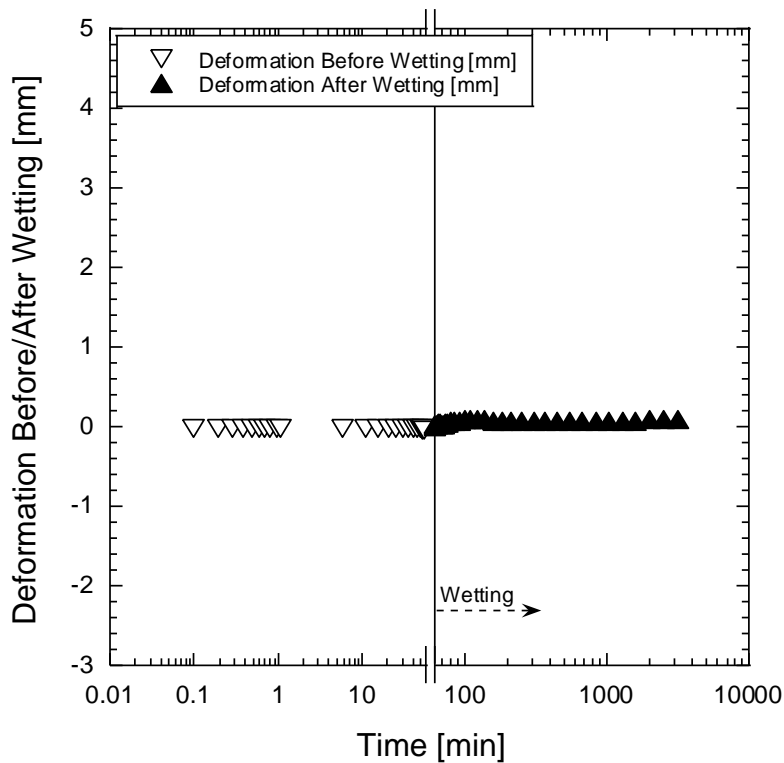


Figure A.50 Swell test under 1 kPa on soil treated with 17% fly ash - first trial

#### A.4.5.8 Fly Ash Content of 17% - Second Trial

Table A.55 Swell test under 1 kPa on soil treated with 17% fly ash - second trial

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	523.24	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	165.5	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, W <sub>s</sub> =	140.9	g	
Height of solids, h <sub>s</sub> =	16.5	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.019	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.033	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.1	%	% swell

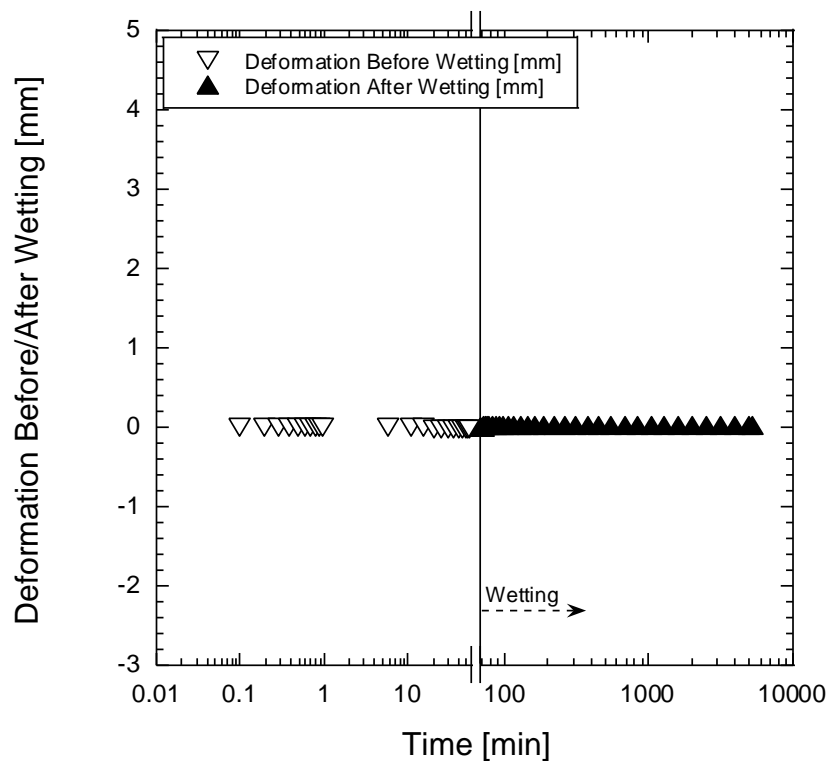


Figure A.51 Swell test under 1 kPa on soil treated with 17% fly ash - second trial

### A.4.5.9 Fly Ash Proportion vs. Swell Potential

Table A.56 Fly ash proportion vs. swell potential

Fly Ash Proportion [%]	Swell Potential [%]
9	0.7
11	0.2
11	0.5
13	0.2
15	0.5
15	0.1
15	0.7
17	0.3
17	0.1

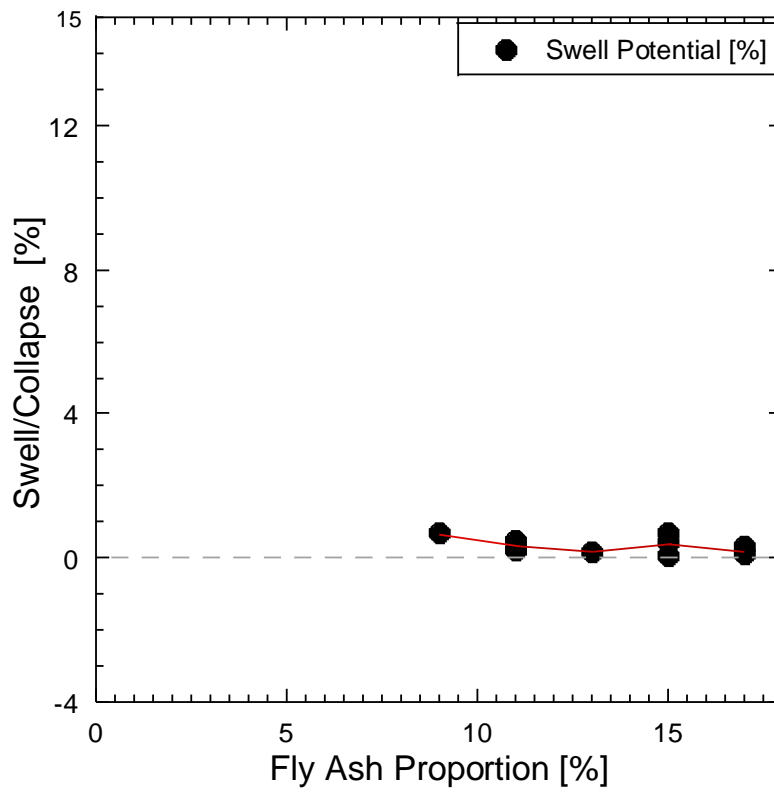


Figure A.52 Fly ash proportion vs. swell potential

### A.4.6 Lime Treated Soil Swell Test (3%)

#### A.4.6.1 Swell Test Under 1 kPa

Table A.57 Swell test under 1 kPa on lime treated soil

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	523.2	g	
Water content, w%=	19.86	%	~Optimum water content
Soil Total weight, Wt=	158.7	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	132.4	g	
Height of solids, hs=	15.5	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.006	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.006	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.0	%	% swell

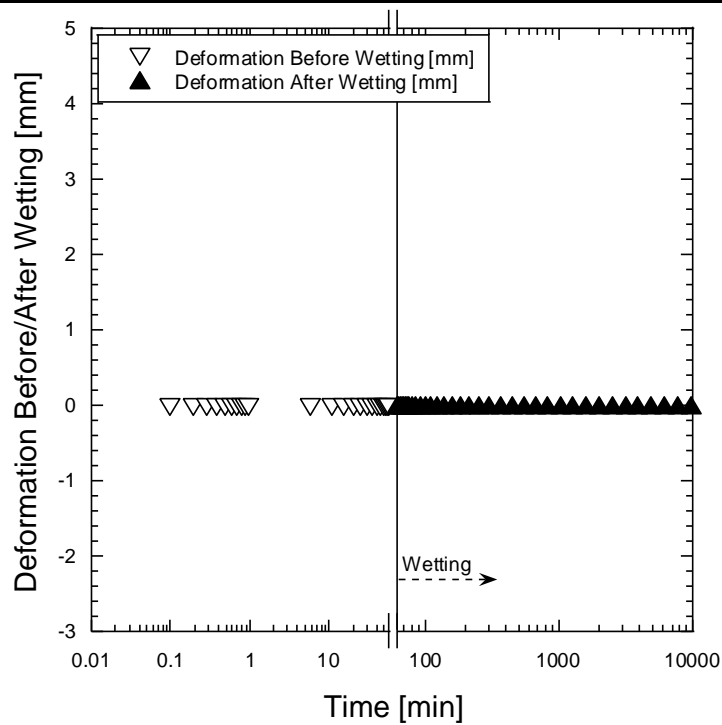


Figure A.53 Swell test under 1 kPa on lime treated soil

### A.4.6.2 Swell Test Under 1 kPa – Second Trial

Table A.58 Swell test under 1 kPa on lime treated soil – second trial

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	521.05	g	
Water content, w%=	19.91	%	~Optimum water content
Soil Total weight, Wt=	161.1	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	134.4	g	
Height of solids, hs=	15.7	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.022	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.005	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.0	%	% swell

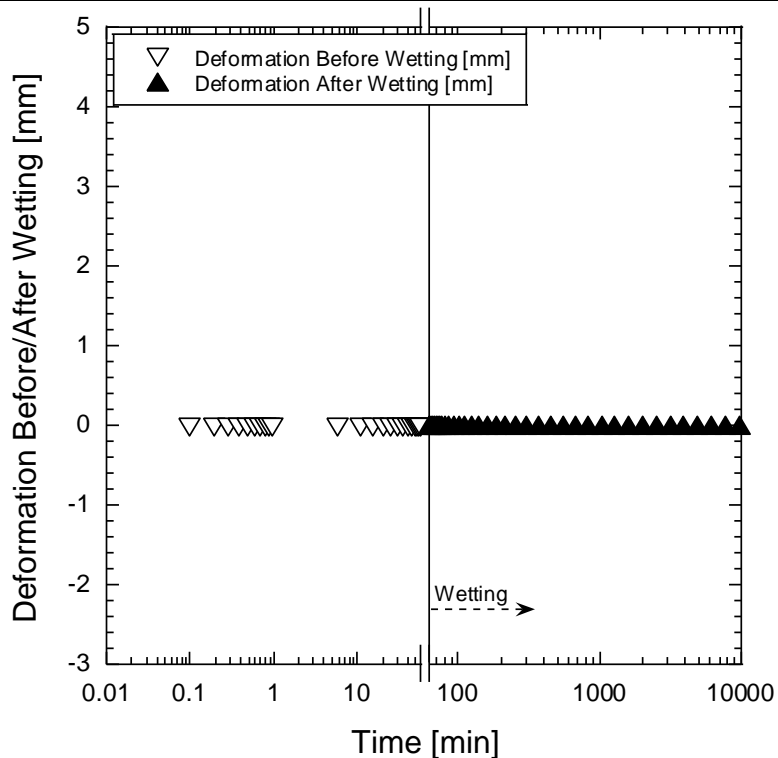


Figure A.54 Swell test under 1 kPa on lime treated soil – second trial



### A.4.6.3 Swell Test Under 10 kPa

Table A.59 Swell test under 10 kPa on lime treated soil

Swell Test	Values	Units	Comments
Pressure to be applied=	10	kPa	
Load to be placed=	322.8	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	519.69	g	
Water content, w%=	19.77	%	~Optimum water content
Soil Total weight, Wt=	158.56	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	132.4	g	
Height of solids, hs=	15.5	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.044	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	-0.072	mm	Linear Collapse
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	-0.3	%	% Collapse

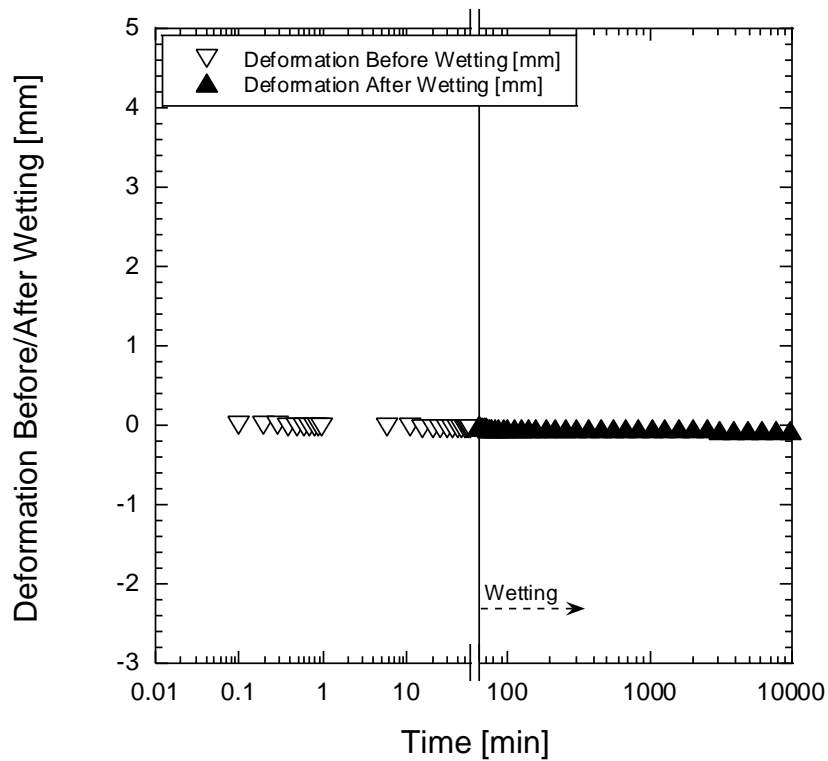


Figure A.55 Swell test under 10 kPa on lime treated soil

#### A.4.6.4 Swell Test Under 50 kPa

Table A.60 Swell test under 50 kPa on lime treated soil

Swell Test	Values	Units	Comments
Pressure to be applied=	50	kPa	
Load to be placed=	1614.1	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	518.13	g	
Water content, w%=	19.73	%	~Optimum water content
Soil Total weight, Wt=	158	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, W <sub>s</sub> =	132.0	g	
Height of solids, h <sub>s</sub> =	15.4	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.172	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.2	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	-0.002	mm	Linear Collapse
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.0	%	% Collapse

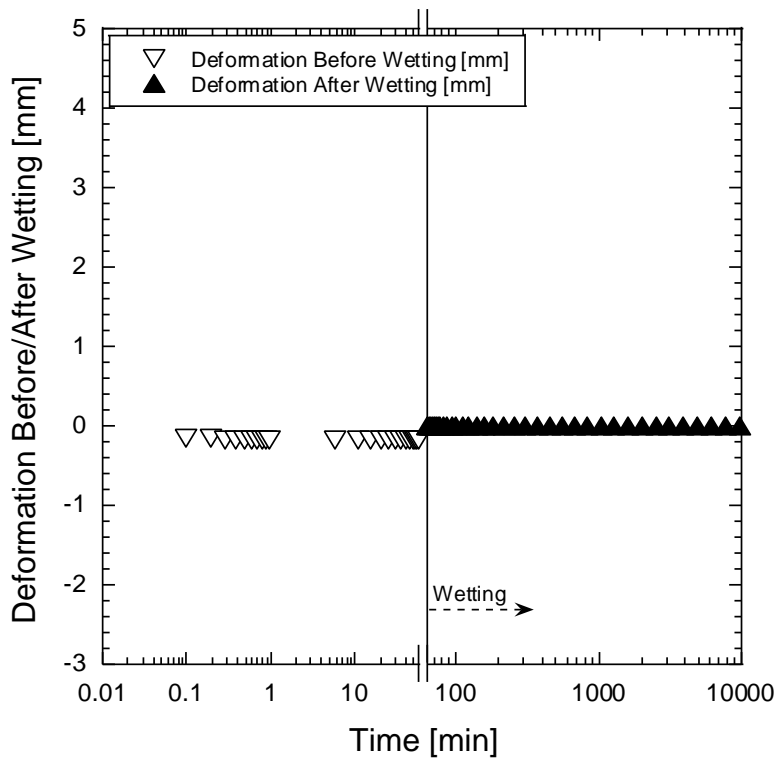


Figure A.56 Swell test under 50 kPa on lime treated soil

### A.4.6.5 Swell Test Under 100 kPa

Table A.61 Swell test under 100 kPa on lime treated soil

Swell Test	Values	Units	Comments
Pressure to be applied=	100	kPa	
Load to be placed=	3228.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	525.77	g	
Water content, w%=	19.46	%	~Optimum water content
Soil Total weight, Wt=	160.28	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	134.2	g	
Height of solids, hs=	15.7	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.215	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.2	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	-0.016	mm	Linear Collapse
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	-0.1	%	% Collapse

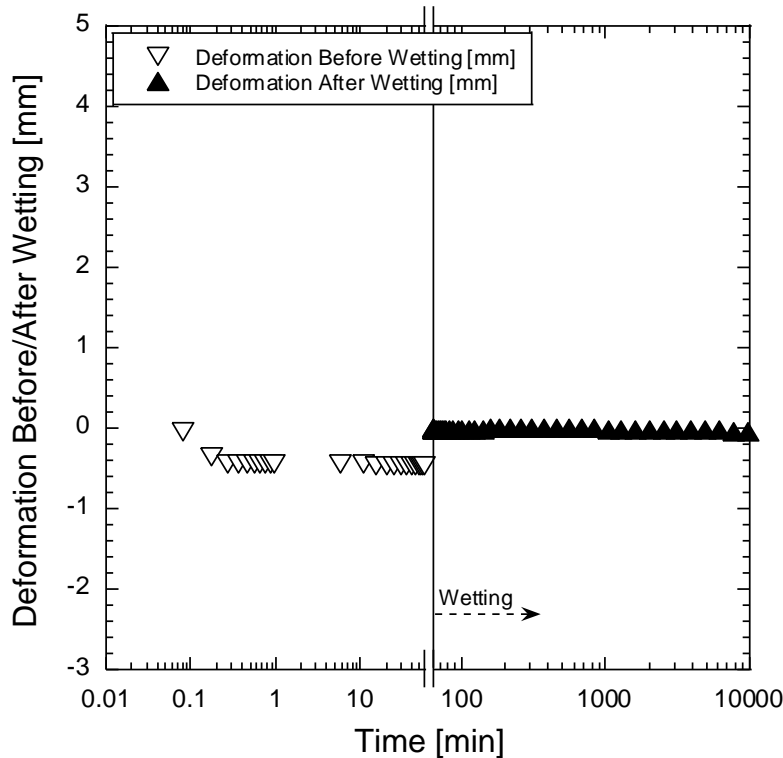


Figure A.57 Swell test under 100 kPa on lime treated soil

#### A.4.6.6 Swell Test Under 207.3 kPa

Table A.62 Swell test under 207.3 kPa on lime treated soil

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	518.21	g	
Water content, w%=	20.24	%	~Optimum water content
Soil Total weight, Wt=	160.61	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	133.6	g	
Height of solids, hs=	15.6	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.473	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	24.9	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	-0.041	mm	Linear Collapse
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	-0.2	%	% Collapse

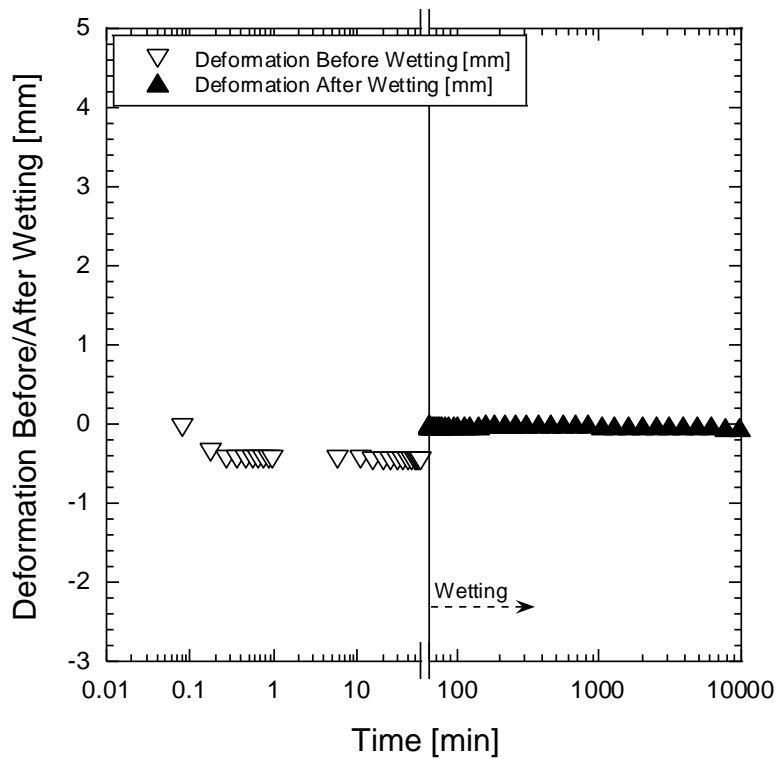


Figure A.58 Swell test under 207.3 kPa on lime treated soil

#### A.4.6.7 Swell Pressure and Swell Potential

Table A.63 Swell pressure and swell potential of lime treated soil

Swell Pressure [kPa]	Swell Potential [%]
1	0.0
1	0.0
1	0.0
10	-0.3
50	0.0
100	-0.1
207.3	-0.2

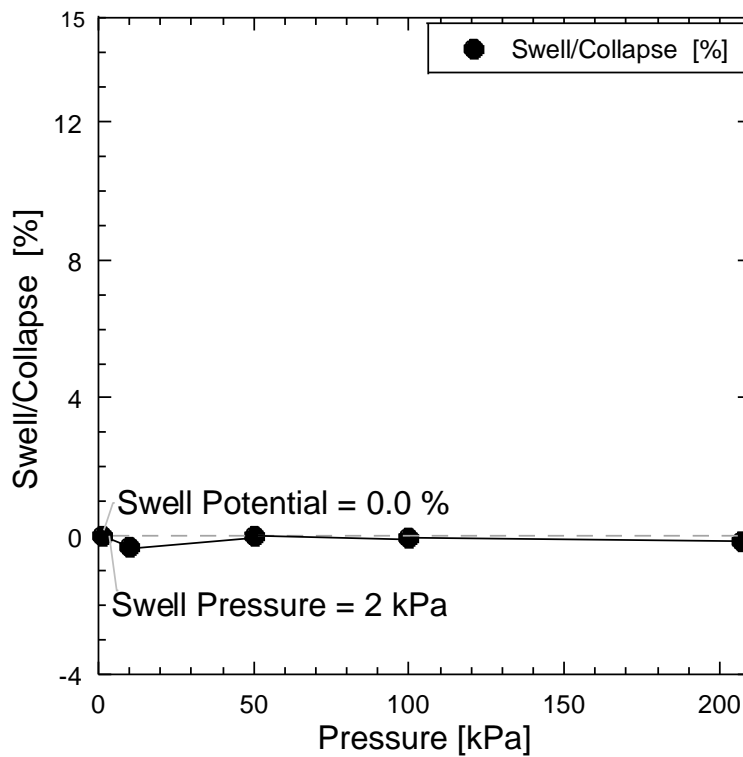


Figure A.59 Swell pressure and swell potential of lime treated soil

### A.4.7 Swell Tests for Determining Optimum Lime Content

#### A.4.7.1 Lime Content of 2%

Table A.64 Swell test under 1 kPa on soil treated with 2% lime

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	0	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	156.18	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	132.9	g	
Height of solids, hs=	15.5	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.012	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.031	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.1	%	% swell

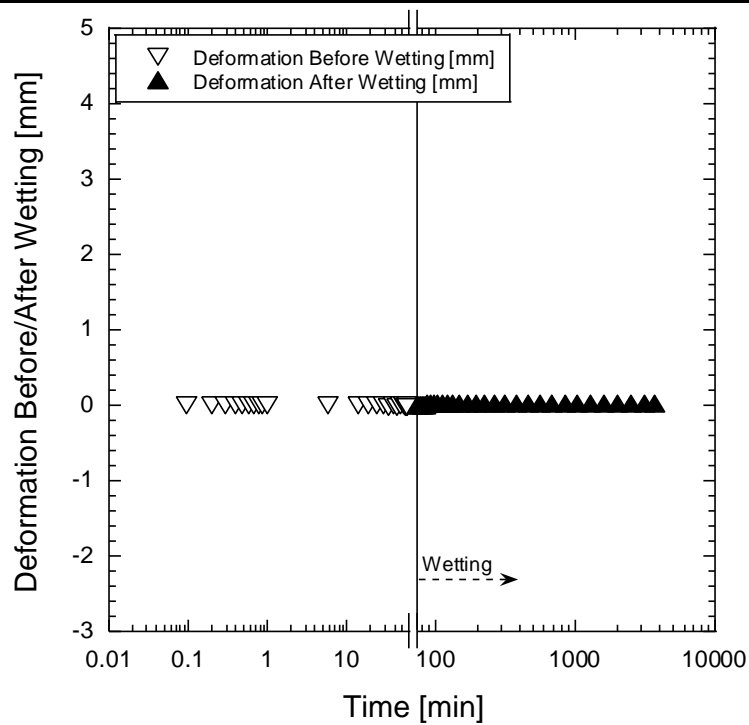


Figure A.60 Swell test under 1 kPa on soil treated with 2% lime

### A.4.7.2 Lime Content of 3% - Second Trial

Table A.65 Swell test under 1 kPa on soil treated with 3% lime – second trial

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	519	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	155.11	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	132.0	g	
Height of solids, hs=	15.4	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.005	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.001	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.0	%	% swell

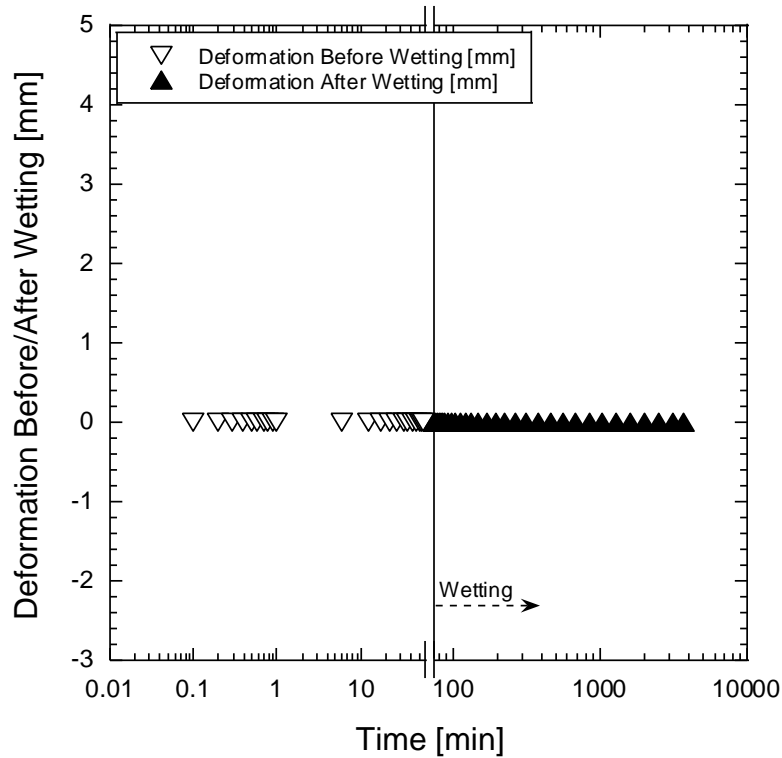


Figure A.61 Swell test under 1 kPa on soil treated with 3% lime – second trial

### A.4.7.3 Lime Content of 4%

Table A.66 Swell test under 1 kPa on soil treated with 4% lime

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	518.34	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	153.07	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	130.3	g	
Height of solids, hs=	15.2	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.007	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.012	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.0	%	% swell

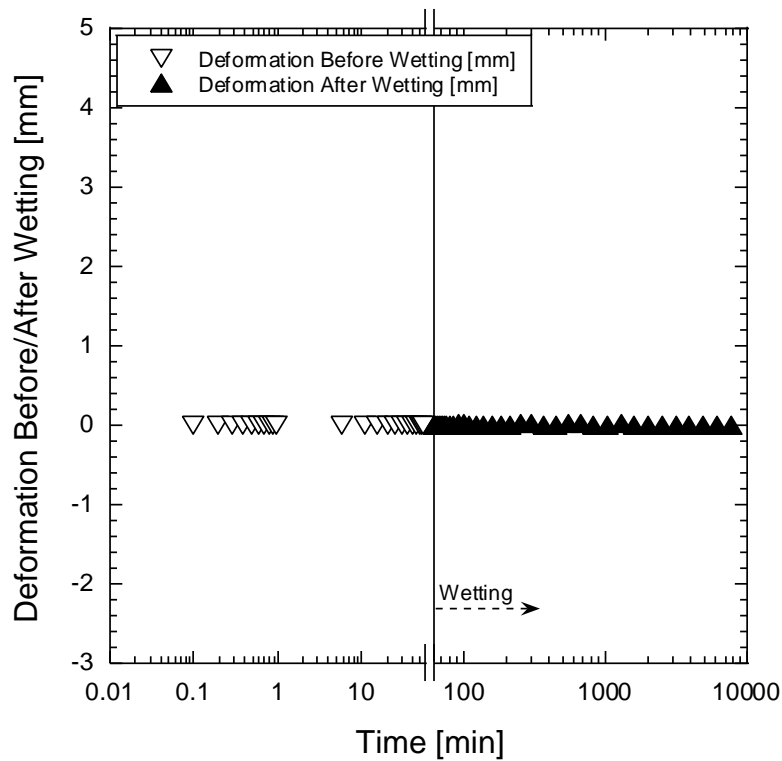


Figure A.62 Swell test under 1 kPa on soil treated with 4% lime



#### A.4.7.4 Lime Content of 5%

Table A.67 Swell test under 1 kPa on soil treated with 5% lime

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	523.24	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	150.74	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, W <sub>s</sub> =	128.3	g	
Height of solids, h <sub>s</sub> =	15.0	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.003	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.004	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.0	%	% swell

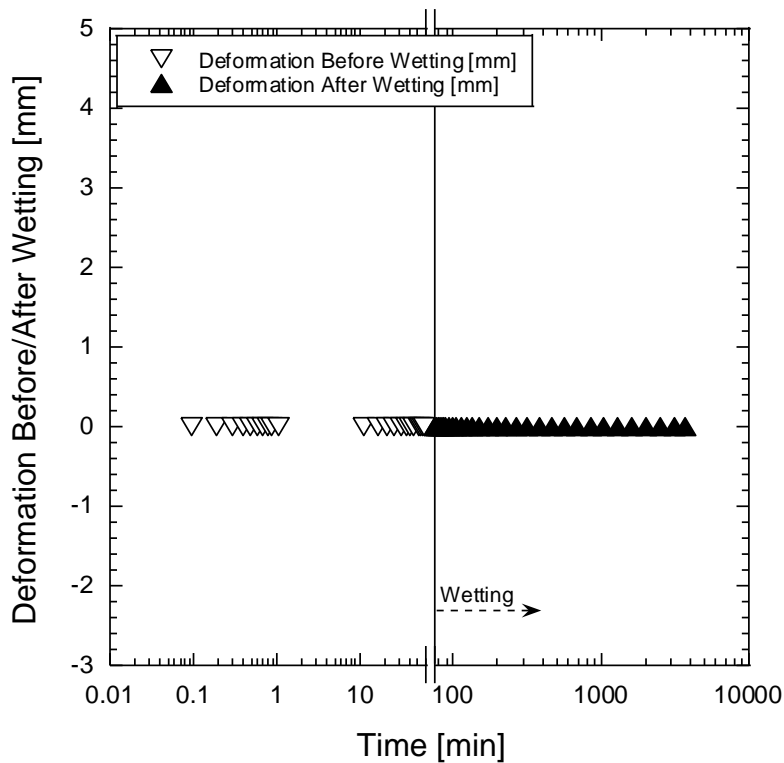


Figure A.63 Swell test under 1 kPa on soil treated with 5% lime

### A.4.7.5 Lime Content of 6%

Table A.68 Swell test under 1 kPa on soil treated with 6% lime

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	523.24	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	151.34	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	128.8	g	
Height of solids, hs=	15.1	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.013	[ - ]	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.001	[ - ]	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.0	%	% swell

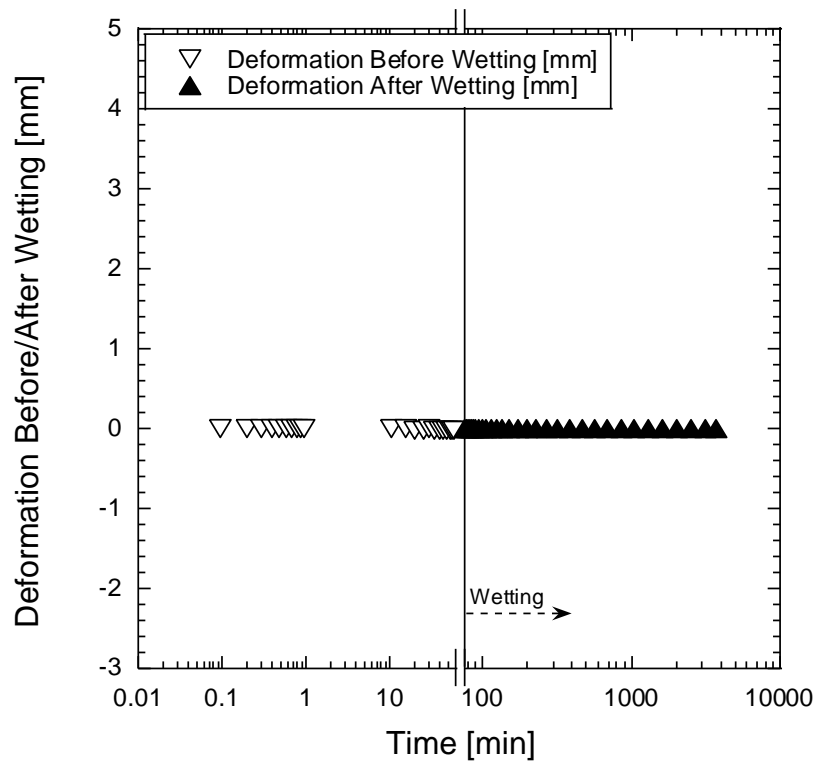


Figure A.64 Swell test under 1 kPa on soil treated with 6% lime

#### A.4.7.6 Lime Proportion vs. Swell Potential

Table A.69 Lime proportion vs. swell potential

Lime Proportion [%]	Swell Potential [%]
2	0.1
3	0.0
3	0.0
4	0.0
5	0.0
6	0.0

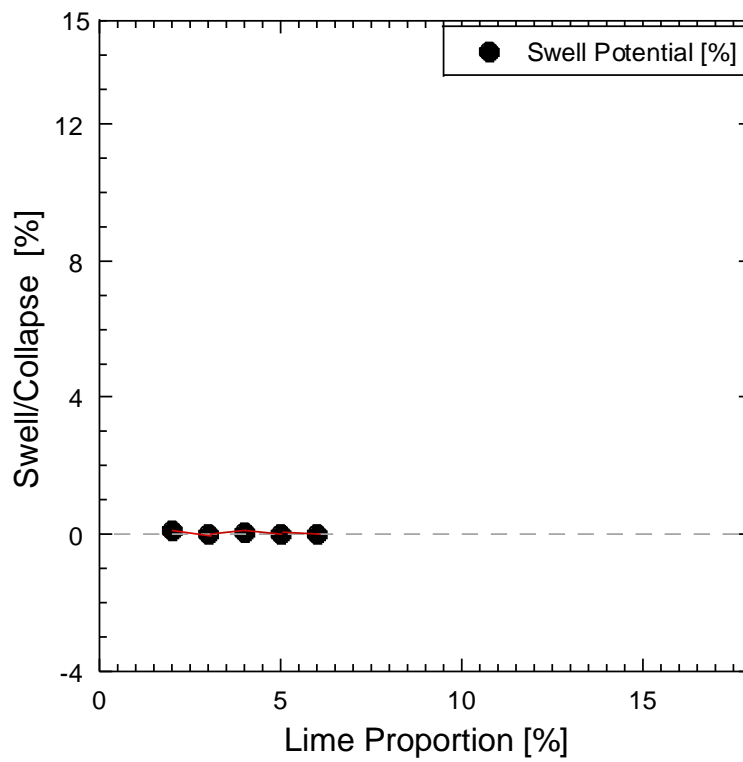


Figure A.65 Lime proportion vs. swell potential

### A.4.8 Swell Potential for Analyzing Variability of P1 Polymer Content – 7-d Curing

#### A.4.8.1 Swell Potential of expansive soil

Table A.70 Swell test under 1 kPa on expansive soil with seven days of curing

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	523.2	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	160.52	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	136.6	g	
Height of solids, hs=	16.0	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.019	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	1.030	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	4.1	%	% swell

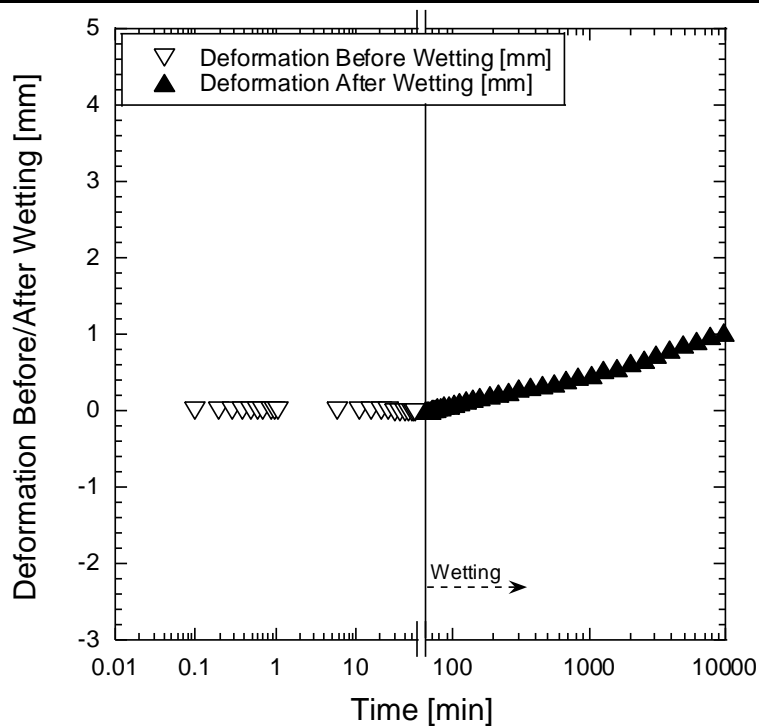


Figure A.66 Swell test under 1 kPa on expansive soil with seven days of curing

### A.4.8.2 Swell Potential of the Fly Ash Treated Soil

Table A.71 Swell test under 1 kPa on fly ash treated soil with seven days of curing

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	519.69	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	162.96	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	138.7	g	
Height of solids, hs=	16.2	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.016	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.024	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.1	%	% swell

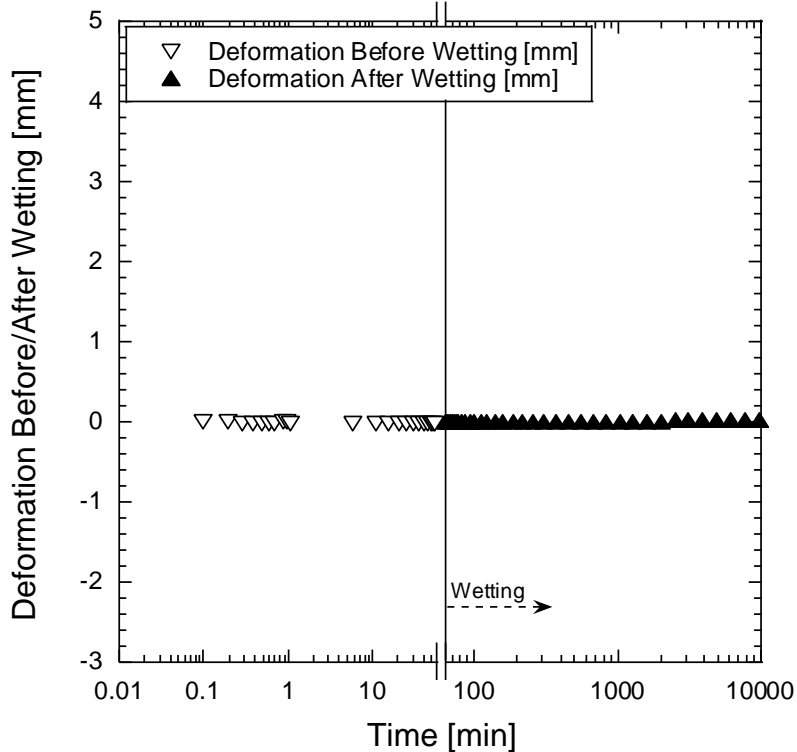


Figure A.67 Swell test under 1 kPa on fly ash treated soil with seven days of curing

### A.4.8.3 Swell Potential of the Lime Treated Soil

Table A.72 Swell test under 1 kPa on lime treated soil with seven days of curing

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	518.13	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	162.46	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	138.3	g	
Height of solids, hs=	16.2	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.002	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	-0.007	mm	Linear Collapse
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.0	%	% Collapse

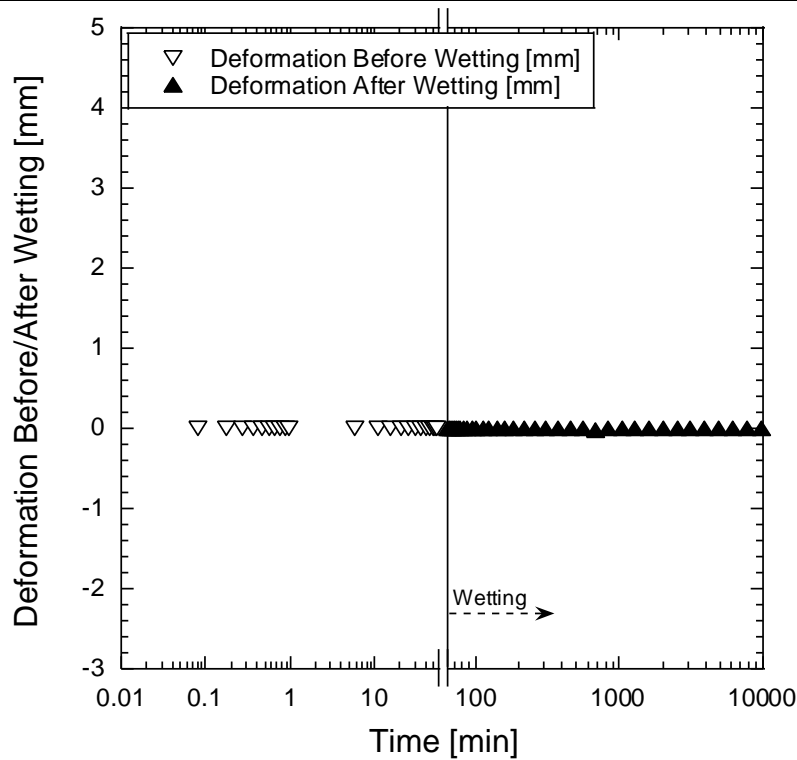


Figure A.68 Swell test under 1 kPa on lime treated soil with seven days of curing

#### A.4.8.4 Swell Potential of 18.4% P1 Treated Soil

Table A.73 Swell test under 1 kPa on 18.4% P1 treated soil with seven days of curing

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	525.77	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	149.17	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	127.0	g	
Height of solids, hs=	14.8	mm	
Compression prior to wetting, dh <sub>1</sub> =	0.002	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.438	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	1.7	%	% swell

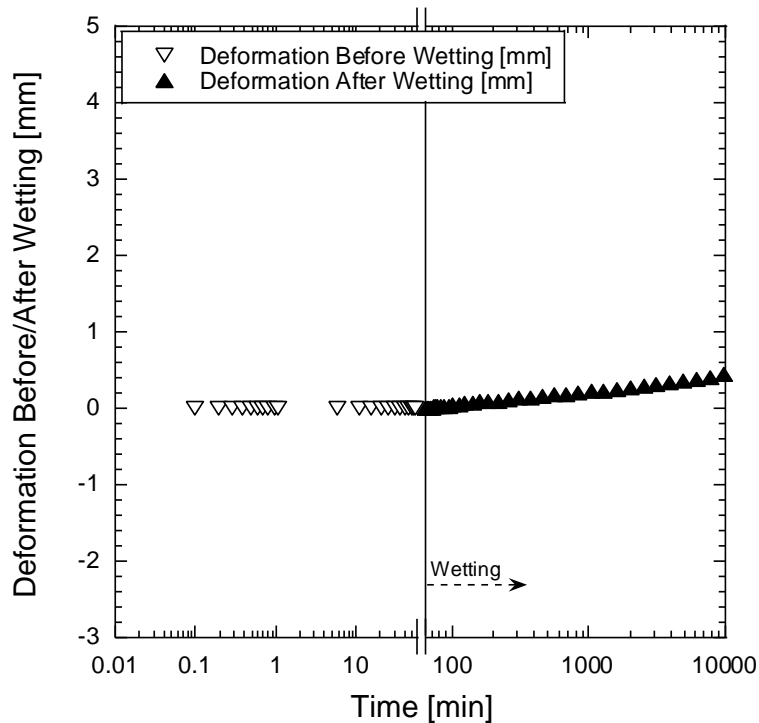


Figure A.69 Swell test under 1 kPa on 18.4% P1 treated soil with seven days of curing

#### A.4.8.5 Swell Potential of 9.2% P1 Treated Soil

Table A.74 Swell test under 1 kPa on 9.2% P1 treated soil with seven days of curing

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	518.21	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	154.77	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	131.7	g	
Height of solids, hs=	15.4	mm	
Compression prior to wetting, dh <sub>1</sub> =	0.032	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	1.326	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	5.2	%	% swell

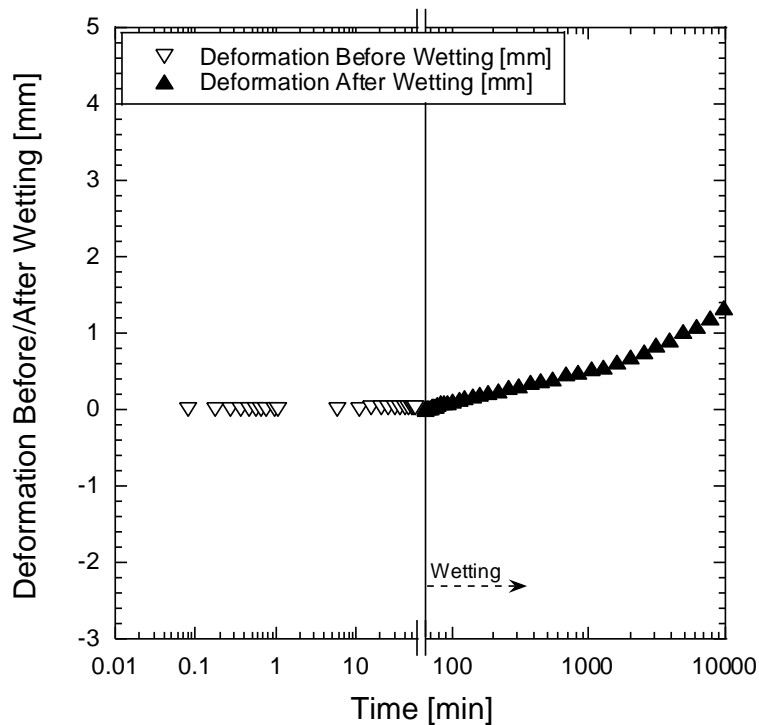


Figure A.70 Swell test under 1 kPa on 9.2% P1 treated soil with seven days of curing



#### A.4.8.6 Swell Potential of 4.6% P1 Treated Soil

Table A.75 Swell test under 1 kPa on 4.6% P1 treated soil with seven days of curing

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	521.05	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	156.51	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	133.2	g	
Height of solids, hs=	15.6	mm	
Compression prior to wetting, dh <sub>1</sub> =	0.002	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.669	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	2.6	%	% swell

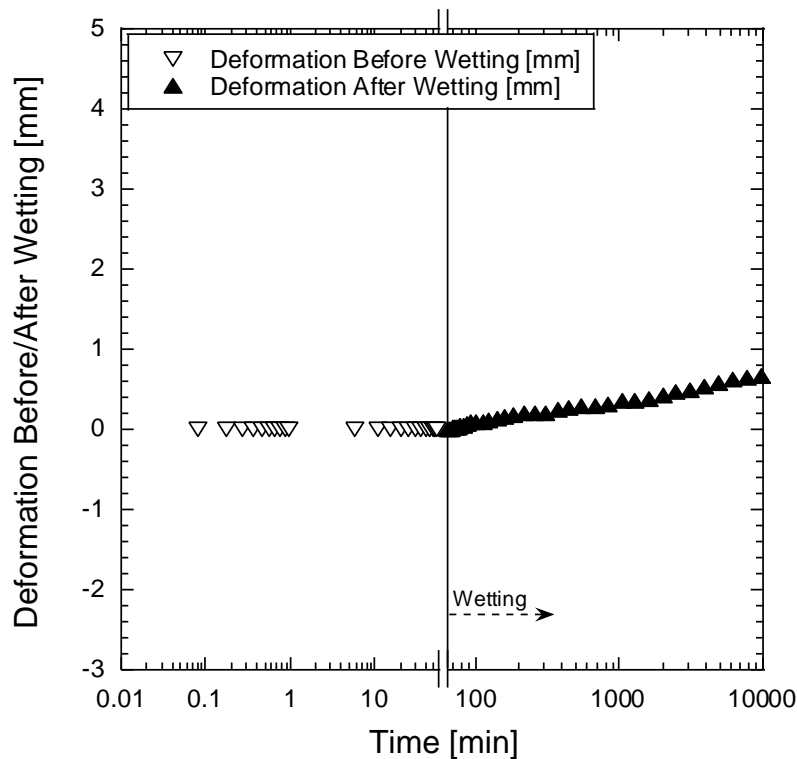


Figure A.71 Swell test under 1 kPa on 4.6% P1 treated soil with seven days of curing

#### A.4.8.7 Effect of P1 Compared with Lime, Fly Ash, and Expansive Soil

Table A.76 Effect of P1 compared with lime, fly ash, and expansive soil

	P1 9.2% Treated	P1 4.6% Treated	P1 18.4% Treated	Fly Ash 15% Treated	Lime 3% Treated
Swell Potential [%]	5.2	2.6	1.7	0.1	0.0

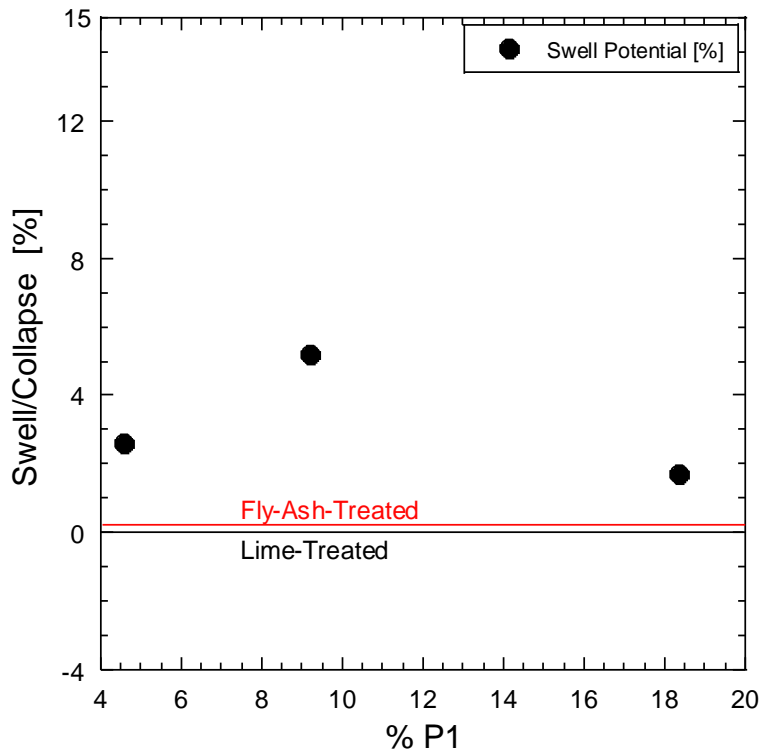


Figure A.72 Effect of P1 compared with lime, fly ash, and expansive soil

### A.4.9 Swell Potential for Assessing Four Polymer Types with 24 Hour Evaporation

#### A.4.9.1 Swell Potential of the Fly Ash Treated Soil

Table A.77 Swell test under 1 kPa on fly ash treated soil with 24 hours of air-drying

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	523.2	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	155.15	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	132.0	g	
Height of solids, hs=	15.4	mm	
Compression prior to wetting, dh <sub>1</sub> =	0.000	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.787	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	3.1	%	% swell

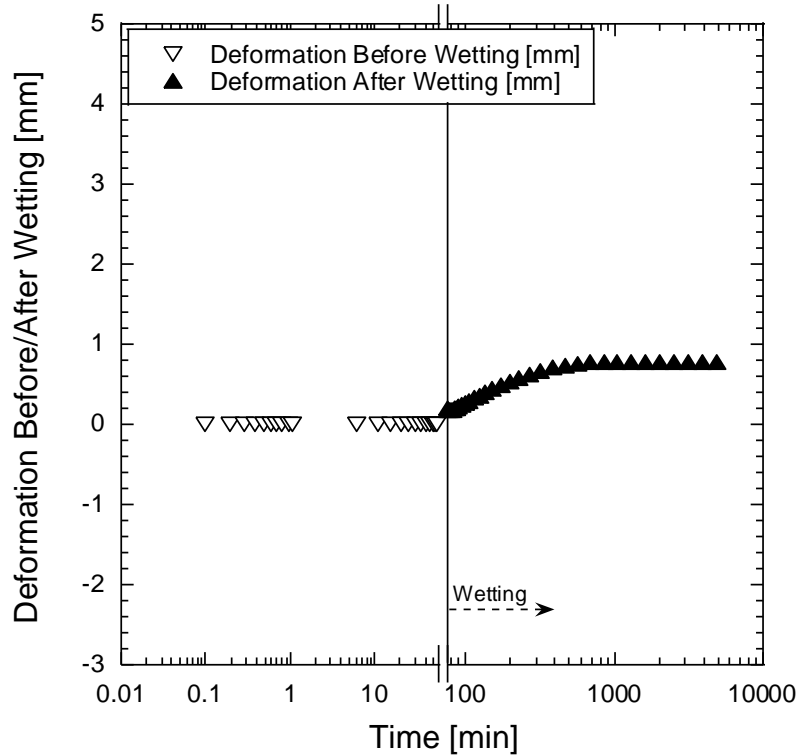


Figure A.73 Swell test under 1 kPa on fly ash treated soil with 24 hours of air-drying

### A.4.9.2 Swell Potential of the Lime Treated Soil

Table A.78 Swell test under 1 kPa on lime treated soil with 24 hours of air-drying

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	519.69	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	148.6	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	126.5	g	
Height of solids, hs=	14.8	mm	
Compression prior to wetting, dh <sub>1</sub> =	0.000	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.196	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.8	%	% swell

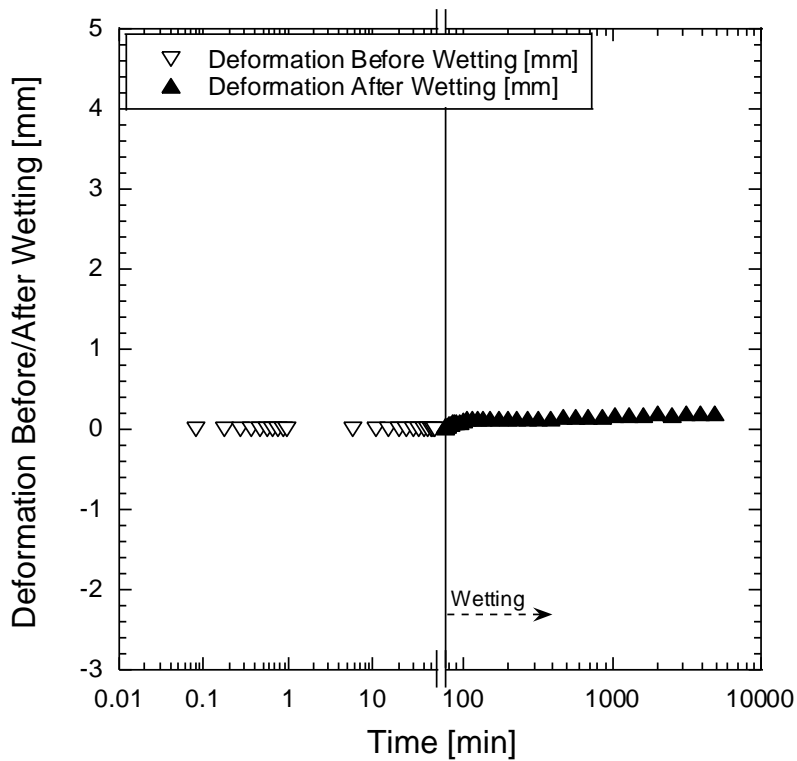


Figure A.74 Swell test under 1 kPa on lime treated soil with 24 hours of air-drying

### A.4.9.3 Swell Potential of the 4.6% of the P1 Treated Soil

Table A.79 Swell test under 1 kPa for the 4.6% on P1 treated soil with 24 hours of air-drying

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	518.13	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	152.26	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	129.6	g	
Height of solids, hs=	15.1	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.039	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	2.207	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	8.7	%	% swell

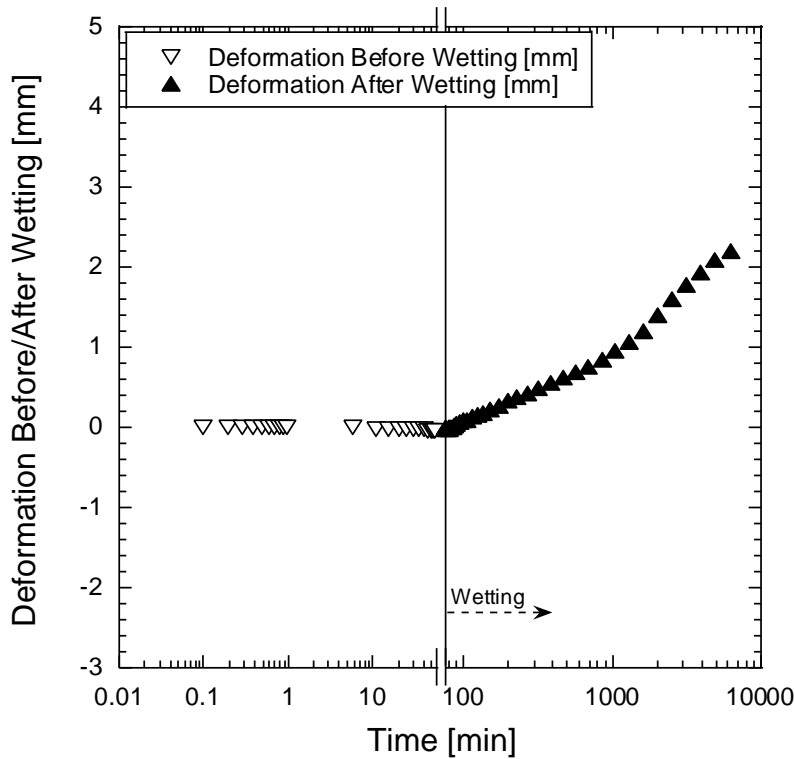


Figure A.75 Swell test under 1 kPa on 4.6% P1 treated soil with 24 hours of air-drying

#### A.4.9.4 Swell Potential of the 4.6% of P2 Treated Soil

Table A.80 Swell test under 1 kPa on 4.6% P2 treated soil with 24 hours of air-drying

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	525.77	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	153.55	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	130.7	g	
Height of solids, hs=	15.3	mm	
Compression prior to wetting, dh <sub>1</sub> =	0.000	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	2.040	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	8.0	%	% swell

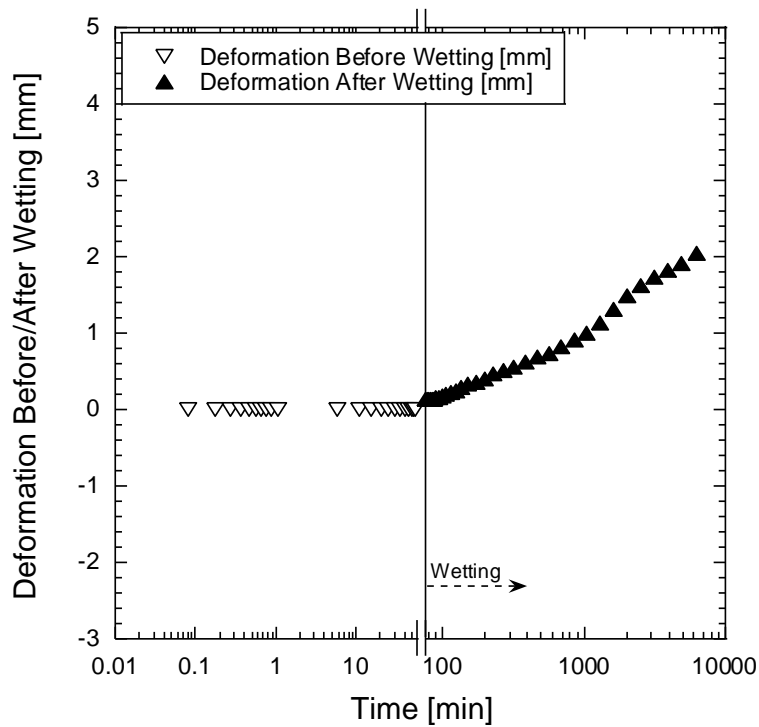


Figure A.76 Swell test under 1 kPa on 4.6% P2 treated soil with 24 hours of air-drying

#### A.4.9.5 Swell Potential of 4.6% P3 Treated Soil

Table A.81 Swell test under 1 kPa on 4.6% P3 treated soil with 24 hours of air-drying

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	518.21	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	150.7	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	128.3	g	
Height of solids, hs=	15.0	mm	
Compression prior to wetting, dh <sub>1</sub> =	0.000	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	2.013	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	7.9	%	% swell

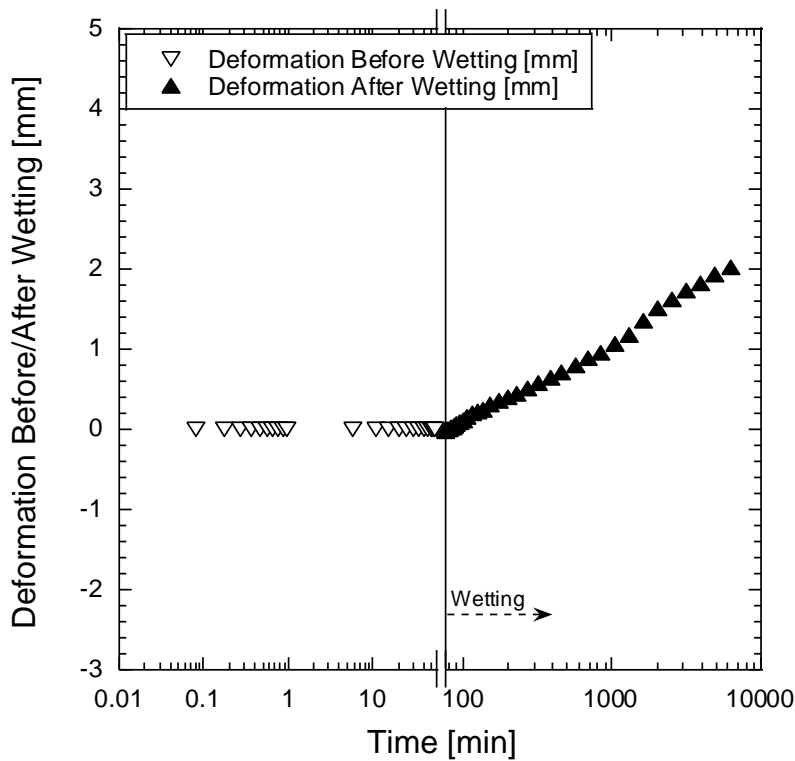


Figure A.77 Swell test under 1 kPa on 4.6% P3 treated soil with 24 hours of air-drying

#### A.4.9.6 Swell Potential on 4.6% P4 Treated Soil

Table A.82 Swell test under 1 kPa on 4.6% on P4 treated soil with 24 hours of air-drying

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	521.05	g	
Water content, w%=	17.5	%	~Optimum water content
Soil Total weight, Wt=	152.62	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, W <sub>s</sub> =	129.9	g	
Height of solids, h <sub>s</sub> =	15.2	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.007	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	1.203	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	4.7	%	% swell

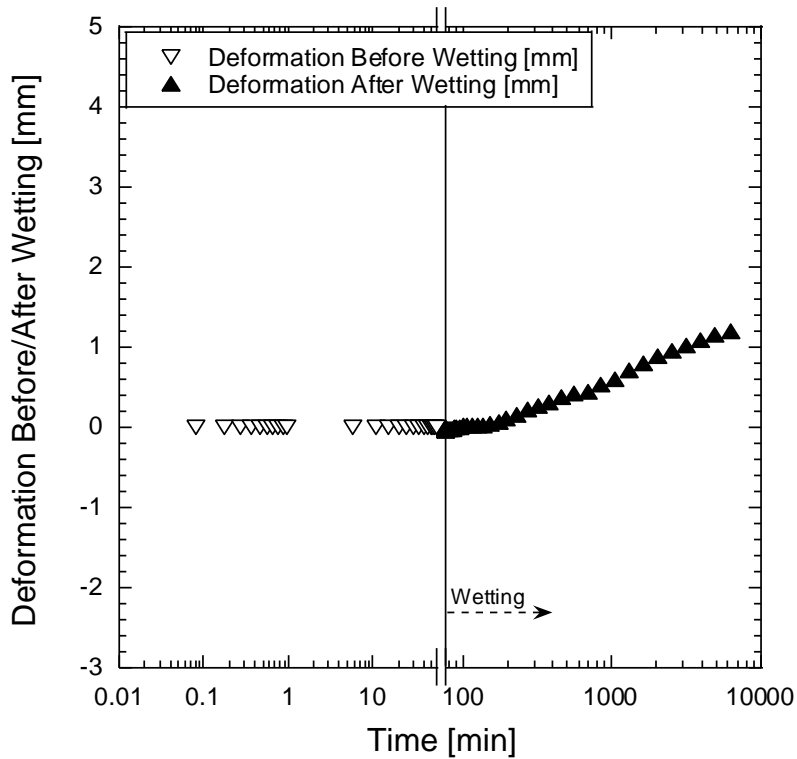


Figure A.78 Swell test under 1 kPa on 4.6% P4 treated soil with 24 hours of air-drying



**A.4.9.7 Effect of Four Polymer Types Compared with Lime And Fly Ash with 24 Hours of Evaporation**

Table A.83 Effect of four polymer types compared with lime and fly ash with 24 hours of air-drying

	P1 4.6% Treated	P2 4.6% Treated	P3 4.6% Treated	P4 4.6% Treated	Fly Ash Treated	Lime Treated
Swell Potential [%]	8.7	8.0	7.9	4.7	3.1	0.8

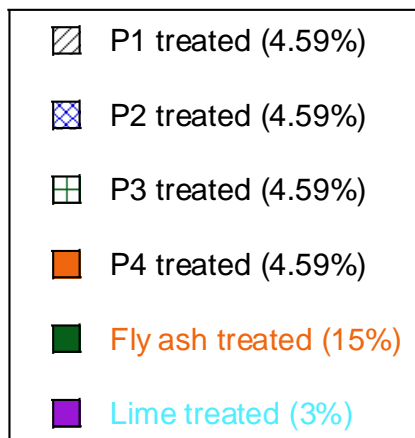
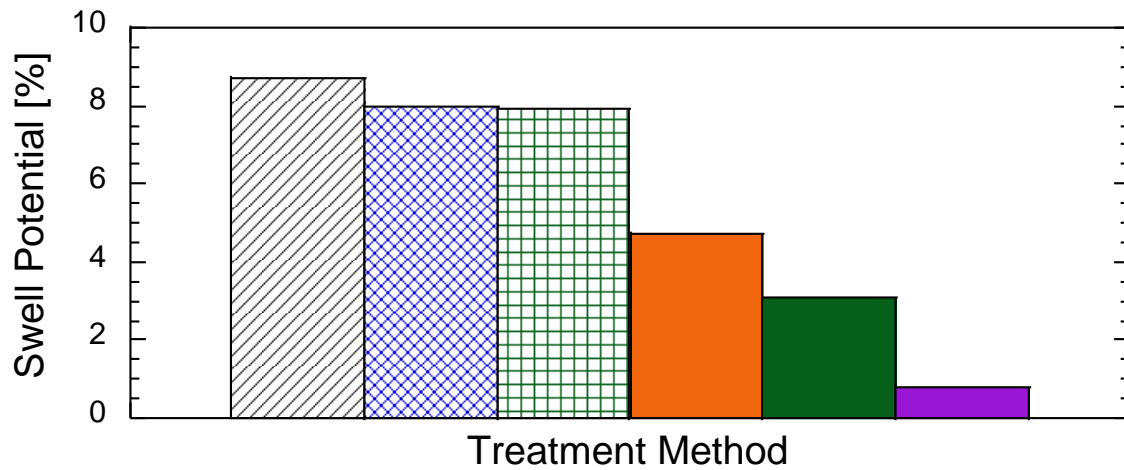


Figure A.79 Effect of four polymer types compared with lime and fly ash with 24 hours of air-drying

#### A.4.10 P4 Treated Swell Test (5%)

##### A.4.10.1 Swell Test Under 1 kPa

Table A.84 Swell test under 1 kPa on P4 treated soil

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	523.2	g	
Water content, w%=	20.2	%	~Optimum water content
Soil Total weight, Wt=	154.15	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	128.2	g	
Height of solids, hs=	15.0	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.036	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	1.247	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	4.9	%	% swell

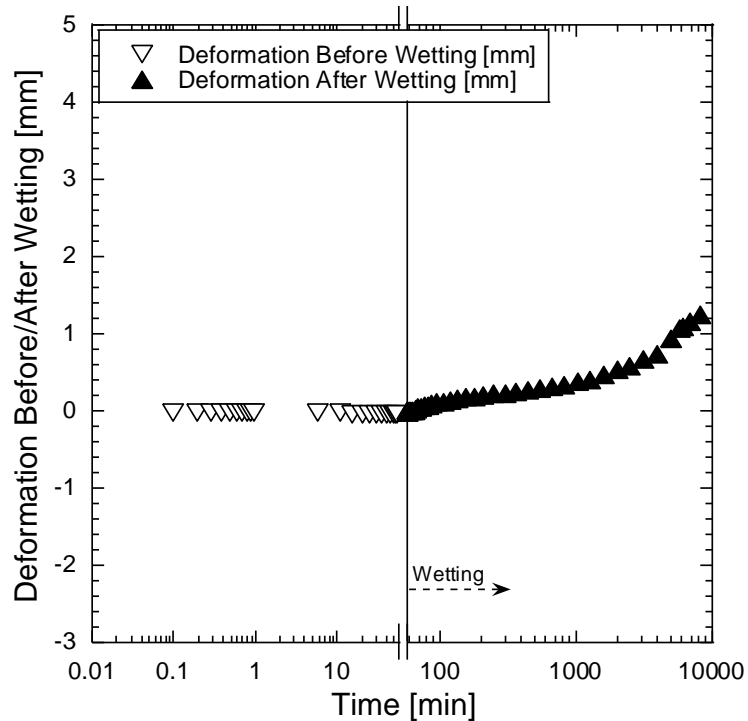


Figure A.80 Swell test under 1 kPa on P4 treated soil

#### A.4.10.2 Swell Test Under 1 kPa – Second Trial

Table A.85 Swell test under 1 kPa on P4 treated soil – second trial

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	521.05	g	
Water content, w%=	19.53	%	~Optimum water content
Soil Total weight, Wt=	156.75	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, W <sub>s</sub> =	131.1	g	
Height of solids, h <sub>s</sub> =	15.3	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.031	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	1.012	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	4.0	%	% swell

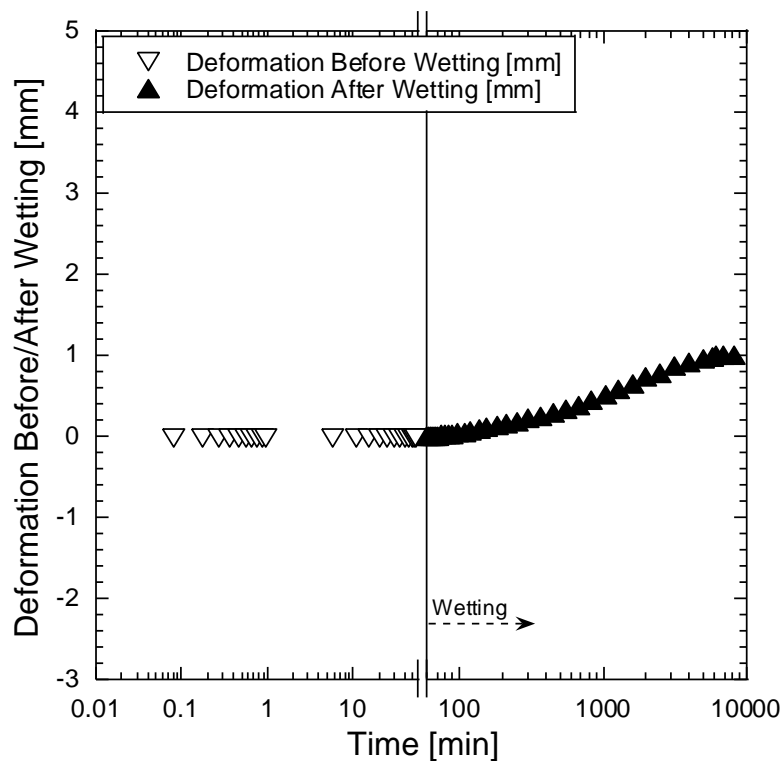


Figure A.81 Swell test under 1 kPa on P4 treated soil – second trial

### A.4.10.3 Swell Test Under 10 kPa

Table A.86 Swell test under 10 kPa on P4 treated soil

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	519.69	g	
Water content, w%=	19.18	%	~Optimum water content
Soil Total weight, Wt=	155.31	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	130.3	g	
Height of solids, hs=	15.2	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.067	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.3	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.251	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	1.0	%	% swell

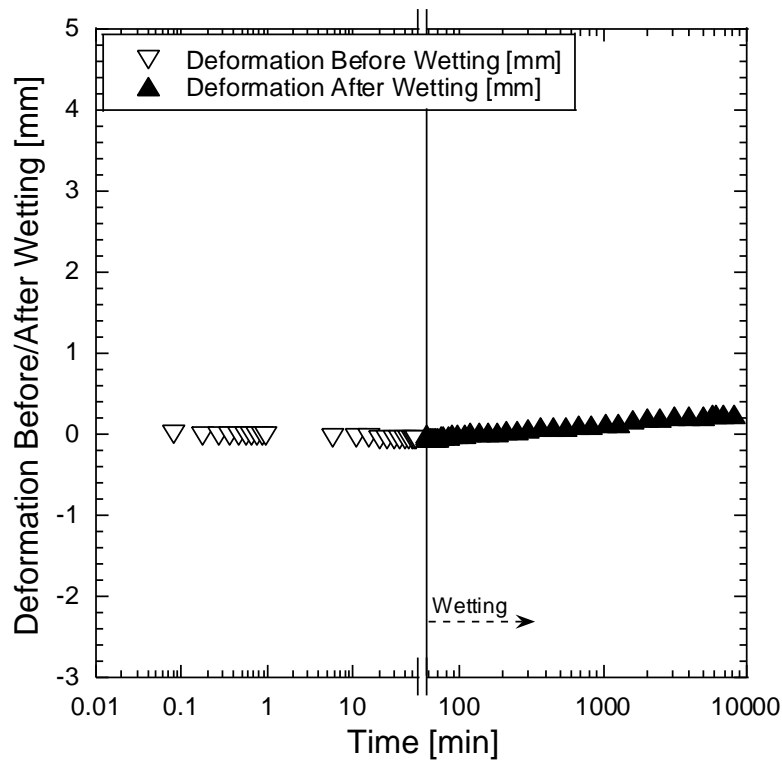


Figure A.82 Swell test under 10 kPa on P4 treated soil

#### A.4.10.4 Swell Test Under 50 kPa

Table A.87 Swell test under 50 kPa on P4 treated soil

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	518.13	g	
Water content, w%=	18.87	%	~Optimum water content
Soil Total weight, Wt=	155.76	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	131.0	g	
Height of solids, hs=	15.3	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.330	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	25.1	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.142	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.6	%	% swell

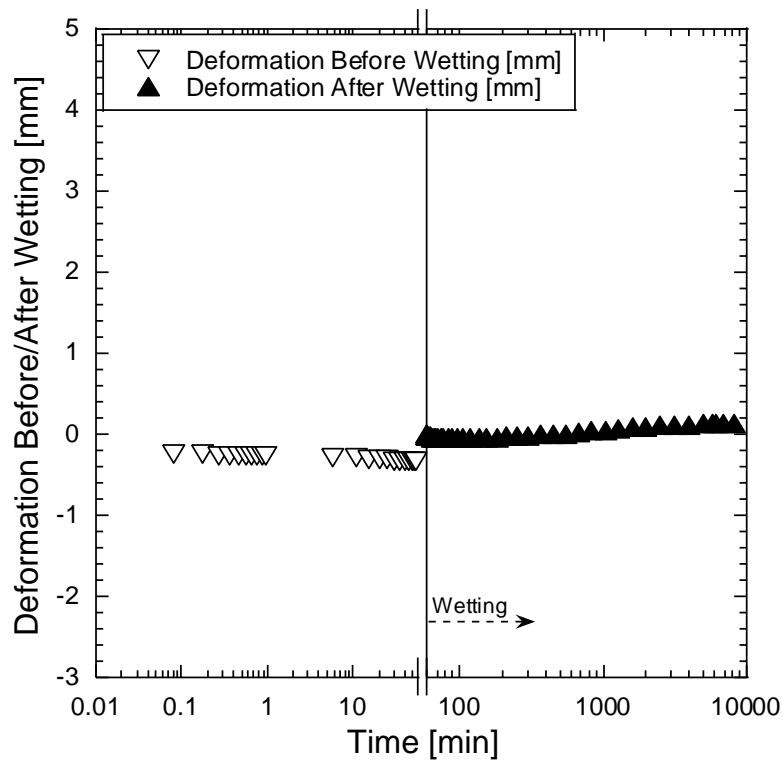


Figure A.83 Swell test under 50 kPa on P4 treated soil

### A.4.10.5 Swell Test Under 100 kPa

Table A.88 Swell test under 100 kPa on P4 treated soil

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	525.77	g	
Water content, w%=	19.1	%	~Optimum water content
Soil Total weight, Wt=	156.85	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	131.7	g	
Height of solids, hs=	15.4	mm	
Compression prior to wetting, dh <sub>1</sub> =	-0.631	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	24.8	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	0.030	mm	Linear swell
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	0.1	%	% swell

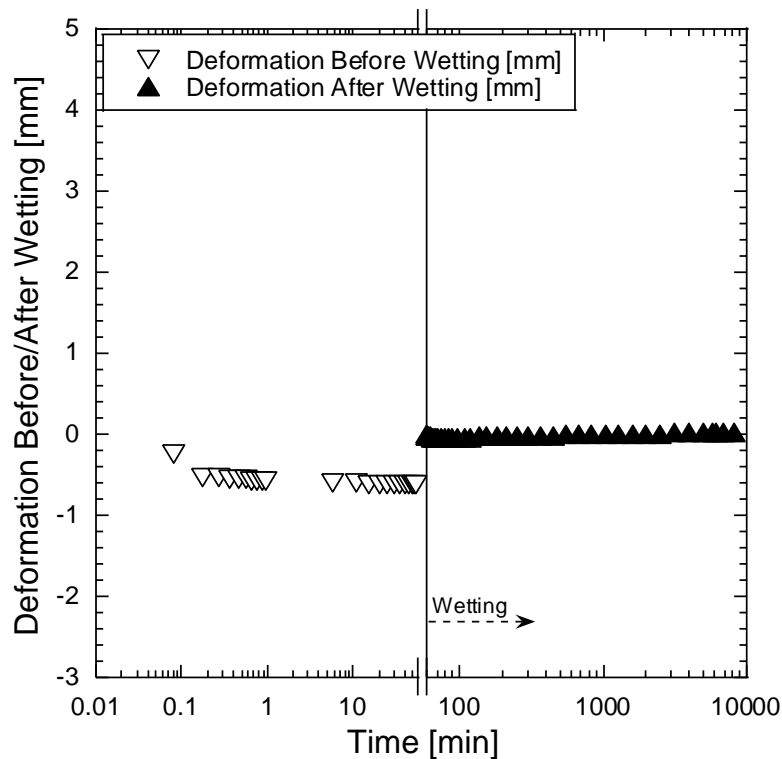


Figure A.84 Swell test under 100 kPa on P4 treated soil

#### A.4.10.6 Swell Test Under 207.3 kPa

Table A.89 Swell test under 207.3 kPa on P4 treated soil

Swell Test	Values	Units	Comments
Pressure to be applied=	1	kPa	
Load to be placed=	32.3	g	Arm= 10x
Cell diameter, d=	63.5	mm	Two inches
Cell height, h=	25.4	mm	One inch
Cell weight=	518.21	g	
Water content, w%=	18.84	%	~Optimum water content
Soil Total weight, Wt=	156.56	g	
Volume of soil, V=	80439.8	mm <sup>3</sup>	
Weight of solids, Ws=	131.7	g	
Height of solids, hs=	15.4	mm	
Compression prior to wetting, dh <sub>1</sub> =	-1.025	mm	Immediately before wetting
Specimen height prior to wetting, h <sub>1</sub> =	24.4	mm	Immediately before wetting
Swell/Collapse caused by wetting dh <sub>2</sub> =	-0.145	mm	Linear Collapse
Swell/Collapse= 100 x dh <sub>2</sub> /h <sub>1</sub> =	-0.6	%	% Collapse

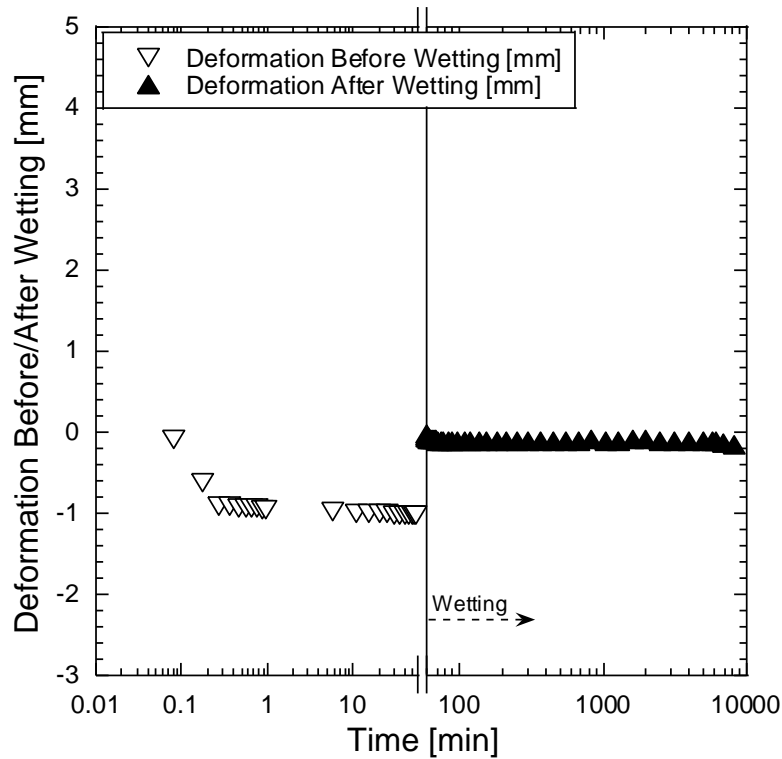


Figure A.85 Swell test under 207.3 kPa on P4 treated soil

#### A.4.10.7 Swell Pressure vs. Swell Potential

Table A.90 Swell pressure vs. swell potential on P4 treated soil

Swell Pressure [kPa]	Swell Potential [%]
1	4.9
1	4.0
10	1.0
50	0.6
100	0.1
207.3	-0.6

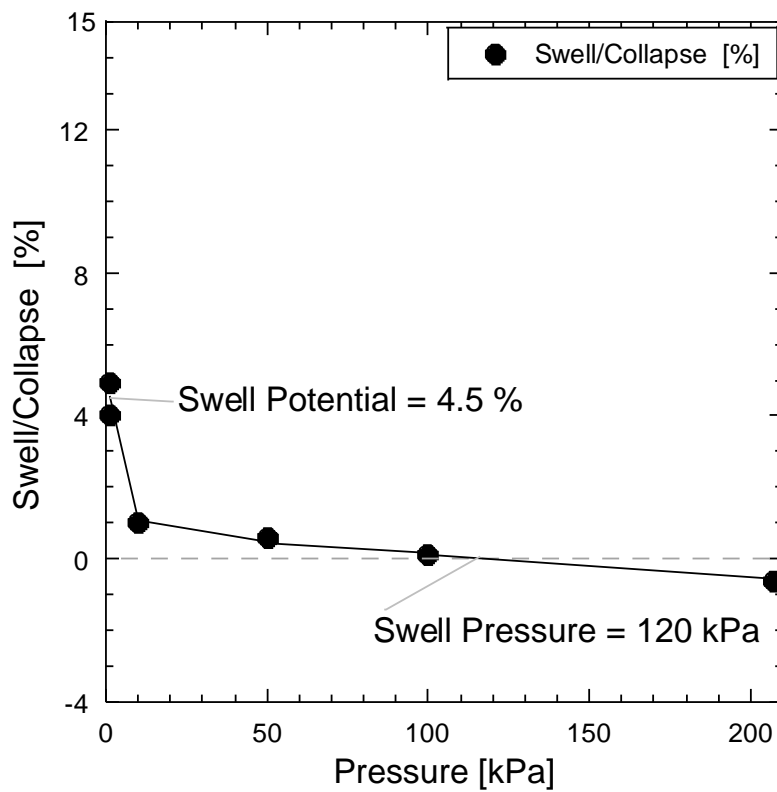


Figure A.86 Swell pressure vs. swell potential for the P4 treated soil



## A.5 Hydraulic Conductivity Tests

### A.5.1 Expansive soil hydraulic conductivity

Table A.91 Hydraulic conductivity test on expansive soil

Hydraulic Conductivity Test	Values	Units	Comments
Volume of soil, V=	429.27	mm <sup>3</sup>	Compacted with 1 layer & 19 blows
Water content, w%=	18.4	%	~Optimum water content
Specimen height, h=	29.3	mm	
Specimen diameter, d=	101.6	mm	
Diameter of Influent pipet d <sub>i</sub> =	10.52	mm	
Correction factor, influent pipet, CF <sub>i</sub>	1.135	[ - ]	
Diameter of effluent pipet d <sub>o</sub> =	10.52	mm	
Correction factor, effluent pipet, CF <sub>o</sub>	1.134	[ - ]	
Degree of saturation, S=	64.0	%	
Hydraulic gradient, i=	30.2	[ - ]	
Cell pressure applied, P <sub>c</sub> =	75.0	psi	
Head backpressure applied, P <sub>h</sub> =	55.4		
Tail backpressure applied, P <sub>t</sub> =	54.2		
Average effective stress, σ' <sub>1</sub> =	20.2	psi	to prevent swelling
Average effective stress, σ' <sub>2</sub> =	139.3	kPa	~Swell pressure
Hydraulic conductivity=	2.9E-11	m/s	

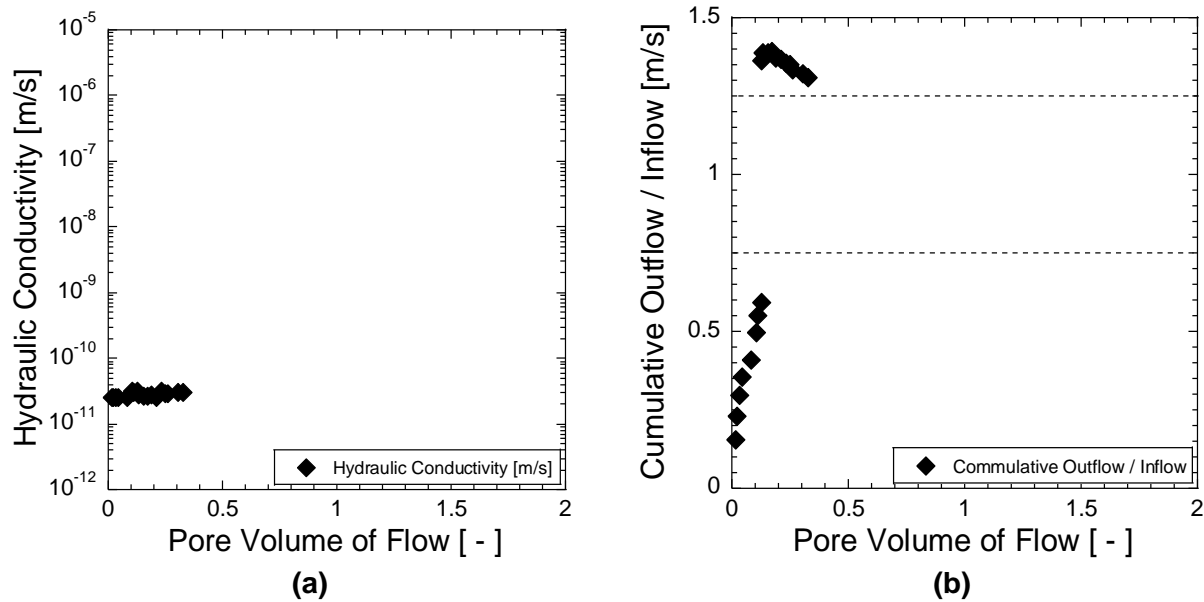


Figure A.87 Hydraulic conductivity of expansive soil; (a) hydraulic conductivity vs. pore volumes of flow, and (b) cumulative outflow /inflow vs. pore volumes of flow.

### A.5.2 Fly Ash Treated Hydraulic Conductivity Test (15%)

Table A.92 Hydraulic conductivity of fly ash treated soil

Hydraulic Conductivity Test	Values	Units	Comments
Volume of soil, V=	436.72	mm <sup>3</sup>	Compacted with 1 layer & 19 blows
Water content, w%=	17.6	%	~Optimum water content
Specimen height, h=	29.15	mm	
Specimen diameter, d=	101.6	mm	
Diameter of Influent pipet d <sub>i</sub> =	10.3	mm	
Correction factor, influent pipet, CF <sub>i</sub>	1.1428	[ - ]	
Diameter of effluent pipet d <sub>o</sub> =	10.1	mm	
Correction factor, effluent pipet, CF <sub>o</sub>	1.1561	[ - ]	
Degree of saturation, S=	65.5	%	
Hydraulic gradient, i=	9.6	[ - ]	
Cell pressure applied, P <sub>c</sub> =	75.0	psi	
Head backpressure applied, P <sub>h</sub> =	70.4		
Tail backpressure applied, P <sub>t</sub> =	70.0		
Average effective stress, σ' <sub>1</sub> =	4.8	psi	to prevent swelling
Average effective stress, σ' <sub>2</sub> =	33.1	kPa	~Swell pressure
Hydraulic conductivity=	3.1E-08	m/s	

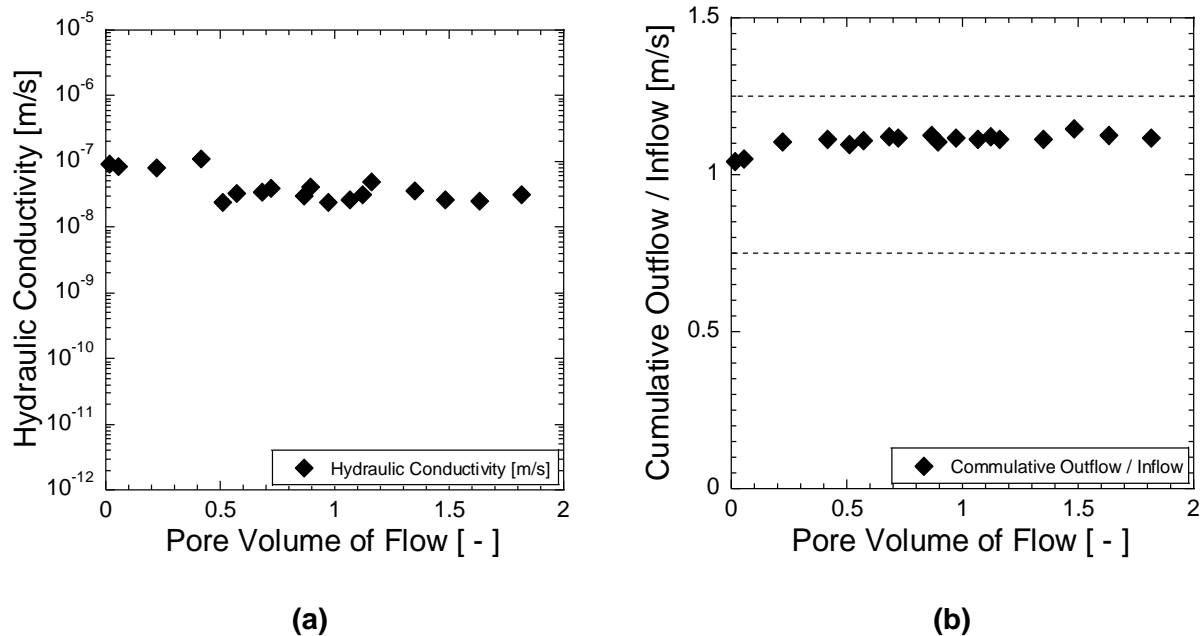


Figure A.88 Hydraulic conductivity of fly ash treated soil; (a) hydraulic conductivity vs. pore volumes of flow, and (b) cumulative outflow /inflow vs. pore volumes of flow.

### A.5.3 Lime Treated Hydraulic Conductivity Test (3%)

Table A.93 Hydraulic conductivity of the lime treated soil

Hydraulic Conductivity Test	Values	Units	Comments
Volume of soil, V=	451.5	mm <sup>3</sup>	Compacted with 1 layer & 19 blows
Water content, w%=	19.96	%	~Optimum water content
Specimen height, h=	29.3	mm	
Specimen diameter, d=	101.6	mm	
Diameter of Influent pipet d <sub>i</sub> =	10.3	mm	
Correction factor, influent pipet, CF <sub>i</sub>	1.1428	[ - ]	
Diameter of effluent pipet d <sub>o</sub> =	10.1	mm	
Correction factor, effluent pipet, CF <sub>o</sub>	1.1561	[ - ]	
Degree of saturation, S=	75.8	%	
Hydraulic gradient, i=	4.8	[ - ]	
Cell pressure applied, P <sub>c</sub> =	71.1	psi	
Head backpressure applied, P <sub>h</sub> =	70.2		
Tail backpressure applied, P <sub>t</sub> =	70.0		
Average effective stress, σ' <sub>1</sub> =	1	psi	almost no swelling
Average effective stress, σ' <sub>2</sub> =	6.9	kPa	~Swell pressure
Hydraulic conductivity=	1.5E-06	m/s	

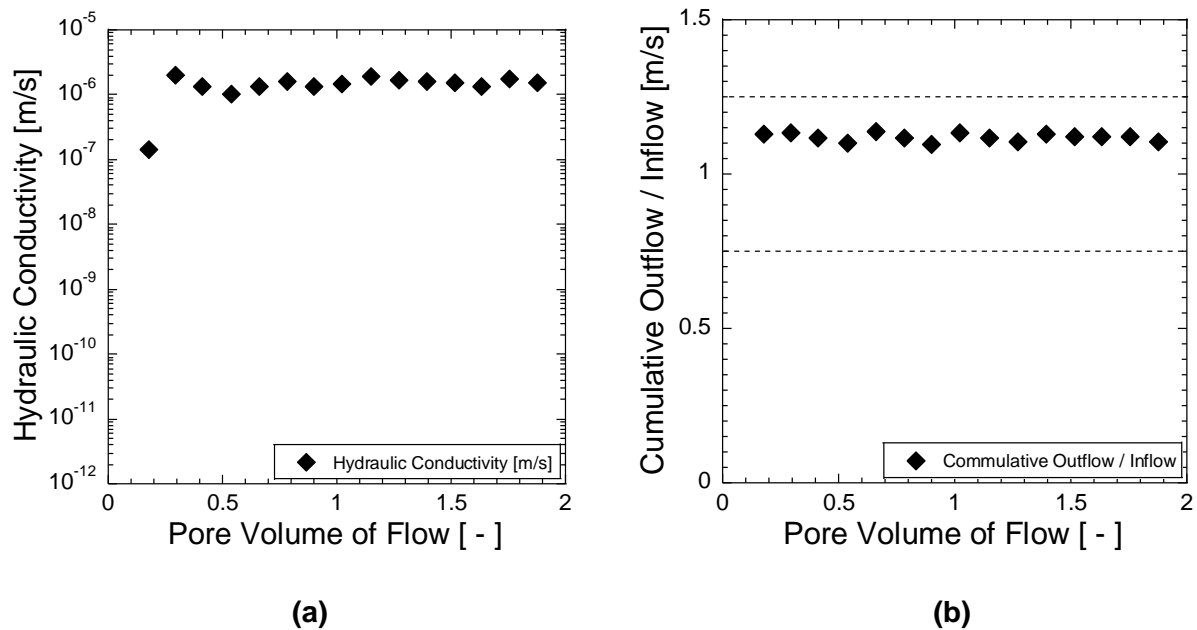


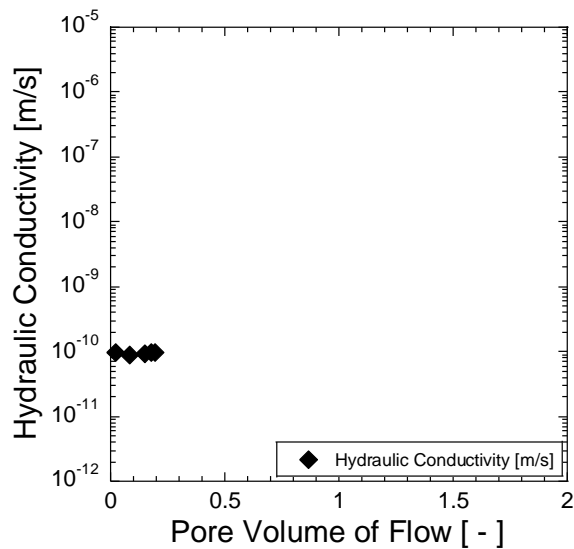
Figure A.89 Hydraulic conductivity of lime treated soil; (a) hydraulic conductivity vs. pore volumes of flow, and (b) cumulative outflow /inflow vs. pore volumes of flow.

### A.5.4 P4 Polymer Treated Soil Hydraulic Conductivity Test (5%)

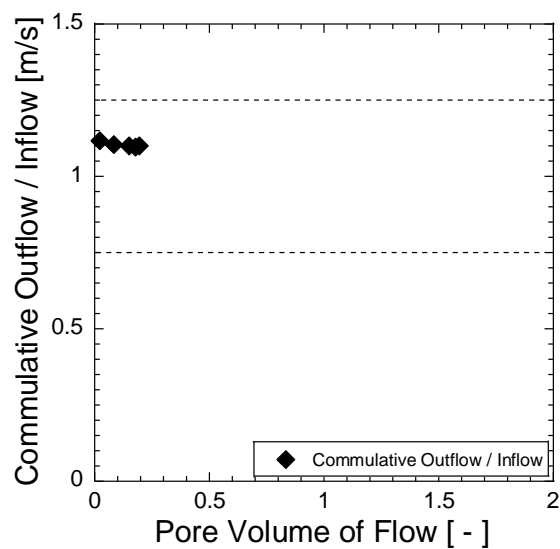
#### A.5.4.1 Assumed Swell Pressure

Table A.94 Hydraulic conductivity of P4 treated soil with assumed swell pressure

Hydraulic Conductivity Test	Values	Units	Comments
Volume of soil, V=	456.14	g	Compacted with 1 layer & 19 blows
Water content, w%=	18.55	%	~Optimum water content
Specimen height, h=	29.3	mm	
Specimen diameter, d=	101.6	mm	
The influent and effluent pipet diameters and correction factors are the same as Table A.93			
Degree of saturation, S=	74.3	%	
Hydraulic gradient, i=	28.8	[ - ]	
Cell pressure applied, P <sub>c</sub> =	75.0	psi	
Head backpressure applied, P <sub>h</sub> =	65.6		
Tail backpressure applied, P <sub>t</sub> =	64.4		
Average effective stress, $\sigma'_v$ =	<b>10</b>	psi	to prevent swelling
Average effective stress, $\sigma'_v$ =	68.9	kPa	~Swell pressure
Hydraulic conductivity=	9.4E-11	m/s	



(a)



(b)

Figure A.90 Hydraulic conductivity of P4 treated soil with assumed swell pressure; (a) hydraulic conductivity vs. pore volumes of flow, and (b) cumulative outflow /inflow vs. pore volumes of flow.

### A.5.4.2 Measured Swell Pressure

Table A.95 Hydraulic conductivity of P4 treated with actual swell pressure

Hydraulic Conductivity Test	Values	Units	Comments
Volume of soil, V=	456.14	g	Compacted with 1 layer & 19 blows
Water content, w%=	18.55	%	~Optimum water content
Specimen height, h=	29.3	mm	
Specimen diameter, d=	101.6	mm	
The influent and effluent pipet diameters and correction factors are the same as Table A.93			
Degree of saturation, S=	74.3	%	
Hydraulic gradient, i=	28.8	[ - ]	
Cell pressure applied, Pc=	77.4	psi	
Head backpressure applied, Ph=	60.6		
Tail backpressure applied, Pt=	59.4		
Average effective stress, $\sigma'$ =	17.4	psi	to prevent swelling
Average effective stress, $\sigma'$ =	120.0	kPa	~Swell Pressure
Hydraulic conductivity=	7.2E-11	m/s	

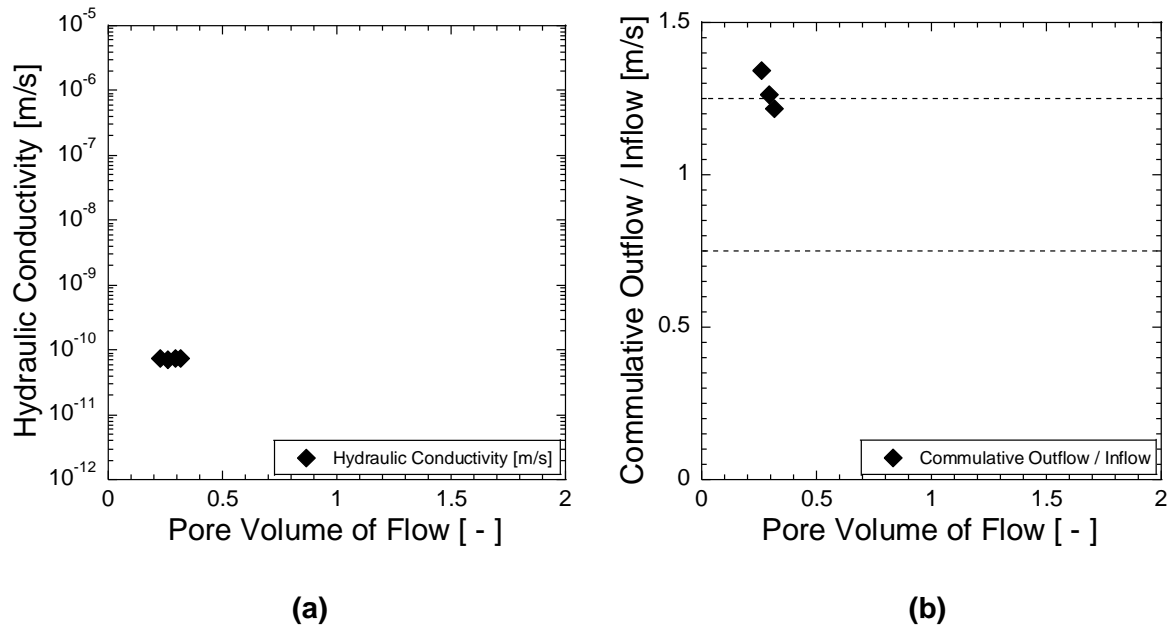


Figure A.91 Hydraulic conductivity of P4 treated soil with measured swell pressure; (a) hydraulic conductivity vs. pore volumes of flow, and (b) cumulative outflow /inflow vs. pore volumes of flow.

### A.5.4.3 Combined Assumed and Measured Swell Pressure Hydraulic Conductivities

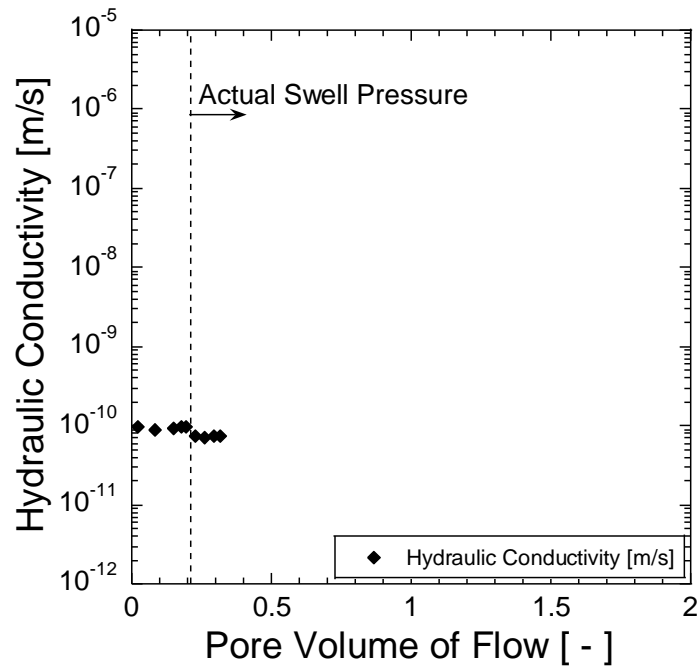


Figure A.92 Combined hydraulic conductivity results for P4 treated soil

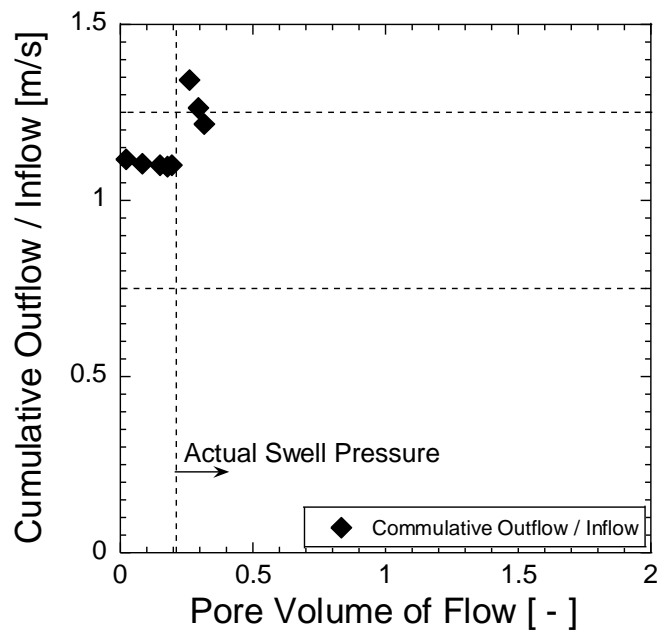


Figure A.93 Combined hydraulic conductivity results for P4 treated soil

## A.6 Unconfined Compressive Strength (UCS) Tests

### A.6.1 Expansive soil UCS Tests

#### A.6.1.1 Unsoaked UCS Test

Table A.96 Unconfined compressive strength of expansive soil – all tests unsoaked

Unconfined Compressive Strength Test	Values	Units	Comments
Volume of soil, V=	18581.4	mm <sup>3</sup>	Compacted with 6 layer & 25 blows
Water content, w%=	18.4	%	~Optimum water content
Specimen height, h=	232.86	mm	
Specimen diameter, d=	101.6	mm	
Speed of strain control=	2.0	% / min	2% as assumed brittle
Speed of strain control=	4.7	mm / min	
Unconfined Compressive Strength, qu=	242.7	kPa	Only unsoaked was doable
Undraind shear strength, Su=	121.3	kPa	

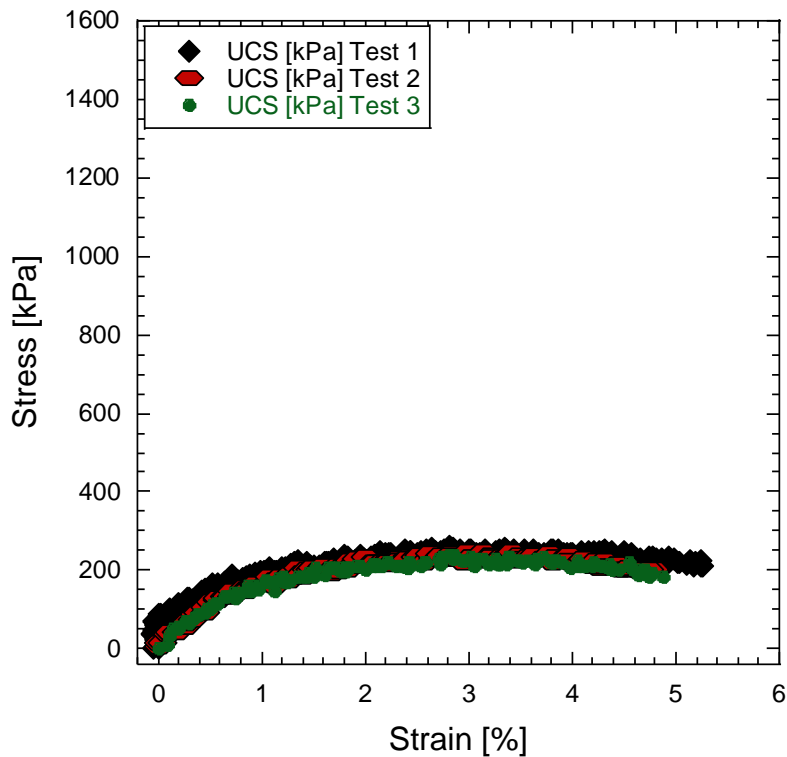


Figure A.94 Unconfined compressive strength of expansive soil - all tests unsoaked

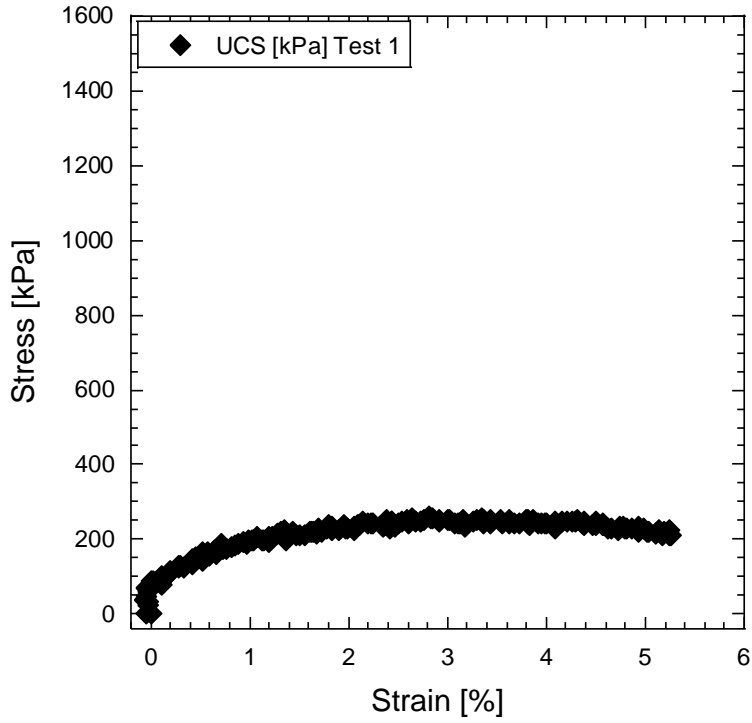


Figure A.95 Unconfined compressive strength of expansive soil – test 1 unsoaked

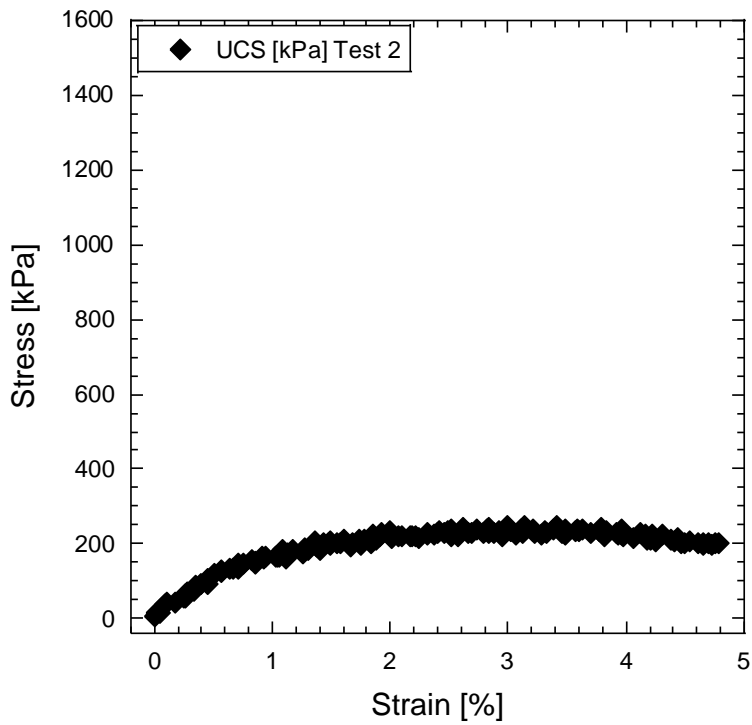


Figure A.96 Unconfined compressive strength of expansive soil – test 2 unsoaked



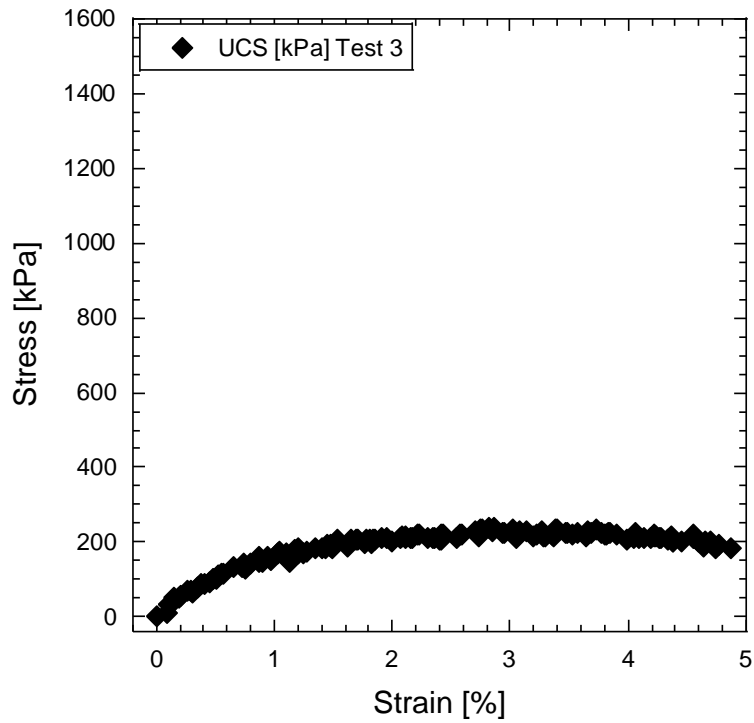


Figure A.97 Unconfined compressive strength of expansive soil – test 3 unsoaked

#### A.6.1.2 Soaked UCS Test

Test could not be completed. The soaked specimen could not be moved and placed under the UCS machine because of the weakness of the specimen caused by excessive swelling. The result of the UCS is assumed to equal zero.

## A.6.2 Fly Ash Treated UCS Tests (15%)

### A.6.2.1 Combined Soaked and Unsoaked UCS Tests

Table A.97 Unconfined compressive strength of fly ash treated soil

Unconfined Compressive Strength Test	Values	Units	Comments
Volume of soil, V=	18581.4	mm <sup>3</sup>	Compacted with 6 layer & 25 blows
Water content, w%=	18.4	%	~Optimum water content
Specimen height, h=	232.86	mm	
Specimen diameter, d=	101.6	mm	
Speed of strain control=	2.0	% / min	2% as assumed brittle
Speed of strain control=	4.7	mm / min	
Unconfined Compressive Strength, qu=	547.0	kPa	unsoaked
Undraind shear strength, Su=	273.5	kPa	
Unconfined Compressive Strength, qu1=	380.0	kPa	soaked
Unconfined Compressive Strength, qu2=	335.0	kPa	soaked
Unconfined Compressive Strength, qu3=	430.0	kPa	soaked
Unconfined Compressive Strength, qu <sub>ave</sub> =	381.7	kPa	soaked
Undraind shear strength, Su=	191.0	kPa	soaked

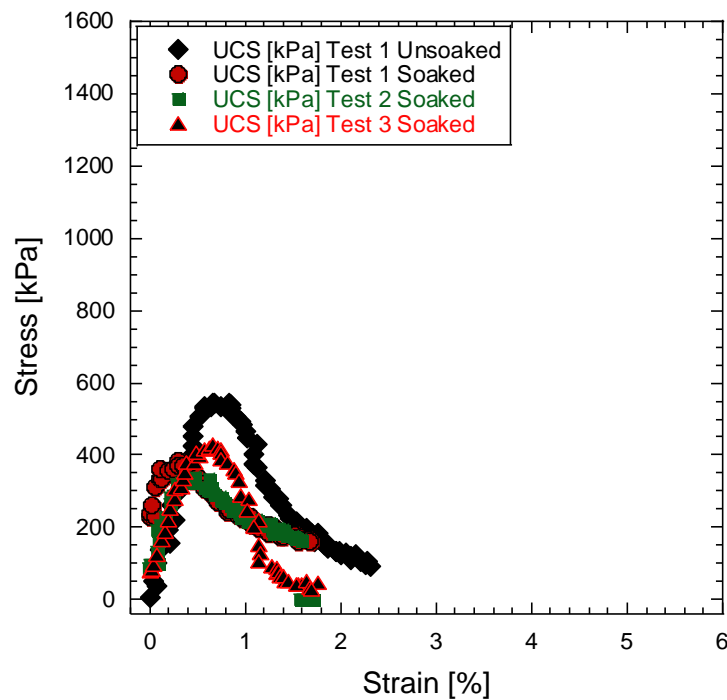


Figure A.98 Unconfined compressive strength of fly ash treated soil – all tests

### A.6.2.2 Unsoaked UCS Tests

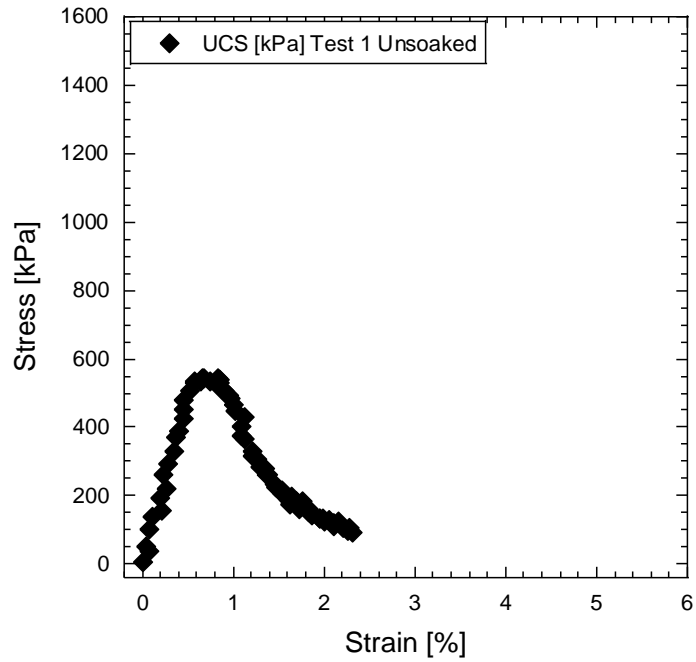


Figure A.99 Unconfined compressive strength of fly ash treated soil – test 1 unsoaked

### A.6.2.3 Soaked UCS Tests

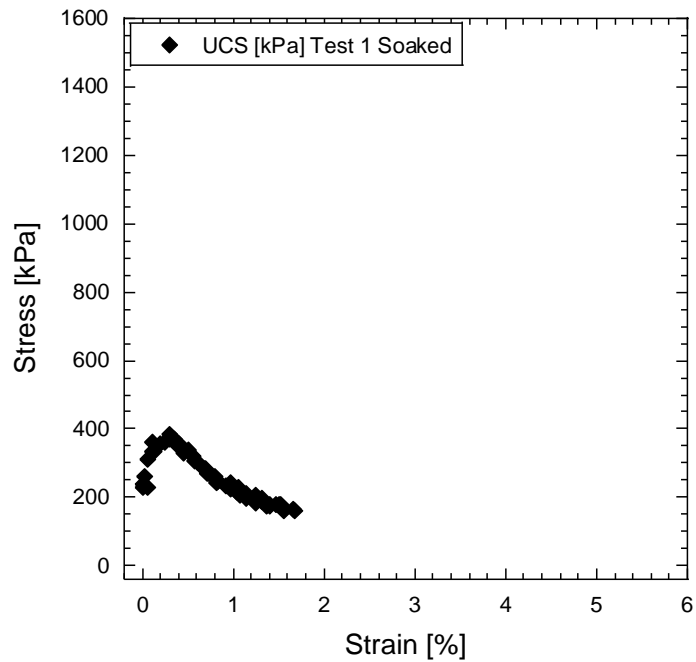


Figure A.100 Unconfined compressive strength of fly ash treated soil – test 1 soaked

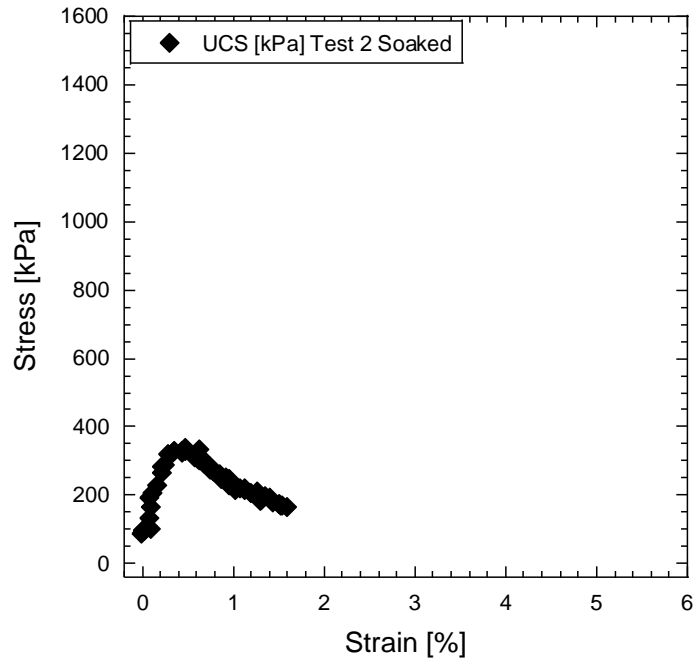


Figure A.101 Unconfined compressive strength of fly ash treated soil – test 2 soaked

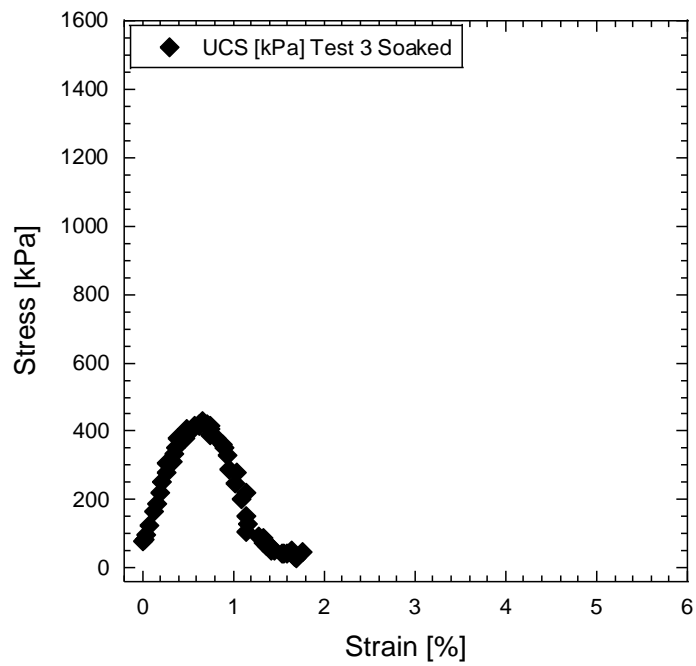


Figure A.102 Unconfined compressive strength of the fly ash treated soil – test 3 soaked

### A.6.3 Lime Treated UCS Tests (3%)

#### A.6.3.1 Combined Soaked and Unsoaked UCS Tests

Table A.98 Unconfined compressive strength of lime treated soil

Unconfined Compressive Strength Test	Values	Units	Comments
Volume of soil, V=	18581.4	mm <sup>3</sup>	Compacted with 6 layer & 25 blows
Water content, w%=	18.4	%	~Optimum water content
Specimen height, h=	232.86	mm	
Specimen diameter, d=	101.6	mm	
Speed of strain control=	2.0	% / min	2% as assumed brittle
Speed of strain control=	4.7	mm / min	
Unconfined Compressive Strength, qu=	1600.0	kPa	unsoaked
Undraind shear strength, Su=	800.0	kPa	
Unconfined Compressive Strength, qu1=	1160.0	kPa	soaked
Unconfined Compressive Strength, qu2=	1300.0	kPa	soaked
Unconfined Compressive Strength, qu3=	1320.0	kPa	soaked
Unconfined Compressive Strength, qu <sub>ave</sub> =	1260.0	kPa	soaked
Undraind shear strength, Su=	630.0	kPa	soaked

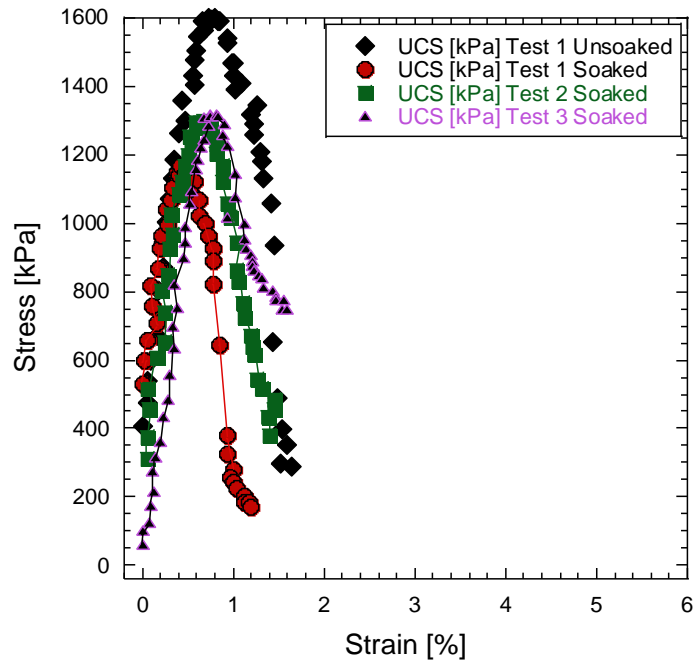


Figure A.103 Unconfined compressive strength of lime treated soils – all tests

### A.6.3.2 Unsoaked UCS Tests

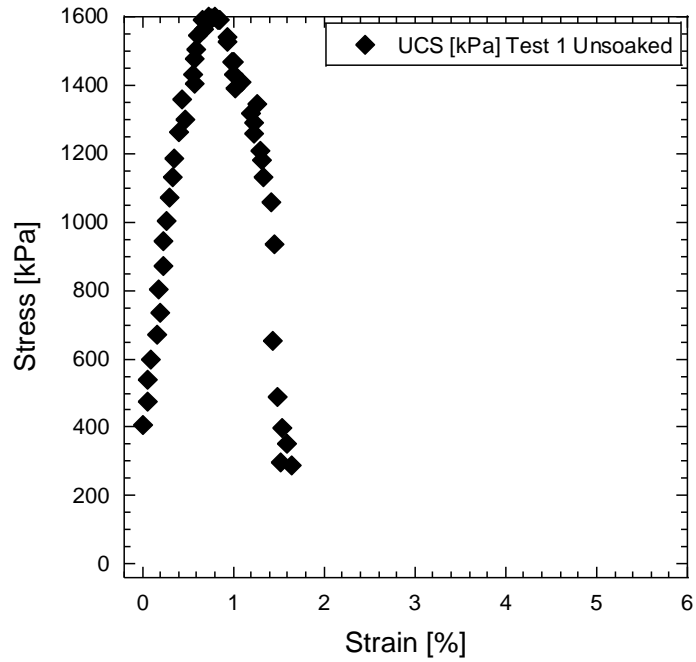


Figure A.104 Unconfined compressive strength of lime treated soil – test 1 unsoaked

### A.6.3.3 Soaked UCS Tests

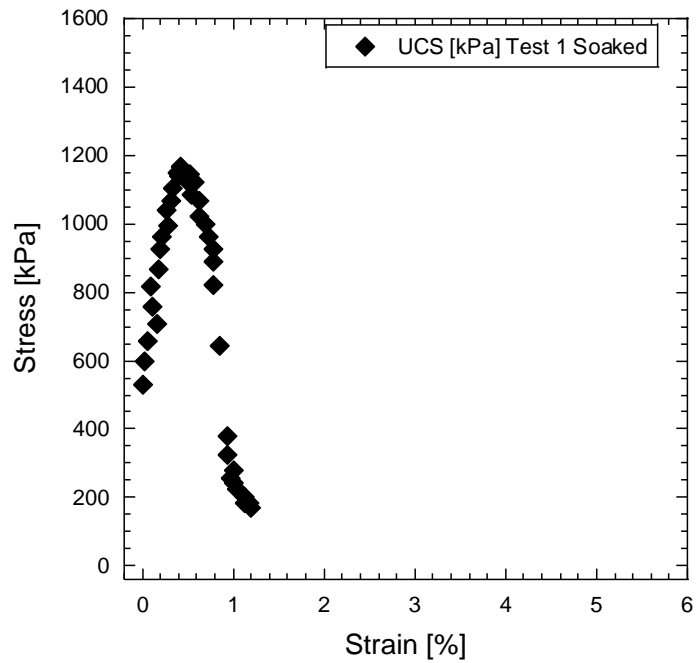


Figure A.105 Unconfined compressive strength of lime treated soil – test 1 soaked

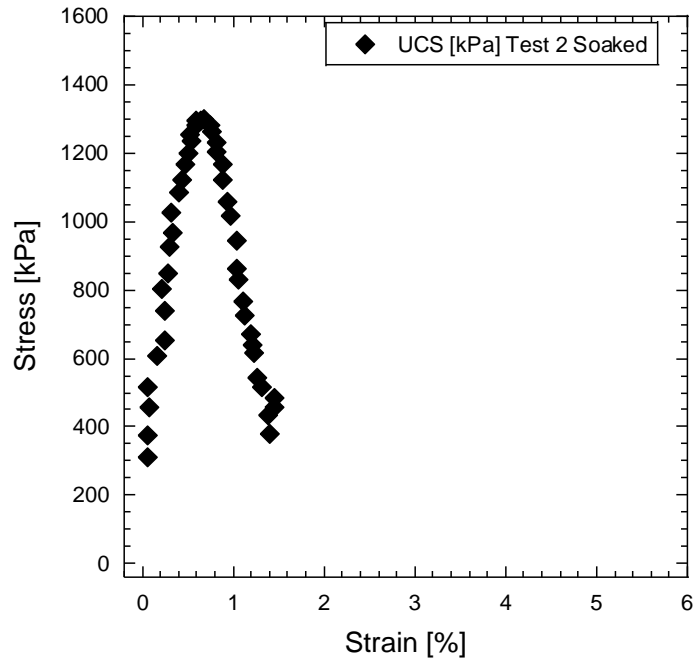


Figure A.106 Unconfined compressive strength of lime treated soil – test 2 soaked

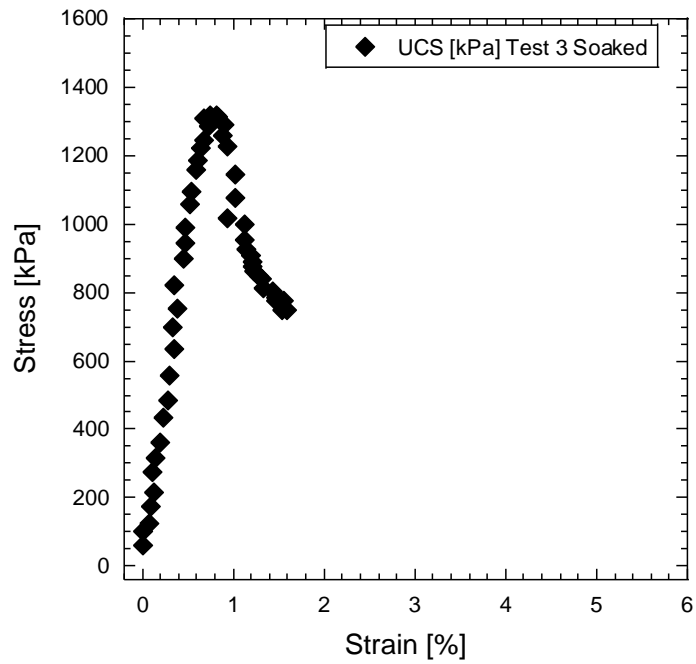


Figure A.107 Unconfined compressive strength of lime treated soil – test 3 soaked

## A.6.4 P4 Treated UCS Tests (5%)

### A.6.4.1 Combined Soaked and Unsoaked UCS Tests

Table A.99 Unconfined compressive strength of the P4 treated soil

Unconfined Compressive Strength Test	Values	Units	Comments
Volume of soil, V=	18581.4	mm <sup>3</sup>	Compacted with 6 layer & 25 blows
Water content, w%=	18.4	%	~Optimum water content
Specimen height, h=	232.86	mm	
Specimen diameter, d=	101.6	mm	
Speed of strain control=	2.0	% / min	2% as assumed brittle
Speed of strain control=	4.7	mm / min	
Unconfined Compressive Strength, qu=	260.0	kPa	unsoaked
Undraind shear strength, Su=	130.0	kPa	
Unconfined Compressive Strength, qu1=	20.0	kPa	soaked
Unconfined Compressive Strength, qu2=	62.0	kPa	soaked
Unconfined Compressive Strength, qu3=	56.0	kPa	soaked
Unconfined Compressive Strength, qu <sub>ave</sub> =	46.0	kPa	soaked
Undraind shear strength, Su=	23.0	kPa	soaked

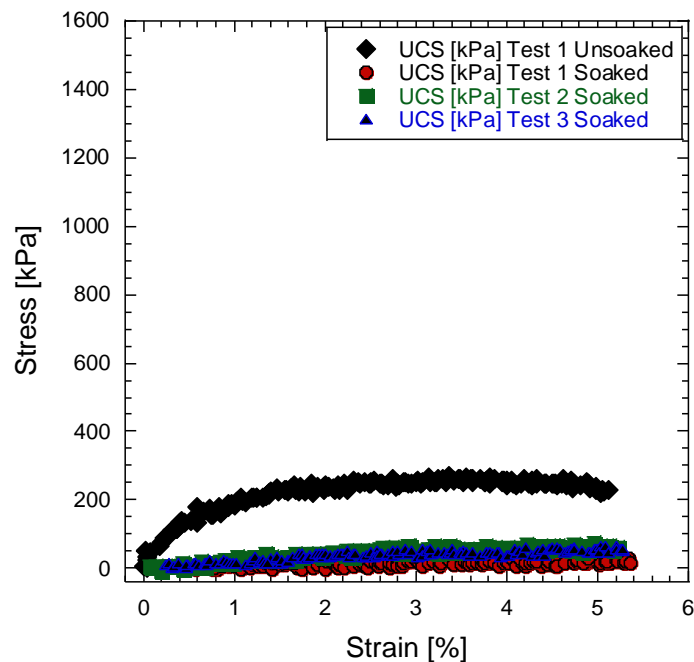


Figure A.108 Unconfined compressive strength of P4 treated soil – all tests



#### A.6.4.2 Unsoaked UCS Tests

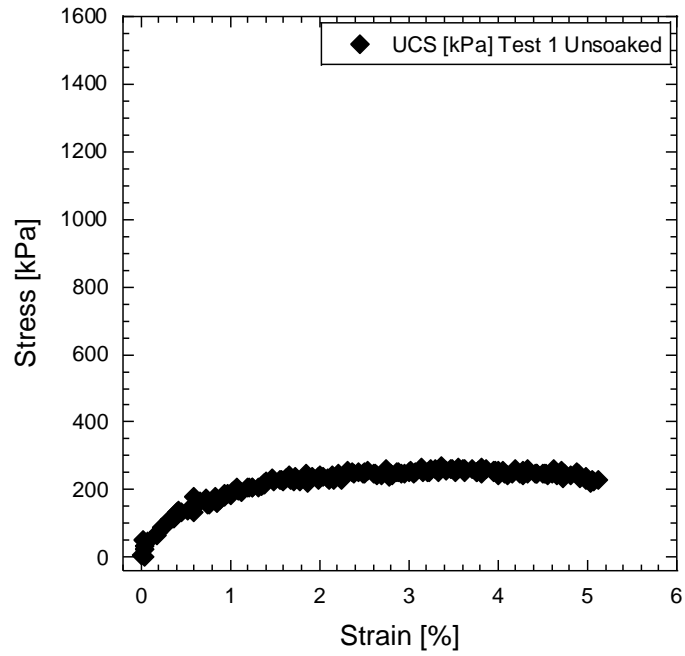


Figure A.109 Unconfined compressive strength of P4 treated soil – test 1 unsoaked

#### A.6.4.3 Soaked UCS Tests

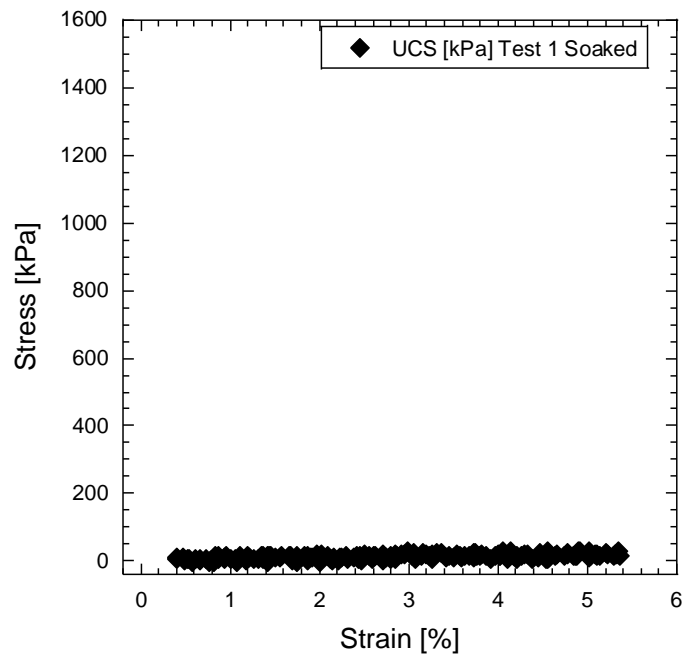


Figure A.110 Unconfined compressive strength of P4 treated soil – test 1 soaked

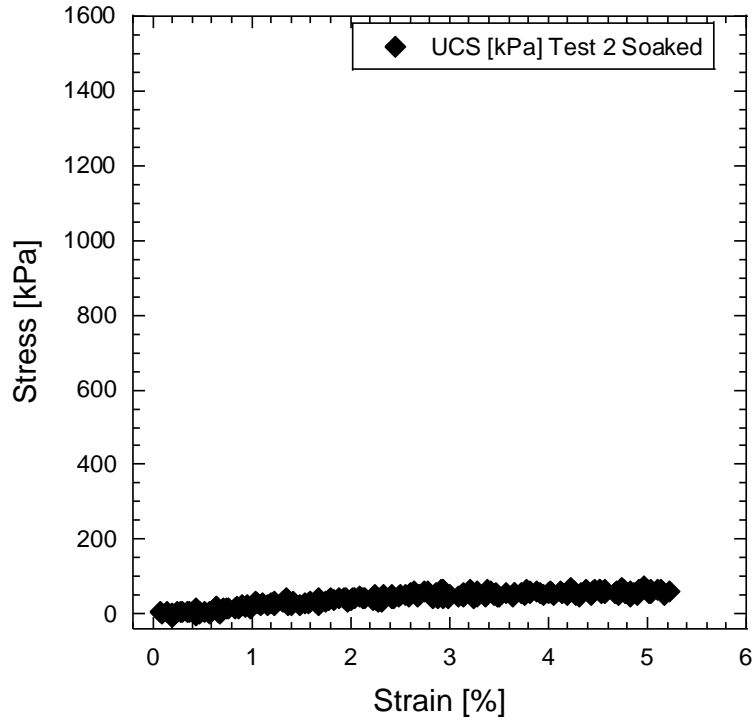


Figure A.111 Unconfined compressive strength of P4 treated soil – test 2 soaked

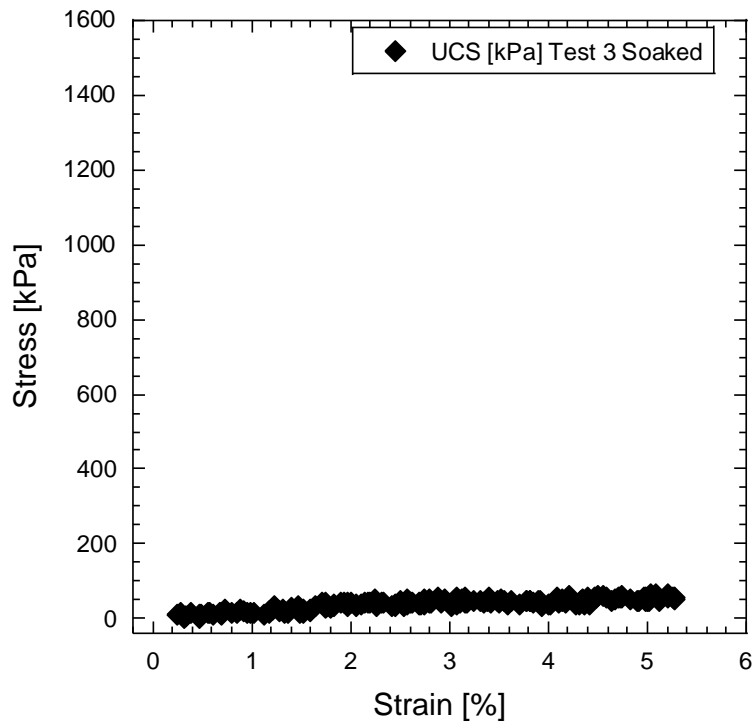


Figure A.112 Unconfined compressive strength of P4 treated soil – test 3 soaked

## **APPENDIX B: AMOUNT OF WATER GAINED BY THE SWELL TEST SPECIMENS**

This appendix provides a method for estimating the swell potential of a specimen from the amount of water gained without measuring the swell potential. This appendix presents the relationship between the amount of water gained versus swell test results of the cured expansive soil specimen and the specimens cured with fly ash, lime, and P1 polymers. For further expanding this relation, P4 polymer swell test data are also presented. The curing used in this research was an accelerated seven days with specimens in a humid oven at 40 °C. The result of the amount of water gained by each swell test specimen in the odometer rings, and the corresponding swell/collapse values are presented in Table B.1 and Fig. B.1. The water gained is calculated as the amount of water gained by the specimen after the swell test finishes (a small amount of water loss occurs after seven days of curing). Therefore, the amount of water gained is measured after the swell tests.

Table B.1 Amount of water gained by the swell test specimens – test one

Treatment Methods	Amount of Water Gained in grams	Swell/Collapse Percentage
Expansive Soil	7.1	4.1
The Fly Ash Treated	1.64	0.1
The Lime Treated	1.55	0
The 4.6% P1 Treated	4.1	2.6
The 9.2% P1 Treated	7.44	5.2
The 18.4% P1 Treated	5.77	1.7

Note: All the specimens cured for seven days at 40 °C

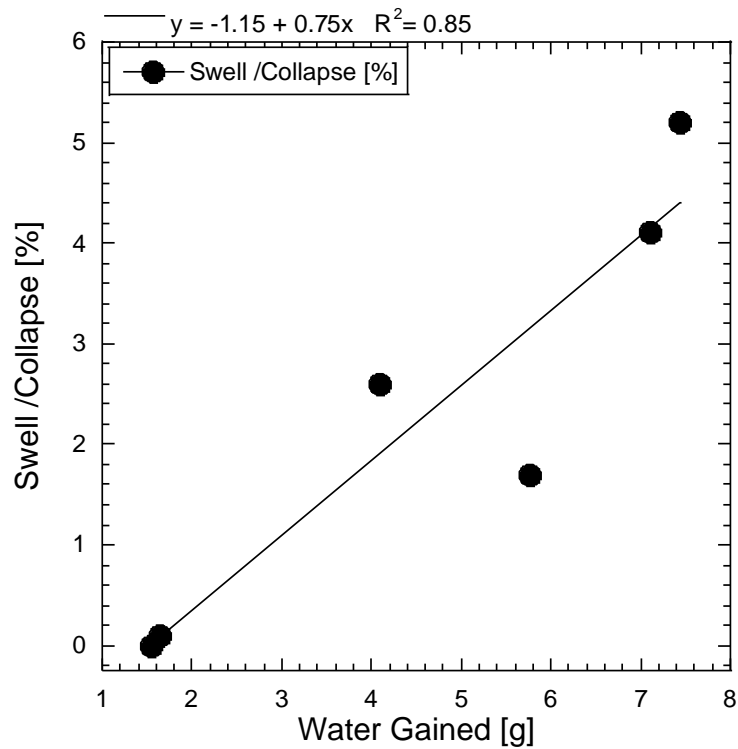


Figure B.1 Amount of water gained by swell test specimens – test one

There is relation between the amount of water gained by the soil specimens and the swell potential values. The  $R^2$  value is 0.85 supports a potential correlation between water gained and swell potential. Table B.2 and Fig. B.2 present companion data for swell tests on P4.

Table B.2 Amount of water gained by swell test specimens – test two

5% P4	Amount of Water Gained in grams	Swell/Collapse Percentage
The Swell specimen under 1 kPa	8.9	4.9
The Swell specimen under 10 kPa	6.04	1
The Swell specimen under 50kPa	3.82	0.6
The Swell specimen under 100 kPa	3.99	0.1
The Swell specimen under 207.3 kPa	2.98	-0.6
The Swell specimen under 1 kPa	9.26	4

Note: All the specimens cured for seven days under 40 °C

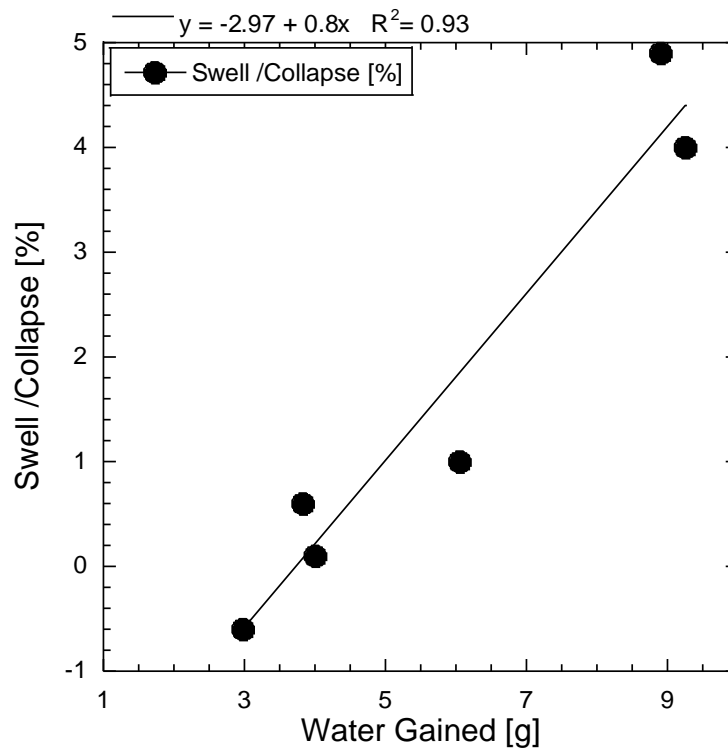


Figure B.2 Amount of water gained by swell test specimens on P4 treated soil – test two

Figure B.2 shows a close relation between swell and water gained and has a  $R^2$  of 0.93.

## APPENDIX C: SWELL SPECIMEN MODIFIED COMPACTION METHOD

This appendix describes the compaction method used in this research. The number of blows and weight of the hammer is provided in Chapter 3. In this appendix, the choice of 10 blows for the first layer and 25 blows for the second layer will be discussed.

Fig. 3.1. and Fig. 3.2 show the soil to be compacted in the first layer is inside the odometer ring having a diameter of 63.50 mm and an area of 3166.9 mm<sup>2</sup>. The second soil layer is the full area of the standard compaction mold and requires 25 blows. Therefore, the second soil layer will have a diameter of 101.60 mm and an area of 8107.3 mm<sup>2</sup>. The number of blows required was determined based on the ratio between the two areas mentioned. The ratio of the area of the first layer divided by that of the second layer  $3166.9/8107.3 = 0.39$ . This ratio (0.39) multiplied by 25 blows is rounded to 10.

For the standard compaction test, the amount of energy that is needed to be applied to a specimen is:

$$E = \frac{\text{Number of blows per layer} \times \text{Number of layers} \times \text{hammer weight} \times \text{hammer drop height}}{\text{volume of mold}}$$

$$E = \frac{25 \times 3 \times 5.5 \text{ lb} \times 1 \text{ ft}}{30^{-1} \text{ ft}^3} = 12400 \text{ ft-lb/ft}^3$$

To determine the amount of energy applied in the swell tests, two separate energies for each layer should be calculated and the average should be taken as the followings.

- The height of the soil needed to fix the ring is around half an inch in height.
- For the energy required for the first layer, the amount of loose soil added into the odometer ring should be considered. In this study, an amount of soil was added into the odometer ring for the first layer that created a small curvature on top of the ring. When compacted, around half to three quarters of the volume of the ring was filled. This gave an average compacted volume of around  $550^{-1} \text{ ft}^3$  that resulted in an energy of 30250 ft-lb/ft<sup>3</sup>.

- For the energy required for the second layer, an amount of soil was added into the compaction mold that created a curvature on top of the ring such that when completely compacted, the ring was safe. The amount of the soil above the ring was estimated to be around half an inch. This gave an approximate height of  $0.5'' + 0.5'' = 1''$ . Ignoring the volume of the ring inside the second layer (because of possible excess soil above the ring), the volume of the second layer was  $135^{-1} \text{ ft}^3$  that resulted in an energy of  $18550 \text{ ft-lb/ft}^3$ . Combining these two energy values, an average energy of  $24400 \text{ ft-lb/ft}^3$  was obtained.

Theoretically, the above energy is around two times higher than that of the standard compaction test. The numbers of blows to fulfil the standard compaction energy would be 4 blows and 17 blows for the first layer and the second layer, respectively. However, when practically measuring maximum dry unit weight with varying number of blows per layer, the accuracy in applying the theoretical energies mentioned did not guarantee that the specimen was compacted close to maximum dry density. The laboratory data for a variety of compaction tests with different blows per layer is presented in this appendix. Results show that changing the number of the blows after the soil was completely compressed did not change maximum dry unit weight significantly (change in the compaction energy was not linear with change of the ratio of actual dry unit weight to maximum dry unit weight). The validity of this compaction method was checked on expansive soil and fly ash treated soils. The results for fly ash treated soils are presented in Table C.1.

Table C.1 Trial and error for determining optimum number of compaction blows for the fly ash treated soil

Trial number	Number of blows for the first layer	Number of blows for the second layer	Actual dry unit weight / maximum dry unit weight ( $\gamma_d/\gamma_{d \text{ max}}$ )
1	2	6	0.955
2	3	8	1.008
3	7	19	1.034
4	10	25	<b>1.014</b>
5	13	33	1.027

As shown in Table C.1, the number of the blows for trial 2 and trial 4 are highly different but their effects on  $\gamma_d/\gamma_{d_{max}}$  are very close. Theoretically trial 2 would generate an average energy of 7670 ft-lb/ft<sup>3</sup> that is around 62% of the standard compaction effort, however, trial 4 would generate an average energy of 24400 ft-lb/ft<sup>3</sup> that is 197% of the standard compaction effort. Therefore, the choice of 10 blow for the first layer and 25 blow for the second layer would guarantee that  $\gamma_d/\gamma_{d_{max}}$  would be at least 1.00 (average is 0.998 from trial 4 and 5) as confirmed by the data of expansive soil shown in Table C.2.

Table C.2 Trial and error for determining optimum number of compaction blows for expansive soil

Trial Number	Number of blows for the first layer	Number of blows for the second layer	Actual Dry Unit Weight / Maximum Dry Unit Weight ( $\gamma_d/\gamma_{d_{max}}$ )
1	7	19	0.996
2	7	19	1.009
3	7	19	0.989
4	10	25	<b>1.007</b>
5	10	25	0.986



## APPENDIX D: WEIGHT AND PRESSURE CALCULATIONS IN SWELL TEST

The following pressures are used in this study to assess the swelling behavior of the treated and untreated specimens: 1, 10, 20, 50, 100, and 207.3 kPa (Table D.1). The magnifying factor of the consolidation assembly should be considered to calculate the amount of mass required to be placed on the loading bar of the consolidation assembly. In this study, the consolidation assembly had a magnifying factor of 10. Meaning, any mass placed at the end of the loading bar acted 10 times more on the specimen than the actual amount. To calculate the mass required for one kPa, for instance, the area of the odometer ring and the magnifying factor should be considered. The typical diameter of the rings in this research was 63.50 mm that makes an area of 0.00317 m<sup>2</sup> (also, the rings were 1" high). One kPa equals 101936.8 g/m<sup>2</sup>, which when multiplied by the ring area, results in a mass of 322.8 g. Due to the magnifying factor, this mass would be divided by 10 that results in a required mass of 32.28 g. Table D.1 reports the masses used in the swell tests performed in this study.

Table D.1 Amount of mass required for varying pressures in swell test

Pressure needed [kPa]	Mass to be placed [g]
1	32.28
10	322.83
20	645.65
50	1614.13
100	3228.26
207.3	6692.18

## **APPENDIX E: HYDRAULIC CONDUCTIVITY INFLUENT AND EFFLUENT BURETTES READING CORRECTIONS**

This appendix provides steps to measure units and diameters of influent and effluent burettes used in the hydraulic conductivity test to calculate accurate inflow and outflow volumes. In other words, the diameter and the reading on the influent and effluent burettes were measured in certain units to avoid inaccurate hydraulic conductivity results, as follows.

1. Burette reading: the influent and effluent burettes readings were measured with a ruler, and the units on the burettes were converted to the units on the ruler.
2. Burette area: at a certain reading of the influent or effluent burettes, the level of the water in the pipet was recorded. This reading was corrected based on step 1. Then, an amount of water was taken out from burette. The water was put in a can of known weight, and the weight of the water was measured. The lowered level of the water in the burette was recorded. The difference between the second and the first burette readings was calculated in millimeters. The weight of the water, which was measured in the can, was converted to volume based on an ideal density of water of  $1 \text{ g/cm}^3$  (At room temperature, density of water is  $1.00 \text{ g/cm}^3$ ). By knowing the volume of the extracted water and the height obtained from the two readings, the areas of the burettes were calculated.

## **APPENDIX F: INFLUENCE OF CHANGING EFFECTIVE STRESS ON HYDRAULIC CONDUCTIVITY**

As per ASTM 5084-16a, volume change of a hydraulic conductivity specimen should be minimized by applying an effective stress that prevents swell and collapse. In this study, effective stresses, which was equalized to swell pressure, were used to prevent volume change (the total pore water pressure was assumed to be negligible as discussed in chapter 4, Section 4.4.3).

The purpose of this appendix is to assess whether or not changing effective stress will cause substantial change in hydraulic conductivity. For this purpose, different swell pressure values used for the soil treated with the P4 polymer (Table E.1) will be analyzed. Firstly, the value of the swell pressure was assumed based on the soil's swell potential compared with that of fly ash. The assumed swell pressure was 10 psi or 69 kPa. The hydraulic conductivity with this assumed swell pressure was  $9.4 \times 10^{-11}$  m/s. For the second swell pressure, the actual measured swell pressure was used. The actual swell pressure was 17.4 psi or 120 kPa. The hydraulic conductivity with this actual swell pressure was  $7.2 \times 10^{-11}$  m/s.

By increasing the swell pressure by 74%, the hydraulic conductivity decreased only 23%. This change in the hydraulic conductivity is negligible (considering the effect of the treatments) as the change is within one order of magnitude. However, increasing the swell pressure by 74% is significant. Therefore, hydraulic conductivity of the treated soils was dependent on the influence of the treatment more than the effective stress.

## APPENDIX G: PHOTOGRAPHS OF SPECIMENS

### G.1 Swell Test for Analyzing Shrinkage, Collapse, and Swelling of Different Treatments

#### G.1.1 Photos of the Specimens after Seven Days of Curing and Before the Swell Test

The photos in this section were taken immediately before the Swell test. In this section, photographs of the untreated specimen and the specimens treated with lime, fly ash, and three addition ratios of P1 polymer (4.59%, 9.18%, and 18.36%) are presented. All of the specimens were cured for seven days at 40 °C.

The shrinkage of the specimens can be related to the results of the swell potential of each specimen after the swell test. For example, the shrinkage of the specimen treated with lime is unnoticeable. Meanwhile, the lime treated soil had a swell potential value of 0.0 %. Therefore, before running the swell test, the swell potential can be estimated based on the specimen's shrinkage. Further studies can be done to measure shrinkage and build relationships between shrinkage and swelling of the specimens.

##### G.1.1.1 Photograph of expansive soil



Figure G.1 Expansive soil after seven days of curing and before the swell test

### G.1.1.2 Photograph of Fly Ash Treated Soil

FA Treated Soil



Figure G.2 Fly ash treated soil after seven days of curing and before the swell test

### G.1.1.3 Photograph of Lime Treated Soil

Lime Treated Soil



Figure G.3 Lime treated soil after seven days of curing and before the swell test

**G.1.1.4 Photograph of 18.36 % P1 Polymer Treated Soil**



Figure G.4 18.36% P1 treated soil after seven days of curing and before the swell test

**G.1.1.5 Photograph of 9.18 % P1 Polymer Treated Soil**



Figure G.5 9.18 % P1 treated soil after seven days of curing and before the swell test

**G.1.1.6 Photograph of 4.59 % P1 Polymer Treated Soil**



Figure G.6 4.59 % P1 treated soil after seven days of curing and before the swell test

### **G.1.2 Photos of The Specimens After Seven Days of Curing and After the Swell Test**

The photos in this section were taken after the swell test finished. In this section, photographs of the untreated specimen and the specimens treated with lime, fly ash, and three addition ratios of P1 polymer (4.59%, 9.18%, and 18.36%) are presented. All of the specimens were cured for seven days at 40 °C.

All of the specimens were loaded with one kPa pressure to determine their swell potential values. The swelling of the specimens in the photographs matches the measured amount of the water gained and the results of the swell potential values. The stabilizing effect of the lime and fly ash can be seen on the treated soils.

#### **G.1.2.1 Photograph of expansive soil**



Figure G.7 Expansive soil after seven days of curing and after the swell test



### G.1.2.2 Photograph of Fly Ash Treated Soil



Figure G.8 Fly ash treated soil after seven days of curing and after the swell test

### G.1.2.3 Photograph of Lime Treated Soil



Figure G.9 Lime treated soil after seven days of curing and after the swell test

**G.1.2.4 Photograph of 18.36 % P1 Polymer Treated Soil**



Figure G.10 18.36% P1 treated soil after seven days of curing and after the swell test

**G.1.2.5 Photograph of 9.18 % of the P1 Polymer Treated Soil**



Figure G.11 9.18 % P1 treated soil after seven days of curing and after the swell test

**G.1.2.6 Photograph of 4.59 % P1 Polymer Treated Soil**

4.59% Polymer- Soil



Figure G.12 4.59 % P1 treated soil after seven days of curing and after the swell test

### G.1.3 Additional Photographs Related to Shrinkage, Swelling, and Collapse

#### G.1.3.1 P1 Polymer, Lime, and Fly Ash Treated Soils Cured with 24 Hour Air-Drying

An addition of 18.36% P1 polymer with four different water contents based on the optimum water content was compared with 3% of lime for assessing the shrinkage amount. As shown in Figure G.13, the lower the water addition to polymer treated soils the lower the amount of shrinkage occurred. This is because of evaporable liquid within the fluidic polymer. The water (liquid) content of the polymer should be accounted for while adding polymer as an additive.

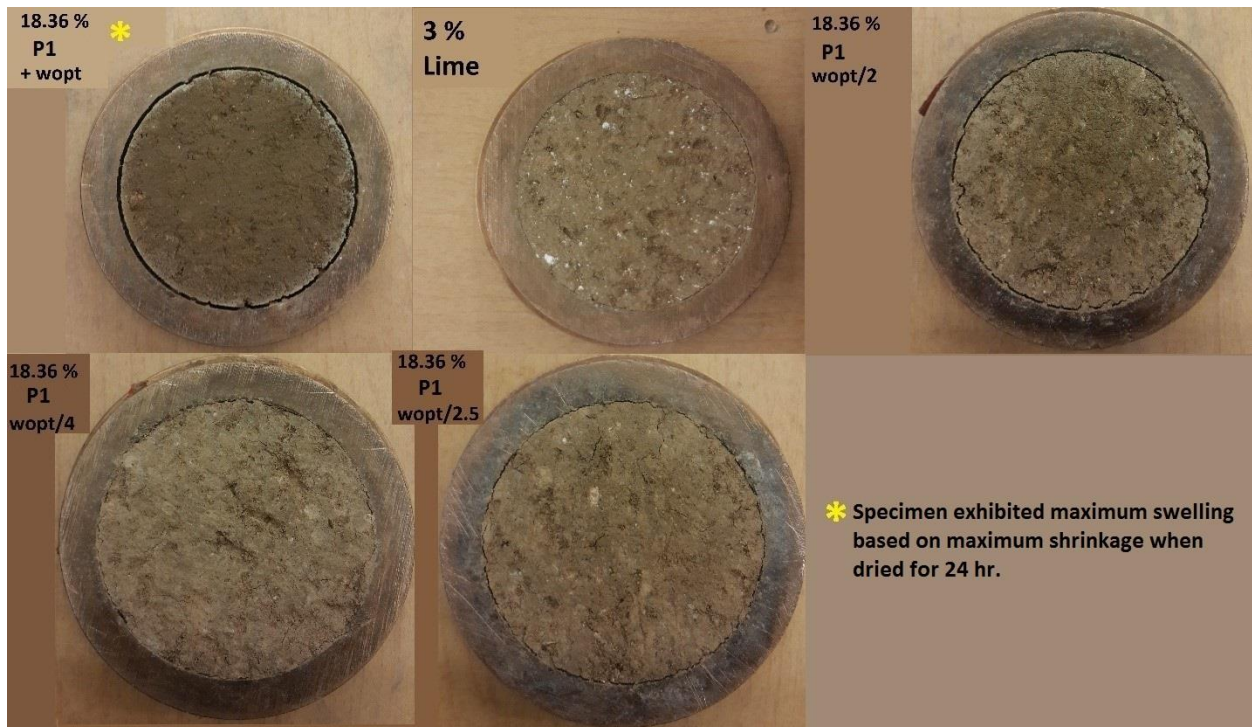


Figure G.13 P1 polymer, lime, and fly ash treated soils with 24 hour drying

### G.1.3.2 P1 Polymer, Lime, and Fly Ash Treated Soils Wetted for Six Hours

The same soil specimens that were air dried for 24 hours in Section G.1.3.1, were wetted for six hours under no pressure to investigate their free swellings. The lime treated soil whose shrinkage was minimum exhibited the lowest swelling. The P1 treated specimens swelled substantially. The specimen treated with 18.36% P1 polymer with optimum water content showed the highest shrinkage.



Figure G.14 P1 polymer, lime, and fly ash treated soils wetted for six hours after 24-hour air drying

### G.1.3.3 Collapse of a Specimen Treated with 2.34% P1 Polymer

Figure G.15 shows the significant amount of collapse of a dried swell specimen taken out from the ring and wetted for an hour (treated with 2.34% P1). This collapse (with complete wetting of the specimen) was expected from the weakness of the UCS polymer-treated specimens when soaked for 24 hours.

The dry specimen shown in the photo represents the specimens before wetting. The level of the water to wet the specimens was kept at half of the specimen heights but caused the whole specimens to collapse.



Figure G.15 Collapse of specimens treated with 2.34% P1 polymer

#### G.1.3.4 Collapsibility of the Expansive soil Specimen vs. the Fly Ash Treated Specimen

Figure G.16 shows collapsibility of expansive soil compared to fly ash treated soil exhibiting ductility.



Figure G.16 Collapsibility of an expansive soil specimen vs. a fly ash treated specimen

## G.2 Effect of Air Drying and Oven Drying on P1 Polymers

### G.2.1 Effect of Air Drying On P1 Polymers

A quick experiment was done to test how polymers stabilize expansive soils. Unlike lime and fly ash, polymers have gluey effect on soils as shown on Figure G.17. Due to this gluey effect, a great amount of polymers is required to stabilize a small amount of soil. Also, if polymers stabilize only the top surface of the soil when applied in the field, the expansive soil up to the active zone depth would still be untreated, and may undergo significant potential swelling.



Figure G.17 The effect of air drying on P1 polymers (1/8" penetrated - undiluted)

### **G.2.2 Polymer Water Content and the Effect of Oven Drying on P1 Polymers**

The tests requiring compaction in this study were run at optimum water content, therefore, the evaporable liquid inside the polymers was required to be considered for adding water into the soils. An amount of the liquid polymers was put in a can and oven dried for at least 24 hours to measure the water content.

Similar to the air drying, when the polymer was oven dried, the polymer created a gluey structure that was brittle. The water content of the P1 polymer was 123.34%. The water content of the P4, which was selected for this study, was 246.37% (P4 was the most dilute polymer among the four polymer types studied in this research).

Since a great portion of the P4 polymer was evaporable, the ratio of water to polymer was found, and based on that ratio, the remaining water to reach the optimum water content was calculated. For P4, the ratio of water to polymer was 2.5:1 (2.4637:1), or 71.1% of the polymer was evaporable liquid.

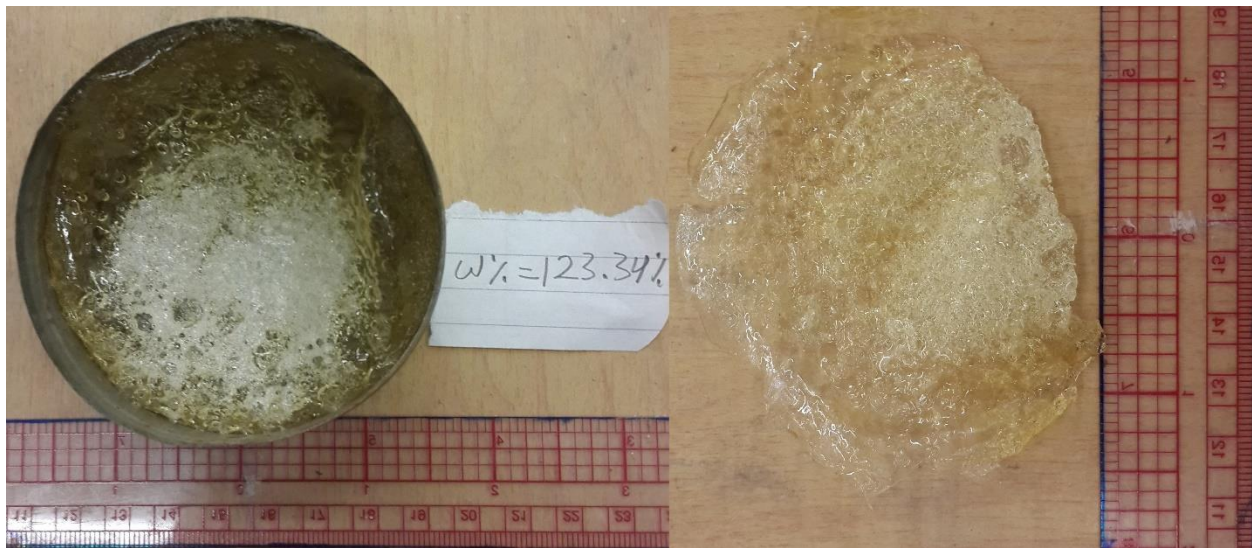


Figure G.18 Polymer water content and the effect of oven drying on P1 polymers



### G.3 Effect of Soaking On UCS Specimens of P4 and Lime Treated Soils

Figure G.19 shows incapability of P4 in providing a durable specimen under  $qu$  testing compared to the  $qu$  of the specimens treated with lime. After seven days of curing and 24 hours of soaking, the weakness of the specimen treated with P4 was comparable to that of expansive soil. However, the lime treated specimen was not considerably weakened by the 24 hour soaking. Therefore, under the UCS test, the specimen treated with P4 exhibited a low  $qu$  as compared to lime treated soil.

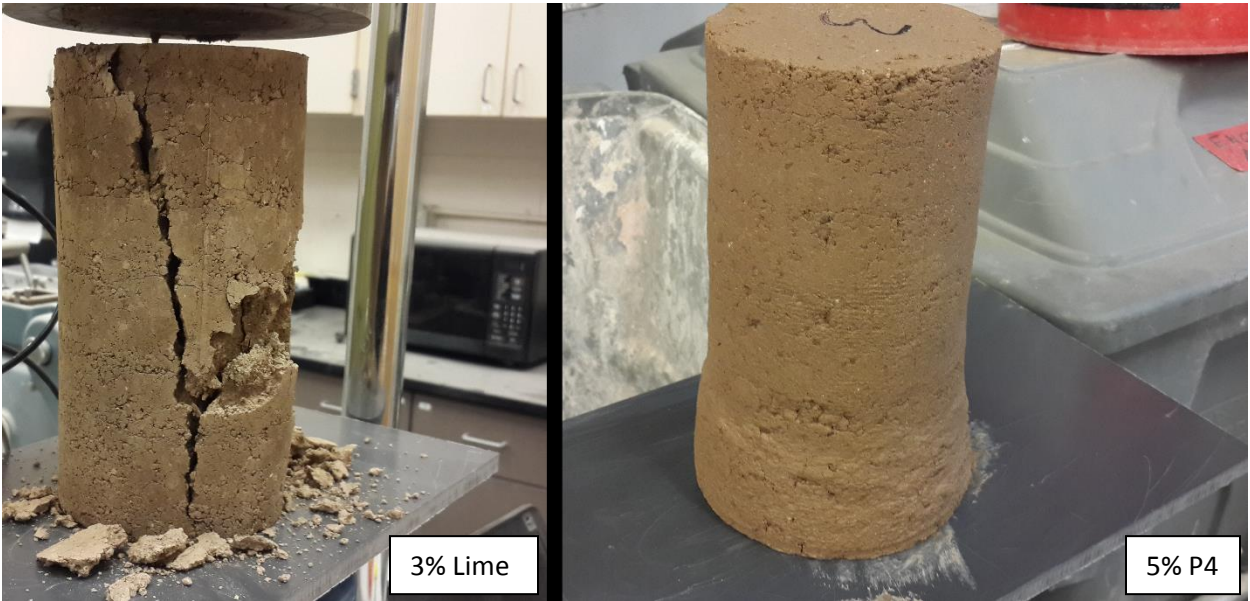


Figure G.19 The effect of soaking on UCS specimens of P4 and lime soils after failure

## G.4 Photos of the Materials Used in This Study (Polymers Not Shown)

### G.4.1 Nelson Farm Clay (NFC)



Figure G.20 Nelson Farm Clay (NFC)

### G.4.2 Bentonite

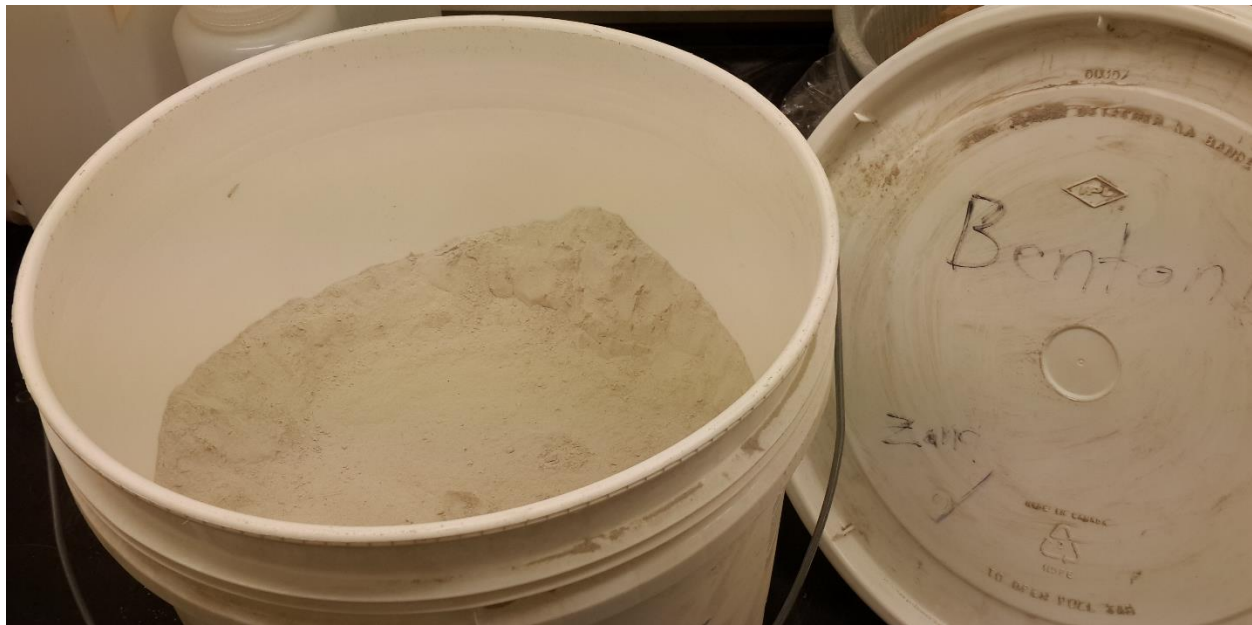


Figure G.21 Sodium bentonite

**G.4.3 Fly Ash Class-C**

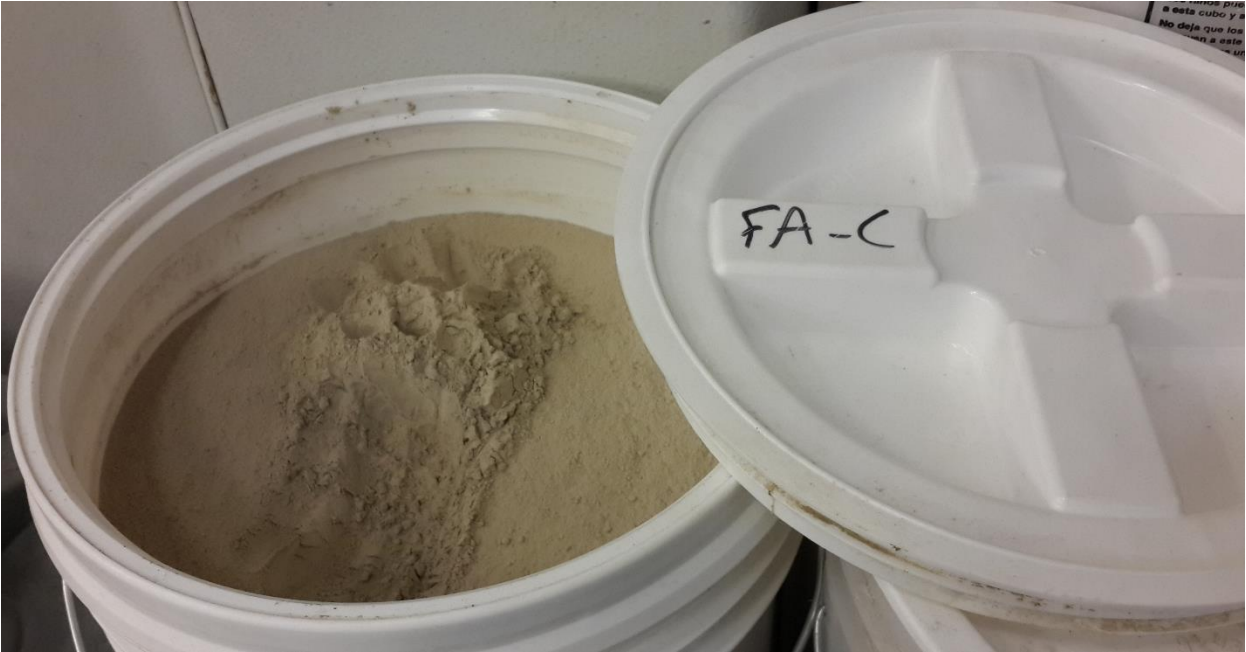


Figure G.22 Fly ash Class-C

**G.4.4 Hydrated Lime**

This lime was not used in this study. The “Eades-Grim” test on this lime was performed, but similar to quicklime, the “Eades-Grim” test was not successful (Appendix I).

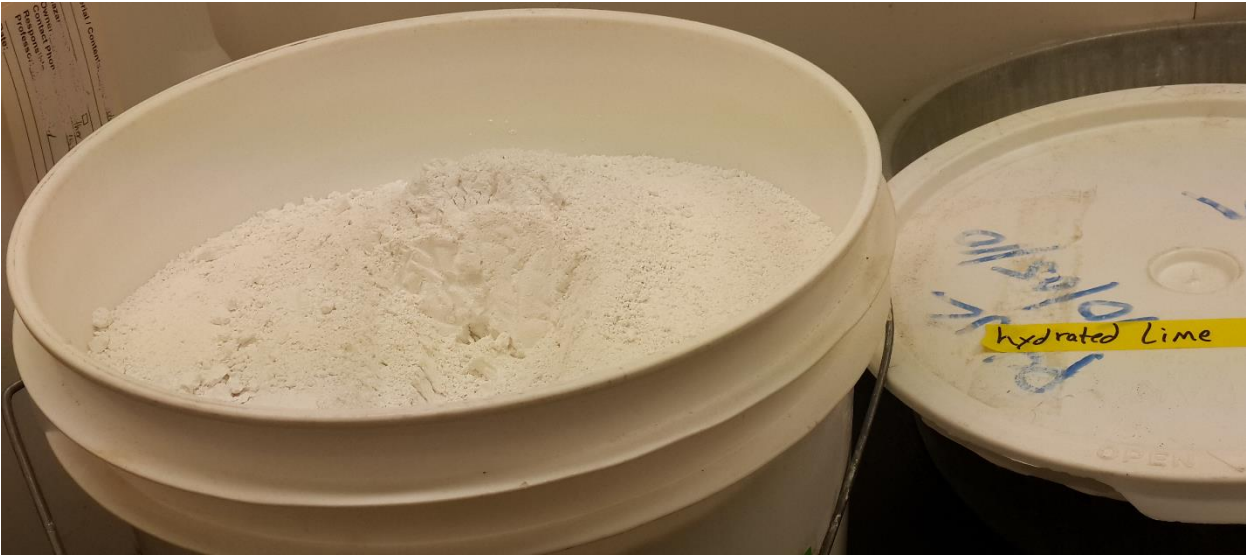


Figure G.23 Hydrated lime

#### G.4.5 Quicklime

##### G.4.5.1 Crushed Quicklime through Sieve #8

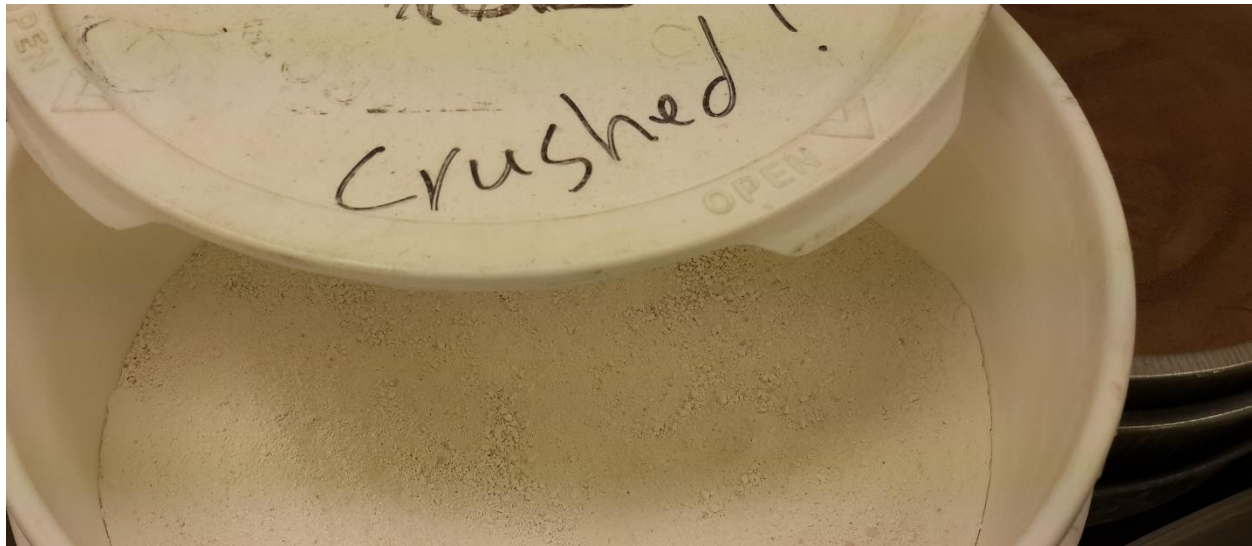


Figure G.24 Crushed quicklime through Sieve #8

##### G.4.5.2 Original Quicklime

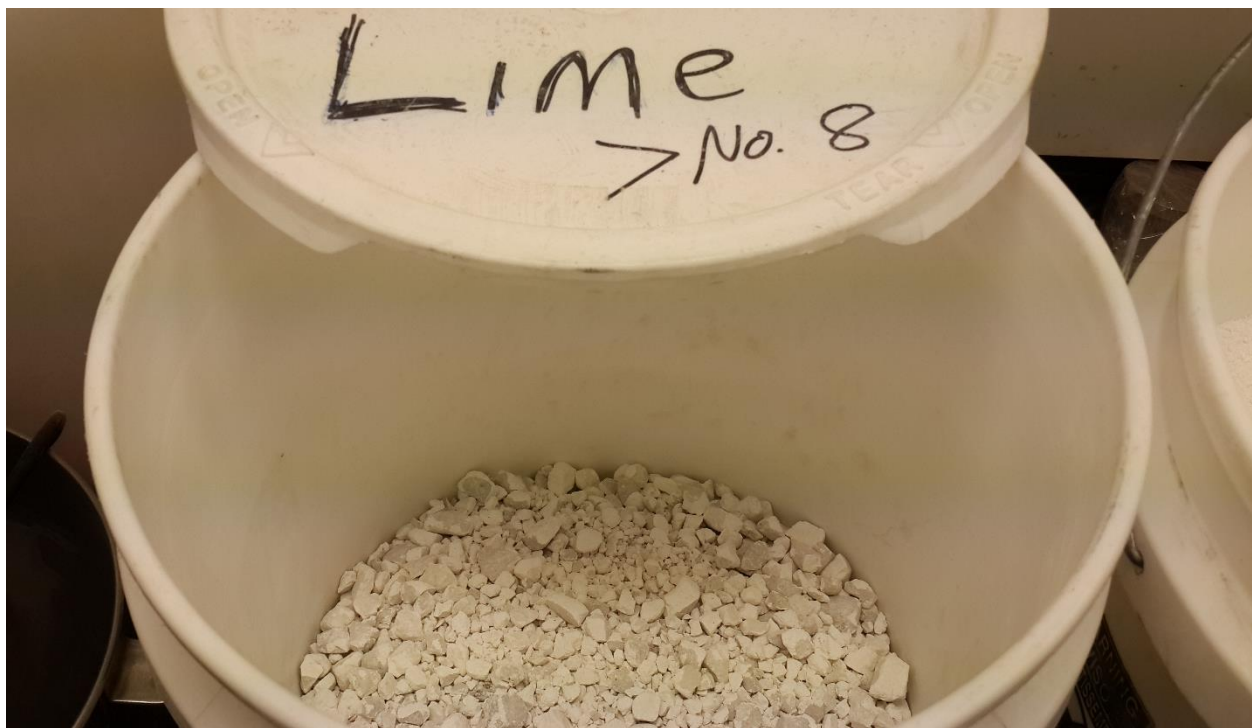


Figure G.25 Original quicklime

## APPENDIX H: POTENTIOMETER AND LOAD CELL CALIBRATIONS FOR UCS TEST

### H.1 Potentiometer Calibration

The potentiometer calibration used in this research based on the relation between the voltage resistance of the potentiometer used and the corresponding length. Meaning, the voltage measured from the potentiometer should be calibrated such that each voltage reading would give exact length of the potentiometer in unit length, for instance in millimeters. To do so, a linear relation between the voltage and the corresponding length measured with the device below should be recorded in a similar table presented below. This relation will have a slope, an intercept, and an  $R^2$  to check accuracy. This process can be done twice (forward and backward), and take the average to make the procedure more accurate.

Figure H.1 is a photo of the device used to measure precise length of the potentiometer. The potentiometer was connected to a medium (software) to calculate the voltage of the potentiometer.

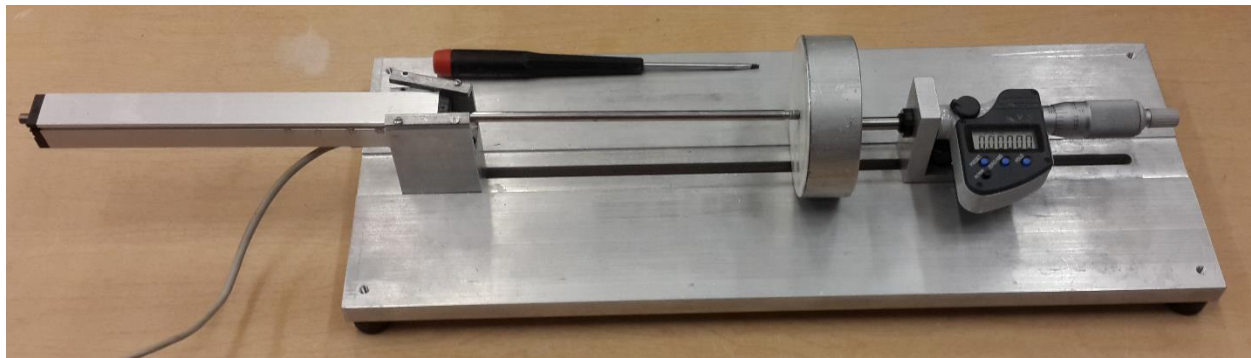


Figure H.1 The potentiometer-calibrating device

A computer software that is shown in Figure H.2 was used, with which the voltage of the potentiometer was calculated. Table H.1 reports the relationship between voltage and length for the potentiometer used in this study. The intercept and the slope gained from this relationship was used to convert voltage to length. Figure H.3 shows the potentiometer calibration graph for the data presented in Table H.1.

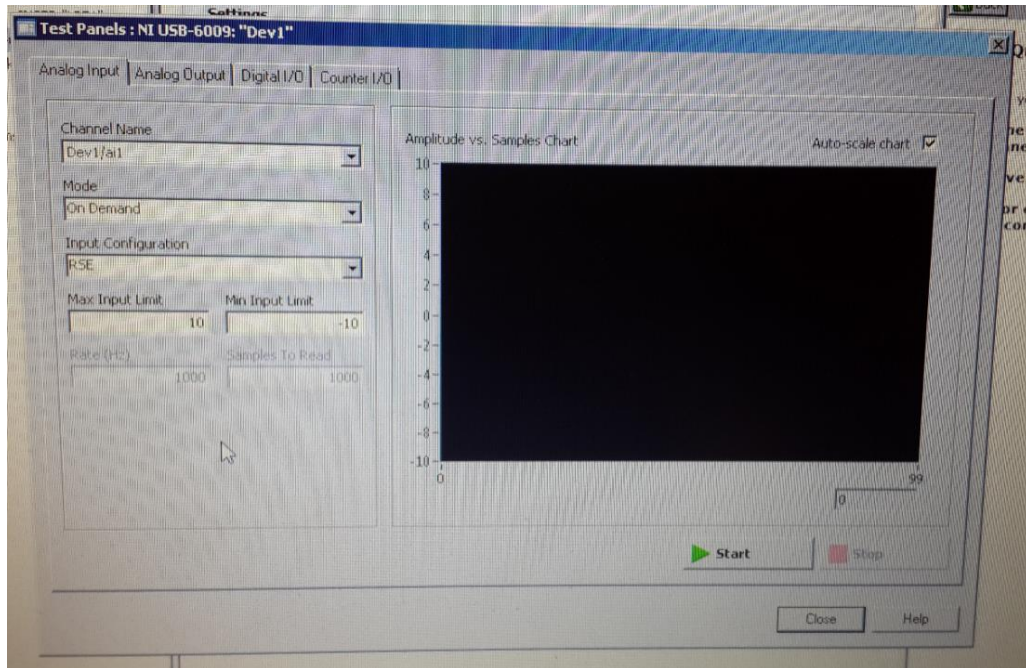


Figure H.2 Software that measured the potentiometer voltage

Table H.1 Conversion of the potentiometer voltage to length

The Potentiometer Length [mm]	The Potentiometer Voltage [Mv/V]
0	0.00214
2.924	0.287
6.003	0.679
9.124	1.08
12.533	1.51
15.169	1.89
18.787	2.35
24.056	3.01
31.057	3.9
37.674	4.76
44.787	5.68
53.844	6.84
62.819	8.03
69.541	8.91
75	9.66
<b>Average Slope</b>	<b>7.740</b>
<b>Average Intercept</b>	<b>0.656</b>
<b>R<sup>2</sup></b>	<b>1.000</b>

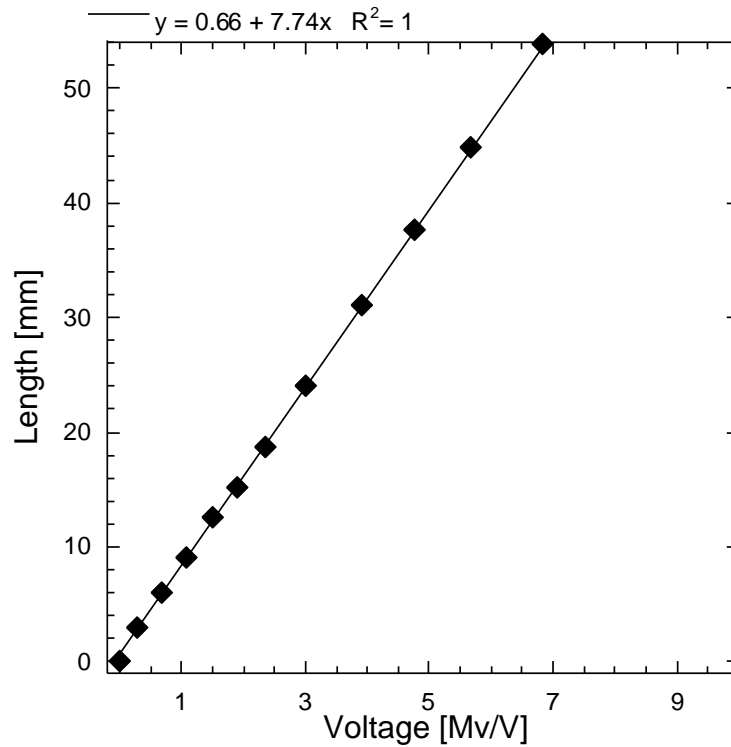


Figure H.3 Calibration and conversion of the potentiometer voltage to length

## H.2 Load Cell Calibration

To calibrate the load cell of the UCS machine used in this research, a proving ring calibrator was utilized with a capacity of 5000 lb (Fig. H.5.). Two conversion steps were required while doing the calibration. First, the divisions on the proving ring was required to be converted to pounds (weight) and a corresponding calibration factor was required. The calibration factor (17.25) for the proving ring used in this study, shown in Figure H.4, was provided by Dr. Christopher Bareither at CSU. The second conversion was from the load in pounds (that would be applied to the UCS machine's load cell) to the voltage of the load cell (that requires a software to be measured; the same software was used as for the potentiometer).

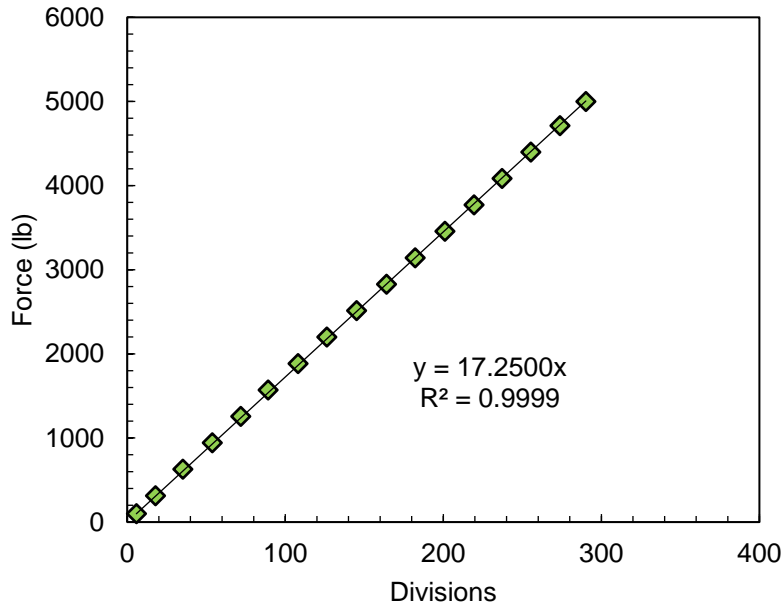


Figure H.4 Conversion between the proving ring divisions to force in pounds

The proving ring, shown in Figure H.5, was used to measure the load applied to the load cell by the UCS machine. The load from the proving ring and the voltage from the software were recorded. This recorded relationship is shown in Table H.2. The slope, intercept, and  $R^2$  should be obtained. The slope and the intercept were input into the UCS machine to convert voltages into loads in pounds.

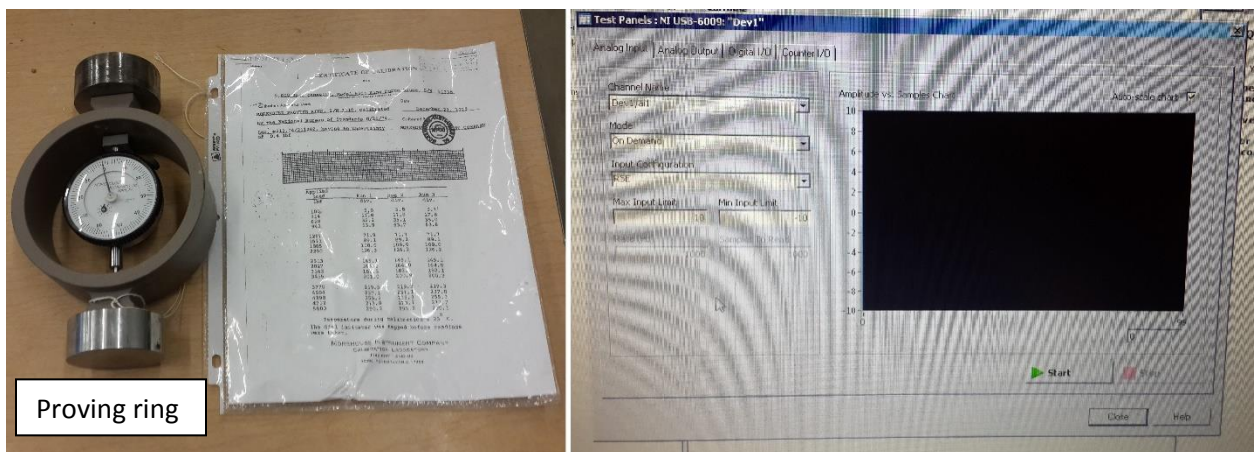


Figure H.5 The proving ring and the software that measure the proving ring voltage



Several trials of loading and unloading were performed while the proving ring was under the load cell to get accurate slope and intercept. Table H.2 shows the calibration process in a loading procedure. The average values of the slope and intercept are also presented in Table H.2.

Table H.2 Load Cell calibration data for the unconfined compression testing machine

Proving Ring - Test 1 (div)	Load - Test 1 (lb)	Voltage - Test 1 (mV/V)
0	0	0.0118
12	207.0	0.121
22	379.5	0.208
65	1121.3	0.556
110	1897.5	0.915
175	3018.8	1.41
243	4191.8	1.95
289.1	4987.0	2.31
Average Slope	<b>2195.0</b>	
Average Intercept	<b>-57.807</b>	

## APPENDIX I: FAILURE TO CONDUCT “EADES-GRIM” TEST FOR LIME DEMAND

ASTM D6276 also called “Eades-Grim” test is used to determine the minimum lime proportion required to stabilize a soil mixture based on pH value. ASTM D6276 reports the whole procedure for this test, and details about this test are provided in (National Lime Association 2006). This test is used to determine lowest percentage of lime that exhibits a pH value of 12.4 be selected as the percentage of lime for stabilization. The buffer solutions available in the geotechnical and geoenvironmental lab at CSU included pH buffers of 2.00, 4.01, 7.00, 10.01, 12.00, and 12.45 (Fig. I.1.). The pH-meter was capable for more than one-point calibration, i.e., more than one buffer solution could be used to calibrate the pH-meter. Buffer solutions of 12.45 and 10.01 were used as a two-point calibration, and a buffer solution of 12.00 was used as a one-point calibration. Despite using different buffer solutions to calibrate the pH-meter in the lab, no reasonable pH values were obtained as expected for the lime soil mixtures in the “Eades-Grim” test.



Figure I.1 Buffer solutions for pH-meter calibration

Unfortunately, even no reasonable pH for the lime-water solution (alone without soil) was obtained. The pH of lime in water should have been at least 12.4 but as shown in Figure I.2, the pH-meter measured a pH value of 11.73. Even with increasing lime in the lime water solution, ironically, the pH-meter measured lower pH values. Therefore, this test was not relied on. A practical 3% lime content was used in this study based on current practice and swell tests.



Figure I.2 Soil-lime mixtures in 25 g of water and the Ph-Meter device

## **APPENDIX J: CORRECTION OF SWELL PRESSURE AND SWELL POTENTIAL**

Fredlund (1969) mentions compressibility problems associated with the consolidometers and their accessories to cause change in the swell pressure. Fredlund (1969) states that corrections are required for the swell pressure when the following problems are encountered: i) friction in the consolidometer ring, especially for low pressures, and ii) compressibility of the consolidometer, accessories, and the porous stone used on top of the specimens. To minimize the need for swell pressure corrections, Fredlund (1969) suggested using the same accessories for repeated swell tests.

However, ASTM D4546-14 “Standard Test Methods for One-Dimensional Swell or Collapse of Soils” only mentions correction for oversize particles. Oversize particle corrections have been presented in this ASTM standard to substitute for the effect of oversize particles such as gravel, which the 2.5”-diameter odometer ring cannot contain. ASTM D4546 suggests using larger molds for the swell test when the soil has more than 40% of particles larger than 4.75 mm. In this research, all of the soils tested were clays, and thus no oversize corrections were required.

Regarding friction of the consolidometer, Section 7.1.1 of ASTM D4546-14 sets a number of conditions for the consolidometer ring that should be stiff, non-corrosive, and the consolidometer ring inner surface should be polished or coated to reduce the wall friction of the specimen. This is the only condition where this ASTM focuses on regarding friction. In this research, brass rings that fulfill the stiffness, non-corrosiveness, and low friction characteristics conditions of the ASTM D4546-14 were used. Therefore, no corrections for the swell potential and swell pressure values were considered in this research, as ASTM D4546-14 states none.

## APPENDIX K: POLYMER RATE AND DEPTH OF PENETRATION CALCULATIONS

A 5% P4 was selected based on a ½” penetration and an approximate polymer rate of 1.16 L/m<sup>2</sup> as suggested by the manufacturer for “heavy use.” The calculations for these assumptions are included below:

1. Assume a 160 g mass of soil that an odometer cell of 2.5” in diameter by 1” high can contain. This assumption is based on a unit weight of 19.5 KN/m<sup>3</sup>. The range of the unit weight of all of the treated and untreated soils in this research is from 19.0 – 19.5 KN/m<sup>3</sup>.

$$\text{-The volume of the cell} = \frac{2.5^2 * \pi}{4} * 1 * \frac{(25.4)^3}{(1000)^3} = 0.000080440 \text{ m}^3$$

$$\text{-Assumed unit mass} = \frac{19.5 * 1000 * 1000}{9.81} = 1987767.584 \text{ g/m}^3$$

$$\text{-The average mass of soil in the cell} = 0.00008044 \text{ m}^3 * 1987767.584 \text{ g/m}^3 = 159.89 \text{ g or approximately 160 g.}$$

2. Assume a polymer application rate of 1.16 L/m<sup>2</sup> as mentioned before.

$$3. \text{ The area of the odometer cell} = \frac{2.5^2 * \pi}{4} * \frac{(25.4)^2}{(1000)^2} = 0.00316692 \text{ m}^2$$

4. Assume the polymer can penetrate the whole height of the cell which is 1”.

$$5. \text{ Then the amount of the polymer needed} = \text{the application rate} * \text{the odometer area} \\ = 1.16 * 0.00316692 = 0.003673629 \text{ L}$$

6. The density of the polymer as measured in the lab is 1.02 g/mL. This makes the polymer amount to be  $\frac{0.003673629}{1000} * 1.02 = 3.747101808 \text{ g}$ .

7. Then, the ratio of the polymer, by weight of the soil, based on a 1” penetration and an approximate polymer rate of 1.16 L/m<sup>2</sup> is  $\frac{3.747101808}{160} * 100 = 2.34\%$

8. Finally, the ratio of the polymer, by weight of the soil, based on a ½” penetration and an approximate polymer rate of 1.16 L/m<sup>2</sup> is  $2 * 2.34\% = 4.68\% \approx 5\%$ . The ½”

penetration was assumed because the soil tested was bentonite-rich clay that swells when hydrated.

According to several testimonials from different agencies shown on the P2, P3, and P4 website, use of P2, P3, and P4 as dust controller and soil stabilizer is recommended. In this study, P4 substantially reduced swell potential as compared to P2 and P3. The effectiveness of P4 found in this study matches the testimonials. A county department, in Arizona, states in one of the testimonials, that by applying one of the P2, P3, and P4 products at a rate of 0.25 gallon/yd<sup>2</sup> (1.13 L/m<sup>2</sup>), they could stabilize a 2" road base. This rate and penetration depth gives a polymer addition ratio of 2.22% which is less than half of the polymer ratio used in this study. Most of the testimonials on the manufacturer's website appreciate P2, P3, and P4 for their dust controlling effect.

## APPENDIX L: HISTORY OF RESEARCH ON EXPANSIVE SOILS

Table L.1 A brief history of identification, study and research on expansive soils

<b>Year</b>	<b>Development in expansive soils knowledge</b>
1955	(Altmeyer 1955) discussed engineering properties of expansive clays
1956	(Holtz and Gibbs 1956) also introduced expansive soils
1960	(Lambe 1960) developed a method to identify swelling clays
1961	(Jennings 1961) studied heave in desiccated soils
1962	(Seed et al. 1962a) discussed swelling characteristics of compacted clays
1962	(Seed et al. 1962b) worked prediction of swelling in compacted clays
1964	The Colorado Department of Highways and the University of Colorado (1964) published a review of the literature on swelling soils.
1968	Building Research Advisory Board (BRAB 1968) published the first widely accepted design criteria for lightly loaded foundations on expansive clays, and this is still a reliable reference.
1970	In the 1970s, researchers improved the ways to better define water suction correlated with expansive soils, and established methods to measure suction.
1976	(Mitchell 1976) provided a comprehensive text on soil behavior including a significant number of treatments for expansive soils.
1960-1992	Seven international conferences on expansive soils took place.
1990	(McKeen and Johnson 1990) studied the active zone of soils.
1994	(Lytton 1994) described simple laboratory tests to identify expansive soils. These tests included the compression indices due to matrix suction and mean principal stress, the slope of suction versus water content curve, and unsaturated permeability and diffusivity.
1997	(Lytton 1997) described constitutive equations for volume change that can be used in design, and established methods to define the depth of the active zone.
2000 -now	Numerous papers published; research is being done on expansive soils.

## **APPENDIX M: MECHANISM OF SWELLING OF EXPANSIVE SOILS**

### **M.1 Causes of Volume Change**

The two main factors that trigger swelling of soils are unloading and addition of water, both of which reduce the soil's effective stress. (Mitchel & Soga 2005) listed the following factors to cause volume change in soils.

- Physical interaction between particles
- Pore Water Chemistry
- Physiochemical interaction between particles: this is important for swelling behavior of clays as swelling includes inter-particle attractive forces.
- Chemical and organic reactions: these reactions role in cementing or reducing surface forces that influence plasticity and compressibility.
- Mineralogical effects: the mineralogical characteristics of expansive clays can have large-scale effects on motivating swelling.
- Fabric and structure: The fabric of a soil greatly influences the amount of swelling of expansive soils. For example, soils with flocculated structures tend to swell more than those soils having dispersed structures as explained by (Seed et al 1962a).
- Stress History: With the same void ratio, an overconsolidated soil tends to swell more than the same soil normally consolidated.
- Temperature: The effective stress of a soil increases when temperature increases. These increases cause soils to have lower tendency to swell.
- Stress path: periodical loading and unloading could change the swelling and compression behaviors of soils.

### **M.2 Ion Distribution within Clay Particles and Double Layer Theories**

(Bohn et al 1979) listed the following two theories related to arrangement of ions around negatively charged clay particles and in the whole clay-water system:



1. Gouy-Chapman Double Layer Theory: assumes that there is a diffuse concentration of ions that changes with a smooth gradient from high at the surface of the clay mineral to low far from the mineral surface (Jury & Horton 2004) (Fig. 2.1).

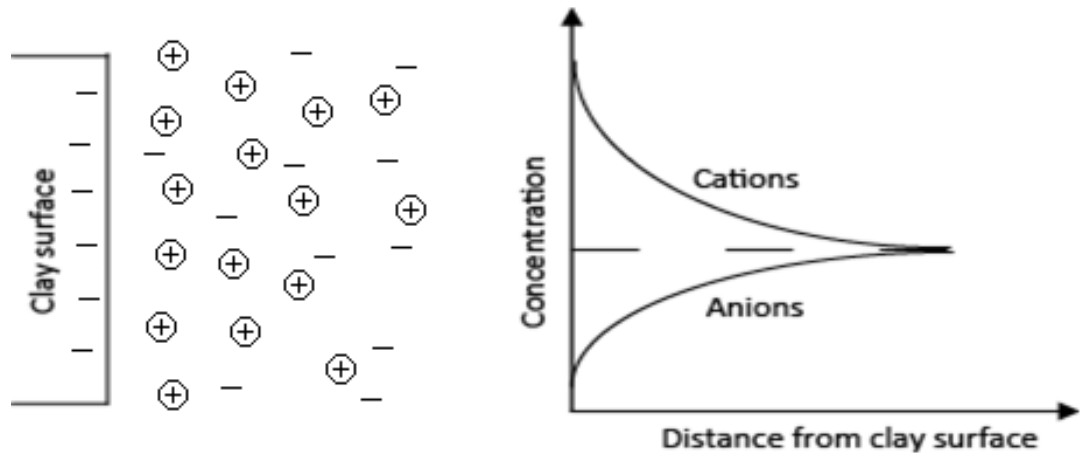


Figure M.1 Gouy-Chapman Double Layer theory

2. Stern Double Layer Theory: This theory assumes that there is a rigid layer of ions surrounding the clay mineral that is loosely connected to a diffuse layer within the clay-water system (Fig. 2.2).

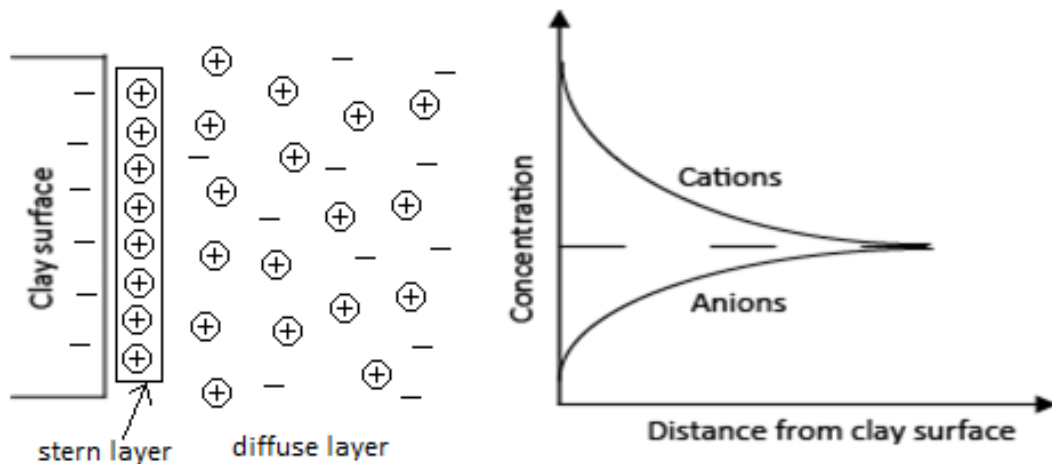


Figure M.2 Stern Double Layer theory

### **M.3 Cation Exchange Capacity**

In both of the double layer theories mentioned above, cations available in both of the clay surface and the diffuse layer must neutralize anions on the surface and anions in the diffuse layer. Cations that role to neutralize anions within the mineral surface are exchangeable with cations that role to neutralize anions in the diffuse layer. This process is called cation exchange (Jury & Horton 2004). The common cations that are usually exchanged in a soil are calcium ( $\text{Ca}^{2+}$ ), magnesium ( $\text{Mg}^{2+}$ ), sodium ( $\text{Na}^+$ ), and potassium ( $\text{K}^+$ ).

Cation exchange capacity (CEC) of clay minerals is the number of charges that the mineral can exchange range from 1-150 meq/100 g. The more the CEC of a mineral, the more the tendency of the mineral to cause the soil to swell. Montmorillonite, that is commercially called bentonite, has a CEC of 80-150 meq/100 g, Illites has a CEC of 10-40 meq/100 g. Kaolinite has a CEC of 3-15 meq/100 g (Mitchel & Soga 2005).

### **M.4 Clay Mineral Flocculation**

Clay particle surface charges are responsible for flocculation and swelling of soils through attraction and repulsion by the other particles. Flocculation happens when cations in the double layer join into one positive layer that is ready to attract the negatively charged mineral surfaces of the clay platelets. This joining process continues until numerous clay platelets stack together to form layers that lead to flocculation and stability of the soils. Divalent exchangeable cations are the main type of cations that motivate flocculation because of the ability of these divalent cations to increase attraction between the particles (Jury & Horton 2004). These divalent cations include  $\text{Ca}^{2+}$  and  $\text{Mg}^{2+}$ .

When two clay particles approach each other, the first interaction between them would be repulsion. This because both of the two clay particles have high concentration of cations, that are positively charged, by their surfaces. This lead to repulsion occurs in low electrolyte

concentrations where attraction energy is low. However, when the clay particles are further compressed towards each other, attraction energy increases and the concentration of the electrolytes becomes higher. This leads to reduction in the repulsive forces gradually, and the attraction energy becomes dominant till flocculation occurs (Sparks 1999).

### **M.5 Osmotic Swelling Theory**

Referring to the scenario above, dispersion occurs when the clay mineral is richly surrounded by monovalent exchangeable cations, such as Sodium  $\text{Na}^+$ , that do not have the capability to join the clay platelets together. This dispersion leads to swelling and a weak structure of the soil (Jury & Horton 2004). Therefore, the repulsive forces that lead to swelling are more dominant than the attraction energy in clay-water systems rich in monovalent cations such as sodium.

The swelling that occurs because of this dominance of the repulsive forces within the double layer at this Nano-scale (less than 1 nm) is referred to Osmotic swelling. Moreover, the pressure needed to prevent this osmotic swelling could be correlated to the clay's directly measured Swell Pressure when the repulsive forces are the dominant factors of swelling (Sparks 1999). In the general osmotic pressure theory, there should exist a semi-permeable membrane to cause osmosis happen from a low solvent concentration solution to a high solvent concentration.

In soils, this separating membrane does not exist between high and low salt-concentration. However, the high concentration of cations surrounding each clay particle is strictly attached to the particle's surface and take the role of that semi-impermeable membrane that prevents diffusion of the cations. When two clay particles get close to each other, the cations (that are strictly adsorbed by the surface of each particle) cause the double layers between the particles to have high ion concentrations relative to the space outside the contact area of the particles (bulk

solution). Therefore, water will be drawn to into this high ion-concentration to balance the water free energy, which leads to repulsion and finally swelling of the soil. The pressure needed to stop this swelling is called Osmotic pressure or, if externally applied; the Swell Pressure (Mitchel & Soga 2005).

### **M.6 Surface Hydration Theory of Swelling**

This is an alternative theory of osmotic pressure also called Water Adsorption Theory (Low 1992). This theory states that when water is added into a clay paste, the reaction of water with the clay particle surfaces changes the chemical potential of the clay-water system that can hold and absorb more water. This theory does not directly consider the electrolyte concentration, surface charge density, and cation valence, which have great influence on overall swelling. Instead, there are empirical features in the Water Adsorption Theory that nearly substitute these factors (Mitchel & Soga 2005).