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In-situ Evaluation of Design Parameters and Procedures for Cementitiously Treated Weak Subgrades using Cyclic Plate Load Tests

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13. Abstract
This study aimed at evaluating the performance-related properties (e.g., resilient modulus and permanent deformation) of cementitious treated/stabilized subgrade soils under repeated loading test conditions. For this purpose, both laboratory and field testing programs were performed on treated/stabilized specimens/test sections. The laboratory testing program comprised of two phases: Phase I focused on evaluating the resilient modulus (M_R) and permanent deformation (PD) of the current subgrade treatment/stabilization schemes for hauled soils using cement and/or lime stabilizer as provided in DOTD's standard specifications. Three soil types with different plasticity indices (PI) were used in Phase I. The results of Phase I showed that the M_R for treated/stabilized soil specimens ranged from 25 ksi to 36 ksi, and that the values of PD are negligible. Phase II focused on examining the proper treatment/stabilization recipe (lime, fly ash, and cement) for very weak subgrade soils at high moisture contents, and evaluating the corresponding M_R and PD. Four soil types with different

PIs were considered in Phase II that were prepared at three different moisture contents correspond to a raw unconfined compressive strength (UCS) of 25 psi or less. All soil specimens in Phase II were treated with different combinations of class C fly ash (or Portland cement type I) and hydrated lime to achieve a 7-day target UCS values of 50 psi to create a working platform and 100 psi for subbase application. Repeated load triaxial tests (RLT) tests were performed in the laboratory testing program to evaluate the M_R and PD of the laboratory treated/stabilized specimens. The results of Phase II demonstrated good correlation between the water/additive ratio and the M_R/PD, such that the soil specimens prepared at low water/additive ratio showed better performance than those prepared at high water/additive ratio. The M_R for treated/stabilized very weak soil specimens at 50 psi UCS for working platform ranged from 5.5 ksi to 11 ksi. However, for the 100 psi UCS (subbase application), the M_R for treated/stabilized very weak soil specimens ranged from 9.5 ksi to 14.5 ksi. The results of laboratory tests showed that there is no correlation between M_R and UCS for the treated/stabilized soils as well as no trend between M_R/PD and PI. The field testing program involved constructing and performing cyclic plate load tests (CPLT) on 10 test sections at the Accelerated Load facility (ALF) site in which six sections were selected based on the results of Phase II, while the other four sections were selected from a previous study on micro cracks of cement stabilized soils. The CPLTs were performed to measure the in-situ composite resilient modulus ($M_{R\text{-comp}}$) of the ten test sections at ALF site. The measured M_R of the different treated/stabilized layers were back-calculated based on the assumption that ratio $(E1/E2)_{DCP}$ equal to the ratio $(E1/E2)_{CPLT}$ for the two-layer system. The results of field tests showed that the back-calculated M_R values for the treated/stabilized test sections are in good agreement with the laboratory-measured M_R values for Phase II.

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This study aimed at evaluating the performance-related properties (e.g., resilient modulus and permanent deformation) of cementitious treated/stabilized subgrade soils under repeated loading test conditions. For this purpose, both laboratory and field testing programs were performed on treated/stabilized specimens/test sections. The laboratory testing program comprised of two phases: Phase I focused on evaluating the resilient modulus (M_R) and permanent deformation (PD) of the current subgrade treatment/stabilization schemes for hauled soils using cement and/or lime stabilizer as provided in DOTD's standard specifications. Three soil types with different plasticity indices (PI) were used in Phase I. The results of Phase I showed that the M_R for treated/stabilized soil specimens ranged from 25 ksi to 36 ksi, and that the values of PD are negligible. Phase II focused on examining the proper treatment/stabilization recipe (lime, fly ash, and cement) for very weak subgrade soils at high moisture contents, and evaluating the corresponding M_R and PD. Four soil types with different PIs were considered in Phase II that were prepared at three different moisture contents correspond to a raw unconfined compressive strength (UCS) of 25 psi or less. All soil specimens in Phase II were treated with different combinations of class C fly ash (or Portland cement type I) and hydrated lime to achieve a 7-day target UCS values of 50 psi to create a working platform and 100 psi for subbase application. Repeated load triaxial tests (RLT) tests were performed in the laboratory testing program to evaluate M_R and PD of the laboratory treated/stabilized specimens. The results of Phase II demonstrated good correlation between the water/additive ratio and the M_R/PD, such that the soil specimens prepared at low water/additive ratio showed better performance than those prepared at high water/additive ratio. The M_R for treated/stabilized very weak soil specimens at 50 psi UCS for working platform ranged from 5.5 ksi to 11 ksi. However, for the 100 psi UCS (subbase application), the M_R for treated/stabilized very weak soil specimens ranged from 9.5 ksi to 14.5 ksi. The results of laboratory tests showed that there is no correlation between M_R and UCS for the treated/stabilized soils as well as no trend between M_R/PD and PI. The field testing program involved constructing and performing cyclic plate load tests (CPLT) on 10 test sections at the Accelerated Load facility (ALF) site in which six sections were selected based on the results of Phase II, while the other four sections were selected from a previous study on micro cracks of cement stabilized soils. The CPLTs were performed to measure the in-situ composite resilient modulus ($M_{R\text{-}comp}$) of the ten test sections at ALF site. The measured M_R of the different treated/stabilized layers were back-calculated based on the assumption that ratio $(E_1/E_2)_{DCP}$ equal to the ratio

$(E1/E2)_{CPLT}$ for the two-layer system. The results of field tests showed that the back-calculated M_R values for the treated/stabilized test sections are in good agreement with the laboratory measured M_R values for Phase II.

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Implementation Statement

The results of this study demonstrated the potential benefits of improving the performance-related properties of treating/stabilizing subgrade layer with lime, cement, and/or fly ash, especially for flexible pavements built over weak subgrade soils, in terms of increasing the composite resilient modulus ($M_{R\text{-comp}}$) and decreasing the permanent deformation of the pavement section under repeated loading conditions. These benefits can be realized in the design of pavement structures through giving a credit to the treated/stabilized subgrade soil layer by incorporating the $M_{R\text{-comp}}$ as input design parameter, which represents the two-layer system of the treated/stabilized subgrade layer and the underneath untreated subgrade within the influence loading depth.

The findings of this research study can be implemented by DOTD pavement engineers in the design of flexible pavement structures built over weak subgrade soils using the following steps.

For treating/stabilizing hauled soils with cement and/or lime according to DOTD standard specifications (Table 8):

a) Evaluate the resilient modulus (M_R) for the treated/stabilized hauled soil layer using the MEPDG M_R model in equation (23) and the corresponding model constants (k_1, k_2, and k_3) presented in Table 20, or directly select M_R directly from Table 22;
b) Calculate the composite resilient modulus ($M_{R\text{-comp}}$) for the two-layer system, the treated/stabilized layer and the underneath untreated subgrade soil, based on thickness of treated/stabilized layer, influence depth and using either equations (29) or (30);
c) The M_R for the untreated subgrade soil can be evaluated using the dynamic cone penetration test data (i.e., DCP index); and
d) Use $M_{R\text{-comp}}$ as an input design parameter for the two-layer subgrade system in the design of flexible pavements using either the MEPDG or the 1993 AASHTO Pavement design.

For treating/stabilizing very weak and wet in-situ soil:

a) Select the proper additive combination (lime, fly ash, and cement) and the corresponding contents by volume as presented in Table 24 based on the soil's PI value and the in-situ moisture content;
b) Evaluate the resilient modulus (M_R) for the treated/stabilized weak soil layer using Table 27 for working platform (50 psi UCS) and Table 28 for subbase application (100 psi UCS);
c) Calculate the composite resilient modulus ($M_{R\text{-}comp}$) for the two-layer subgrade system, the treated/stabilized layer and the underneath untreated subgrade soil, based on thickness of treated/stabilized layer, influence depth and using either equations (29) or (30); and
d) Use $M_{R\text{-}comp}$ as an input design parameter for the two-layer subgrade system in the design of flexible pavements using either the MEPDG or the 1993 AASHTO Pavement design.

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Introduction

Due to the soft nature of subsurface soil condition in southern Louisiana, roads often have to be constructed on very weak subgrade soils with high in-situ moisture contents that do not have the sufficient strength/stiffness to support the construction/traffic loads, which poses a significant challenge for the construction of highway projects. To solve this challenge, cementitious additives such as cement, lime, fly ash, cement kiln dust (CKD), and slag, alone or in combination, can be used to treat/stabilize these soils to improve their engineering properties such as strength and stiffness, thus providing the needed support for pavement construction. The long-term performance of treated/stabilized soils is influenced by the characteristics of the parent soil, type, and quantity of stabilization additive, construction practices, frequency and magnitude of loading, and environmental conditions.

Depending upon the objective, cementitious material can be used to treat soils to either provide a working platform for the construction of the pavement shortly after mixing, or to enhance the strength and stiffness of soils in the long-term for subbase application. The former is referred to as soil modification/treatment, while the latter is referred to as soil stabilization.

The common practice in Louisiana is to treat/stabilize in-situ weak subgrade soils or hauled soils with cement, lime, or a combination, depending on the soil's plasticity index (PI), thus enhancing their engineering strength/stiffness properties. While the Louisiana Department of Transportation and Development's (DOTD) engineers provide recommendations on the selection of stabilizer type and content for treating hauled soils (controlled layer), the selection of stabilizer type and content for treating in-situ weak subgrade layer (under in-situ moisture and density conditions) is based on achieving a specific compressive strength (50 psi for working platform and 100 psi for subbase application).

Many studies have shown that well-engineered and constructed cementitiously treated/stabilized subgrade soils can provide strong, stiff, and durable support to pavement structures, and hence enhancing the short- and long-term performance of pavements under traffic loads. However, no credit is given to the treated/stabilized subgrade soils in the pavement design process.

To give a reliable credit and include the structural contribution of the treated/stabilized subgrade soils, one of the major considerations in the design of flexible pavements, requires proper evaluating the performance of treated/stabilized soil layer in terms of resilient modulus (M_R) and permanent deformations. The composite resilient modulus ($M_{R\text{-comp}}$) representing the treated/stabilized subgrade soil layer and the underneath untreated soil need to be evaluated depending on layers' thicknesses within the influence depth, which will be used as the input in the pavement design.

This study aimed at evaluating the resilient modulus (M_R) of treated/stabilized subgrade soils of different plasticity, different moisture contents and different additive type/doses, for both hauled (controlled layer) subgrade soil and in-situ subgrade soil. It is also aimed at providing guidance on how to calculate and incorporate $M_{R\text{-comp}}$ that represent the treated and untreated subgrade soil layers in the pavement design process for use in both the Mechanistic-Empirical Pavement Design Guide (MEPDG) and the 1993 AASHTO (American Association of State Highway and Transportation Officials) Pavement Design Guide. The study consists of two phases. Phase I: evaluate the current subgrade treatment/stabilization schemes for hauled soil as provided in DOTD specifications and Phase II: examine the appropriate treatment schemes for very weak in-situ subgrade soils at high water content.

Both the laboratory and field-testing programs were performed in this study to evaluate the performance of cementitious treated/stabilized subgrade soils in terms of M_R and PD characteristics. For Phase I, only laboratory tests were performed on three selected soil types with different PIs (low PI, medium PI, and high PI) that were stabilized based on the PI values according to DOTD specifications (i.e., cement and/or lime). However, Phase II program included both laboratory and field-testing programs, in which four soil types with different plasticity values (low PI, medium PI, high PI, and heavy clay) were considered. Phase II testing focused on examining the appropriate treatment/stabilization schemes for very weak subgrade soils at high water content conditions (i.e., at wet side of optimum of nearly saturated condition) with unconfined compressive strength (UCS) < 25 psi. Several cementitious additives, including cement, lime, lime-cement, or lime-fly ash, were considered in Phase II to determine the proper dose for use as working platform (minimum 7-day strength of 50 psi) and subbase layer (minimum 7-day strength of 100 psi) applications.

The field testing program involved constructing and conducting cyclic plate load tests (CPLT) on ten test sections at the ALF site, in which six sections were selected based on the results of Phase II; while the other four sections were selected from a previous study on micro cracks of cement stabilized soils. The CPLT tests were performed to evaluate the composite resilient modulus ($M_{R\text{-}comp}$) and the PD characteristics of the treated/stabilized sections at the ALF site. The measured M_R values of different test sections were back-calculated based on the assumption that the ratio of $(E1/E2)_{DCP}$ for the two-layers system obtained from the DCP data is the same as the ratio of $(E1/E2)_{CPLT}$ obtained from the CPLT test. The back-calculated M_R values for the different ALF test sections were compared with the laboratory-measured M_R values for Phase II.

Literature Review

Background

Subgrade is the lowest supporting layer in the pavement structure underlying the base layer. Generally, the subgrade consists of locally-available soil deposits that sometimes can be very soft and/or very wet, therefore lacking the strength/stiffness to support the pavement's traffic loading. The replacement of such soil with better quality of borrow soil fill (cut and fill) is not always a good option, especially in pavement construction, due to the associated extra cost of excavation and hauling of the materials. The use of cementitious materials to treat/stabilize the soft subgrade is a widely accepted practice by many state highway agencies. Portland cement, lime, and fly ash are the most common types of cementitious materials used to treat/stabilize soft subgrade soils. A well-engineered and constructed cementitiously treated/stabilized subgrade layer usually requires achieving a threshold compressive strength that is capable of providing a strong and durable support to construction loading and the pavement structures. This treated/stabilized subgrade layer can be incorporated into structural design of pavements through increasing the resilient modulus of the composite subgrade layer or by considering it as a separate subbase layer.

Mechanism of Stabilization

Before the selection of the specific stabilizer, it is necessary to understand the behavior and mechanism of the stabilizer interaction with the soil. The soil stabilization mechanism, depending on the types of stabilizer, may vary from the formation of new compounds binding the finer soil particles to coating particle surface by the stabilizer to limit the moisture sensitivity [1]. The overall stabilization/treatment process in the presence of water can be summarized into four different processes: cation exchange, flocculation and agglomeration, cementitious hydration, and pozzolanic reaction [2], [3]. Portland cement and lime are both calcium-based products; however, their differences may include important properties such as strength, time dependency on strength development, curing, and durability and performance of the treatment [2]. All four aforementioned processes will occur in cement treated/stabilized soils, whereas in case of lime treated/stabilized soils cementitious hydration will be absent.

For soil-lime mixtures, cation exchange and flocculation-agglomeration are the primary reactions, which take place immediately after mixing. During these reactions, the divalent calcium ions supplied by the lime replace the monovalent cations that are generally associated with clay mineral. These reactions bring about immediate change in textural, plasticity, and workability because the exchange of cations causes a reduction in the size of the diffused double water layer, thereby allowing clay particles to clump together into large-sized aggregates. The pozzolanic reaction process is a long and slow process. It occurs between lime and the silica and alumina of the clay mineral and produces cementitious materials such as calcium-silicate-hydrates and calcium-alumina-hydrates. Studies have shown that when the pH of the soil increases to 12.4, which is the pH of saturated lime water, the solubility of silica and alumina increase significantly [4]. Therefore, as long as enough calcium from the lime remains in the mixture and the pH remains at least 12.4, the pozzolanic reaction will continue to occur. The basic pozzolanic reactions are described in the following equations:

$$Ca(OH)_2 + SiO_2 \rightarrow CaO \cdot SiO_2 \cdot H_2O \text{ (C-S-H)} \quad [1]$$

$$Ca(OH)_2 + Al_2O_3 \rightarrow CaO \cdot Al_2O_3 \cdot H_2O \text{ (C-A-H)} \quad [2]$$

Portland cement is comprised of calcium-silicates and calcium-aluminates that hydrate to produce cementitious materials, which bind the soil particles together. For soil-cement mixtures, the hydration of cement is the most important contributor to the improvement of engineering properties of soil [5]. Cement hydration is relatively fast and causes immediate strength gain in soil. The hydration behavior of calcium-silicates in cement can be described by the following equations; while the hydration of calcium-aluminates is somewhat more complex:

$$2\ Ca_3SiO_5 + 7\ H_2O \rightarrow 3\ CaO \cdot 2SiO_2 \cdot 4H_2O \text{ (C-S-H)} + 3\ Ca(OH)_2 \quad [3]$$

$$2\ Ca_2SiO_4 + 5\ H_2O \rightarrow 3\ CaO \cdot 2SiO_2 \cdot 4H_2O \text{ (C-S-H)} + Ca(OH)_2 \quad [4]$$

Much of the tricalcium silicate (Ca_3SiO_5) hydration occurs during the first few days, leading to substantial strength gains. The dicalcium silicate (Ca_2SiO_4) hydration contributes little to the early strength of cement soil but makes substantial contributions to the strength of mature cement paste. Similar to soil-lime mixtures, the cation exchange and flocculation-agglomeration also take place immediately after mixing of the soil and cement, resulting in reduction in soil plasticity. The lime generated during hydration of the cement helps increasing the binding between the soil particles through the pozzolanic reactions.

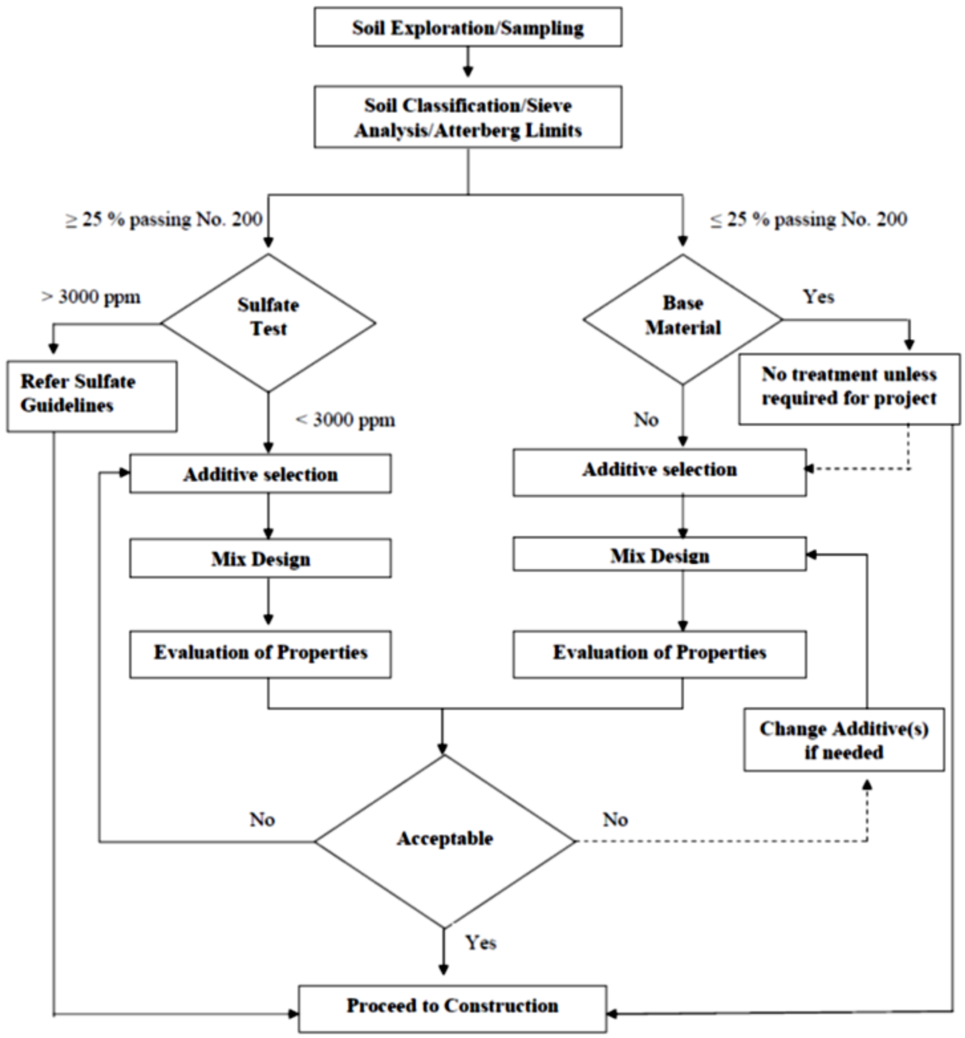
Fly ash stabilization of the soil is similar to cement; however, the strength provided is less than the cement. Depending upon the reactivity, the fly ashes are classified as self-cementing (C-class) and/or non-self-cementing (F-class). Generally, F-class fly ash is applied with either cement or lime; whereas, C-class does not include any activating stabilizers.

Mixture Design

Constructing of asphalt concrete pavements on weak subgrade soils has been a challenge for engineers for decades. Frequently, cementitious additives such as cement, lime, fly ash, cement kiln dust (CKD) and asphalt, alone or in combination, are used to treat these soils to improve their engineering properties. The long-term performance of stabilized soils is influenced by the characteristics of the parent soil, type, and quantity of stabilization additive, construction practices, frequency and magnitude of loading, and placement environment. The appropriate design of mixture is crucial for the successful applications of treated/stabilized subgrade soils.

A generalized procedure, which allows for evaluating the effectiveness of a stabilizer, is recommend by Little and Nair [1] in their draft of AASHTO "Standard Recommend Practice for Stabilization of Subgrade Soils and Base Materials," which is presented in Figure 1.

Figure 1. Guideline for stabilization of subgrade soil and base materials [1]



Selection of Stabilizers. In the selection of stabilizers, the type of soil to be stabilized must be considered. Most criteria for the preliminary selection of soil stabilizers are based on basic soil index properties such as plasticity index (PI) and soil gradation characteristics. This methodology was developed by Solanki et al. [6]. The unified facilities criteria (UFC) [7] provided a guideline for preselection of additives for soil stabilization in pavement. In this method, the soil is classified into different areas based on soil grain size characteristics, as shown in Figure 2. The candidate stabilizers for each area of soil in Figure 2 are then selected based on Table 1, giving special attention to restrictions based on liquid limit (LL), PI, and percent passing No. 200 sieve.

Figure 2. Gradation triangle [7]

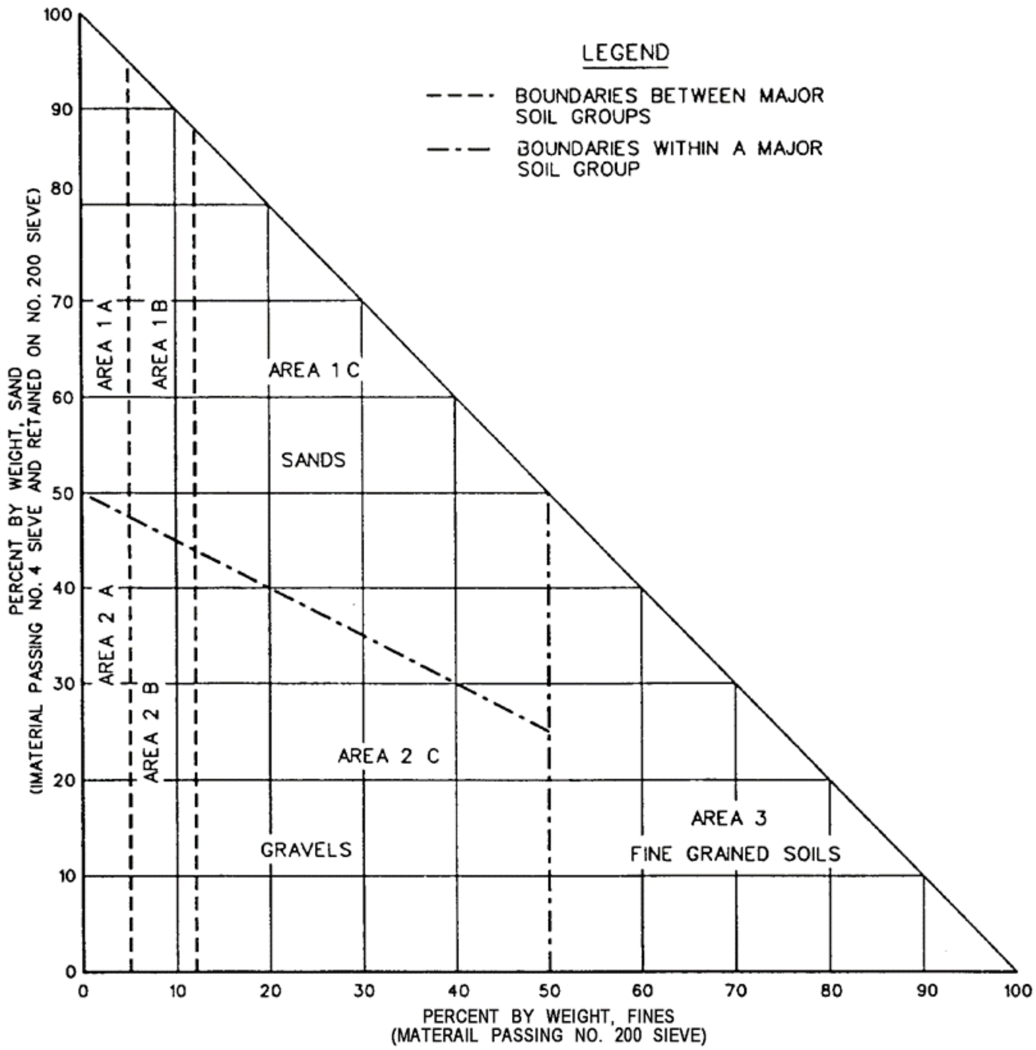


Table 1. Guide for stabilizer selection [7]

Area	Soil Class.[a]	Type of Stabilizing Additive Recommended	Restriction on LL and PI of Soil	Restriction on Percent Passing No. 200 Sieve[a]	Remarks
1A	SW or SP	(1) Bituminous			
		(2) Portland cement			
		(3) Lime-cement-fly ash	PI not to exceed 25		
1B	SW-SM or SP-SM or SW-SC or SP-SC	(1) Bituminous	PI not to exceed 10		
		(2) Portland cement	PI not to exceed 30		
		(3) Lime	PI not to exceed 12		
		(4) Lime-cement-fly ash	PI not to exceed 25		
1C	SM or SC or SM-SC	(1) Bituminous	PI not to exceed 10	Not to exceed 30% by weight	
		(2) Portland cement	--[b]		
		(3) Lime	PI not less than 12		
		(4) Lime-cement-fly ash	PI not to exceed 25		
2A	GW or GP	(1) Bituminous			Well-graded material only
		(2) Portland cement			Material should contain at least 45% by weight of material passing No. 4 sieve
		(3) Lime-cement-fly ash	PI not to exceed 25		
2B	GW-GM or GP-GM or GW-GC or GP-GC	(1) Bituminous	PI not to exceed 10		Well-graded material only
		(2) Portland cement	PI not to exceed 30		Material should contain at least 45% by weight of material passing No. 4 sieve
		(3) Lime	PI not less than 12		
		(4) Lime-cement-fly ash	PI not to exceed 25		
2C	GM or GC or GM-GC	(1) Bituminous	PI not to exceed 10	Not to exceed 30% by weight	Well-graded material only
		(2) Portland cement	--[b]		Material should contain at least 45% by weight of material passing No. 4 sieve
		(3) Lime	PI not less than 12		
		(4) Lime-cement-fly ash	PI not to exceed 25		
3	CH or CL or MH or ML or OH or OL or ML-CL	(1) Portland	LL less than 40 and PI less than 20		Organic and strongly acid soils falling within this area are not susceptible to stabilization by ordinary means
		(2) Lime	PI not less than 12		

[a] Soil classification corresponds to MIL-STD-619B. Restriction on liquid (LL) and plasticity index (PI) is in accordance with Method 103 in MIL-STD-621A.

[b] $PI \le 20 + \frac{50 - \text{percent passing No. 200 sieve}}{4}$

Different state transportation agencies across the country may have their own criteria to identify the candidate stabilizer based on soil index properties (e.g., [8], [9]). In Little and Nair's [1] draft of AASHTO "Standard Recommend Practice for Stabilization of Subgrade Soils and Base Materials," The Texas Department of Transportation's (TxDOT) adopted the criteria described in Figure 3 below for selection of stabilizer type. The

DOTD recommends the criteria presented in Table 2 for the selection of the stabilizer based on the soil characteristic [10].

Figure 3. Stabilizer selection for subgrade soils [8]

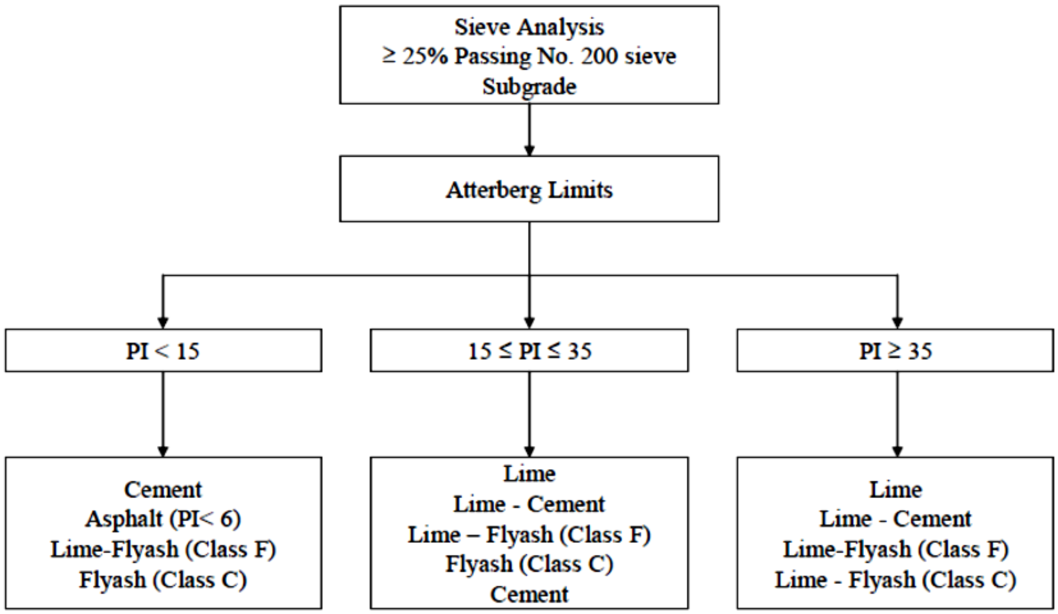


Table 2. Stabilizer selection criteria [10]

S.N.	Plasticity Index (PI)	Lime/Cement (% Volume)
1	0-15	9% Cement
2	16-25	6% Lime + 9% Cement
3	26-35	9% Lime + 9% Cement

Determination of Stabilizer Content. Once the stabilizer is selected, the dosage of stabilizer needs to be determined. Stabilized layers are generally required to achieve a minimum compressive strength to ensure benefits from stabilization. The minimum compressive strength requirement for stabilized layers varies considerably from agency to agency. Thompson [11] recommended the minimum desired unconfined compressive strength for lime-soil mixture with consideration of the anticipated use of stabilized layer, the thickness of pavement over the stabilized layer, exposure to soaking conditions, and the expected number of freezing and thawing cycles (Table 3). This criterion is adopted in the National Lime Association mixture design guideline [12] and Little and Nair's

AASHTO standard draft [1], as one of the lime-soil mixture design criteria. The unconfined compressive strength requirement for subgrade treated with cement, lime, lime-cement, and lime-cement-fly ash stabilized soils, as shown in Table 4, is suggested by the UFC [7] and recommended by the MEPDG. In Louisiana, Gautreau et al. [13] recommended a minimum compressive strength value of the 50 psi at 7 days curing time for working platform and a minimum compressive strength value of 150 psi at 7 days curing time for stabilized subbase layer that is identical criteria with the Indiana Department of Transportation (InDOT) criteria for selection of stabilizers.

Table 3. Soil-lime mixture minimum unconfined compressive strength recommendations [11]

Anticipated Use	Unconfined Compressive Strength Recommendations for Various Service Conditions			
	Extended Soaking for 8 Days (psi)	Cyclic Freeze-Thaw[b]		
		3 Cycles (psi)	7 Cycles (psi)	10 Cycles (psi)
Subbase				
Rigid Pavement	50	50	90	120
Flexible Pavement (> 10 in.)[a]	60	60	100	130
Flexible Pavement (8 in. - 10 in.)[a]	70	70	100	140
Flexible Pavement (< 8 in.)[a]	90	90	130	160
Base				
	130	130	170	200

[a]Total pavement thickness overlying the subbase. [b]Number of freeze-thaw cycles expected in soil-lime layer during the 1st winter of exposure.

Table 4. Minimum unconfined compressive strength recommendations [7]

Stabilized Soil Layer	Minimum Unconfined Compressive Strength (psi)[a]	
	Flexible Pavement	Rigid Pavement
Base Course	750	500
Subbase, Select Material, or Subgrade	250	200

[a]7 days for cement stabilization and 28 days for lime, lime-fly ash, and lime-cement-fly ash stabilization.

For lime-stabilized soil, it is generally accepted that the initial amount of lime (i.e., minimum amount of lime) required for stabilization can be determined by Eades-Grim PH test (ASTM D 6276). The percentage of lime in soil that achieves the design PH of 12.4 at 77°F is the minimum lime percentage for soil stabilization. For stabilization with cement, the Portland Cement Association [14] gave the usual range in cement requirements based on the AASHTO soil classification (Table 5) and provides a good starting point for the determination of cement percentage required for stabilization. It should be noted that the requirements provided by Portland Cement Association are based on durability tests (ASTM D 559 and D 560) and that many soils can be successfully stabilized with considerably lower cement contents [1]. The UFC [7] also provided initial estimate cement content by weight based on the USCS soil classification (Table 6).

Table 5. Cement requirements for AASHTO soil classification [14]

AASHTO Soil Classification	Usual Range in Cement Requirement		Estimated Cement Content and that Used in Moisture-Density test, Percent by Weight
	Percent by Volume	Percent by Weight	
A-1-a	5-7	3-5	5
A-1-b	7-9	5-8	6
A-2	7-10	5-9	7
A-3	8-12	7-11	9
A-4	8-12	7-12	10
A-5	8-12	8-13	10
A-6	10-14	9-15	12
A-7	10-14	10-16	13

Table 6. Cement requirements for USCS soil classification [7]

USCS Soil Classification	Initial Estimated Cement Content, Percent by Weight
GW, SW	5
GP, GW-GC, GW-GM, SW-SC, SW-SM	6
GC, GM, GP-GC, GP-GM, Gm-GC, SC, SM, SP-SC, SP-SM, SM-SC, SP	7
CL, ML, MH	9
CH	11

Characterization of Cementitiously Stabilized Subgrade Layers

Atterberg Limits. When highly plastic soil (PI >20-30) is treated with calcium rich additive, the chemical reaction between clay particles and additive results in reduction of the size of the diffused double layer and increase in the inter-particle contact. Consequently, the liquid limit of the soil will be decreased associated with the increase in plastic limit; hence decreasing the plasticity index of the stabilized soil. As a result, strength/stiffness of the stabilized soil will be improved.

Moisture-density Relation. The change in chemical composition of the soil can be noticed by the decrease in the maximum dry density and increase in the optimum moisture content of the soil-stabilizer mixture ([2], [3], [15]). However, Gautreau et al. [13], and Horpibulsuk et al. [12] found different types of results for cement stabilized soils. For heavy clay soil with LL=74% and PI=46%, Horpibulsuk et al. [16] reported an increase in maximum dry density with no significant change in optimum moisture content (Figure 4); while Gautreau et al. [13] found out that for soil with LL=34% and PI=12%, there is no significant difference in the optimum moisture content and maximum dry density with cement content, as shown in Figure 5.

Figure 4. Moisture density of heavy clay with cement [16]

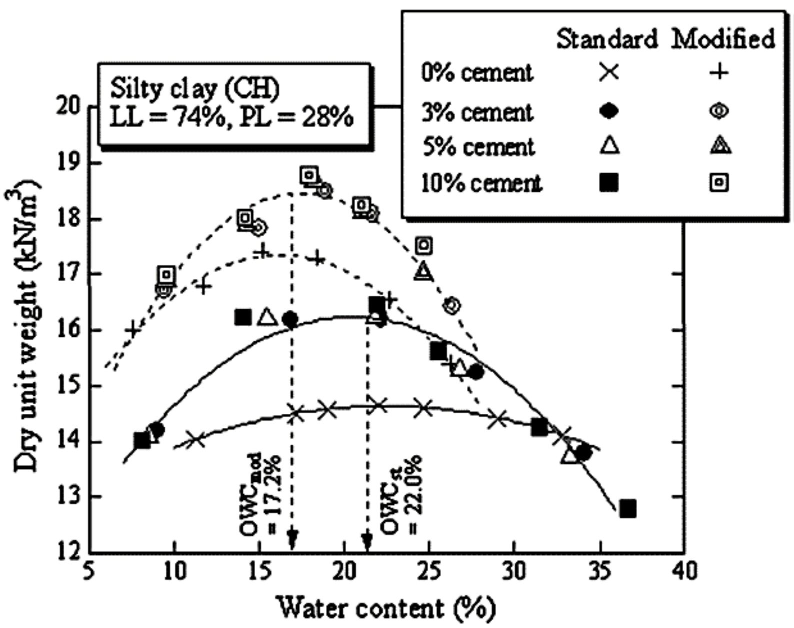
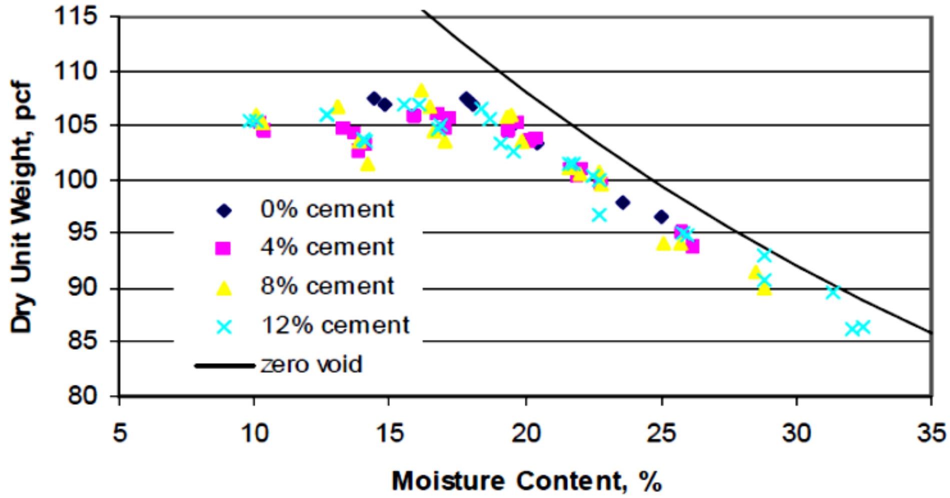
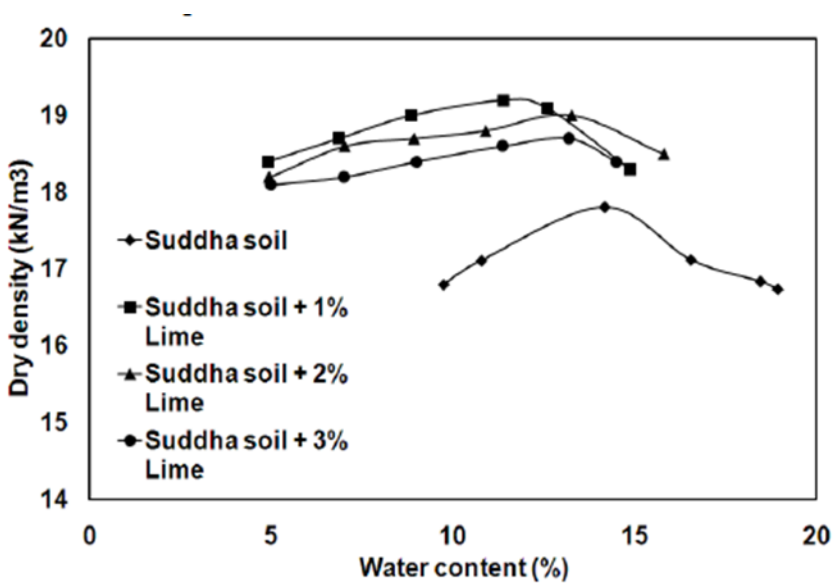


Figure 5. Moisture density relation of soil with cement [13]



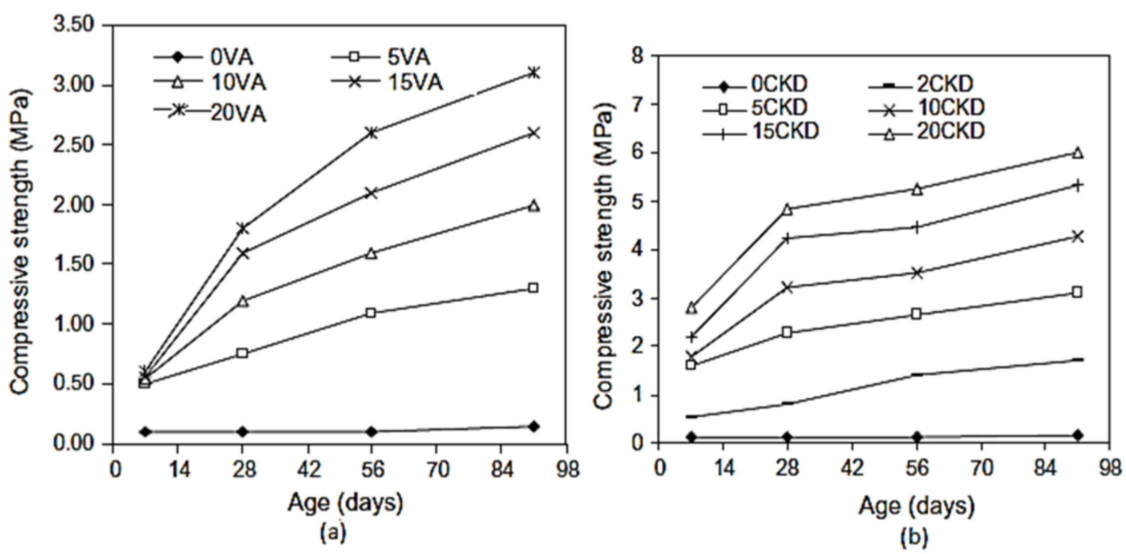
Furthermore, Umesha et al. [17] found even more different results than other researchers reported on using lime to treat/stabilize soils with plasticity limit of 41% and plasticity index of 17 %. They found out that the maximum dry density increased while the optimum moisture content decreased with the addition of lime as compared to those of the raw soil (Figure 6).

Figure 6. Moisture density relation of Suddha soil with lime [17]



Unconfined Compressive Strength (UCS). The unconfined compressive strength (UCS) of the soil increases drastically with the increase of the stabilizer content. Generally, cement stabilized soils possesses higher UCS than the soils treated with other stabilizers for the same stabilizer-moisture ratio. Bhattacharja and Bhatty [18] compared the performance of lime and cement on three different types of soils in Texas with PI of 25%, 37% and 42%, and found out that for all soils, better performance was observed from cement stabilizer. However, there was large decrease in the strength (by more than 50%) of the cement treated soils with delay of compaction by 24 hour. Additionally, Hossain and Mol [19] used natural pozzolans and industrial waste to stabilized the clay soil (A-6) having LL 39% and PI of 19%, and reported almost double strength gain with cement kiln dust (CKD) as compared to volcanic ash (VA) under identical condition as shown in Figure 7. Consoli et al. [20] obtained a linear variation of UCS with the increase in lime content for a soil having LL of 23% and PI of 10%. Furthermore, the effect of the porosity as well as porosity/lime ratio was determined, which showed considerable decline in compressive strength of the soil was observed with the increase of both porosity and the porosity/lime ratio.

Figure 7. Effect of VA (a) and CKD (b) on UCS of soil [15]



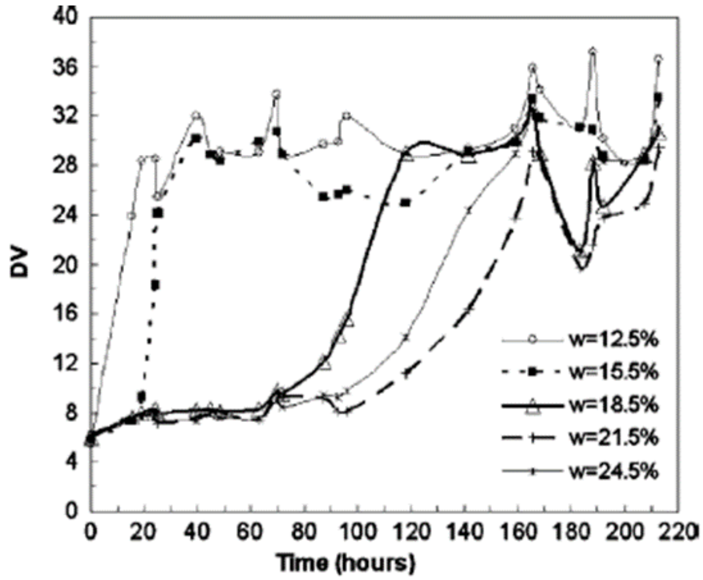
Durability. The durability test of the chemically-stabilized subgrade layers is the main concern during the construction of the pavement structures. It determines the ability of the subgrade to withstand against the extremely adverse environmental conditions during the service life of the pavement. In the past, freeze-thaw, wet-dry, California bearing ratio (CBR) tests were used to evaluate the durability of the soil or aggregates to be used in pavement structures. However, in recent years, a new technology to evaluate the performance of the pavement materials was introduced and termed as tube suction test, which gives a dielectric value of the surface having free moisture on it.

The increase in durability of the stabilized subgrades was experienced in previous research studies ([21], [22], [23]). Parker [22] used two different subgrade soils (SM and CL) and stabilized with cement, lime, and fly ash then tested for series of durability tests including freeze-thaw/wet-dry, tube suction tests, and vacuum saturation. He concluded that the sand specimens stabilized with lime-fly ash showed higher residual UCS results than cement/lime, and class C fly ash stabilized sand soil after freeze-thaw and vacuum saturation tests. Solanki et al. [23] further concluded that freeze-thaw tests were more severe than any other durability tests. Similar results on UCS were also observed with the clay specimens after cycles of durability tests. Unlike the treated sandy soils, the clay soils tested by Parker [22] were moisture susceptible as the results showed no marginal dielectric value. Solanki et al. [23] finally concluded that cement/lime treated/stabilized soils are more vulnerable to durability issue than the fly ash treated/stabilized soil subgrades.

Similarly, Parson and Milburn [24] performed durability tests on four different soil types (CH, CL, ML, and SM) stabilized with lime, cement, fly ash, and enzymes. Durability tests include swelling, freeze-thaw, wet-dry, and leaching test of the stabilized soil samples. Swelling of all soils treated with all stabilizers was almost reduced, except for a soil with small amount of sulfate content (0.41%). The order of soil loss after freeze-thaw was cement < fly ash < lime. However, they found higher strength and lower PI in lime and cement treated soils than in those treated with fly ash after leaching. The result showed that the clayey soils were vulnerable to wet-dry cycles; however different stabilizer performed differently depending upon the type of the soil.

In recent years, many researchers have used tube suction test for evaluating the durability of the pavement materials ([21], [25], [26], [27]). The tube suction test is a non-destructive test that is more time-efficient and economic than the other durability tests like freeze-thaw and wet-dry etc. The test measures the dielectric value of the compacted soil specimens after 10 days of capillarity suction. This test was developed by the joint effort of the Finnish National Road Administration and the Texas Transportation Institute in the late 1990's. Generally, soil samples having dielectric values less than 10 are assumed to perform well, but the dielectric values above 16 are considered moisture susceptible. The soil samples yielding values in between 10 to 16 are considered to have fair strength against the severe environmental condition ([28]). The dielectric value measures the amount of free water available in the soil; not the moisture content of the soil. The typical result from the tube suction test is presented in Figure 8.

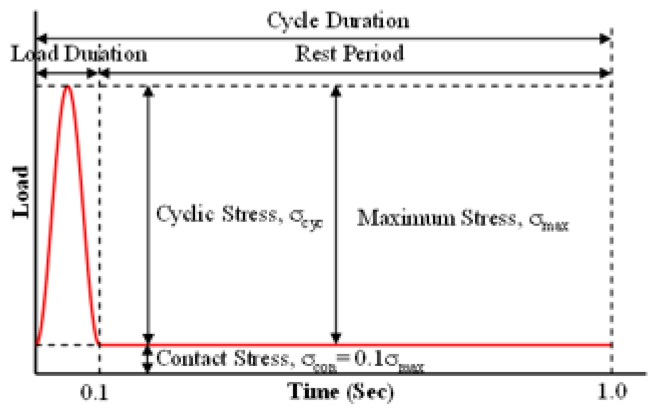
Figure 8. Dielectric value of soil at different water contents [17]



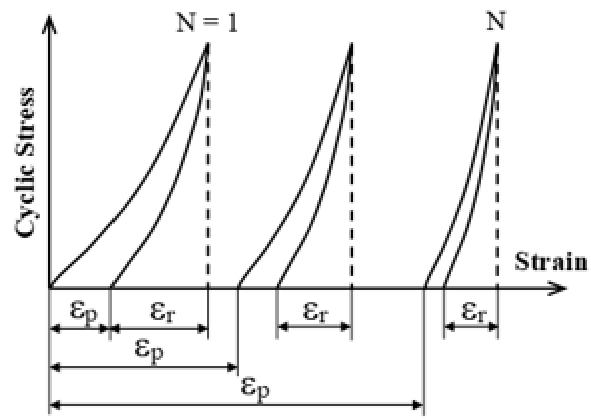
Repeated Load Characteristics. In order to characterize the resilient and permanent deformation behavior of the treated/stabilized soils, the RLT tests are usually performed to determine the M_R and permanent deformation characteristics of the treated/stabilized soils. The RLT test is conducted by applying a repeated axial cyclic stress of fixed magnitude, load duration, and cycle duration to a cylindrical test specimen for a certain number of cycles. While the specimen is subjected to dynamic cyclic stress, it is also subjected to a static confining pressure. The cyclic loading usually consists of repeated cycles of a haversine-shaped load pulse, as shown in Figure 9(a). These load pulses consist of 0.1-second load duration and 0.9-second rest period. The resilient modulus is defined as the ratio of the cyclic stress to the recoverable or resilient strain, as shown in Figure 9(b).

$$M_R = \frac{\sigma_{cyc}}{\varepsilon_r} \quad [5]$$

Figure 9. Repeated load triaxial test



(a)



(b)

Resilient Behavior. Many factors have been identified in the literature to influence the stiffness (or resilient modulus) of cementitiously stabilized soil. These factors include curing time, deviatoric stress, moisture content, curing temperature, cementitious content, stabilizer type, soil properties, density, and delay of compaction time (e.g., [15], [23], [29]).

Both laboratory and field studies available in literature clearly demonstrated that the stiffness (or resilient modulus) of cementitiously stabilized soil continuously increases with time for many years ([30], [31]). Generally, the curing time of lime and lime/fly ash stabilized soil are much slower than the cement stabilized soil [31]. The length of curing period used to determine the acceptable strength/modulus properties of the stabilized soil is an important consideration in design. Several studies in the literature showed a strong double logarithmic linear relationship between the resilient modulus and curing time for lime/cement stabilized soil (e.g., [30] [32]). The required modulus input in MEPDG is the 28-day M_R for stabilized soil cured at room temperature (73°).

The stress state (deviatoric stress and confining pressure) at which the M_R should be estimated can be determined, in general, from the structural analysis of the trial design (after properly accounting for overburden pressure) [33]. In the absence of the capability to perform structural analysis to estimate the stress state within the stabilized soil layer, a M_R value determined at a deviatoric stress of 6 psi is adequate for design purposes [31].

Puppala et al. [15] reported that lime stabilization worked better at wet-of-optimum moisture content than at dry-of-optimum moisture content in terms of unconfined compressive strength.

Achampong et al. [34] investigated the effect of deviatoric stress, molding moisture content, stabilizer type, curing period, and soil type on the resilient modulus on lime and cement stabilized soils. They reported that M_R increases with decreasing the deviatoric stresses, increasing lime and cement content, and extended curing period. They also reported that the Kaolinitic CL soils work better than montmorillonitic CH soils with both lime and cement.

Solanki et al. [23] evaluated engineering properties of cementitiously stabilized subgrade soils common in Oklahoma. In their study, three cementitious additives [hydrated lime (or lime), class C fly ash (CFA), and cement kiln dust (CKD)] were used to stabilize the subgrade soils. Their results showed that at lower dosage of stabilizer (3% to 6%), the lime stabilized soil showed the highest improvement in the M_R values. At higher dosages of stabilizer, CKD stabilization provided the highest improvement in the M_R values. The results also showed that the lime stabilization is more effective as compared to CFA and CKD stabilization in reducing the PI of soils. The results of tube suction tests and three-dimensional swell tests revealed that for non-sulfate bearing soil, lime and CFA stabilization can help reduce the moisture susceptibility of soil and three-dimensional swell. On the other hand, CKD stabilization makes stabilized soil more susceptible to moisture and three-dimensional swell, as compared to parent soils. They believed that it is due to the presence of high sulfate content (28,133 ppm) in CKD causing sulfate-induced heaving (ettringite formation). For sulfate bearing soil, only the CFA stabilization showed promising results in reducing the moisture susceptibility and three-dimensional swell.

Different models were developed in the literature to estimate the resilient modulus of soils. Some of them have also been used by various researchers to predict the resilient modulus of treated/stabilized subgrade soils.

Two different models used to predict the resilient modulus of pavement materials is given by AASHTO 1993 and are presented in equations (6) and (7).

$$M_R = k_1 \theta^{k_2} \text{ (bulk stress)} \quad [6]$$

$$M_R = k_2 \sigma_d^{k_4} \text{ (Deviator stress)} \quad [7]$$

where, M_R = resilient modulus, $\sigma_d = \sigma_1 - \sigma_3$ = deviator stress, θ= bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$ (sum of major, intermediate and minor principal stress respectively), and k_i are material constants to be determined from regression analysis.

The bulk stress model ignores the individual effect of deviatoric stress and confinement on resilient modulus, whereas the deviator stress model fails to define the effect of confining stress on cohesive soil.

Uzan [35] introduced the universal model [equation (8)] to calculate the resilient modulus by considering effect of confining stress as well as deviator stress. This model can effectively use for granular as well as cohesive soils.

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\sigma_d}{p_a}\right)^{k_3} \quad [8]$$

where, p_a reference atmospheric stress (14.7 psi).

Mohammad et al. [36] studied 8 different soil types (with plastic indices PI<64) in Louisiana to predict the realistic model that correlates the soil properties with the resilient modulus. They stated that when the ratio of maximum principal stress to minimum exceeds the value 2.5, the dilation of the soil starts. Furthermore, they assumed that the subgrade soil at lower depth may get the higher ratio of stresses that can experience the dilative behavior. Hence, it is not a good idea to use linear model to predict the resilient modulus that obviated the effect of dilation as well as shearing. Finally, they proposed the following model [equation (9)] to predict the resilient modulus:

$$\frac{M_R}{P_a} = k_1 \left(\frac{\sigma_{oct}}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a}\right)^{k_3} \quad [9]$$

where, $\sigma_{oct} = \frac{1}{3}(\sigma_1 + 2\sigma_3)$ and $\tau_{oct} = \frac{\sqrt{(\sigma_1-\sigma_2)^2+(\sigma_1-\sigma_3)^2+(\sigma_3-\sigma_2)^2}}{3}$ are octahedral normal stress and shear stress respectively.

Ni et al. [37] proposed the modified version of the Uzan model [equation (10)] that replaces the bulk stress by confining stress and eliminates the chance of having zero resilient modulus during zero deviatoric stress.

$$M_R = k_1 P_a \left(1 + \frac{\sigma_3}{P_a}\right)^{k_2} \left(1 + \frac{\sigma_d}{P_a}\right)^{k_3} \qquad [10]$$

Ooi et al. [38] recommended two models [Equation (11) and (12)] after modification to the model predicted by Ni et al. [36].

$$M_R = k_1 P_a \left(1 + \frac{\theta}{P_a}\right)^{k_2} \left(1 + \frac{\sigma_d}{P_a}\right)^{k_3} \qquad [11]$$

$$M_R = k_1 P_a \left(1 + \frac{\theta}{P_a}\right)^{k_2} \left(1 + \frac{\sigma_{oct}}{P_a}\right)^{k_3} \qquad [12]$$

The log-log model recommended by NCHRP 1-28A [39] with five regression parameters is presented in Equation (13), which changes the MEPDG model [33] [equation (14)] after assigning initial value for k6 and k7 as zero and one, respectively as recommended.

$$M_R = k_1 P_a \left(\frac{\theta - 3k_6}{P_a}\right)^{k_2} \left(k_7 + \frac{\sigma_{oct}}{P_a}\right)^{k_3} \qquad [13]$$

$$M_R = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(1 + \frac{\sigma_{oct}}{P_a}\right)^{k_3} \qquad [14]$$

Gupta et al. [40] predicted the a model [equation (15)] to evaluate the resilient modulus with considering the effect of soil-moisture suction. They performed the resilient modulus tests for four different clays (PI ranging from 0 to 52) in Minnesota by using NCHRP 1-28A testing protocol.

$$M_R = k_1 P_a \left(\frac{\theta - 3k_6}{P_a}\right)^{k_2} \left(k_7 + \frac{\sigma_{oct}}{P_a}\right)^{k_3} + \alpha_1(\mu_a - \mu_w) \qquad [15]$$

where, $(\mu_a - \mu_w)$ is the matric suction; α_1, β_1 are regression coefficients depending upon clay content or plastic limit.

Permanent Deformation. The power model proposed by Monismith et al. [41] for silty clay (PI < 15) is a widely accepted model to calculate the permanent deformation or rutting of the pavement with cohesive subgrade soils [equation (16)].

$$\varepsilon_p = AN^b \qquad [16]$$

where, ε_p = permanent or plastic deformation, N = number of repetition of loads, *A* and b are material properties (based on regression analysis).

During this test, samples of 2.8 in. diameter to 6 in. height were loaded for 10,000 cycles, with some receiving up to 100,000. The researchers found that the exponent term b depends only on the soil type, and the parameter *A* has significant effect on the permanent deformation of the pavement. The value of b was ranged from 0.154 to 0.332; while te value of *A* ranged from 0.0467 to 39.5.

Li and Selig [42] modified this power model to predict the accumulated plastic deformation of subgrade soils and proposed the following model [equation (17)]:

$$\varepsilon_p = aN^b\beta^m \qquad [17]$$

The material constants a, b, and m can be calculated by using regression analysis. The researchers also recommended different values of a, b, and m based on clay type. The term β is the ratio of deviator stress to static strength $\left(\frac{\sigma_d}{\sigma_s}\right)$ of soil, which was introduced to indirectly link the effect of moisture-density relationship.

Elliott et al. [43] proposed a linear model to correlate the permanent deformation to the deviator stress with R^2 value of 0.95. From this model [equation (18)], it can be inferred that the permanent deformation is increasing rapidly in higher level of deviator stress (σ_d) than with low deviator stress.

$$\varepsilon_p = -4.4193 + 0.7618\sigma_d \qquad [18]$$

All the tests for the model in equation (18) were performed under constant confining stress of 3 psi (21 kPa) with sinusoidal wave of 0.1 second loading and 0.9 second rest period. The samples were molded at 105%, 110%, 120% of optimum moisture content and 90%, 95%, 100% of maximum dry density. Soils from six different locations were compacted in 4 in. diameter to 5 in. height mold and loaded to maximum load of 10,000 cycles.

Puppala et al. [44] studied the permanent deformation test of silt, sand, and clay in the laboratory, and proposed the four parameter model [equation (19)] that considers the effect of octahedral shear and normal stress along with the number of load cycles.

$$\varepsilon_p = \alpha_1 N^{\alpha_2} \left(\frac{\sigma_{oct}}{\sigma_{atm}}\right)^{\alpha_3} \left(\frac{\sigma_{oct}}{\sigma_{atm}}\right)^{\alpha_3} \qquad [19]$$

Repeated loading triaxial (RLT) tests were performed on the laboratory molded samples at three different moisture contents to predict the model in equation (18). The deviatoric stresses were selected as 20%, 40% and 60% of failure deviatoric stress that obtained from the unconsolidated-undrained (UU) triaxial tests to simulate the different loading condition in the field. They concluded that the permanent deformation not only depends upon the number of repetitions of the loading cycles, but also it can be expressed as a function of octahedral normal and shear stresses.

Wang [45] used the heavy vehicle simulator (HVS) to estimate the permanent deformation in two types of subgrade soils (A-2-4 and A-4), and predicted the model presented in equation (20) for permanent deformation of the subgrade soils. He found significant difference between the deformations predicted from MEPDG model to the measured deformations, and stated that this model may not be accurately applicable to other subgrade soil types.

$$\frac{\varepsilon_p}{\varepsilon_r} = a_1 \left(\frac{\theta}{P_a}\right)^{a_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{a_3} W_c^{a_4} N^b \qquad [20]$$

where, ε_r = resilient strain, $W_c^{a_4}$ = water content (%), a_i, b = regression constants depending on the material properties.

Luo et al. [46] used the probabilistic model [equation (21)] to predict the rutting of subgrade soil under high speed railways that correlated the long-term deformation of subgrade soils with relative compaction, number of cycles, and ratio of cyclic to deviatoric stress.

$$\varepsilon_p = a \left(\frac{\sigma_d}{\sigma_c}\right)^m K^n N^b \qquad [21]$$

where, a, m, n, and b are constants to be determine statistically and K is relative compaction of subgrade soil.

Objective

The objectives of this research study can be summarized as follows:

a) Evaluate the current subgrade treatment/stabilization schemes of hauled soils as provided in the DOTD *Standard Specifications for Roads and Bridges* (Phase I).
b) Examine the appropriate treatment/stabilization schemes for very weak in-situ subgrade soils at high water content condition (i.e., at wet-side of optimum of nearly saturated condition) with unconfined compressive strength < 25 psi (Phase II). Four types of subgrade soils were considered: silty soil, low-plasticity silty clay ($PI \leq 15$), medium-plasticity silty clay ($PI = 16$-25), high-plasticity silty clay ($PI = 26$-35), and heavy clay ($PI > 35$).
c) Identify the properties of cementitiously-treated subgrade soil that significantly influence the design, constructability, and performance of pavement structures. This includes the performance-related properties [e.g., the resilient modulus of treated subgrade layers (M_R) and permanent deformation] for characterizing the cementitiously treated/stabilized subgrade layer for use in the design and analysis of pavements.
d) Provide DOTD pavement engineers with guidance on how to consider the cementitiously-treated/stabilized very weak subgrade layer to create a working platform and/or to structurally enhance the composite subgrade layer in designing flexible pavements, such as defining composite resilient modulus ($M_{R\text{-}comp}$) of treated/stabilized subgrade soil, for use in both the 1993 AASHTO Pavement Design Guide and the new *Mechanistic-Empirical Pavement Design Guide* (MEPDG).

Scope

This research study consists of two phases. Phase I evaluates the current subgrade treatment/stabilization schemes for hauled soil as provided in the DOTD standard specifications. Phase 2 examines the appropriate treatment schemes for very weak in-situ subgrade soils at high water content. In Phase I, stabilizer (cement and/or lime) was selected based on soil type. Repeated load triaxial (RLT) tests were performed to evaluate the resilient modulus (M_R) and permanent deformation characteristics of stabilized specimens.

The first part of Phase II includes the selection of the suitable stabilizer for wide range of soil types and recommends the best stabilizer type and dose for use to a particular type of soil based on strength requirements from unconfined compressive strength and the plasticity indices of the soils. The second part of Phase II includes the evaluation of the repeated loading characteristic of the treated/stabilized weak soil samples of different additive contents and moisture contents that achieve the minimum strength recommended by the DOTD for use as working platform (minimum 7-day strength of 50 psi) as well as subbase layer (minimum 7-day strength of 100 psi). The behavior of the laboratory molded specimens were evaluated using RLT tests in the form of M_R and permanent deformation tests. Regression analysis were conducted on the results of laboratory tests and were analyzed using statistical analysis software (SAS) to evaluate the resilient modulus regression parameters k_1, k_2, k_3; and the results were fitted on the suitable model available for the clayey soil based on the literature review.

The literature review of the repeated loading behavior of the cementitiously stabilized soil was performed. Furthermore, the mechanism of the stabilization of the clayey soil due to addition of the stabilizer is also reviewed. Three naturally soil types (low PI, medium PI, and high PI) were considered for Phase I, and four naturally soil types (low PI, medium PI, high PI, and heavy clay) were used for Phase II of this study. The factorial of the research will be presented later in the Methodology section.

Methodology

This section will present the detailed procedures of all the laboratory tests performed to achieve the objectives of this study. The soils were collected from different locations in Louisiana, and all the samples used in this study were molded in the laboratory according to the available standard procedures by American Society of Testing and Materials (ASTM) or American Association of State Highway and Transportation Officials (AASHTO) or based on the literature review. For the laboratory testing program, lime type-I Portland cement and fly ash were considered as a candidate stabilizer to treat/stabilize the different soil types.

Phase I

Testing Materials

Three different soil types of different plasticity (low-plasticity clay, medium-plasticity clay, high-plasticity clay), as shown in Table 7, were selected for inclusion in Phase I of this study.

Table 7. Properties of soils used in the study (Phase I)

Soil #	Soil Type	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	AASTHO Classification	USCS Classification
1	**Low PI**	30	19	11	A-6	CL
2	**Medium PI**	58	34	24	A-7-5	CH
3	**High PI**	55	21	34	A-7-6	CH

Mixture Design

The available recommendations on the selection of stabilizer type and stabilizer content made by DOTD specifications (Table 8) was considered in Phase I study of this research.

Table 8. Selection of stabilizer on soil properties (DOTD)

Soil property	Additive type
PI≤15	6% cement
16≤PI≤25	6% lime and 6% cement
26≤PI≤35	9% lime and 6% cement

Laboratory Characterization of Raw and Cementitiously Treated/Stabilized Soils

The laboratory tests summarized in Table 9 were performed to characterize the properties of raw and treated/stabilized soils.

Table 9. Laboratory characterization tests (Phase I)

Test	Method
Index Tests	Atterberg limits test
Moist Density (MD) Curve	Standard Proctor test
Strength Tests	Unconfined compressive strength
Moisture Susceptibility Tests	Tube Suction Test
Shrinkage Tests	Linear shrinkage test

Standard Proctor Test. For standard Proctor test, the lime-treated soils were compacted in proctor after few hours of mixing to allow the mellowing period; whereas the cement-treated soil specimens were compacted immediately after mixing.

Unconfined Compressive Strength Test. The molded samples were placed in airtight plastic wrapper and kept in a 100% humid room in accordance with the ASTM standard procedure (ASTM 1632). ASTM D 2166, ASTM D 5102, and ASTM D 1633 were followed to compact and test the raw, lime, and cement treated/stabilized soils, respectively. After 7 or 28 days of curing period, the soil samples were removed from the plastic wrapper. Cement treated/stabilized samples were then submerged in the water bath for approximately 3 to 4 hours (ASTM D 1633) prior to testing; lime treated/stabilized soils were kept above porous stone for capillarity suction for about 8 to 10 hours prior to testing.

Tube Suction Test. For the Tube suction test, the samples were compacted at identical conditions of moisture content and additive contents as the RLT tests. However, the sizes of the specimens were 4 in. in diameter and 7 in. height and compacted using the standard proctor energy. All the samples were cured for 7 days in the humid room. After 7 days of curing, the samples were kept in the oven at 104ºF until the weight of the sample became constant. The samples were then placed inside a plastic mold with holes at the bottom to ensure enough capillarity suction [Figure 10(a)] and then above the porous stone with at least 0.20 in. of water to create submergence conditions at the bottom part of the sample. Five readings were taken at different places at the top of the sample. The lower and higher values were dropped in order to achieve consistency. The Percometer [Figure 10(b)] was used to measure the dielectric value of the soil-stabilizer specimens. The readings were recorded for at least 10 days or until the dielectric value became constant.

Linear Shrinkage Test. A known quantity of stabilized mix according to standard maximum dry unit weight was compacted in two layers into a rectangular steel mold with dimensions of 3 in. × 3 in. × 11 in. A wooden plate was designed and used to compact the soil sample. The wooden plate was used to ensure uniform distribution of the compaction energy on the entire surface of prepared sample. Figure 11(a) shows the metal mold used to prepare the soil specimens. The Proctor hammer was used for compaction. The number of blows to compact the sample was calculated to achieve the standard Proctor compaction of 600 kJ/m^2. Two gauge studs were placed at the middle of the end sections during compaction to facilitate shrinkage measurements. Specimens were cured for 48 hours at 100% relative humidity (RH) and at an air temperature of 73.4°F. Subsequently, the specimens were dried in a controlled environment with 50% RH and at an air temperature of 73.4°F. Figure 11(b) shows a free drying shrinkage test setup. Specimen lengths, li (in.), were recorded at gradually increasing intervals for up to 90 days. Shrinkage strain (ε) at any time can be determined as follows:

$$\varepsilon = \frac{l_i - l_0}{l_0} \qquad [22]$$

Figure 10. (a) Mold with holes at the bottom for capillarity suction (b) Percometer to measure dielectric constant

(a) (b)

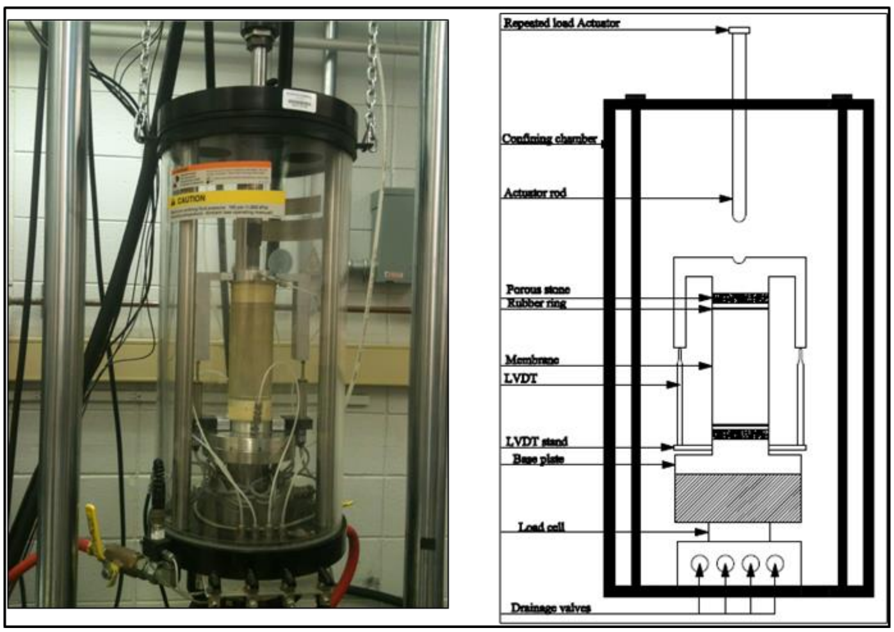
Figure 11. (a) Metal mold used to prepare soil specimens (b) Linear shrinkage test setup

(a) (b)

Repeated Loading Triaxial Tests

All RLT tests were carried out using the Material Testing System (MTS810) with a closed loop and servo hydraulic system. Figure 12 shows the testing equipment. The applied loads were measured using a ±5,000 lbf capacity load cell. The load cell is placed inside the testing chamber. This particular setup eliminates the push-rod seal friction and pressure area errors, thus reducing the testing equipment error. An external load cell is affected by changes in confining pressure and by load rod friction, and the internal load cell therefore gives more accurate readings. The axial deformation was measured using two Linearly Variable Differential Transducers (LVDTs). The two LVDTs were secured to the top plate. The confining pressure was achieved through the use of pressurized air. It was measured using a pressure sensor. The prepared sample was placed on the load cell and secured on to the load cell through a base plate [Figure 12(a)]. The sample was then sealed with the use of o-rings and clamps so that the confining pressure could be applied. Once the sample was safely secured in the pressure chamber, it was conditioned to be prepared for the RLT tests. A series of 2.8 in. (in diameter) × 5.6 in. (in height) cylindrical specimens of cementitiously treated/stabilized subgrade soil samples were molded in the laboratory at optimum and optimum ± 2% moisture content to simulate field controlled subgrade layer construction.

Figure 12. MTS 810 machine (left) with sketch (right) used for the RLT tests



Resilient Modulus Tests. The resilient modulus tests were performed in accordance with the AASHTO-T307 standard method for determining the resilient modulus of subgrade soils [47]. In this test method, the samples are first conditioned by applying 1,000 load cycles with a cyclic stress of 3.6 psi (24.8 kPa) and a confining stress of 6 psi (41.4 kPa). The purpose of conditioning step is to remove any irregularities on the top and bottom surfaces of the specimen, and to suppress most of the initial stage permanent deformation. The conditioning step is followed by a sequence of loading with varying confining and cyclic stresses. The confining pressure is first set as 6 psi (41.4 kPa), and the cyclic stress is increased from 1.8 psi (12.4 kPa) to 3.6 psi (24.8 kPa). The cyclic stress will further increased to 5.4 psi (37.2 kPa), then to 7.2 psi (49.6 kPa), and then to 9 psi (62.1 kPa), with 100 cycles for each load combination. Subsequently, the confining pressure is decreased to 4 psi (27.6 kPa), and then to 2 psi (13.8 kPa). The cyclic stress varies in the same way as with the confining pressure of 6 psi (41.4 kPa). The resilient modulus tests were performed on laboratory molded samples that were cured for 7 days as well as 28 days prior to testing. The proposed testing factorial for resilient modulus is presented in Table 10.

Table 10. Testing factorial for resilient modulus tests (Phase I)

	Untreated*			Treated					
Soil Type	OMC - 2%	OMC#	OMC + 2%	OMC - 2%		OMC		OMC + 2%	
				7 days	28 days	7 days	28 days	7 days	28 days
1	3	3	3	3	-	3	3	3	-
2	3	3	3	3	3	3	3	3	3
3	3	3	3	3	-	3	3	3	-

* = Specimens were tested immediately after preparation, # = Optimum moisture content.

Single-stage Permanent Deformation Tests. For single-state permanent deformation test, the samples were first conditioned by applying a cyclic stress of 2.25 psi (15.5 kPa) and a confining stress of 6 psi (41.4 kPa) for 1,000 cycles. After that, the confining pressure was set as 2 psi (13.8 kPa). A cyclic stress of 5.4 psi (37.2 kPa) was then applied to the specimen for 100,000 cycles. The loading conditions were selected based on the results of previous tests conducted by Mohammad and Hearth [48] on subgrade soils in Louisiana. The permanent deformation tests were performed on laboratory molded samples that were cured for 7 days as well as 28 days prior to testing. The factorial for single-stage permanent deformation test is presented in Table 11.

Table 11. Testing factorial for single-stage permanent deformation tests (Phase I)

	Untreated*	Treated					
Soil Type	OMC#	OMC - 2%		OMC		OMC + 2%	
		7 days	28 days	7 days	28 days	7 days	28 days
1	2	-	2	2	2	-	2
2	2	-	2	2	2	-	2
3	2	-	2	2	2	-	2

* = Specimens were tested immediately after preparation, # = Optimum moisture content.

Multi-stage Permanent Deformation Tests. Multi-stage permanent deformation tests were performed to characterize the behavior of raw soil and treated/stabilized soil specimens under different deviatoric and confining stress levels. Initially, the static triaxial loading of the specimens were performed for soil #3 at different confining stress to develop the p versus q chart (described later) and hence to define the failure line. The stresses were then selected based on the developed p-q chart (Figure 13).

For the multi-stage permanent deformation tests, the AASHTO T-307 standard was followed when taking into consideration the conditioning phase of the sample before testing. Condition consisted of applying 1,000 cycles at a cyclic stress of 13.5 psi (93 kPa) and a confining stress of 15 psi (103.4 kPa). Conditioning is important as it removes unevenness of the top and bottom layers. It also helps in the initial rearrangement of the aggregates, which could cause larger absolute permanent deformation.

Once the conditioning phase was completed, the sample was tested in six different stages in sequence. Each stage included applying 10,000 load cycles at the specific stress level. Each stage differed from the previous one due to an increase in σ_1/σ_3 ratio. In doing so, crossing the static failure line of the sample would be easier to achieve and thus determining the shakedown limits. To increase the σ_1/σ_3 ratio, σ_3 was kept constant and σ_1 was increased. Samples were tested with three different values of σ_3: 6, 4, and 2 psi. The stress levels for each stage are summarized in Table 12. Test 1, 2, and 3 represent the different confining pressure. The proposed factorial for multi-stage permanent deformation tests is presented in Table 13.

Figure 13. p-q plot for multi-stage test for soil #3

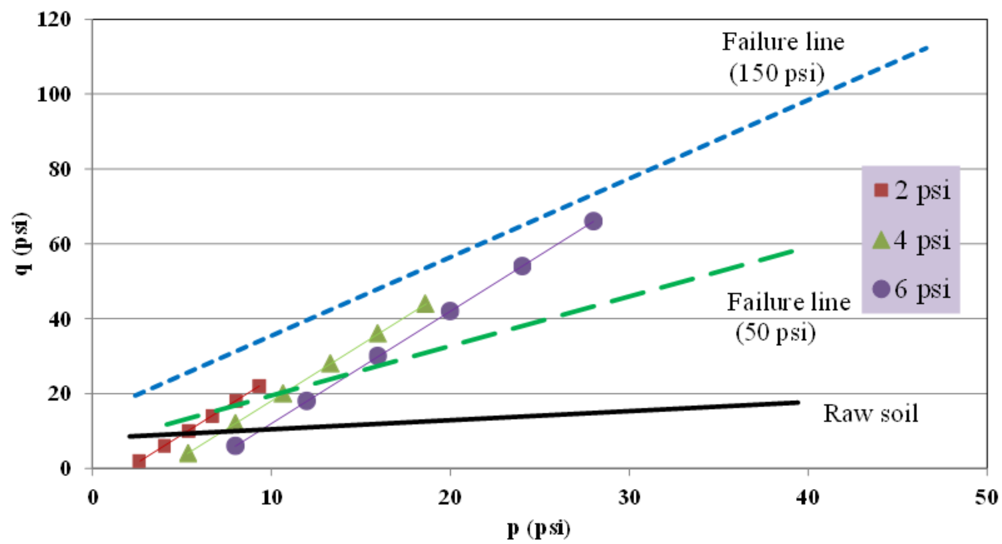


Table 12. Multi-stage RLT tests stress levels

Stage	Test 1			Test 2			Test 3		
	σ_3 (psi)	σ_1 (psi)	σ_1/σ_3	σ_3 (psi)	σ_1 (psi)	σ_1/σ_3	σ_3 (psi)	σ_1 (psi)	σ_1/σ_3
1	6	12	2	4	8	2	2	4	2
2	6	24	4	4	16	4	2	8	4
3	6	36	6	4	24	6	2	12	6
4	6	48	8	4	32	8	2	16	8
5	6	60	10	4	40	10	2	20	10
6	6	72	12	4	48	12	2	24	12

Table 13. Testing factorial for multi-stage permanent deformation tests (Phase I)

Soil Type	Untreated*	Treated					
	OMC#	OMC – 2%		OMC		OMC + 2%	
		7 days	28 days	7 days	28 days	7 days	28 days
1	1	-	1	1	1	-	1
2	1	-	1	1	1	-	1
3	1	-	1	1	1	-	1

* = Specimens were tested immediately after preparation, # = Optimum moisture content.

Phase II-a

Testing Materials

Four different soil types of different plasticity (low-plasticity clay, medium-plasticity clay, high-plasticity clay, heavy clay), as shown in Table 14, were selected for inclusion in Phase II of this study.

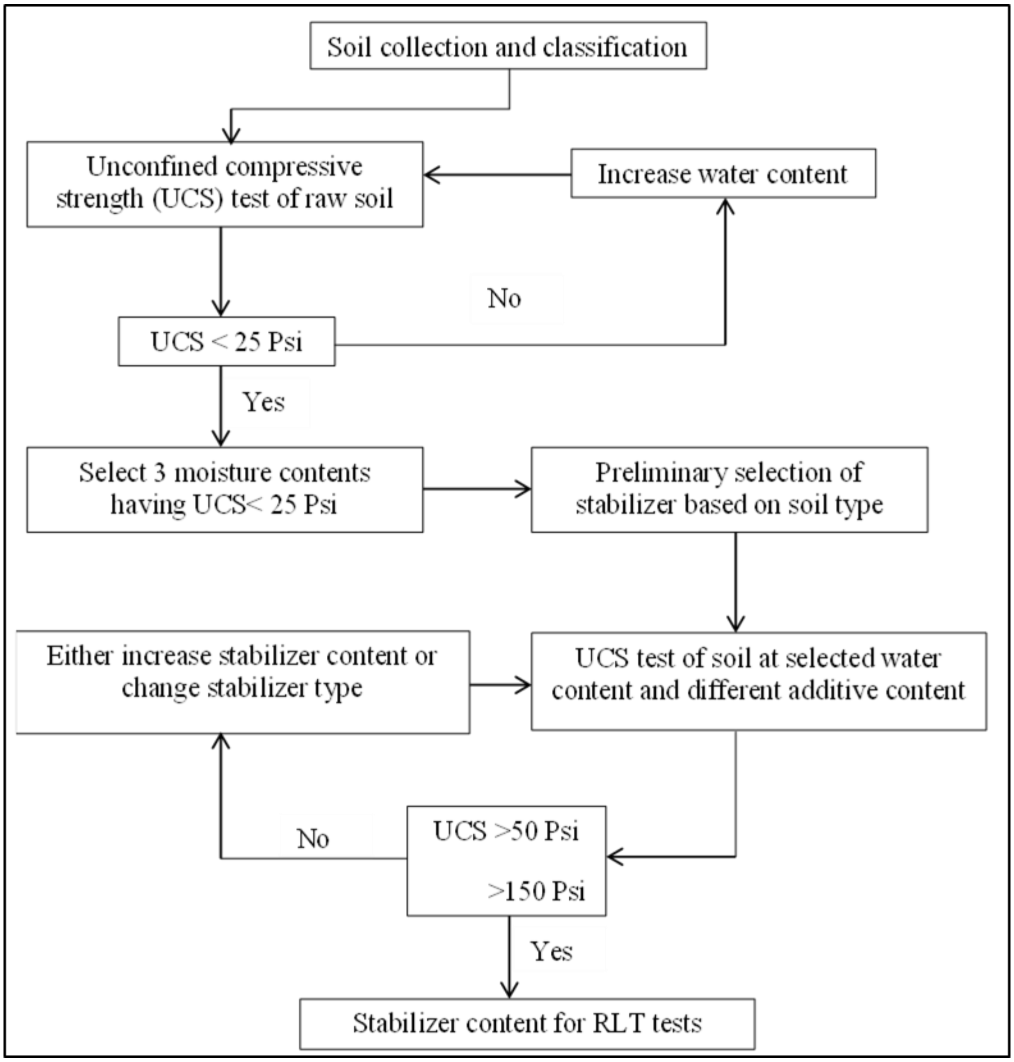
Table 14. Properties of soils used in the study (Phase II)

Soil #	Soil Type	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	AASTHO Classification	USCS Classification
1	Low PI	30	19	11	A-6	CL
2	Medium PI	40	19	21	A-6	CL
3	High PI	46	18	28	A-7	CL
4	Heavy Clay	88	35	53	A-7-6	CH

Mixture Design

UCS tests were first performed on the raw soils at different moisture contents to establish the UCS-moisture content relationship. Three moisture contents at the wet side of optimum producing UCS of 25 psi or less were then selected to simulate the very weak wet soil condition in the field. Preliminary selection of soil stabilizers were made based on basic soil index properties such as PI and soil gradation characteristics. The UCS tests were then conducted on treated/stabilized soils to evaluate the suitability of the selected stabilizer for particular soil and to determine the percentage of stabilizer needed to achieve the target 7-day strength values of 50 psi for working table application and 100 psi for stabilization of subbase. The schematic diagram in Figure 14 presents the detailed procedure flowchart adopted during the selection of the moisture contents and stabilizer contents for the present study.

Figure 14. Mixture design procedure for soils



Laboratory Characterization of Raw and Cementitiously Treated/Stabilized Soils

The laboratory tests summarized in Table 15 were performed to characterize the properties of raw and treated/stabilized soils. The procedures for these tests are the same as those used in Phase I of this research study.

Table 15. Laboratory characterization tests (Phase II)

Test	Method
Index Tests	Atterberg limits test
Moist Density (MD) Curve	Standard Proctor test
Strength Tests	Unconfined compressive strength
Moisture Susceptibility Tests	Tube Suction Test
Shrinkage Tests	Linear shrinkage test

Repeated Loading Triaxial Tests

A series of 2.8 in. (in diameter) × 5.6 in. (in height) cylindrical specimens of cementitiously treated/stabilized subgrade soil samples were molded in the laboratory at selected moisture content to simulate very weak wet soil. Repeated load triaxial tests were not possible for raw soil specimens at high moisture contents since they were too weak to sustain the RLT tests.

Resilient Modulus Tests. The resilient modulus tests were performed in accordance with AASHTO-T307 standard method for determining the resilient modulus of subgrade soils [47]. The resilient modulus tests were performed on laboratory molded samples that were cured for 28 days prior to testing. The proposed testing factorial for resilient modulus at three selected moisture contents is presented in Table 16.

Table 16. Testing factorial for resilient modulus tests (Phase II)

	28 days curing					
Soil Type	**50 psi**			**100 psi**		
	MC1	**MC2**	**MC3**	**MC1**	**MC2**	**MC3**
1	3	3	3	3	3	3
2	3	3	3	3	3	3
3	3	3	3	3	3	3
4	3	3	3	3	3	3

Single-stage Permanent Deformation Tests. For the single-stage permanent deformation tests, the samples were first conditioned by applying a cyclic stress of 2.25 psi (15.5 kPa) and a confining stress of 6 psi (41.4 kPa) for 1,000 cycles. Once the conditioning phase was completed, the confining pressure is set at 2 psi (13.8 kPa). A cyclic stress of 5.4 psi (37.2 kPa) was then applied to the specimen for 100,000 cycles. The loading conditions were selected based on the results of previous tests conducted by Mohammad and Hearth [48] on subgrade soils in Louisiana. The permanent deformation tests were performed on laboratory molded samples that were cured for 7 days as well as 28 days prior to testing. The testing factorial for single-stage permanent deformation test at three selected moisture contents is presented in Table 17.

Table 17. Testing factorial for single-stage permanent deformation tests (Phase II)

	28 days curing					
Soil Type	**50 psi**			**100 psi**		
	MC1	**MC2**	**MC3**	**MC1**	**MC2**	**MC3**
1	2	2	2	2	2	2
2	2	2	2	2	2	2
3	2	2	2	2	2	2
4	2	2	2	2	2	2

Multi-stage Permanent Deformation Tests. For multi-stage permanent deformation tests, the AASHTO T-307 standard was followed when taking into consideration the conditioning phase of the sample before testing. The conditioning phase consisted of applying 1,000 cycles at a cyclic stress of 13.5 psi (93 kPa) and a confining stress of 15 psi (103.4 kPa).

Once the conditioning phase was completed, the sample was tested in six different stages in sequence. Each stage included applying 10,000 load cycles at the specific stress level. Each stage differed from the previous one due to an increase in σ_1/ σ_3 ratio. In doing so, crossing the static failure line of the sample would be easier to achieve and thus determining the shakedown limits. To increase the σ_1/ σ_3 ratio, σ_3 was kept constant and σ_1 was increased. Samples were tested with three different values of σ_3: 6, 4, and 2 psi. The stress levels for each stage are summarized in Table 12. Test 1, 2, and 3 represent the different confining pressure. The proposed factorial for multi-stage permanent deformation tests at three selected moisture contents is presented in Table 18.

Table 18. Testing factorial for multi-stage permanent deformation tests (Phase II)

	28 days curing					
Soil Type	50 psi			100 psi		
	MC1	MC2	MC3	MC1	MC2	MC3
1	1	1	1	1	1	1
2	1	1	1	1	1	1
3	1	1	1	1	1	1
4	1	1	1	1	1	1

Discussion of Results

Phase I

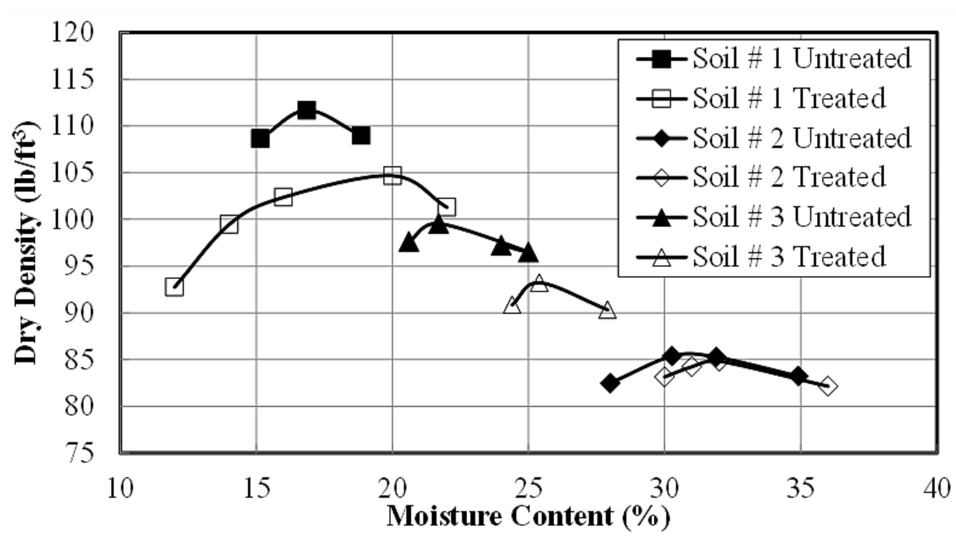
Phase I testing focused on evaluating the current subgrade treatment/stabilization schemes as provided in DOTD *Standard Specifications for Roads and Bridges* (Table 8). Three different soil types of different plasticity [low-plasticity clay (PI=11), medium-plasticity clay (PI=24), high-plasticity clay (PI=34)] were included in Phase I of this study (Table 7).

Moisture-Density Relationship

Standard Proctor tests were performed on the three selected raw subgrade soils as well as the treated/stabilized soils as per DOTD standard specifications. The corresponding Standard Proctor compaction curves obtained for the raw soils subgrade and treated/stabilized soils are presented in Figure 15.

While the compaction curve shifted right-down for the treated/stabilized low and high plasticity soils, there was almost no effect of the stabilizers on the compaction characteristics of the medium plastic soils (PI = 24). The change in optimum moisture content (OMC) and maximum dry density (MDD) for the treated/stabilized soils were observed to be 18.7% and 6.2% for soil #1 and 17% and 6.3% for soil #3.

Figure 15. Compaction curves of raw and treated soils



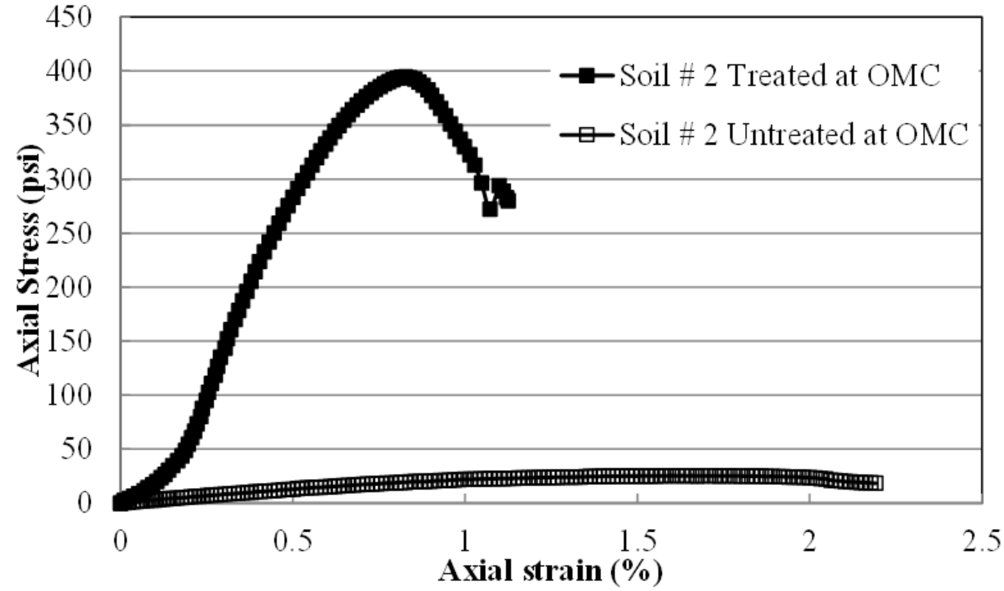
Unconfined Compressive Strength (UCS) Tests

A series of UCS tests were conducted on both the raw soils and treated/stabilized soil specimens at different moisture contents (i.e., OMC-2 %, OMC%, OMC+2%). The results of UCS tests for the three soil types are presented in Table 19. The stress-strain behavior of the raw soil specimens were also compared with the treated/stabilized soil specimens. The addition of stabilizers enhances the strength and stiffness of the raw soils; while at the same time the soil loses its ductile nature or cohesive nature and becomes more brittle as the axial strain reduced considerably. Typical stress-strain curves for the treated and untreated soil #2 obtained at OMC are presented in Figure 16. The figure clearly indicates that the stress-strain curves shift towards the left hand side as the strain at failure reduced with addition of the stabilizer and it is associated with significantly higher compressive strength; hence, increasing the elastic modulus and shear modulus of the treated/stabilized soils.

Table 19. Unconfined compressive strength of specimens (psi)

	Untreated			Treated (28 days curing)		
Soil #	**OMC-2%**	**OMC%**	**OMC+2%**	**OMC-2%**	**OMC%**	**OMC+2%**
I	-	26	-	317	251	196
II	-	29	-	434	386	319
III	-	18	-	220	184	150

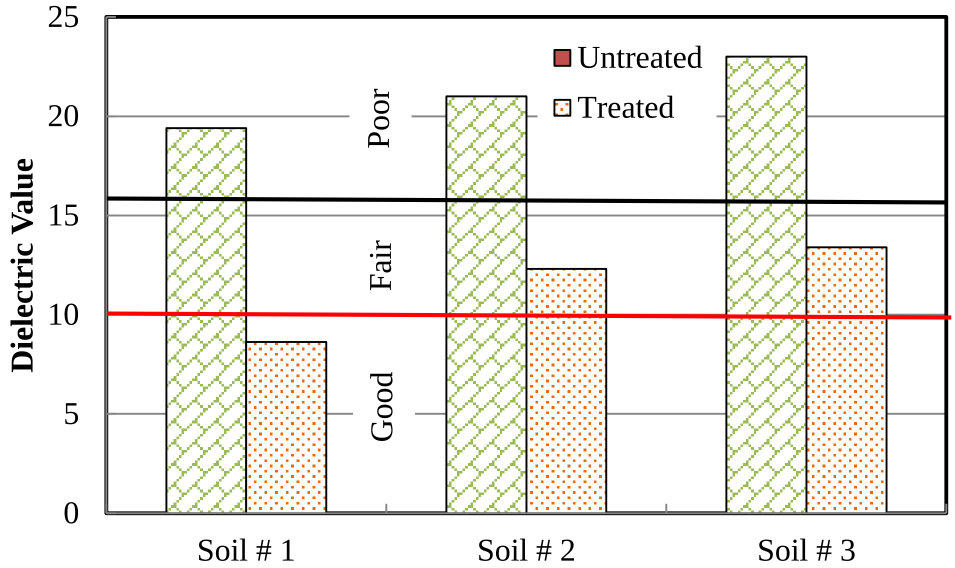
Figure 16. Stress-strain relationships for soil #2 with and without treatment



Results of Tube Suction Tests

Tube suction tests were performed on the raw and treated/stabilized soil specimens molded at OMC to evaluate their moisture susceptibility, since durability is a major concern for stabilized soil particles. The soil specimens were molded under identical conditions of moisture content and additive content to that of the repeated loading triaxial tests. The results of tube suction tests in terms of dielectric values (DVs) for the raw and treated/stabilized soil specimens are presented in Figure 17. The results showed that the final dielectric values (DVs) of the raw soil samples were all above the value of 16. Referring to the criteria recommended by the TxDOT, all three raw soils were water susceptible. For the treated/stabilized soil #1, the maximum DV was less than 10, which means good quality material according to TxDOT's criterial; while for the treated/stabilized soil #2 and soil #3, the maximum DVs were between 10 and 16, which means marginal material based on TxDOT's criterial.

Figure 17. Dielectric values of soil #1 at different moisture contents (7-day curing period)



Results of Linear Shrinkage Tests

Linear shrinkage tests were performed on lime treated soil specimens (for soils #1, #2 and #3) for a duration of 20 days. The shrinkage characteristics of clay soil specimens were improved significantly by the addition of lime (Figure 18 to 20). The addition of lime to such clay soils also reduces, or indeed removes, their potential for swelling. Linear shrinkage usually amounts to around 9 to 12% [48]. The addition of lime reduced the linear shrinkage of all three soils. Increasing the amounts of lime gave rise to increasing reductions in shrinkage, the most noticeable reductions being attained with small additions of lime.

Figure 18. Results of linear shrinkage test for lime treated soil #1

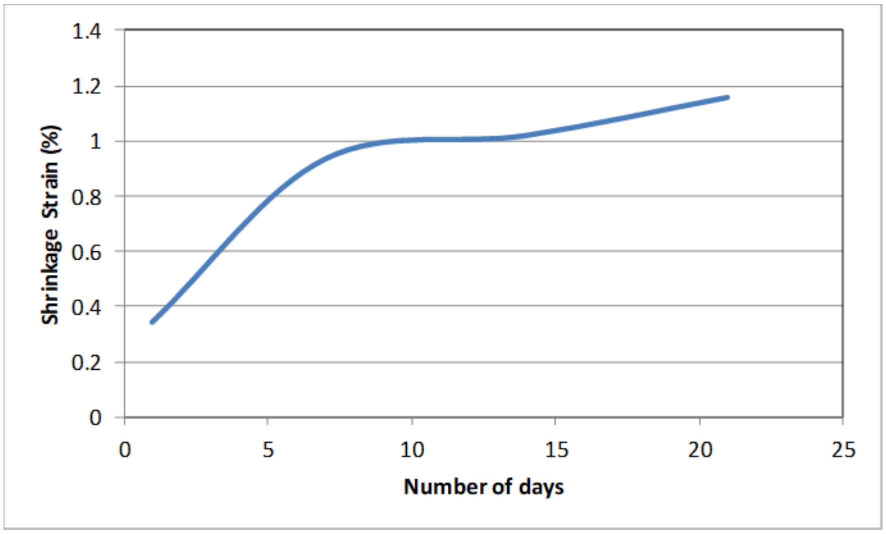


Figure 19. Results of linear shrinkage test for lime treated soil #2

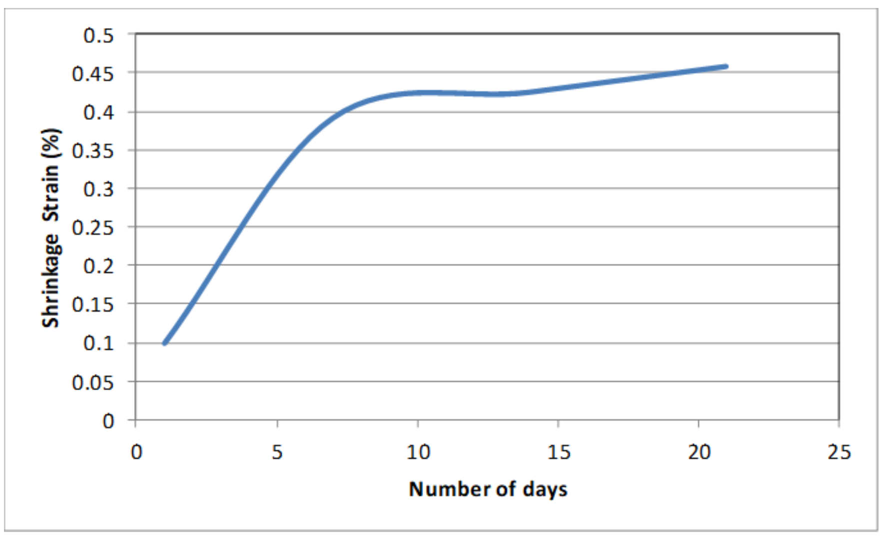
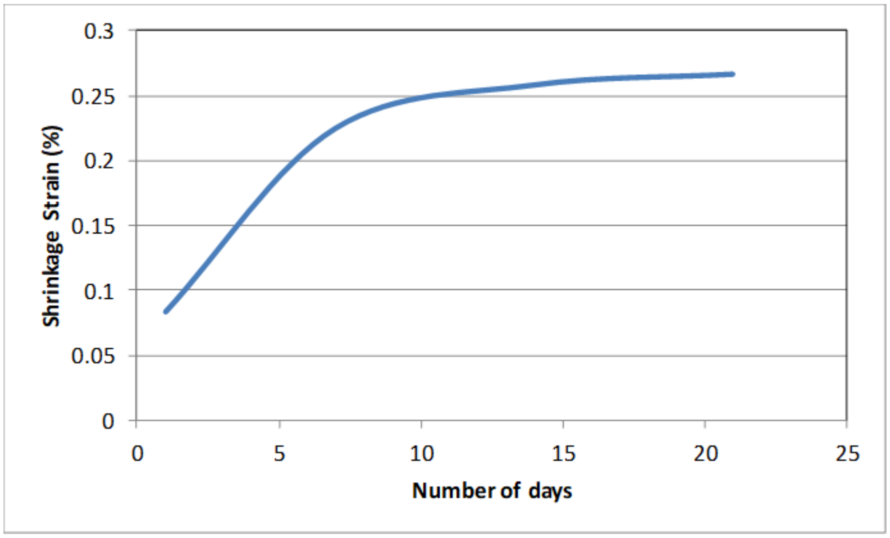


Figure 20. Results of linear shrinkage test for lime treated soil #3

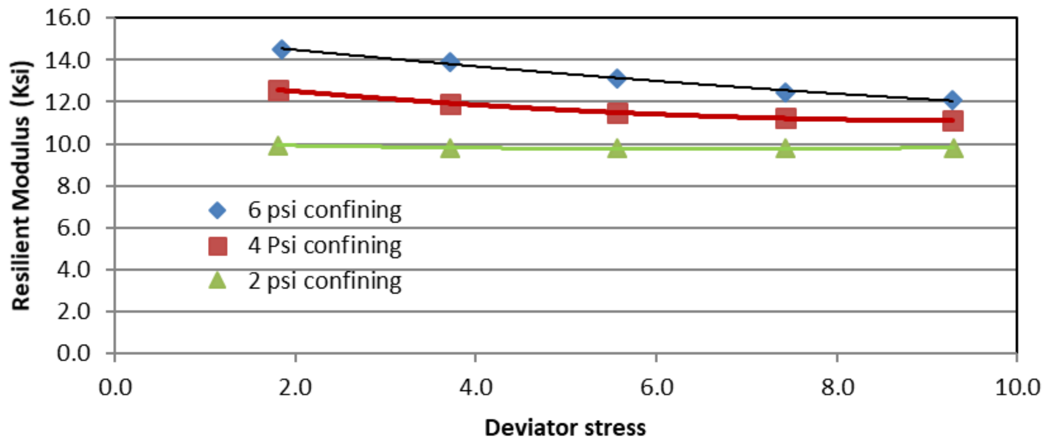


Results of Resilient Modulus Tests

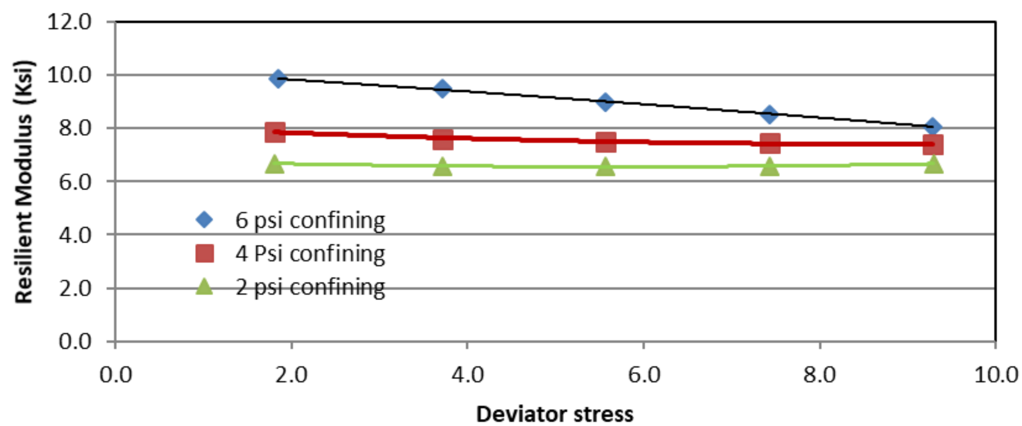
Extensive resilient modulus tests were performed in the laboratory using the MTS machine on the treated/stabilized soil specimens prepared at optimum and optimum ± 2% moisture contents and stabilizer contents according to Table 8. Among the various factors affecting the resilient response of the soil, the effects of stress state and curing time were studied and are discussed here.

Resilient modulus is a key input material property in pavement design. Typical variation of the resilient modulus with stress conditions obtained from the laboratory tests of untreated specimens are presented in Figure 21. From the slope of the curves, it can be inferred that with the increase of deviatoric stress, the resilient modulus of untreated soil decreases. This type of behavior represents strain softening to subgrade materials under increase in deviatoric/cyclic stress. Furthermore, as expected, the confining stress has positive effect on resilient modulus such that an increase in the resilient modulus was observed with the increase in confining stress.

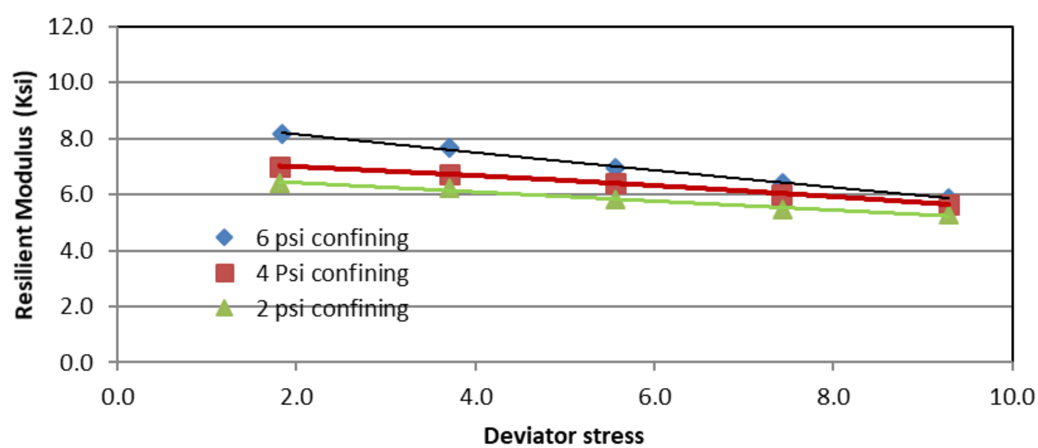
Figure 21. Resilient modulus of untreated soil #2 specimens



(a) OMC - 2%



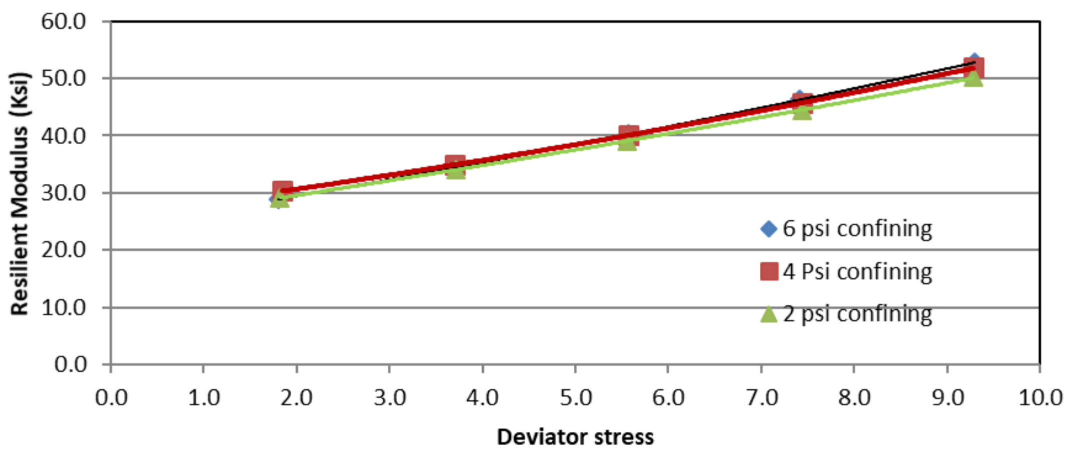
(b) OMC



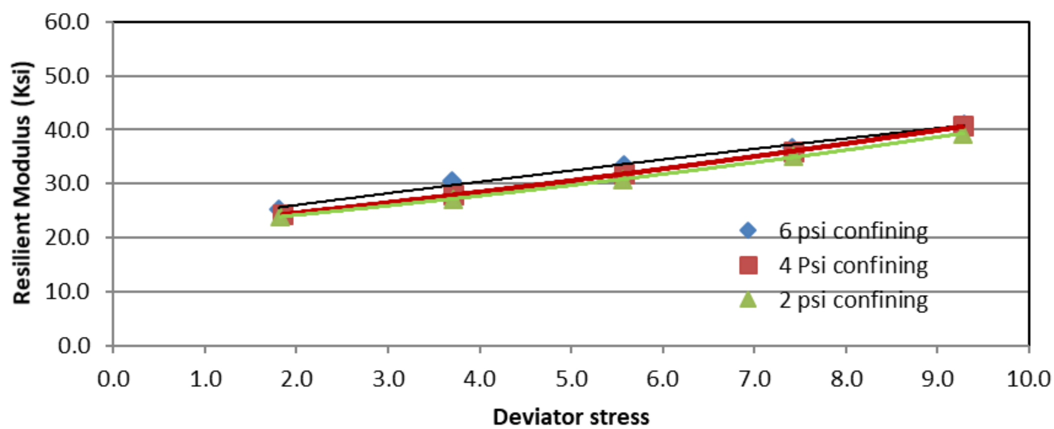
(c) OMC + 2%

Figure 22 and Figure 23 present variation of the resilient modulus with stress conditions for the treated/stabilized soil #2 cured for 7 days and 28 days, respectively. The variation of resilient modulus with stress conditions for the treated/stabilized soil #1 and soil #3 specimens can be found in Appendix A. The results showed that, in contrary to the untreated soil, the resilient modulus of treated soil specimens increases with increasing the deviatoric stress. This type of behavior represents strain hardening under increase in deviatoric/cyclic stress. Furthermore, the confining stress has little effect on the stiffness/modulus of treated soil here.

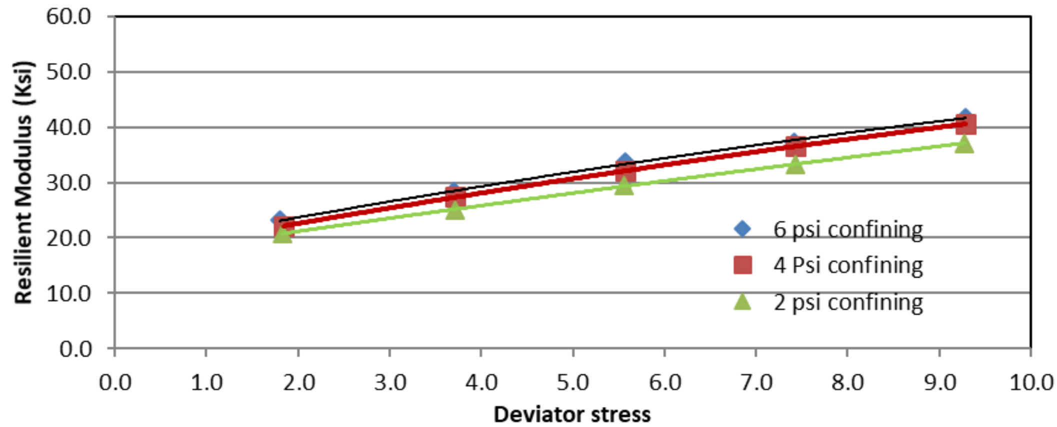
Figure 22. Resilient modulus of treated soil #2 specimens (6% cement + 6% lime, 7 days)



(a) OMC - 2%

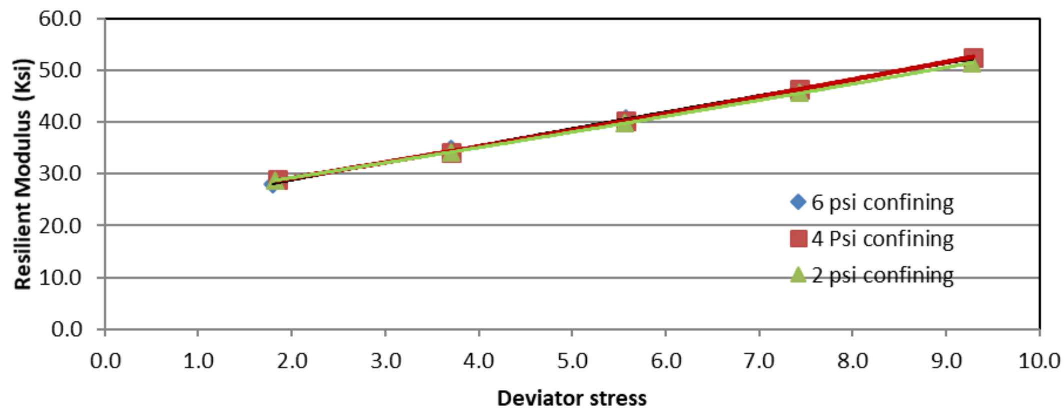


(b) OMC

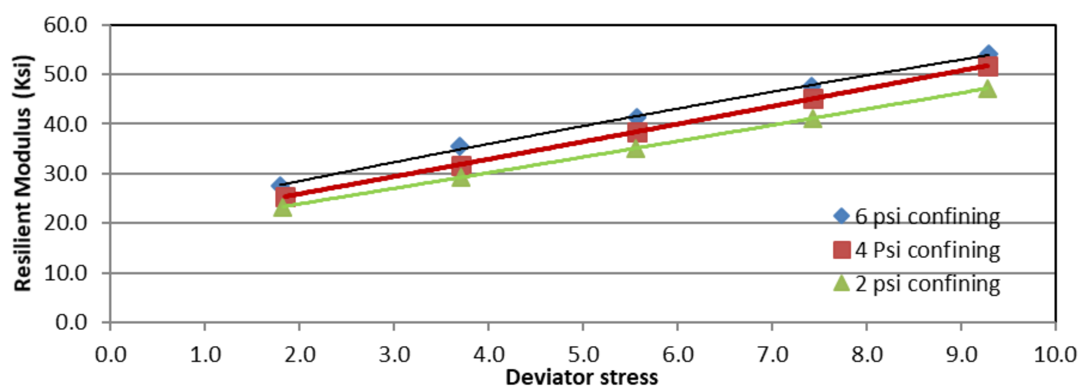


(c) OMC + 2%

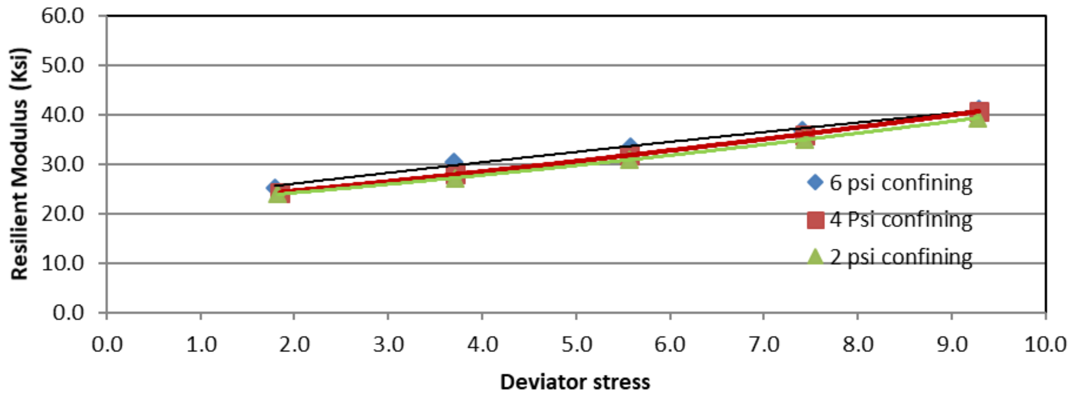
Figure 23. Resilient modulus of treated soil #2 specimens (6% cement + 6% lime, 28 days)



(a) OMC - 2%



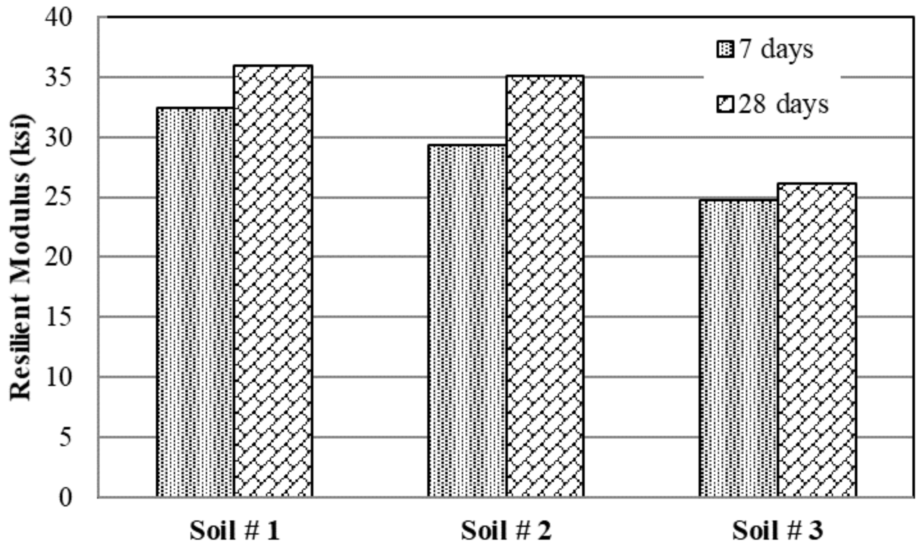
(b) OMC



(c) OMC + 2%

Figure 24 presents the comparison of resilient modulus for soils #1, #2, and #3 obtained at curing periods of 7 days and 28 days. It is apparent that all the three cement and cement-lime treated/stabilized soils showed certain increase in resilient modulus of the specimens tested after 28 days of curing as compared with those cured for 7 days prior to testing. The increase in resilient modulus was about 11%, 18%, and 5% for the cement and cement-lime treated/stabilized soil # 1, soil #2, and #3, respectively, after 28 days of curing time as compared to the resilient modulus obtained after 7 days of curing period.

Figure 24. Variation of resilient modulus with curing time for thee soils



Different types of empirical models have been developed to estimate the M_R at different confining pressures and deviatoric stresses (e.g., [33], [35], [38]). These M_R models account for both external confinement and shear stress effects on the resilient properties. Although all the models were developed for granular and cohesive soils, they have been extended by various researchers for estimating the resilient modulus of cohesive soils and cementitiously treated/stabilized soil [23]. Among all models available in the literature, only four models presented in equations (23), (24), (25), and (26) were considered in this study for evaluating M_R for cementitiously treated/stabilized soils (e.g., [35], [36], [38], [39]).

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(1 + \frac{\tau_{oct}}{p_a}\right)^{k_3} \quad [23]$$

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\sigma_d}{p_a}\right)^{k_3} \quad [24]$$

$$M_R = k_1 p_a \left(\frac{\sigma_{oct}}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a}\right)^{k_3} \quad [25]$$

$$M_R = k_1 p_a \left(1 + \frac{\theta}{p_a}\right)^{k_2} \left(1 + \frac{\sigma_d}{p_a}\right)^{k_3} \quad [26]$$

where, θ is the bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$; τ_{oct} is the octahedral shear stress = $[(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_3 - \sigma_2)^2]^{1/2}/3$; σ_1, σ_2, σ_3 are the major, intermediate, and minor principal stresses, respectively; p_a is atmospheric pressure; and k_1, k_2, k_3 are model constants.

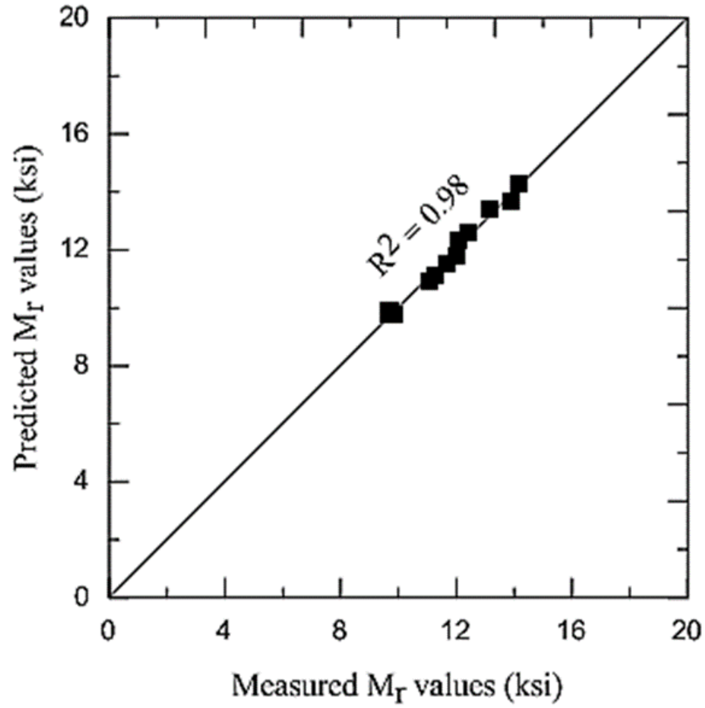
Table 20 presents the average model constants (k_1, k_2, and k_3) obtained for the three soils using the model recommended by the AASHTO 2002 MEPDG [33], i.e. Equation (23). These constants represent the average results of three triplicate specimens. These values can be used for pavement design and analysis provided the state of stress is known from layered elastic analysis, finite element analysis, or any other means. The k_2 coefficient describes the stiffening (higher modulus) of the material with the increase in the bulk stress. It is noted from Table 20 that all k_2 coefficients were less than 1 for all untreated soils. This indicates that the effect of bulk stress decreases with increasing magnitude. It is also noted from Table 20 that the value of most k_2 coefficients for treated soils are trivial. This suggests that the bulk stress has minimal effect on the resilient modulus of treated soils here. Table 20 shows that all k_3 coefficients were negative for untreated soils. This means the weakening of the untreated soils (lower modulus) with the increase in the shear stress. On the other hand, the k_3 coefficients are positive for all treated soils, which suggests the stiffening and hardening of the treated soils (higher modulus) with the increase in the shear stress. Table 20 also shows that the magnitude of regression coefficients k_1, k_2, and k_3 are largely dependent on the soil type and stabilizer content. This suggests that the specimens have similar UCS values but with different types of soil and stabilizer can have different resilient characteristics. As such, the use of direct correlation between the UCS and the resilient modulus for cementitiously treated/stabilized soils can be misleading and should be carefully used in pavement design.

The M_R values predicted using the MEPDG [39] model for untreated and 7 days treated/stabilized soil #2 specimens prepared at OMC ± 2% were compared with the laboratory measured M_R values as shown in Figure 25 and 26, respectively. The figures clearly demonstrate the accuracy of using the MEPDG M_R model and the corresponding correlated k_1, k_2, and k_3 constants for estimating M_R for untreated and treated/stabilized soil specimens ($R^2 > 95\%$).

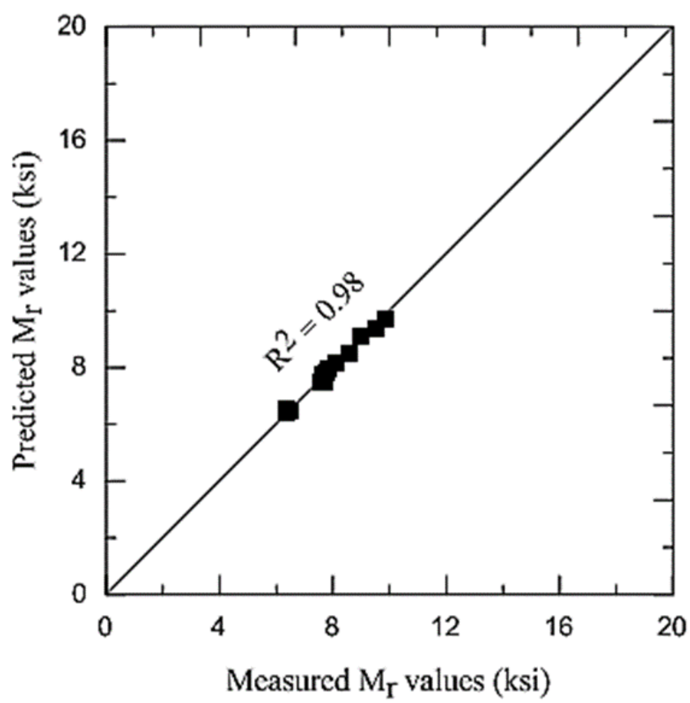
Table 20. AASHTO 2002 [32] MEPDG model constants for the three soils (Phase I)

Soil #	Model Constants	Untreated					
		OMC-2	**OMC**	**OMC+2%**	**OMC-2**	**OMC**	**OMC+2%**
I	k1	975.28	564.28	295.62	-	-	-
	k2	0.45	0.37	0.23	-	-	-
	k3	-2.07	-1.89	-1.61	-	-	-
II	k1	946.70	624.61	564.64	-	-	-
	k2	0.42	0.44	0.26	-	-	-
	k3	-1.47	-1.42	-1.76	-	-	-
III	k1	1032.36	793.45	378.03	-	-	-
	k2	0.36	0.44	-0.01	-	-	-
	k3	-2.68	-3.04	-2.79	-	-	-
		Treated					
		7 days			28 days		
I	k1	3073.22	2603.68	2574.68	-	3367.89	-
	k2	0.01	-0.03	0.02	-	-0.02	-
	k3	2.75	2.91	2.64	-	2.62	-
II	k1	3808.33	2746.80	2715.53	3693.29	3432.35	3281.89
	k2	0.05	0.18	0.04	0.02	0.23	0.10
	k3	2.48	2.52	2.49	2.69	2.59	2.06
III	k1	2107.60	2149.58	1987.64	-	2718.14	-
	k2	-0.01	0.01	0.07	-	0.09	-
	k3	3.53	2.82	2.27	-	1.94	-

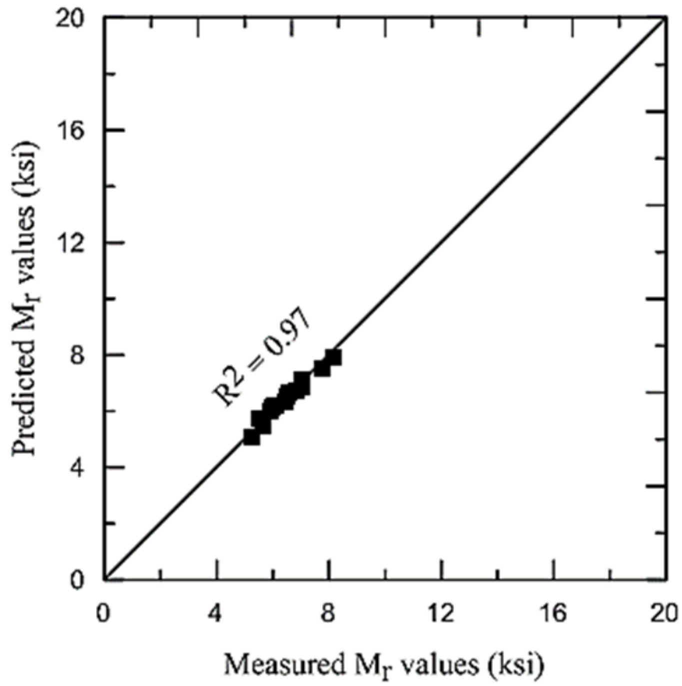
Figure 25. Measured versus predicted resilient modulus for soil #2 (untreated) using MEPDG model [39]



(a) OMC - 2%

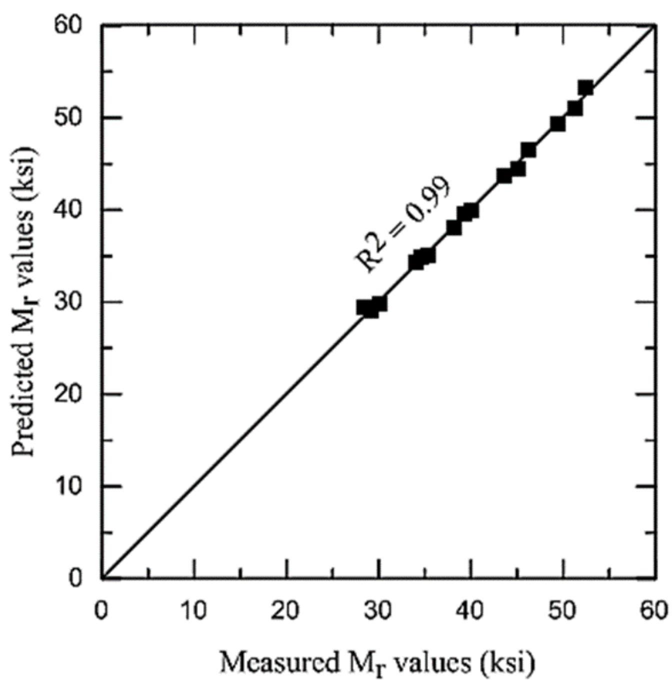


(b) OMC

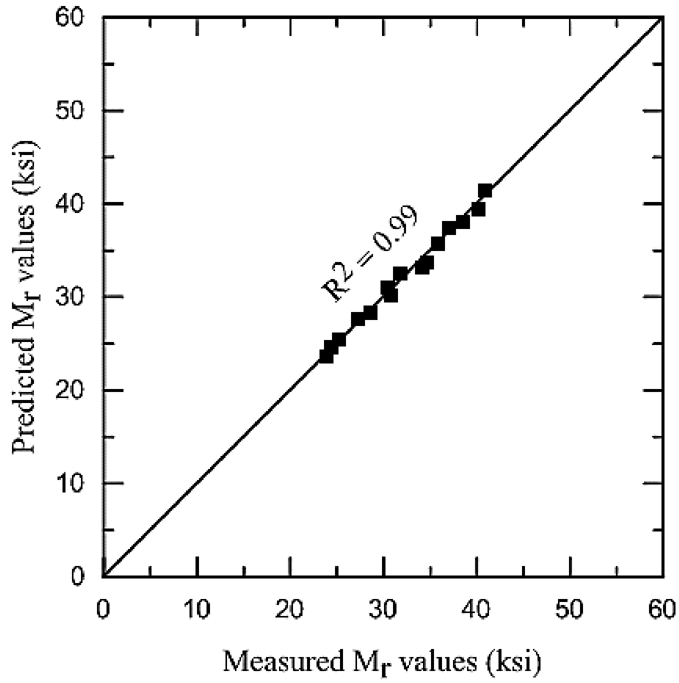


(c) OMC + 2%

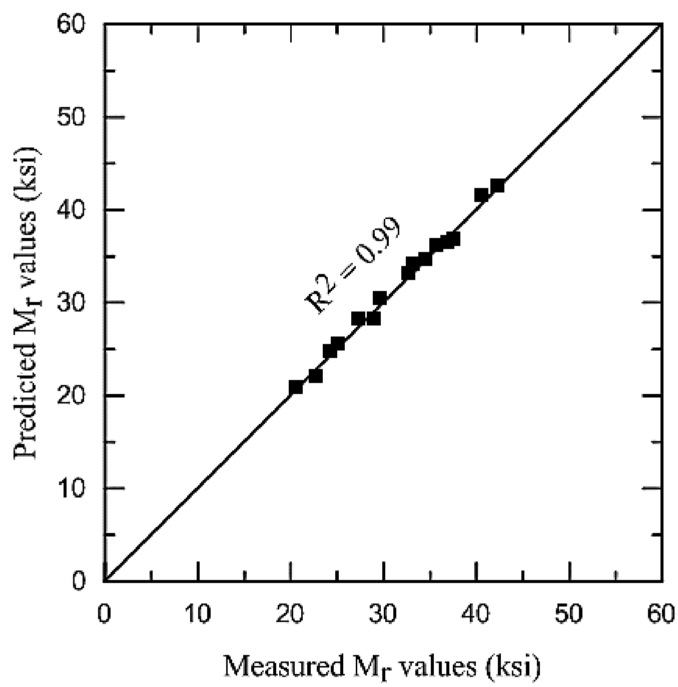
Figure 26. Measured versus predicted resilient modulus for soil #2 (treated 7 days) using MEPDG model [39]



(a) OMC - 2%


R2 = 0.99
Predicted Mr values (ksi)
Measured Mr values (ksi)
0
10
20
30
40
50
60

(b) OMC


R2 = 0.99
Predicted Mr values (ksi)
Measured Mr values (ksi)
0
10
20
30
40
50
60

(c) OMC + 2%

Results of Single-stage Permanent Deformation Tests

Repeated load triaxial tests were performed to evaluate the permanent (axial) deformation behavior of the different untreated and treated/stabilized subgrade soil specimens. Figure 27 and Figure 28 present the typical curves of the average permanent (axial) strain versus the number of cycles obtained for the different RLT cases for the untreated and treated/stabilized soil #2. The results here represent the average of two replicate specimens. The results of single-stage permanent deformation tests for the untreated and treated/stabilized soil #1 and soil #3 specimens can be found in Appendix B. The permanent (axial) deformation curve has two distinct stages. In the first stage (post-compaction stage), the material accumulates a significant amount of permanent deformation. This is most probably due to extra compaction and initial particle bonding breakage induced particle re-arrangement. During the second stage (secondary stage), the material accumulates permanent strain at a much lower rate and even in some cases the permanent deformation approaches a constant value. The permanent (axial) strains observed for all treated soil specimens after 10,000 cycles of loading are summarized in Table 21. The treated/stabilized soils experienced significantly less permanent deformations than the raw soils for all three soil types. The figures also show that the permanent deformation of treated/stabilized soils is somehow negligible (permanent strain < 0.07%) that might be ignored in pavement design. This observation is consistent with the UCS test results, which show that the stress-strain curves of treated/stabilized soils shifted towards the left hand side (brittle behavior) as the percent of stabilizer content increases (Figure 16). This is also in agreement with the MEPDG, which does not consider the deformations of cement stabilized layers in pavement design.

Figure 27. Single-stage permanent deformation results for soil #2 (treated and untreated specimens)

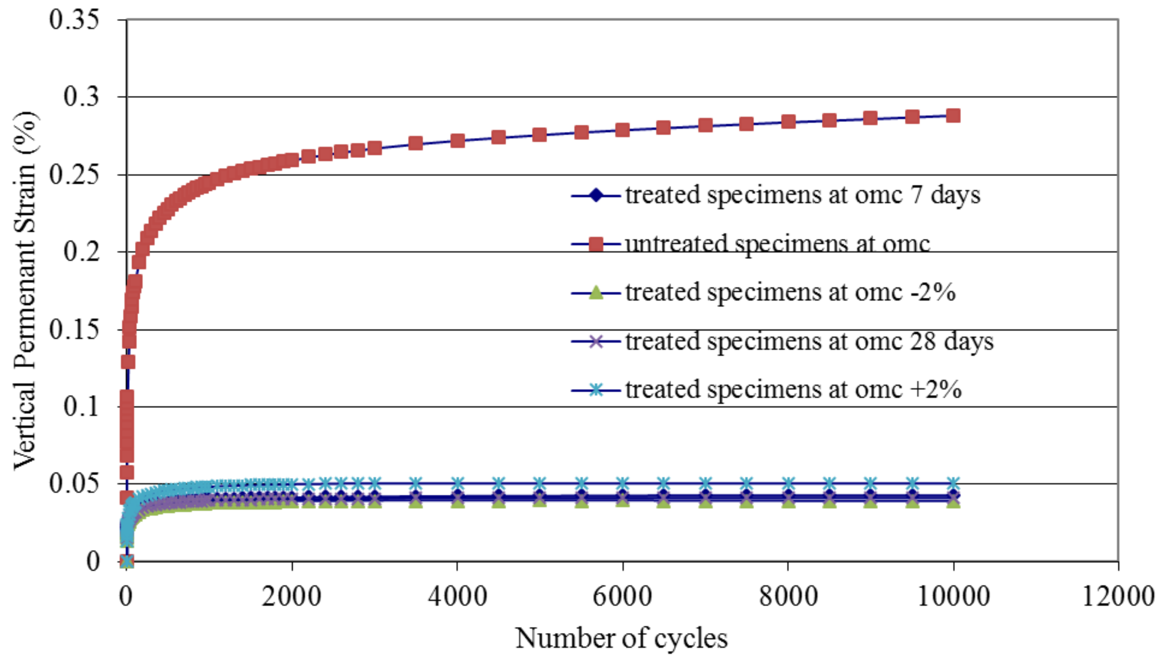


Figure 28. Single-stage permanent deformation results for soil #2 (only treated specimens at different moisture content)

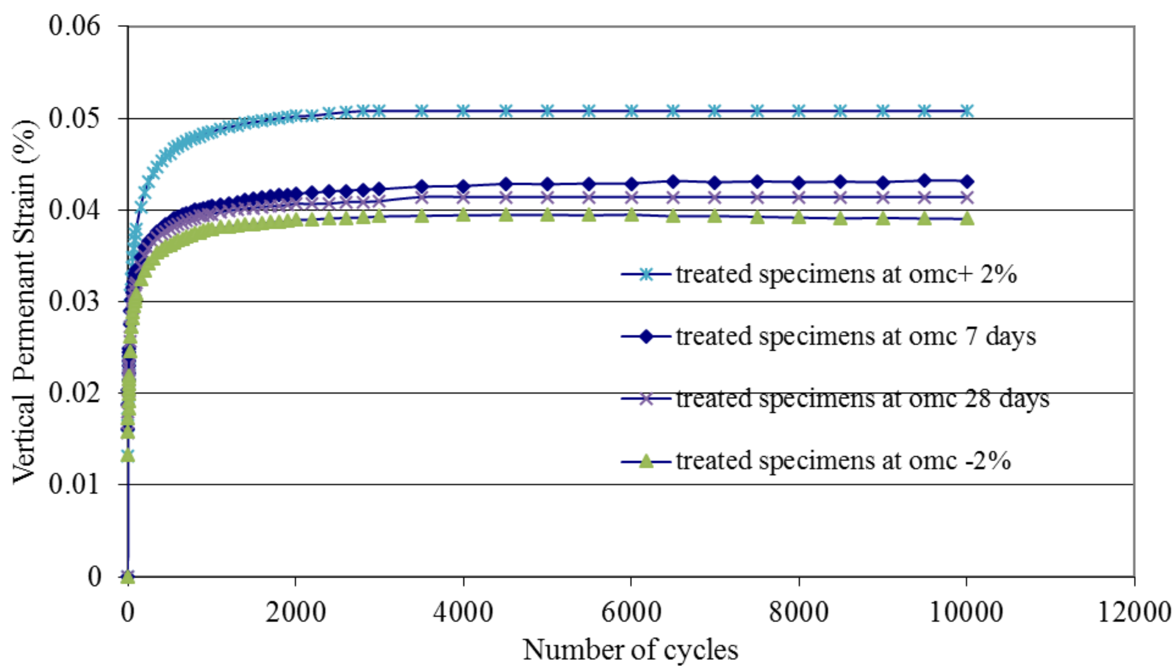


Table 21. Vertical permanent strain of specimens at the 10,000th cycle

	Untreated					
Soil #	**OMC-2%**	**OMC**	**OMC+2%**	**OMC-2%**	**OMC**	**OMC+2%**
I	-	0.580	-	-	-	-
II	-	0.290	-	-	-	-
III	-	3.000	-	-	-	-
	Treated					
	7 days			**28 days**		
I	0.022	0.044	0.058	-	0.030	-
II	0.039	0.043	0.050	-	0.040	-
III	0.042	0.064	0.061	-	0.044	-

Results of Multi-stage Permanent Deformation Tests

Multi-stage permanent deformation tests were conducted to evaluate the behavior of treated/stabilized soil specimens under different stress levels and to determine the shakedown limits by different stages of loading. The vertical permanent strain obtained throughout the six different stages of loading were plotted against the vertical strain rate (strain/ load cycles) to evaluate the shakedown limits of the soil specimens. The general plots of the measured permanent strains versus number of cycles observed strains of specimens at different moisture contents for soil #2 are presented in Figure 29 to Figure 33. The results of multi-stage permanent deformation tests for the untreated and treated/stabilized specimens for soil #1 and soil #3 are presented in Appendix C. Higher permanent strains were observed with higher stress ratio in untreated soils, which is not the case for treated soils. This may be due the change of deviatoric stress under the same confining stress in this study is so small comparing to the strength of treated/stabilized soils that the breakage of new bond between stabilizer and soil caused by applying higher deviatoric stress is less likely. Under identical stress ratio, the specimens tested at higher confining stress showed higher measured permanent strains. This is because under similar stress ratio (σ_1/σ_3), higher confining stress resulted in higher deviatoric stress. For example, at $\sigma_1/\sigma_3 = 6$, the corresponding deviatoric stress applied are 10 psi, 20 psi, and 30 psi for confining stress of 2 psi, 4 psi, and 6 psi, respectively. Therefore, the larger measured permanent strain for high confining stress is mainly due to the application of larger deviatoric stress.

Figure 29. Multi-stage permanent deformation soil #2 untreated at OMC

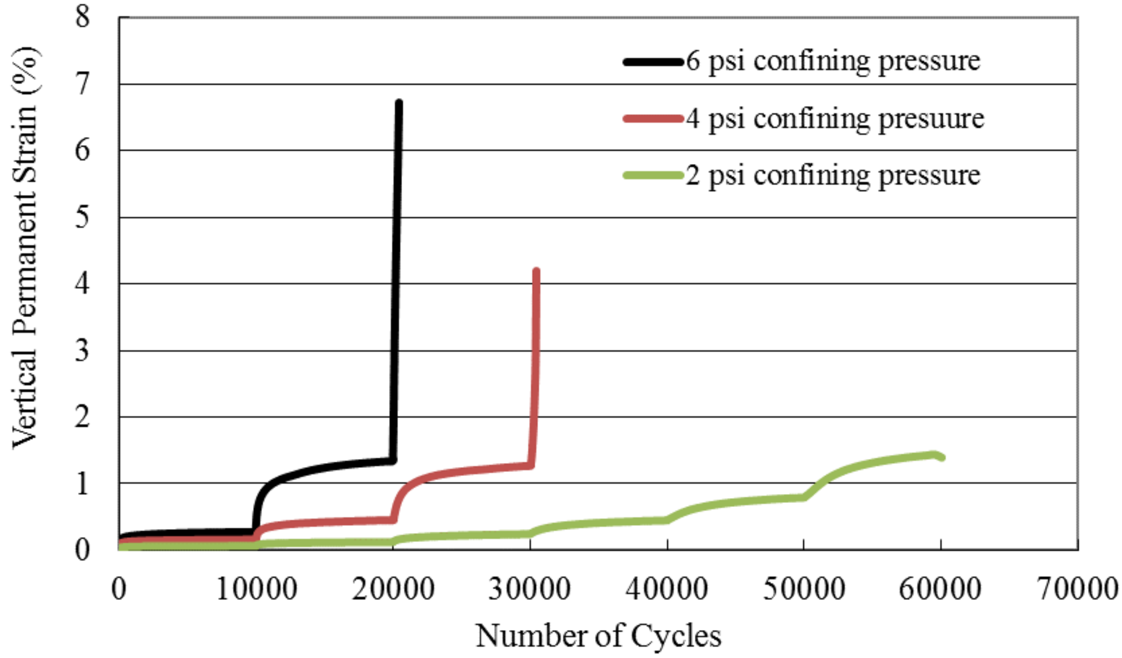


Figure 30. Multi-stage permanent deformation soil #2 treated with 6% cement, 6% lime at OMC (7 days)

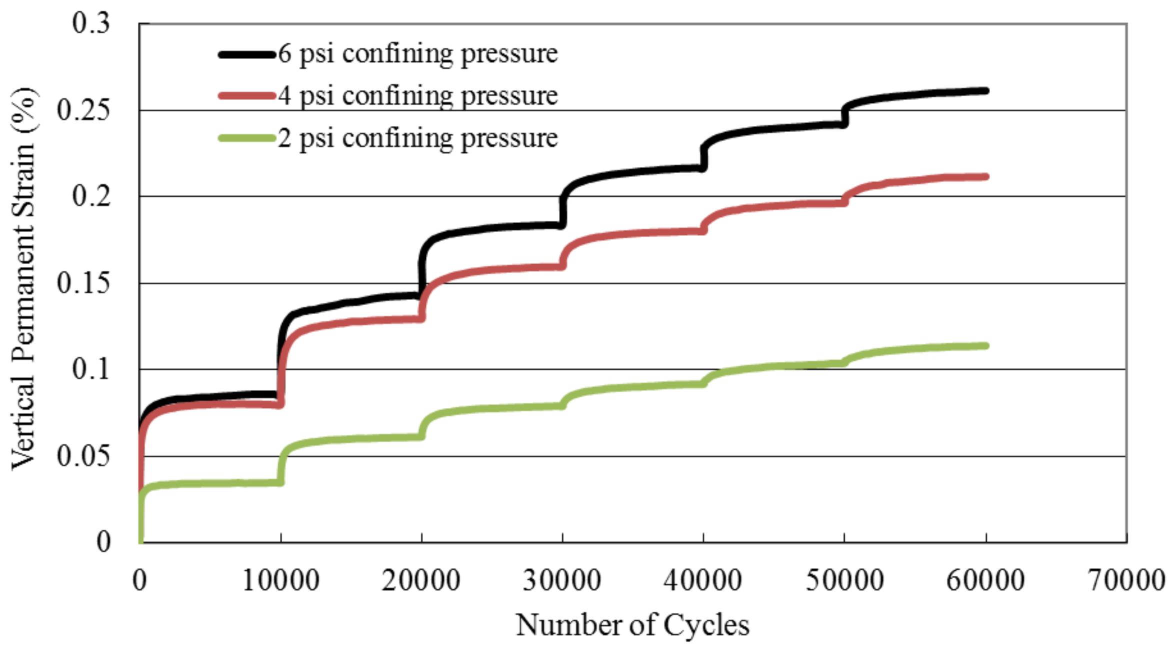


Figure 31. Multi-stage permanent deformation soil #2 treated with 6% cement, 6% lime at OMC+2% (7 days)

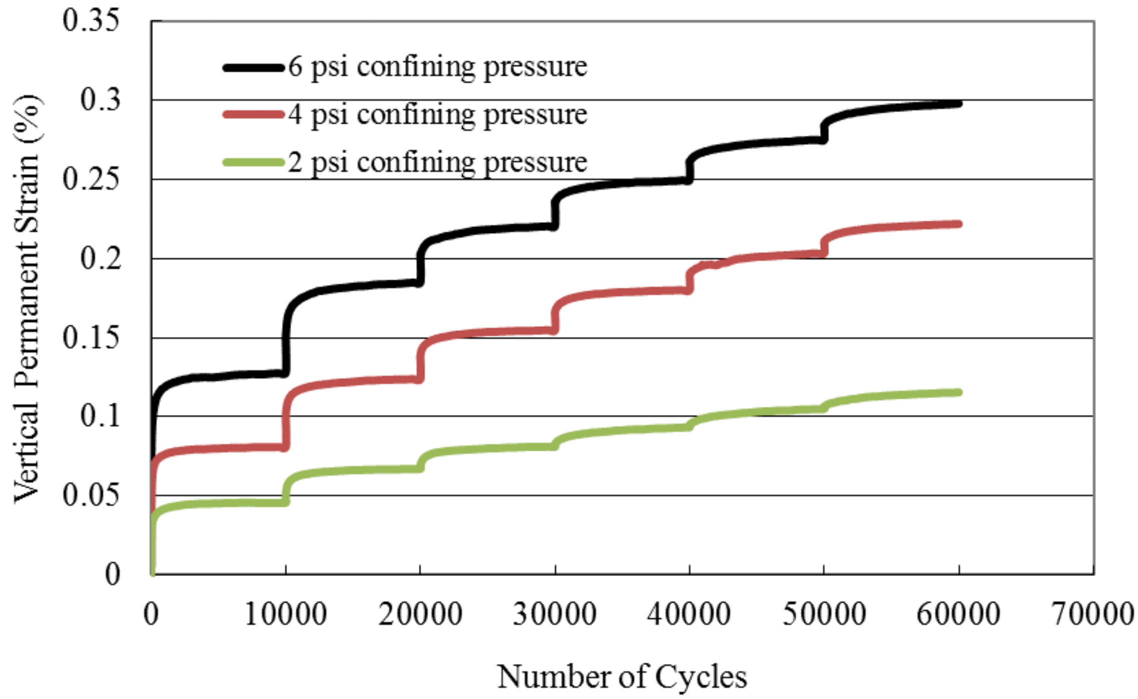


Figure 32. Multi-stage permanent deformation soil #2 treated with 6% cement, 6% lime at OMC-2% (7 days)

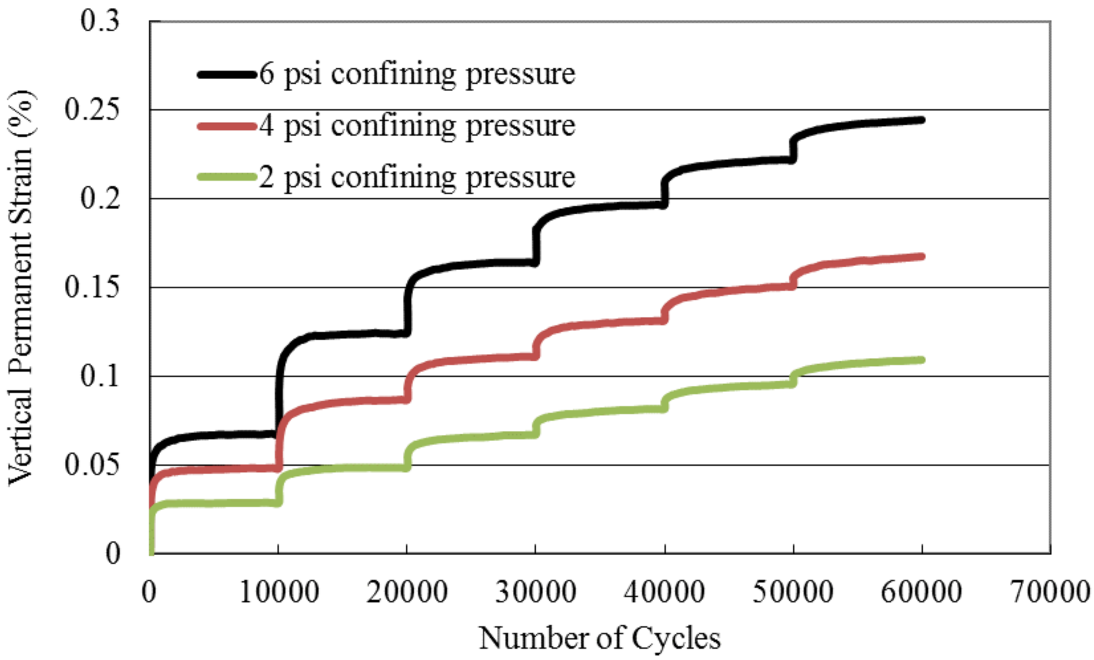
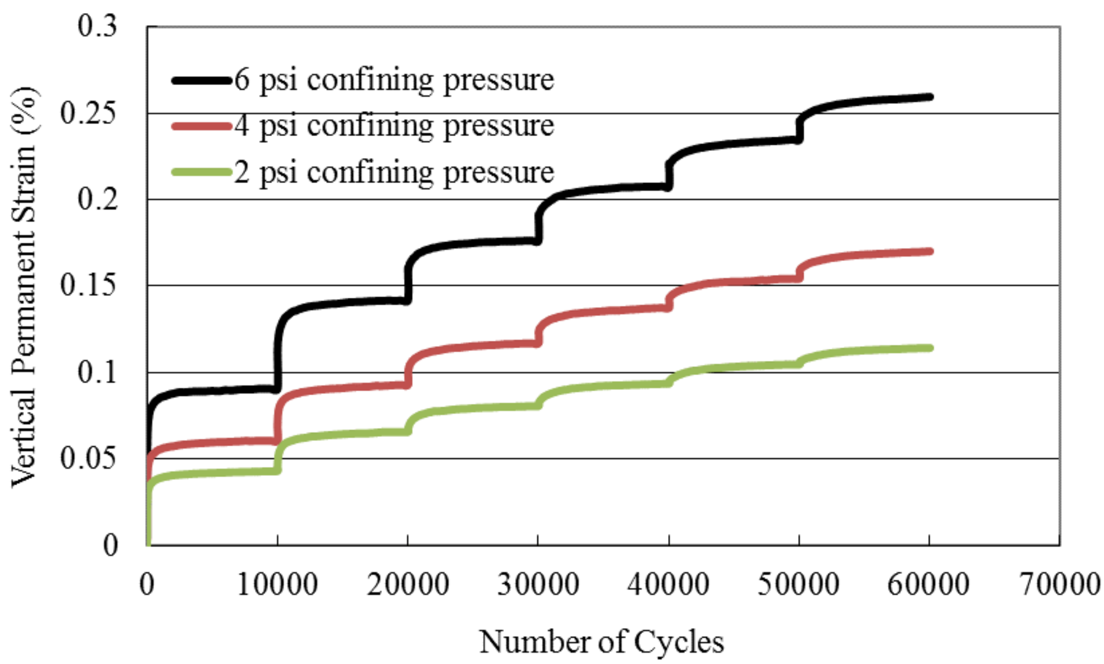


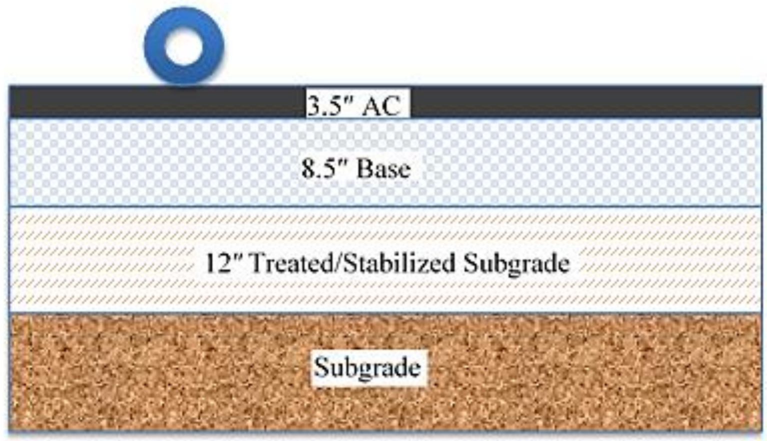
Figure 33. Multi-stage permanent deformation soil #2 treated with 6% cement, 6% lime at OMC (28 days)



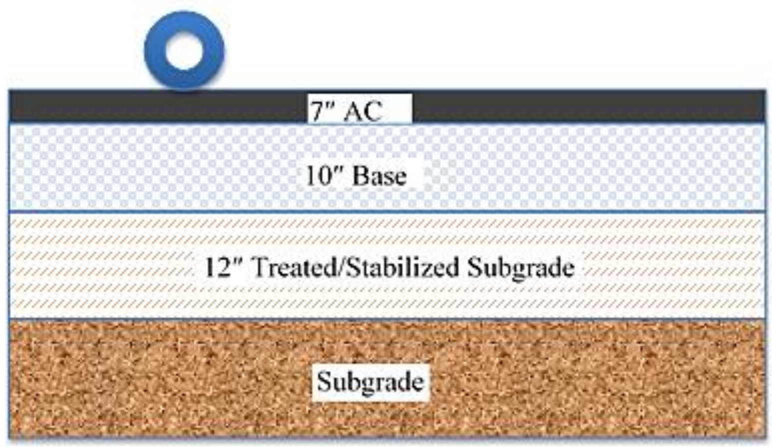
Resilient Modulus Inputs of Flexible Pavement Systems

The results of the resilient modulus tests obtained from laboratory testing of the treated/stabilized subgrade soils can be fed into the MEPDG analysis of flexible pavement sections. The typical flexible pavement sections for use in low-volume, medium-volume, and high-volume roads are presented in Figure 34 with additional stabilized subgrade layer. All the three pavement systems consist of four main layers: asphalt concrete top layer, an unbound aggregate or soil-cement base, a stabilized subgrade layer, and bottom natural subgrade. The confining and deviatoric stresses for the treated/stabilized subgrade layers present in the three typical pavement structures were first calculated using KENLAYER. The resilient moduli were then estimated using the M_R constants (k_1, k_2, and k_3) for the AASHTO 2002 [33] MEPDG model presented in Table 20. The corresponding M_R values are presented in Table 22.

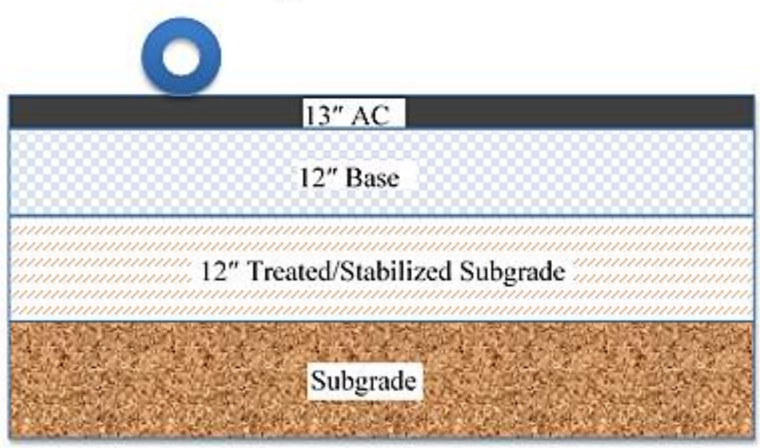
Figure 34. Typical flexible pavement sections



(a) low-volume



(b) Medium-volume



(c) High-volume

Table 22. Recommended resilient modulus (ksi) for treated/stabilized subgrade (Phase I)

Soil Type	Treatment	Low-volume		Medium-volume		High-volume	
		Stone Base	Soil Cement Base	Stone Base	Soil Cement Base	Stone Base	Soil Cement Base
PI≤15	6% cement	30	28	27	26	25	25
16≤PI≤25	6% lime and 6% cement	32	29	28	27	26	26
26≤PI≤35	9% lime and 6% cement	24	22	21	20	20	20

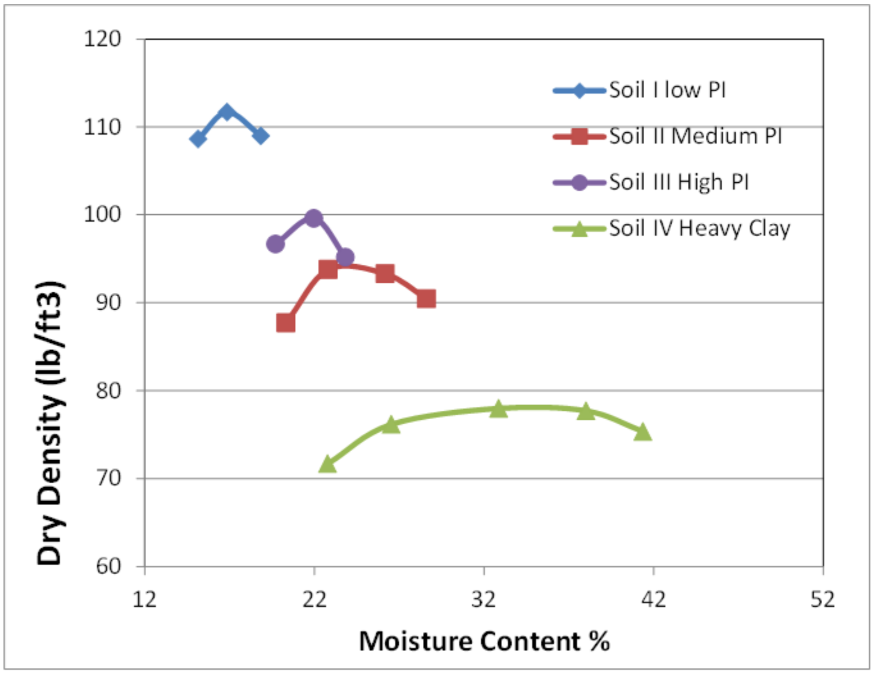
Phase II-a

Phase II testing focused on examining the appropriate treatment/stabilization schemes for very weak subgrade soils at high water content conditions (i.e., at wet-side of optimum of nearly saturated condition) with unconfined compressive strength < 25 psi (Phase 2). Four types of subgrade soils were considered: silty soil, low-plasticity silty clay (PI ≤ 15), medium-plasticity silty clay (PI = 16-25), high-plasticity silty clay (PI = 26-35), and heavy clay (PI > 35), designated as soil I, soil II, soil III, and soil IV, were included in Phase II of this study (Table 14). The performance-related properties (e.g., the resilient modulus and permanent deformation) of the treated/stabilized very weak subgrade layers (M_R)] were characterized at two UCS target values, 50 psi for working platform, and 100 psi for subgrade stabilization, for use in the design and analysis of pavements.

Moisture-density Relationship

Standard Proctor tests were performed on all four raw subgrade soils (i.e., soils I, II, III, and IV). The compaction curves are presented in Figure 35. Their maximum dry unit weights were 111.7 pcf, 99.6 pcf, 93.9 pcf and 78.4 pcf, respectively, at the corresponding optimum moisture content of 16.9 percent, 22.0 percent, 24.0 percent, and 35.5 percent.

Figure 35. Compaction curves of raw soils



Selection of Moisture Contents

A series of UCS tests were conducted on the raw soils prepared at different moisture contents of the wet-side of optimum. Three replicate specimens were tested for each moisture content case. The variation of the average UCS with moisture contents for the four raw soil are presented in Figure 36 to 39. These figures are important in the selection of the moisture contents for sample preparation for later UCS and RLT tests of the treated/stabilized subgrade soils in this study. Based on the results of UCS tests, sets of three moisture contents producing soil strength of 25 psi or less (to represent very weak wet soils) were selected for each of the four raw subgrade soils for use in the treatment/stabilization process and evaluation, as summarized in Table 23.

Figure 36. Variation of UCS with moisture content for soil I

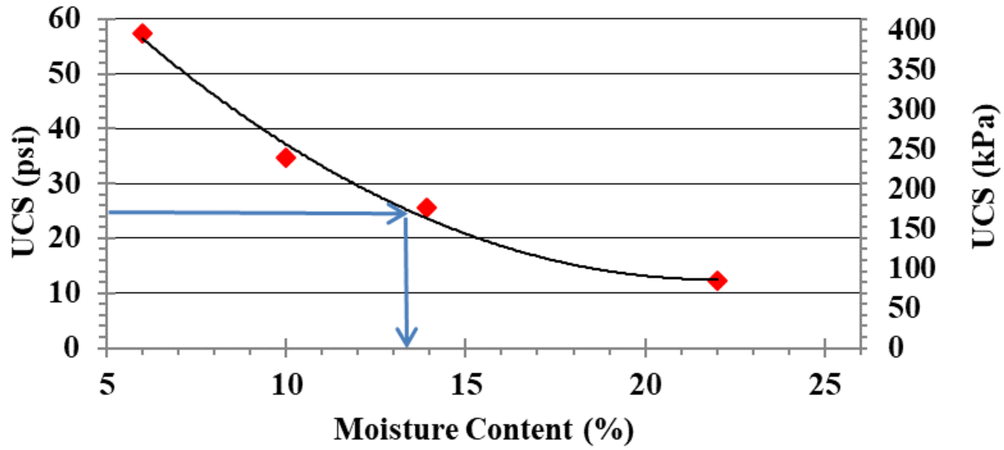


Figure 37. Variation of UCS with moisture content for soil II

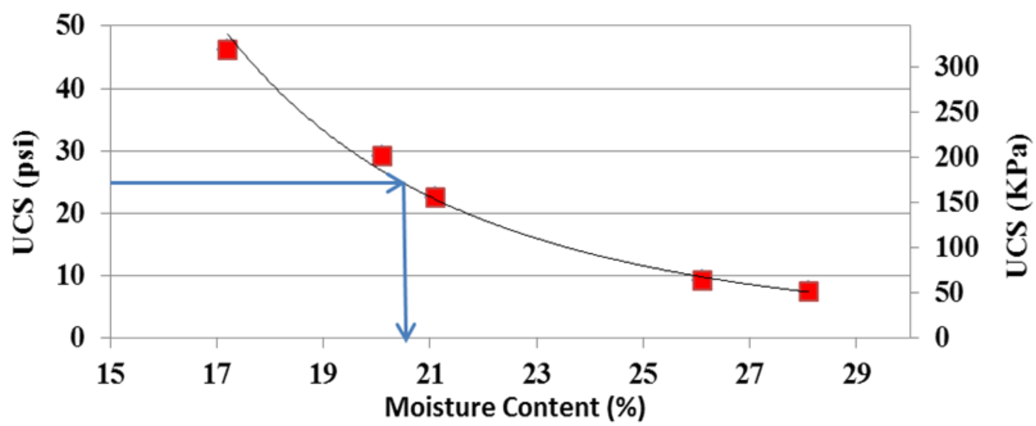


Figure 38. Variation of UCS with moisture content for soil III

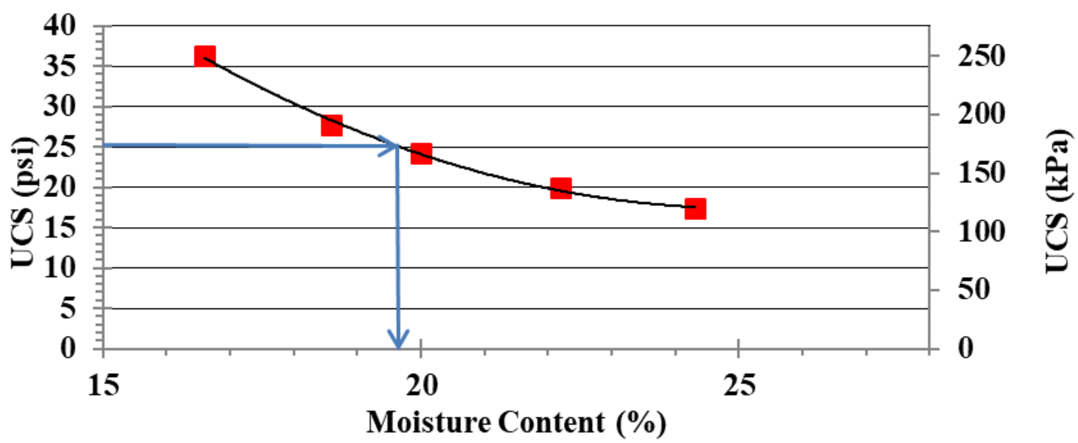


Figure 39. Variation of UCS with moisture content for soil IV

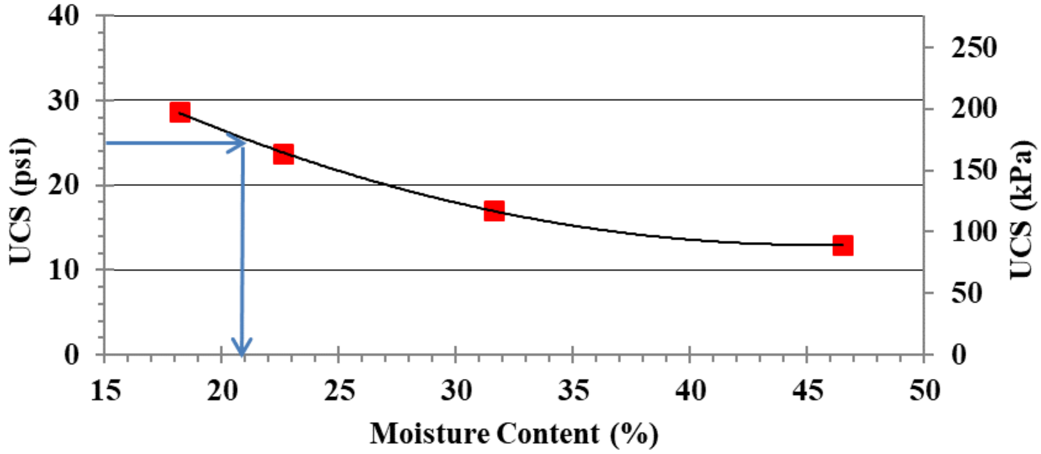


Table 23. Summary of selected working moisture contents

Soil #	Soil Name	MC1	MC2	MC3
I	Low PI	20%	23%	26%
II	Medium PI	26%	28%	30%
III	High PI	24%	28%	32%
IV	Heavy clay	38%	42%	46%

Selection of Stabilizer Contents

The treated/stabilized soil specimens were prepared by mixing raw soils with different percentage of stabilizer (cement, lime, lime-cement, or lime-fly ash) at the selected moisture contents that produce a UCS of 25 psi or less for raw subgrade soil, as shown in Table 23. Part of the screening process, the 7-day UCS tests were then performed on these specimens to determine the percentage of stabilizer doses needed to achieve the target UCS values of 50 psi (for working platform) and 100 psi (for subgrade stabilization). For the low to medium plasticity soils (I and II), Louisiana experience indicated that cement treatment works much better than lime. However, in this study a combination of lime and fly ash was tried to treat the soils for the target UCS value of 50 psi. For the high PI and heavy clayey soils (III and IV), both lime and cement were tried first alone to treat/stabilize the soils. While adding lime alone cannot bring the strength of the soil up to the target values, the cement alone did not improve the workability of the soil and its mixing characteristics significantly. As such, a combination of lime and cement or lime and fly ash with different mixing ratios was tried to treat/stabilize high plasticity soils.

Each soil was first treated by lime and fly ash combination to obtain the required quantities that gives 50 psi and 100 psi UCS, provided that the maximum stabilizer content does not exceed 30% by volume. However, using these criteria, the fly ash and lime combination did not increase the UCS up to 100 psi with stabilizer content less than 30% by volume. In order to achieve the UCS value of 100 psi without violating the maximum stabilizer content of 30% by volume, some soils were treated by a combination of cement and lime.

Figure 40 to 43 present the variations of 7-day UCS values obtained at different combination of stabilizers for soils I to soil IV, respectively, prepared at MC1 in Table 23. Similar UCS tests were also performed on the four soil types prepared at the other selected moisture contents in Table 23 using different combinations of stabilizers. The first screening process was used to identify the best combinations of stabilizers (cement, lime, lime-cement, or lime-fly ash) for each soil at the selected moisture contents. Next, researchers determined the right percent doze by volume of the selected stabilizer combination at the mixing moisture contents in Table 23 for each of two target USC values (50 psi versus 100 psi).

Figure 44 presents the selection of additive contents by volume (for 1 lime: 3 fly ash combination) for soil I prepared at the three selected moisture contents (i.e., 20%, 23% and 26%) and for the 50 psi UCS target value. As seen in the figure, more than 30% per volume of 1 lime : 3 fly ash additive is needed to achieve the 100 psi UCS target value, which is not practical. Therefore, cement was selected to achieve a 100 psi UCS for soil I (low PI soil) as can be seen in Figure 45. The selection of additive contents for soil II for the three moisture contents are presented in Figure 46 and 47 for the target USC values of 50 psi and 100 psi, respectively.

The selection of additive contents for soil III for the moisture contents 24% and 28% are presented in Figure 48 for the target USC values of 50 psi. The additive content exceeds 30% for the moisture content of 32%. Therefore, screening was done again using cement and lime combination, as shown in Figure 49. Figure 50 present the selection of additive contents for soil III at moisture content = 32% for a target UCS of 50 psi, and the additive contents for the three moisture contents at 100 psi UCS value.

The selection of additive contents for soil IV at the moisture contents and for both 50 psi and 100 psi target USC are presented in Figure 51. The mixing ratio, having the least volume while achieving the target value of UCS, was selected to treat/stabilize soil for RLT tests. Based on these results, the final selection of stabilizer type and contents by volume for the four different soils and three different moisture contents are presented on Table 24.

Figure 40. Variation of UCS with stabilizer content per volume and mixing ratio for soil I

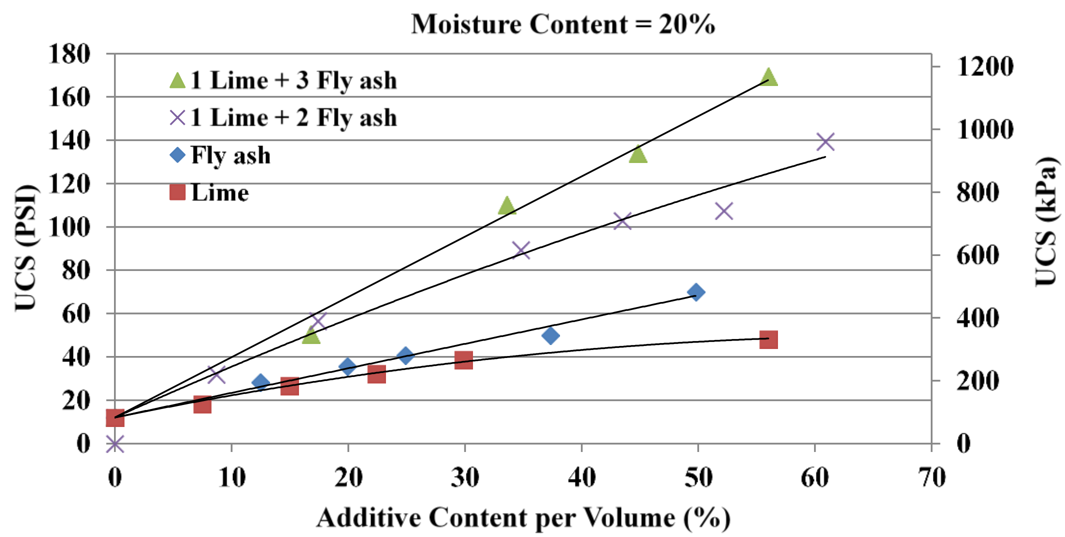


Figure 41. Variation of UCS with stabilizer content per volume and mixing ratio for soil II

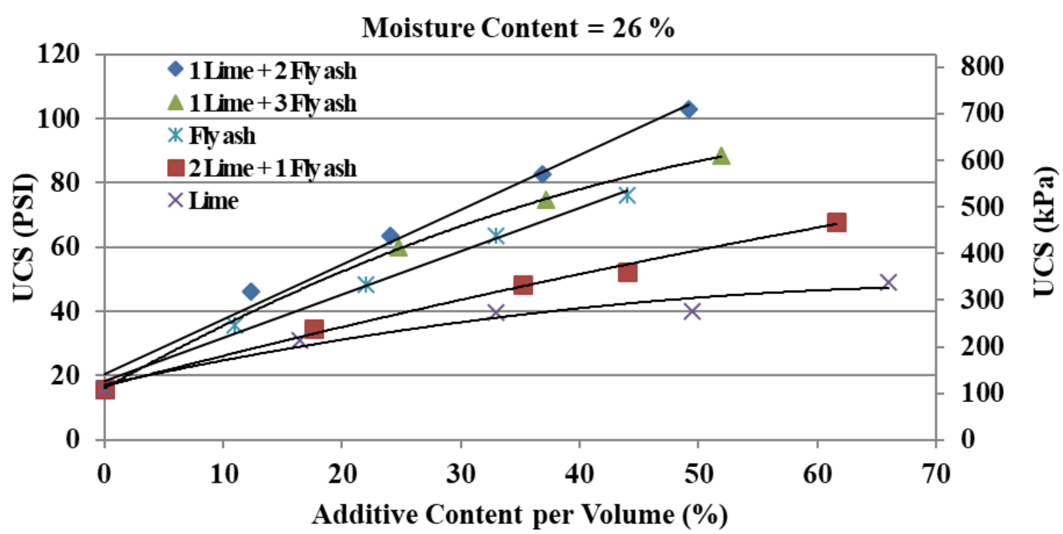


Figure 42. Variation of UCS with stabilizer content per volume and mixing ratio for soil III

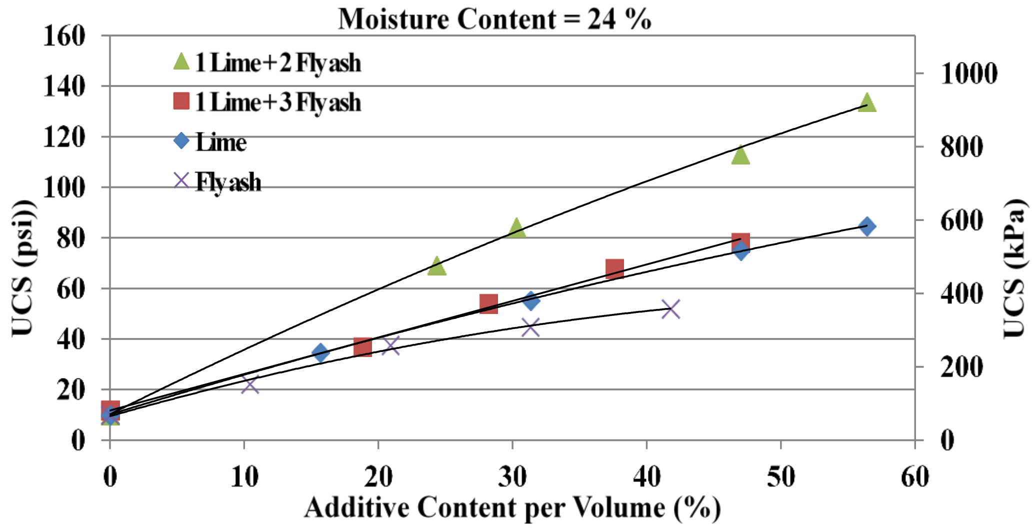


Figure 43. Variation of UCS with stabilizer content per volume and mixing ratio for soil IV

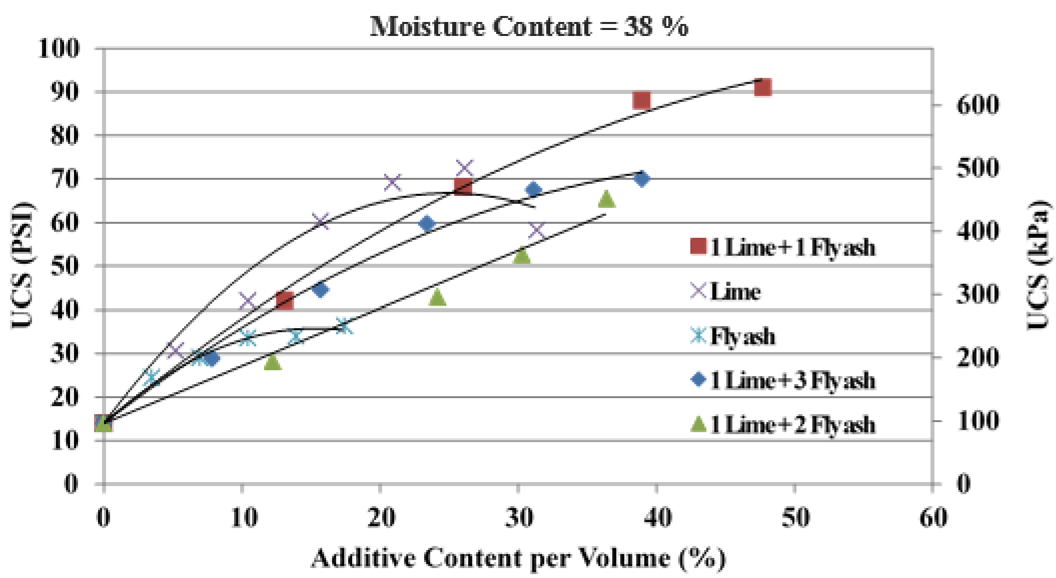


Figure 44. Additive contents for soil I at 50 psi UCS (1 lime: 3 fly ash)

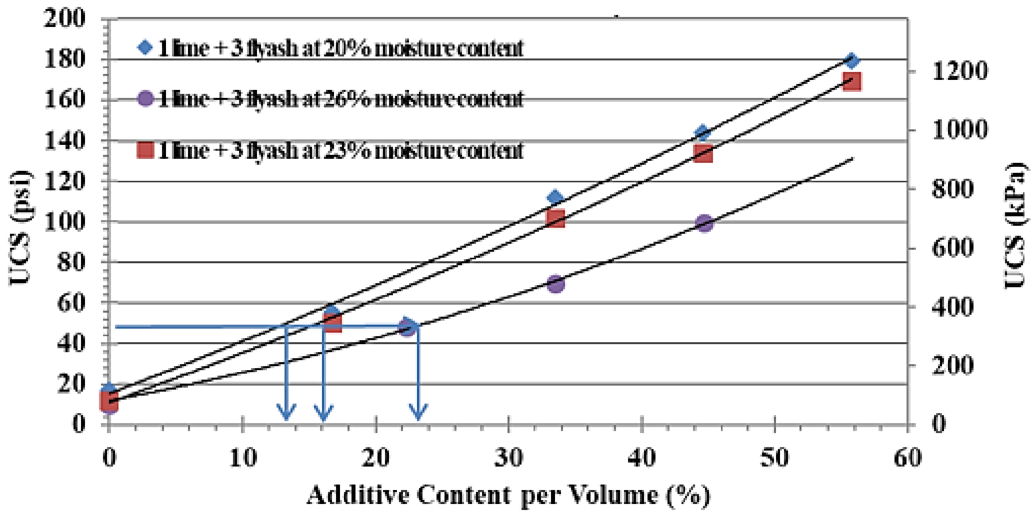


Figure 45. Additive contents for soil I at 100 psi UCS (cement only)

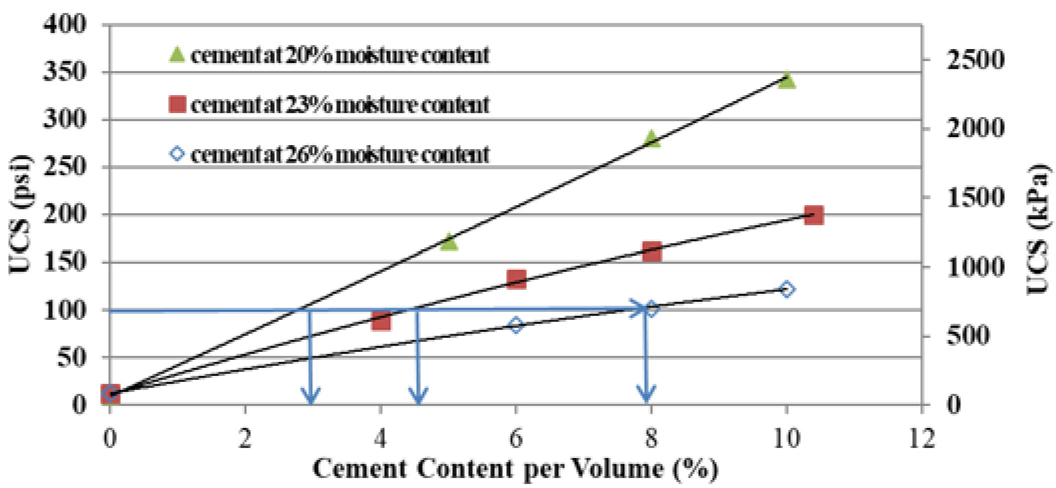


Figure 46. Additive contents for soil II at 50 psi UCS (1 lime: 2 fly ash)

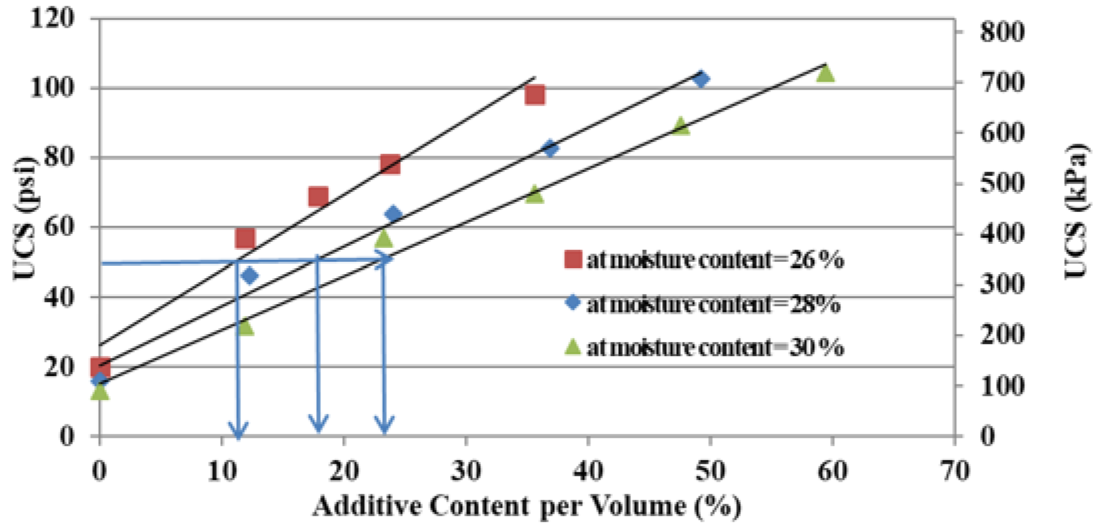


Figure 47. Additive contents for soil II at 100 psi UCS (cement only)

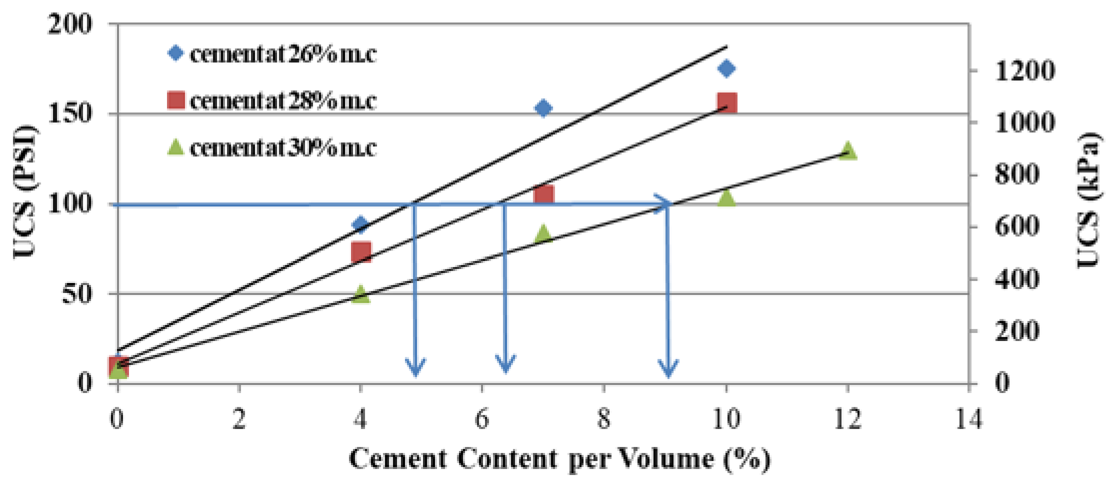


Figure 48. Additive contents for soil III at 50 psi UCS (1 lime: 2 fly ash)

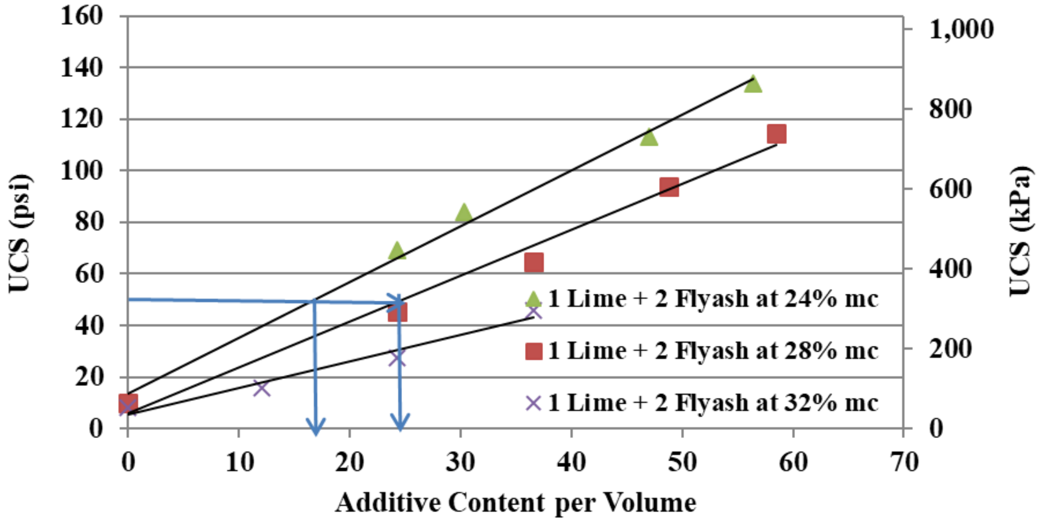


Figure 49. Additive contents for soil III at mc = 32% and 100 psi UCS (cement and lime)

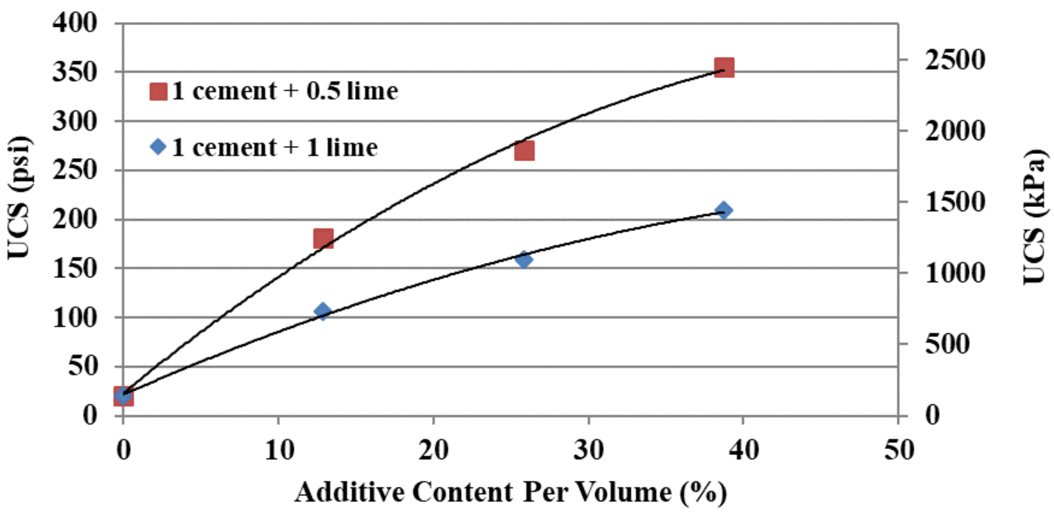


Figure 50. Additive contents for soil III at 100 psi UCS (1 cement: 1 lime)

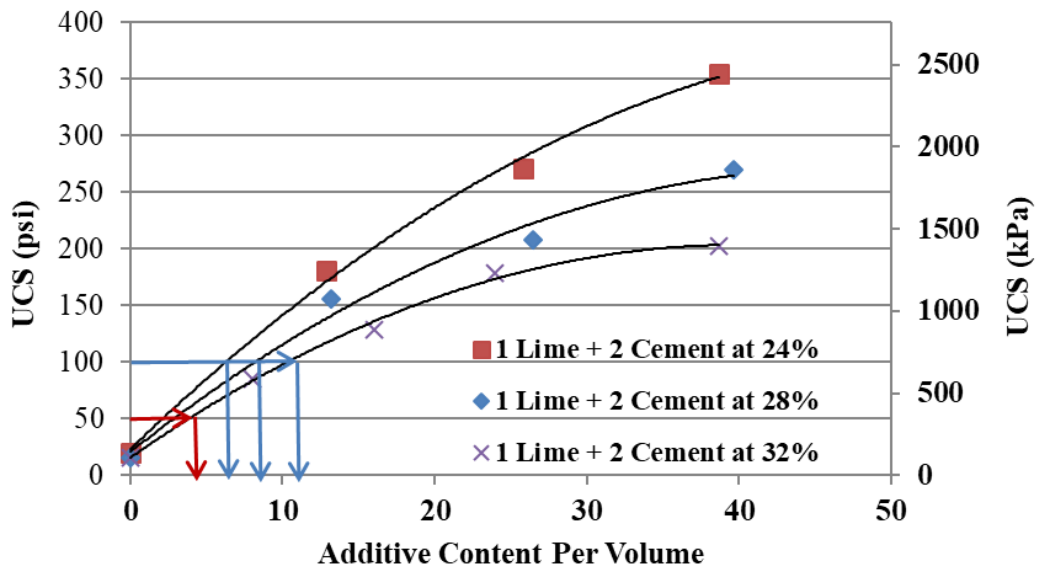


Figure 51. Additive contents for soil IV at 50 and 100 psi UCS (1 cement: 0.5 lime)

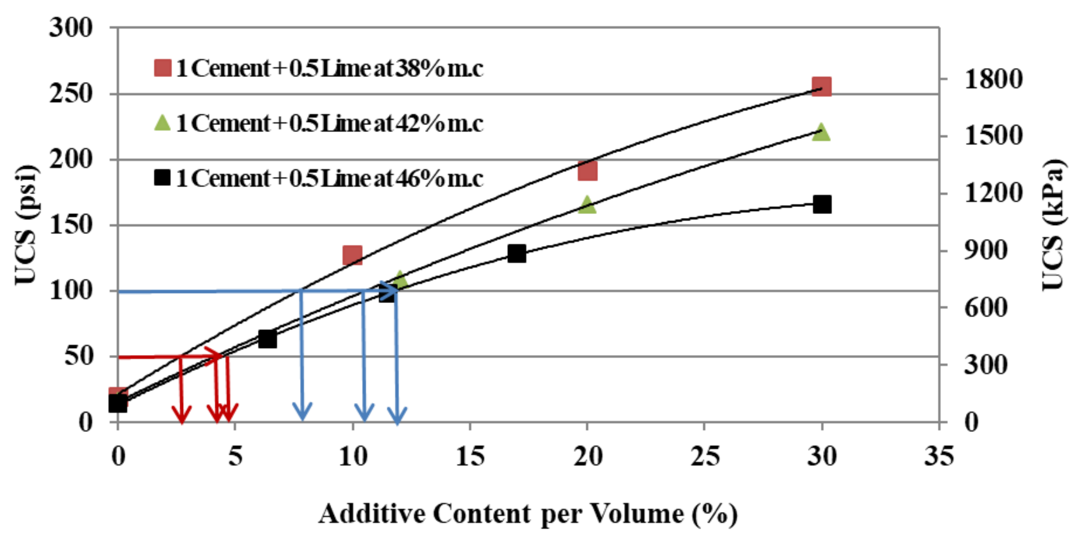


Table 24. Final selected additive type and contents by volume for different soils and different moisture contents

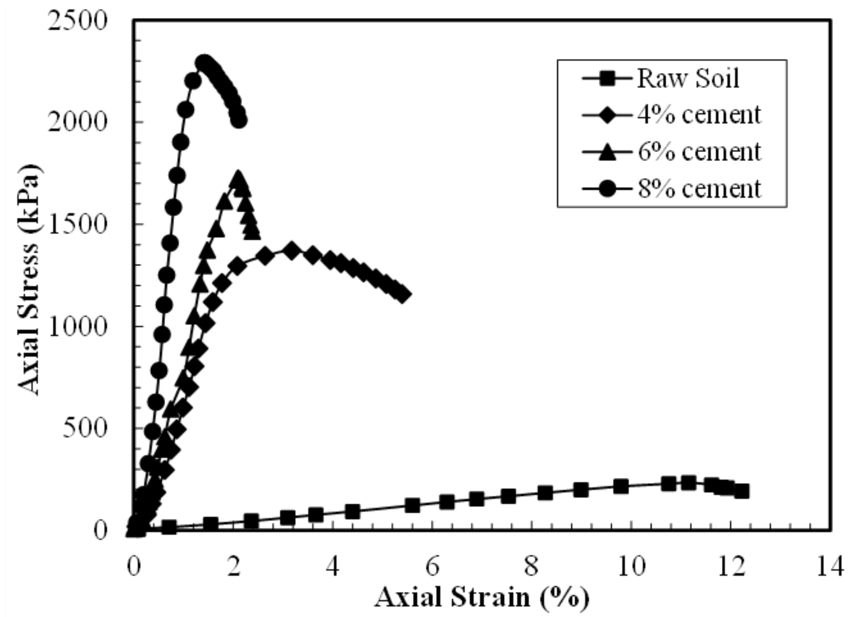
Soil #	Soil Name Low PI	MC1		MC2		MC3	
		50 psi	100 psi	50 psi	100 psi	50 psi	100 psi
I	Low PI	5%L +10%F	4%C	5%L+11%F	5%C	8%L+15%F	7%C
II	Medium PI	6%L+ 10%F	5%C	7%L+11%F	6%C	9%L+15%F	8%C
III	High PI	8%L+11%F	4%L+3%C	10%L+15%F	5%L+4%C	5%L+3%C	6%L+5%C
IV	Heavy clay	4%L+2%C	5%L+4%C	5%L+3%C	6%L+5%C	6%L+4%C	7%L+6%C

L: lime; C: cement; F: fly ash

The stress-strain behavior of the raw soil specimens was also compared with the treated/stabilized soil specimens prepared at the pre-selected moisture contents and additive contents, whenever possible. In most cases, it was not possible to test the raw soil samples at higher moisture contents (very weak). However, the addition of stabilizers enhances the strength and stiffness of the raw soils, while at the same time the soil loses its ductile nature or cohesive nature. The treated/stabilized soils become more brittle as the axial strain reduced considerably with the increase in additive contents.

Typical stress-strain curves of soil specimens with different types of stabilizers are presented in Figure 52 for soil II. The figure clearly indicates that the stress-strain curves shift towards the left hand side as the strain at failure reduced with the increase in stabilizer content and it is associated with higher compressive strength; hence, increasing the elastic modulus and shear modulus of the treated/stabilized soils.

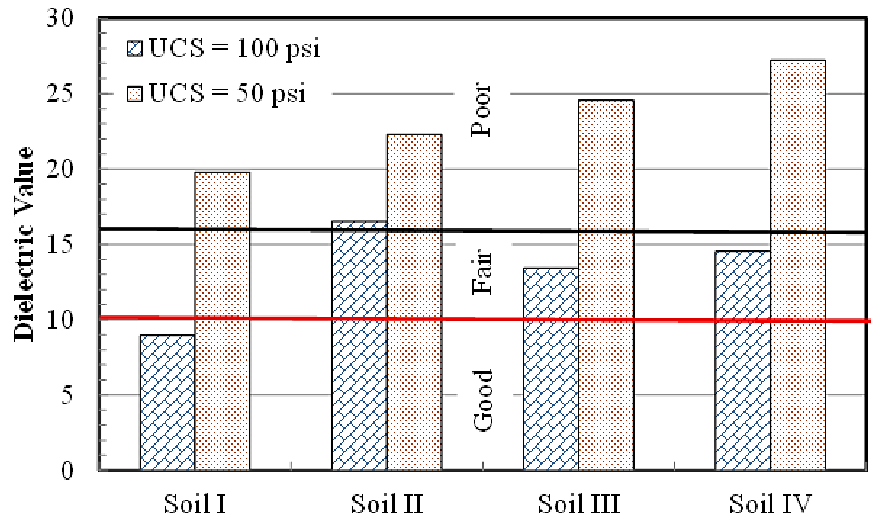
Figure 52. Stress-strain relationships for soil II with and without treatment



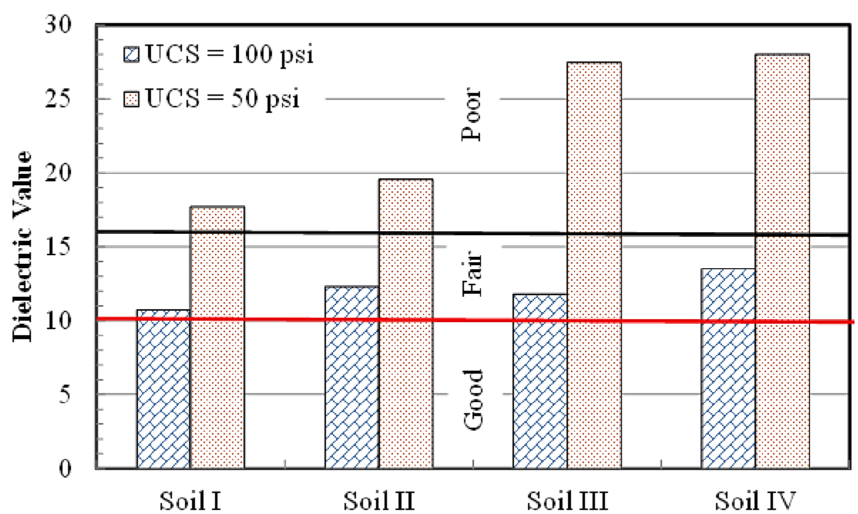
Results of Tube Suction Tests

Tube suction tests were conducted on the treated/stabilized specimens of the four soil types prepared at the three selected moisture contents and for the two UCS target values of 50 psi and 100 psi. The soil specimens were treated/stabilized using the stabilizer corresponding to the selected moisture content and the target UCS value. Figure 53 presents the results of tube suction tests performed on the treated/stabilized soil specimens molded at three different moisture content. The results showed that the final dielectric values (DVs) of the treated soil specimens at the target UCS of 50 psi were all above the value of 16. This means that all treated soil samples at the target UCS of 50 psi were water susceptible. However, for the treated/stabilized soil at the target UCS of 100 psi, the maximum DV were mostly between 10 and 16, which means marginal material based on TxDOT’s criterial.

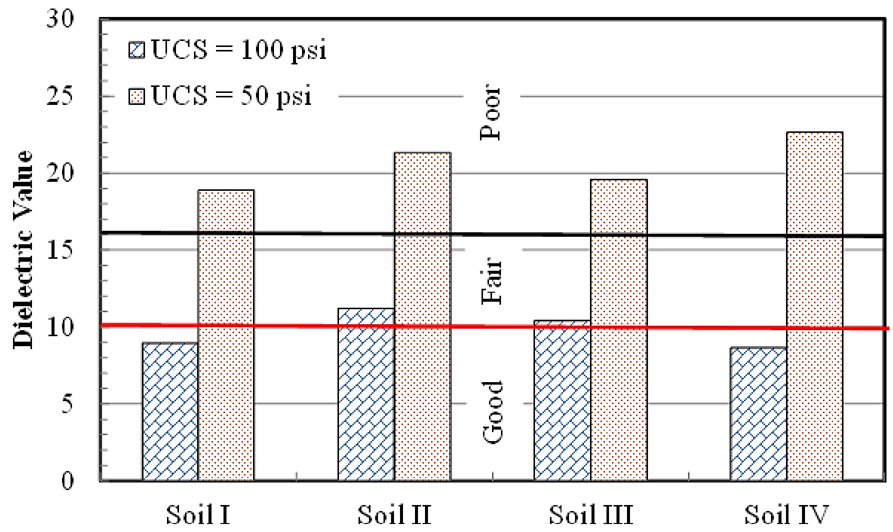
Figure 53. Dielectric values for the treated/stabilized soils in Phase II



(a) MC1



(b) MC2



(c) MC3

Results of Linear Shrinkage Tests

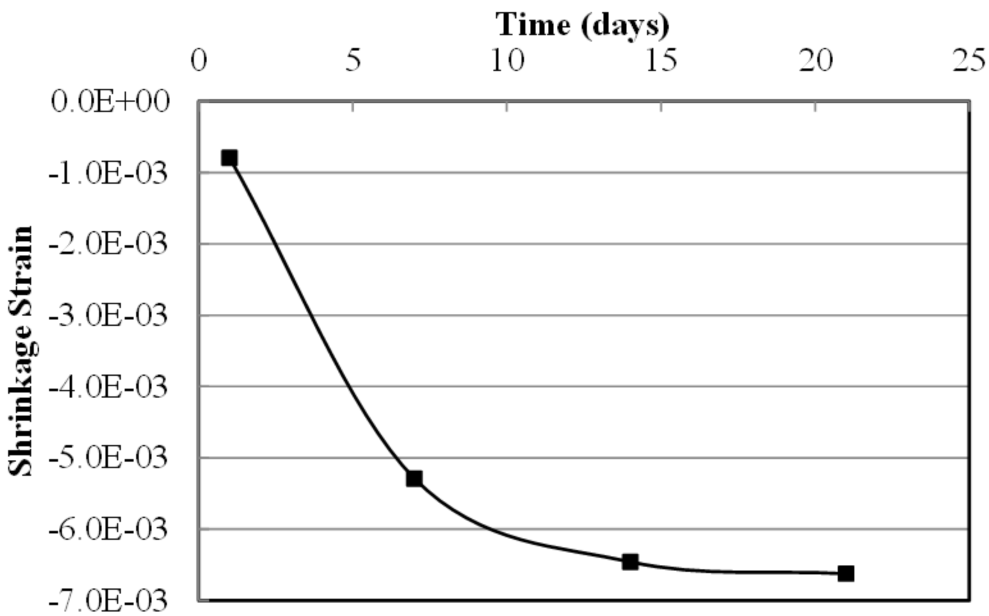
Linear shrinkage tests were performed on the treated/stabilized specimens of the four soil types at the two UCS target values of 50 psi and 100 psi for a 20 days duration time. Figure 54 and Figure 55 present the results of linear shrinkage tests performed on treated/stabilized soils I and IV, respectively. Similar results were obtained for the other soil types. The figures show that the shrinkage of the treated/stabilized soils for Phase II increased very fast initially and then stabilized, similar to what we observed in Phase I. Linear shrinkage usually amounts to around 9 to 12% [48]. The shrinkage characteristics of treated/stabilized specimens are considered very low, hence significantly reducing the treated/stabilized soil's potential for swelling.

Results of Resilient Modulus Tests

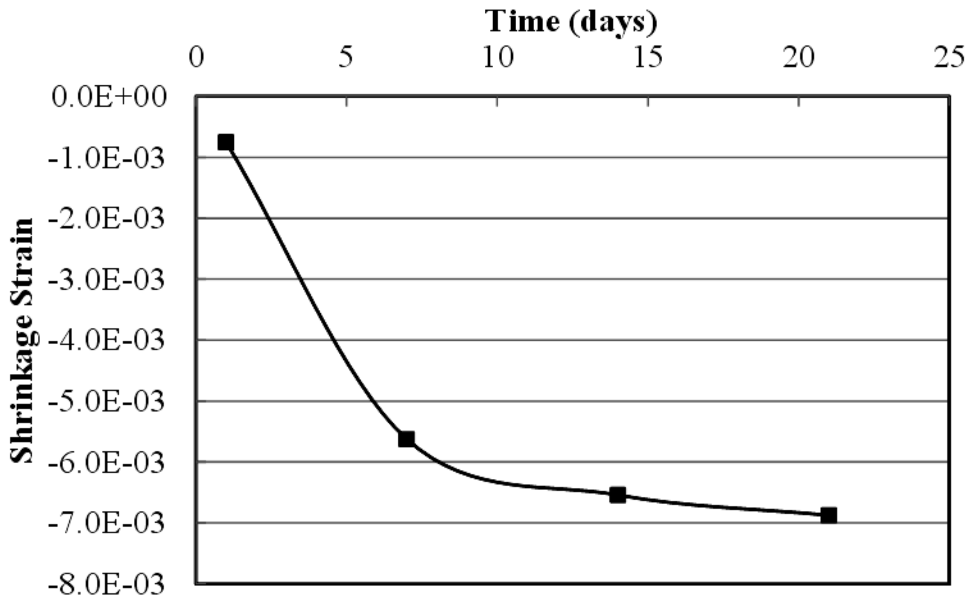
Extensive resilient modulus tests were performed in the laboratory using the MTS machine on the treated/stabilized soil specimens prepared at three different moisture contents as presented in Table 23 and using stabilizer/additive contents according to Table 24. Since the raw soil samples were too wet and very weak, only the performance results of treated/stabilized soil specimens cured at 7 days or 28 days were included in the part of analysis. Among the various factors affecting the resilient response of the soil, the effects of factors like stress state, water/stabilizer ratio, and plasticity index were studied and are discussed here.

The average value of the resilient modulus for the last 10 cycles of each stress sequence was first calculated. The results of all resilient modulus tests for the four treated/stabilized specimens prepared and three moisture contents for the 50 psi and 100 psi UCS values are presented in Appendix D. Regression analyses were then carried out to fit the data of each resilient modulus test case, for the different tested cementitiously treated/stabilized specimens, to determine the corresponding k_1, k_2, and k_3 coefficients for the different M_R models.

Figure 54. Linear shrinkage strain for treated/stabilized soil I

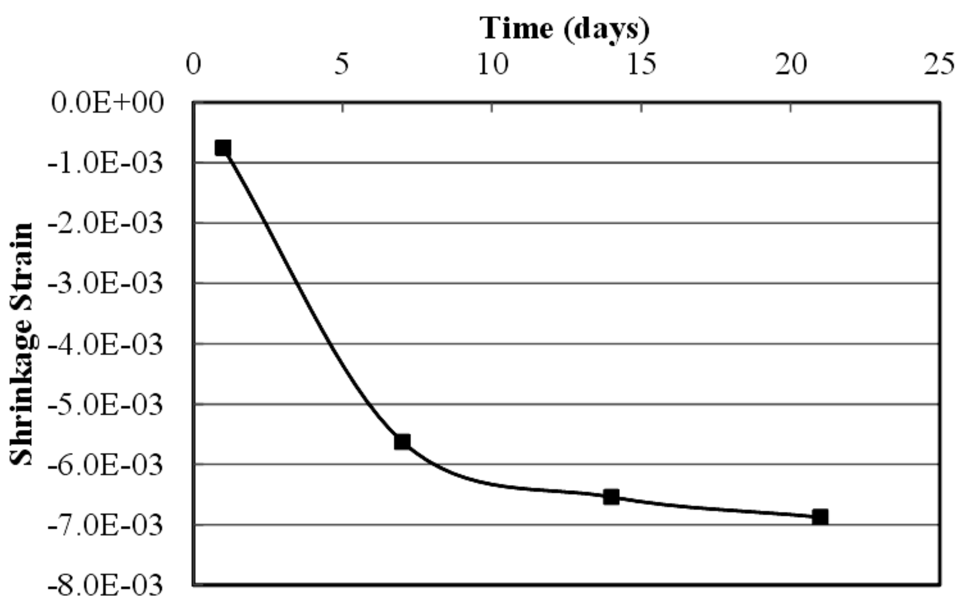


(a) UCS = 50 psi

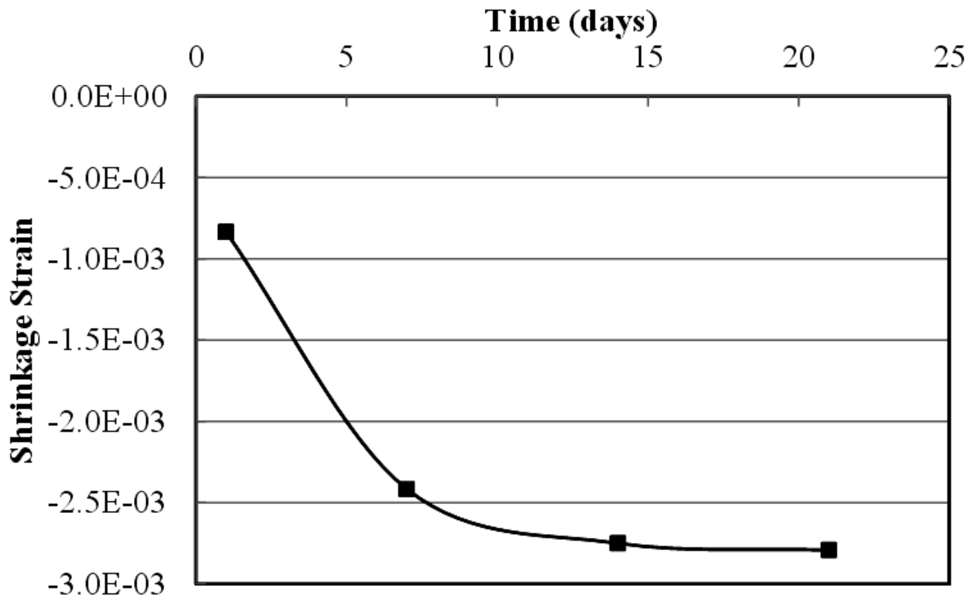


(b) UCS = 100 psi

Figure 55. Linear shrinkage strain for treated/stabilized soil IV



(a) UCS = 50 psi



(b) UCS = 100 psi

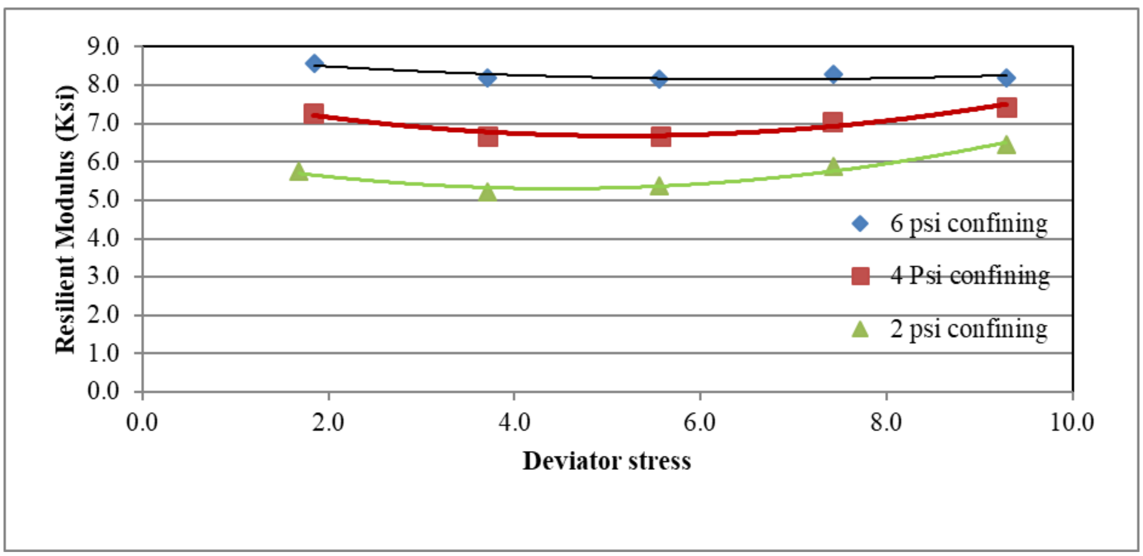
Table 25 presents the average model constants (k_1, k_2, and k_3) obtained for the different soils for the generalized model recommended by the AASHTO 2002 MEPDG [33], i.e. equation (23). Averages consisted of results of M_R on three specimens. These values can be used for pavement design and analysis provided the state of stress is known from layered elastic analysis, finite element analysis, or any other means. The k_2 coefficient describes the stiffening (higher modulus) of the material with the increase in the bulk stress. It is noted from Table 25 that all k_2 coefficients were less than 1. This indicates that the effect of bulk stress decreases with increasing magnitude. Table 25 also show that the magnitude of regression coefficients k_1, k_2, and k_3 is largely dependent on the soil type and water/stabilizer ratio. This suggests that the specimens having similar UCS but with different water/cement or water/stabilizer ratio have shown different resilient characteristics. As such, the use of direct correlation between the UCS and the resilient modulus for cementitiously treated/stabilized soils can be misleading and should be carefully used in pavement design.

Table 25. Model constants for different soils (Phase II)

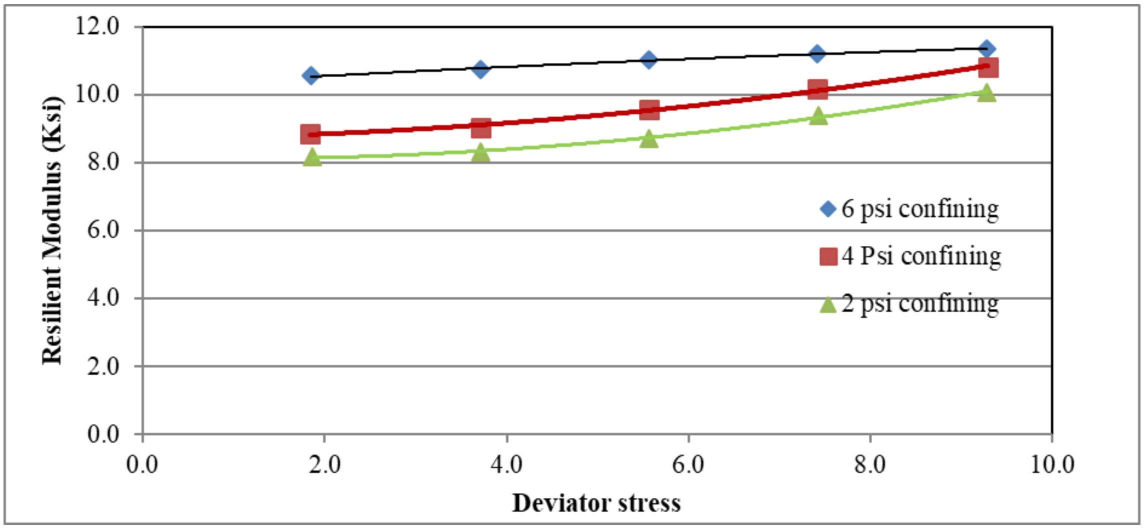
Soil #	Model Constants	7 days					
		50 psi target (7-day UCS)			100 psi target (7-day UCS)		
		MC1	MC2	MC3	MC1	MC2	MC3
I	k_1	527.82	513.56	586.07	466.14	574.46	728.41
	k_2	0.41	0.51	0.38	0.16	0.27	0.11
	k_3	-0.17	-0.93	-0.35	1.72	1.57	1.77
II	k_1	763.73	627.76	655.00	626.02	628.51	701.96
	k_2	0.28	0.29	0.07	0.19	0.16	0.21
	k_3	-0.48	0.10	2.02	1.95	1.74	1.64
III	k_1	673.76	527.26	736.36	1267.51	1479.45	960.05
	k_2	0.34	0.37	0.25	0.71	0.57	0.43
	k_3	-0.68	-0.25	0.24	-1.20	-1.68	0.65
IV	k_1	757.83	643.40	707.91	706.91	687.28	543.39
	k_2	0.44	0.45	0.51	0.58	0.29	0.19
	k_3	-1.46	-1.14	-1.14	-0.49	0.37	1.46
		28 days					
I	k_1	-	472.56	-	-	355.38	-
	k_2	-	0.59	-	-	0.58	-
	k_3	-	-1.43	-	-	-1.43	-
II	k_1	-	641.64	-	-	483.54	-
	k_2	-	0.29	-	-	0.30	-
	k_3	-	0.92	-	-	0.91	-
III	k_1	-	593.65	-	-	447.75	-
	k_2	-	0.15	-	-	0.15	-
	k_3	-	1.23	-	-	1.23	-
IV	k_1	-	503.75	-	-	377.18	-
	k_2	-	0.18	-	-	0.17	-
	k_3	-	0.78	-	-	0.80	-

Effect of Stress State. Typical variation of the resilient modulus with stress conditions obtained from the laboratory tests of treated/stabilized specimens for moisture content MC_2 are presented in 56 and 57 for 50 psi and 100 psi UCS, respectively. The variation of resilient modulus with stress conditions for the other moisture contents (MC_1 and MC_3) can be found in Appendix D. The resilient modulus values of treated/stabilized soils increase with the increasing confining pressure. This is due to the decrease in dilatational properties and the increase in stiffness from the increasing confining pressure. The resilient modulus of treated/stabilized soils shows a mixed behavior with the increase of deviatoric/cyclic stress, such that it almost remains constant or decrease for treated/stabilized samples at 50 psi UCS with increasing deviatoric/cyclic stress. It also shows an increase in resilient modulus with increasing the deviatoric/cyclic stress for specimens treated/stabilized to 100 psi UCS, as indicated in Figure 56, Figure 57, and Appendix D.

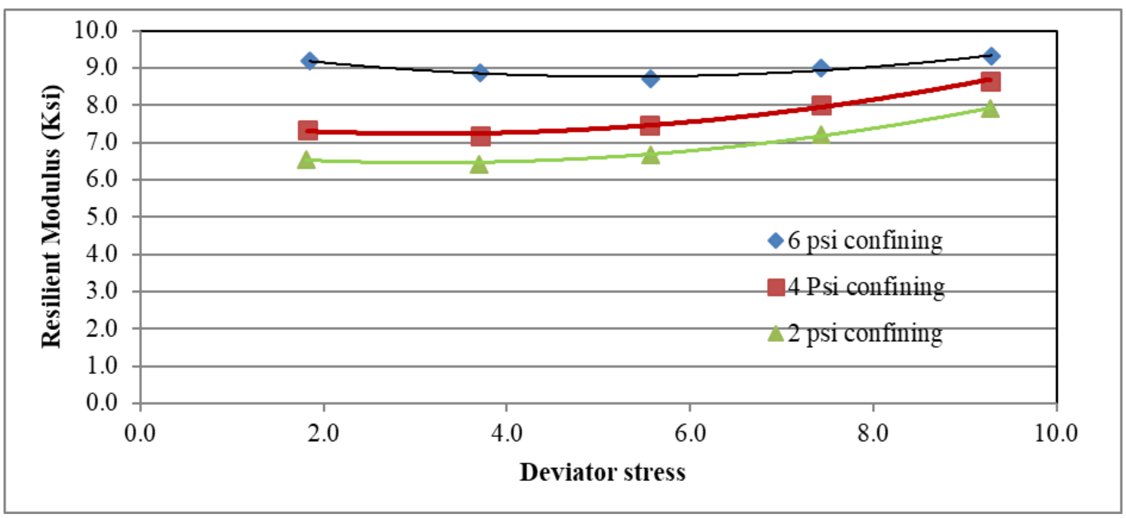
Figure 56. Resilient modulus of treated soil specimens (MC = MC_2 and UCS = 50 psi)



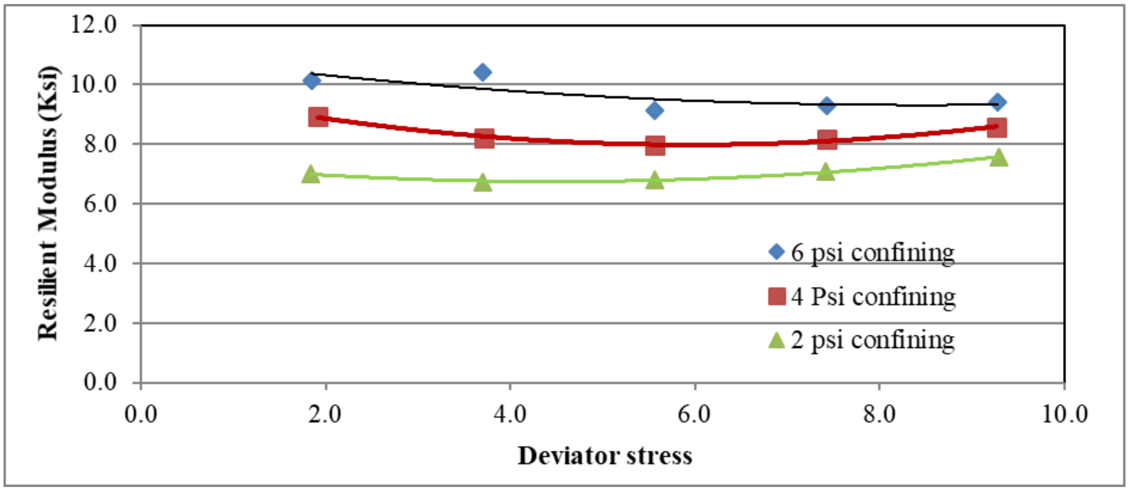
(a) Soil I



(b) Soil II

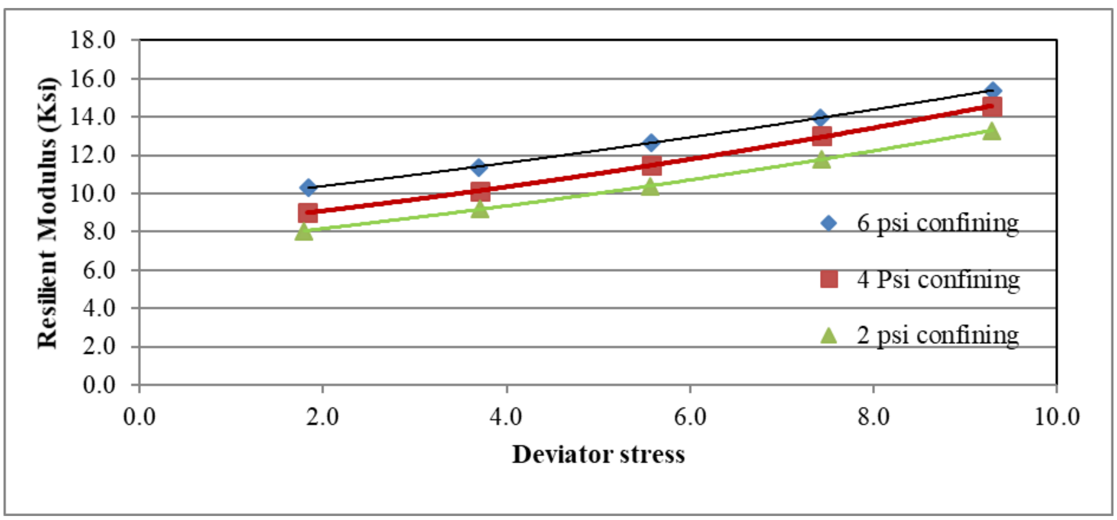


(c) Soil III

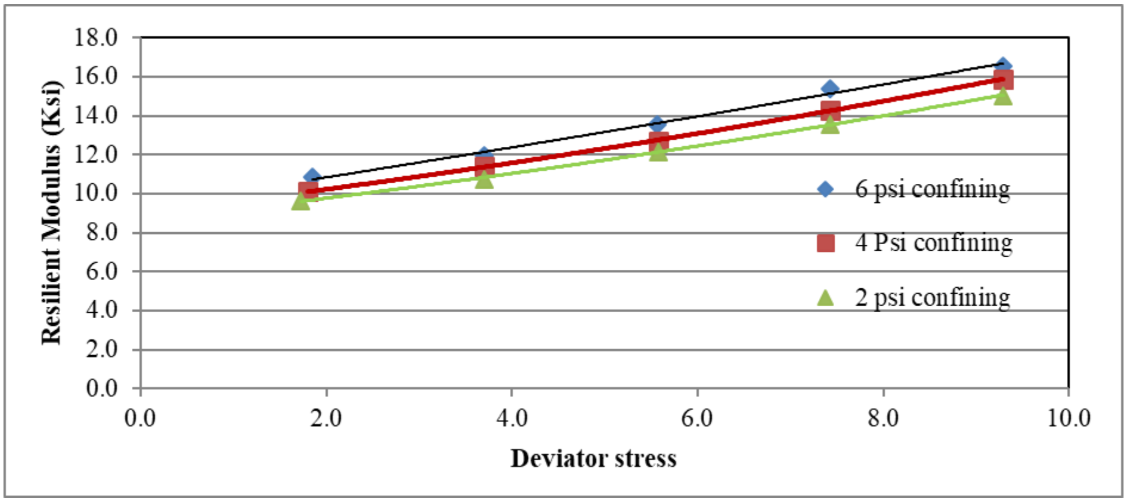


(d) Soil IV

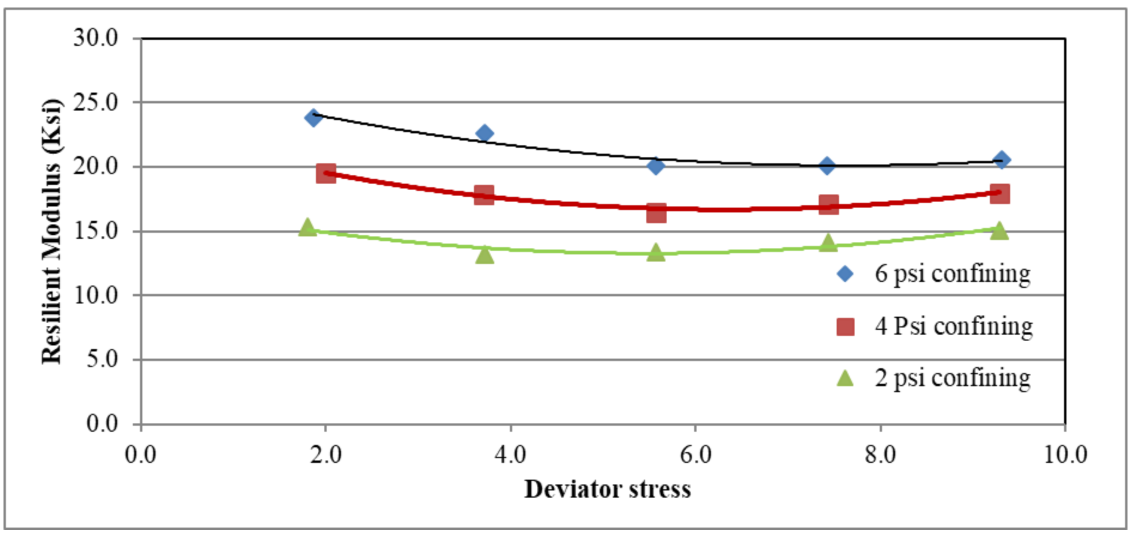
Figure 57. Resilient modulus of treated soil specimens (MC = MC_2 and UCS = 100 psi)



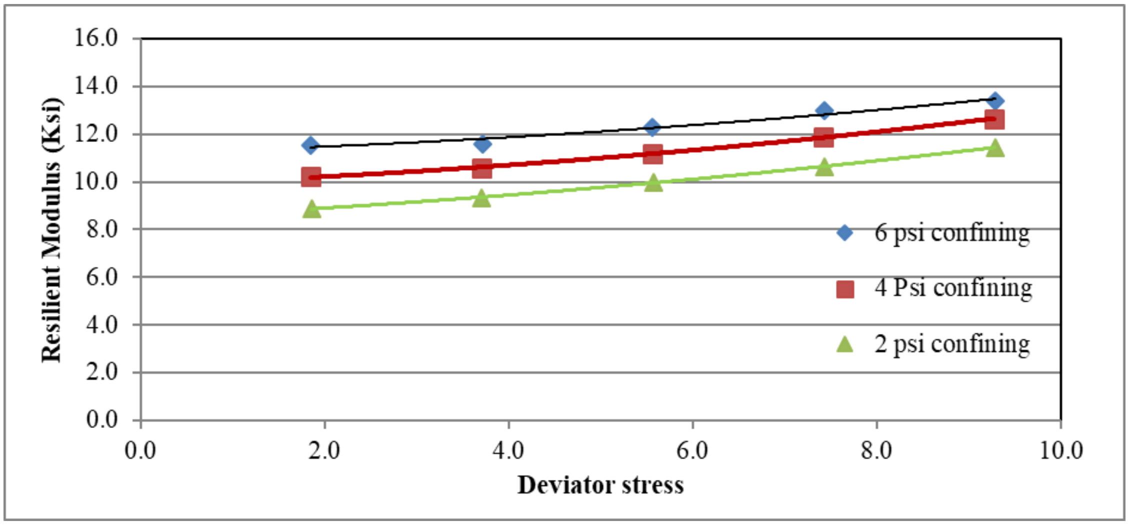
(a) Soil I



(b) Soil II



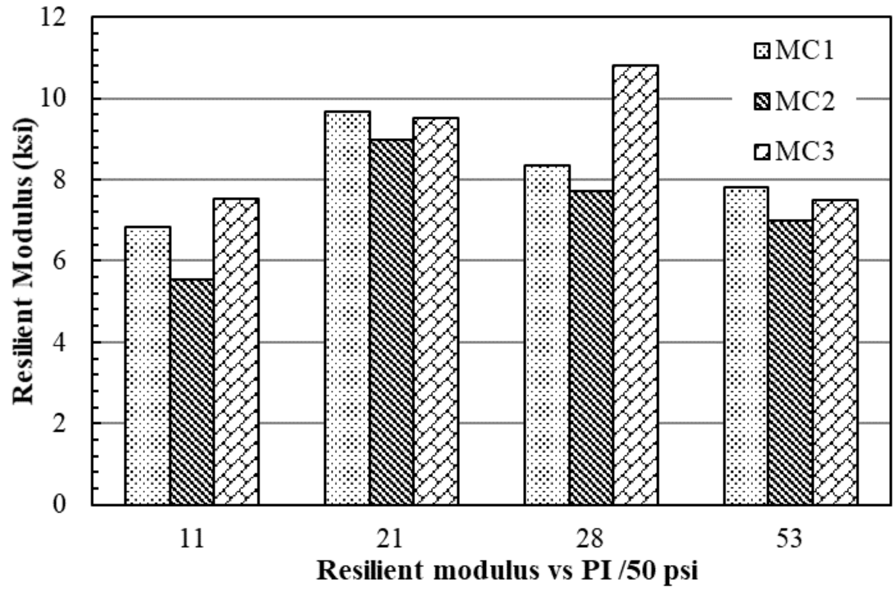
(c) Soil III



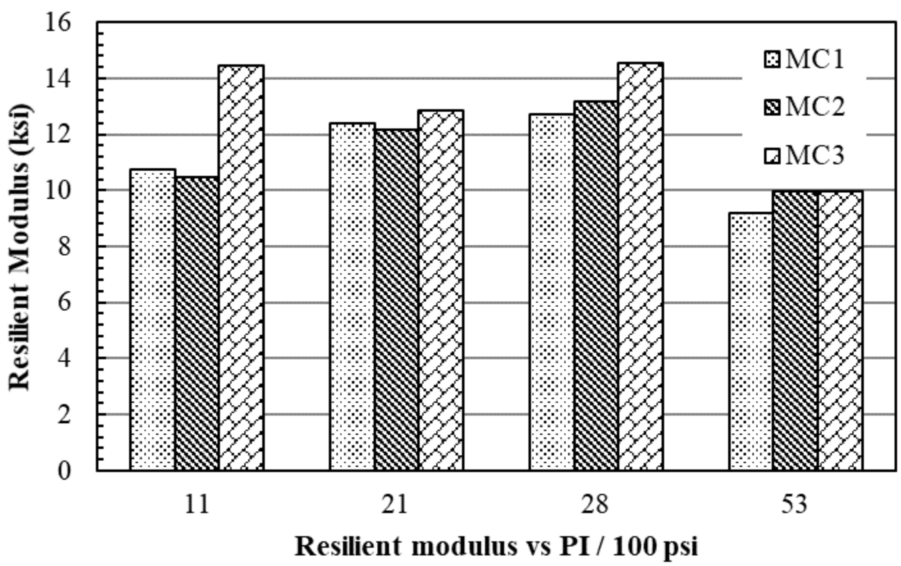
(d) Soil IV

Effect of Plasticity. Figure 58 presents the comparison of M_R variations obtained for the four different soil types with different plasticity indices that were treated/stabilized to reach the same target UCS (i.e., 50 psi and 100 psi). It should be noted here that to achieve the same target UCS, the higher the moisture content, the higher the stabilizer content is added, as shown in Table 24. For the same UCS, no trend was observed between the M_R and the PI. For the same PI, the resilient modulus increases with increasing the additive content, which also correspond to higher moisture content. The general trend shows that for the target UCS of 100 psi, the resilient moduli increases with increasing the additive content for all soils. In contrast, for the target UCS of 50 psi, the resilient moduli of specimens were a little lower at a combination of medium additive content and medium moisture content. The resilient modulus values in Figure 58 were also selected using a cyclic stress level of 5.4 psi and a confining stress of 2 psi, which represents the stress state that a subgrade layer encounters under traffic loading [12].

Figure 58. Variation of resilient modulus with PI of all soils



(a) UCS = 50 psi

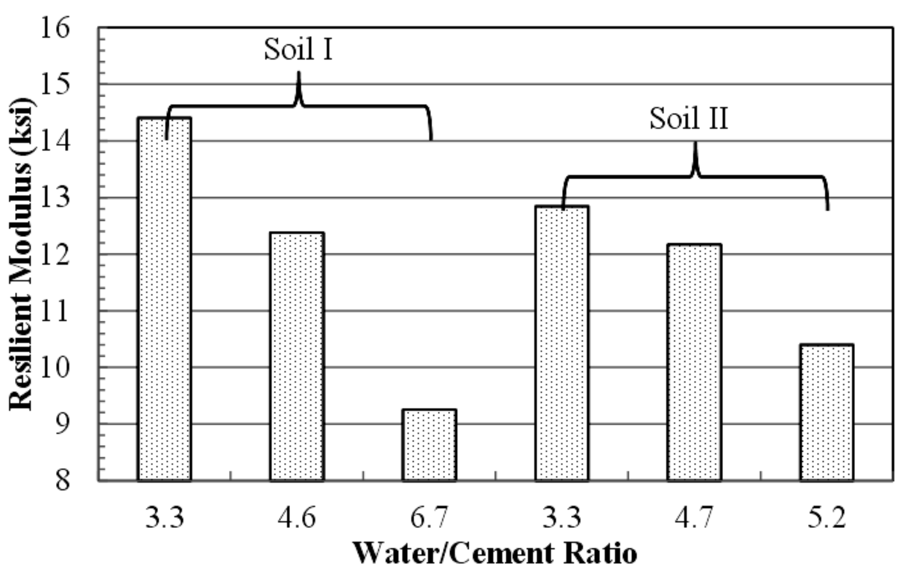


(b) UCS = 100 psi

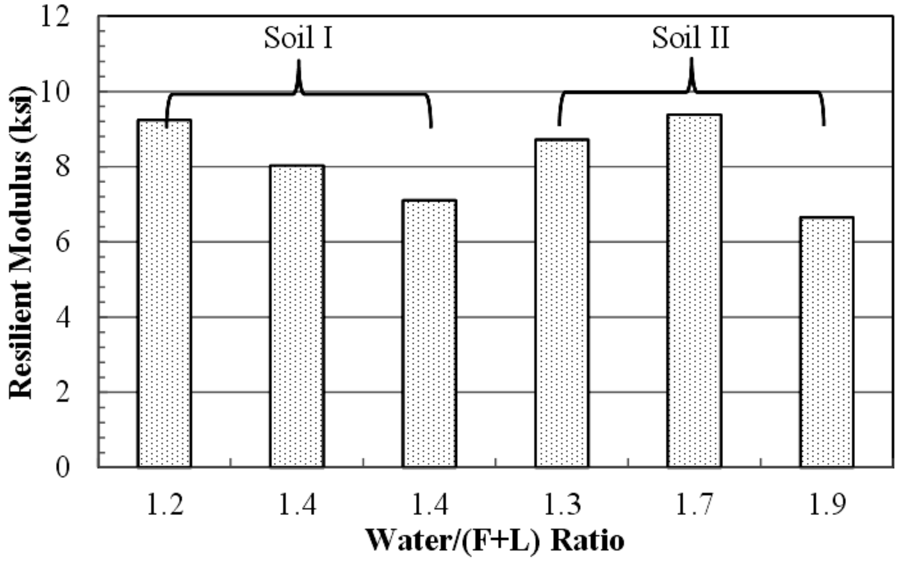
Effect of Water/Stabilizer Ratio. The effect of various combinations of water to stabilizer ratio on the resilient modulus for the four different soil types was observed for the 7 days of the curing period. In the case of the cement-treated/stabilized soil samples (soil #1 and soil #2 at target UCS of 100 psi), having similar UCS, the test result showed a decrease in resilient modulus with the increase in water/cement ratio [Figure 59 (a)] and vice versa. One explanation for this behavior may be partially due to the increase in capillary pressure (suction) as saturation decreases; hence, the material stiffens as the capillary pressure increases. It should be noted here that an increase in resilient modulus for the tested specimens in this study is also associated with an increase in the cement content, as shown in Figure 60. This means the viability of more calcium ions to exchange with monovalent cations present on clay sample during cation exchange process, hence reduce the thickness of diffused double layer, increase the contact between the clay particles, and stiffen the material.

For soil #1 and soil #2, treated/stabilized with both lime and fly ash at the target UCS of 50 psi, no definite relation was observed between the resilient modulus and the water/lime-fly ash ratio (Figure 59b). The lime-cement treated/stabilized soil #3 shows that the resilient modulus decreases with increasing water/lime-cement ratio (Figure 59c). Meanwhile, the lime-cement treated/stabilized soil #4 shows that the resilient modulus decreased with increasing water/lime-cement ratio to a certain value, after which it keeps almost constant. Again, the resilient modulus values in Figure 59. were selected using a deviatoric/cyclic stress level of 5.4 psi and a confining stress of 2 psi, which represent the stress state that a subgrade layer encounters under traffic loading [12].

Figure 59. Resilient modulus of treated soils at different water/stabilizer ratio

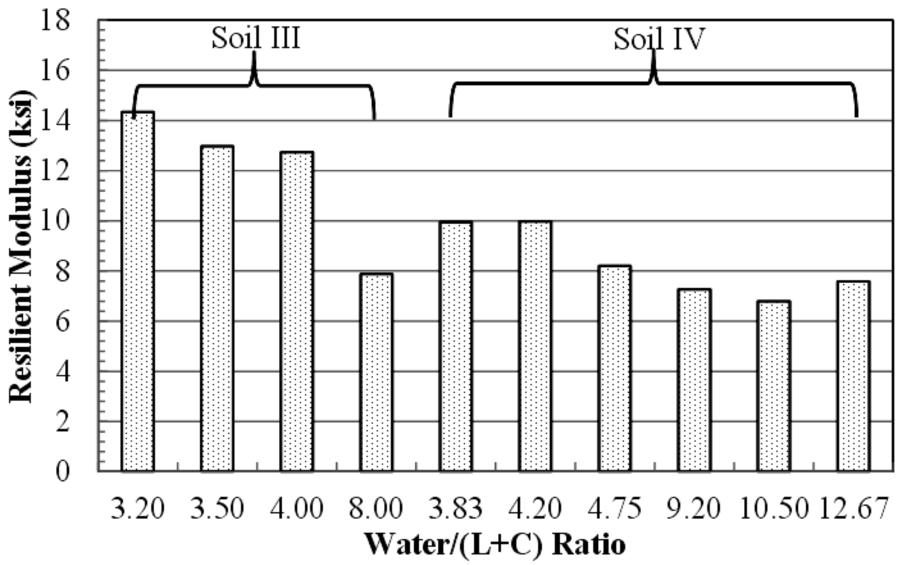


(a)



Soil I
Soil II
Resilient Modulus (ksi)
12
10
8
6
4
2
0
1.2
1.4
1.4
1.3
1.7
1.9
Water/(F+L) Ratio

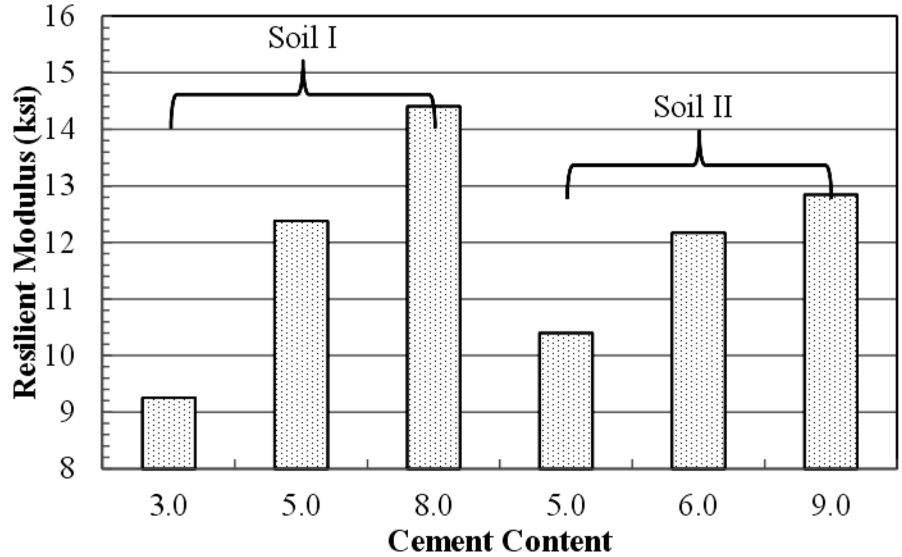
(b)



Soil III
Soil IV
Resilient Modulus (ksi)
18
16
14
12
10
8
6
4
2
0
3.20
3.50
4.00
8.00
3.83
4.20
4.75
9.20
10.50
12.67
Water/(L+C) Ratio

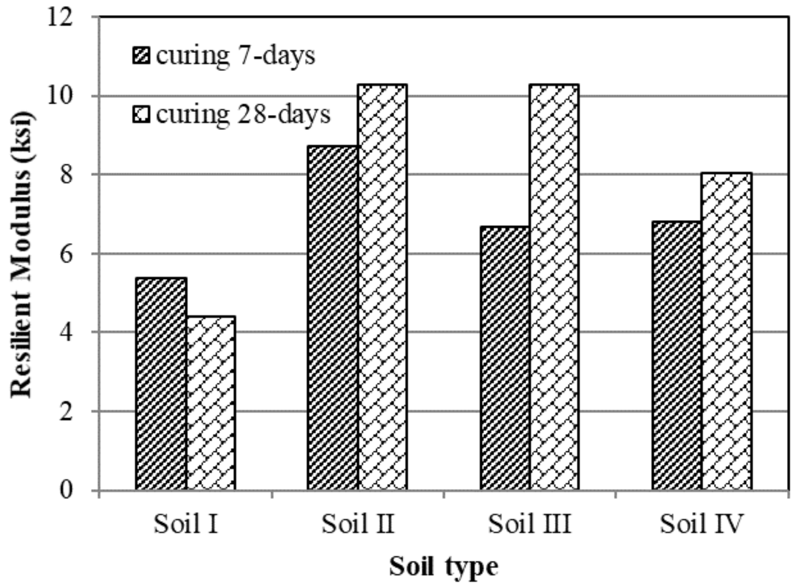
(c)

Figure 60. Resilient modulus of treated soils at different cement content

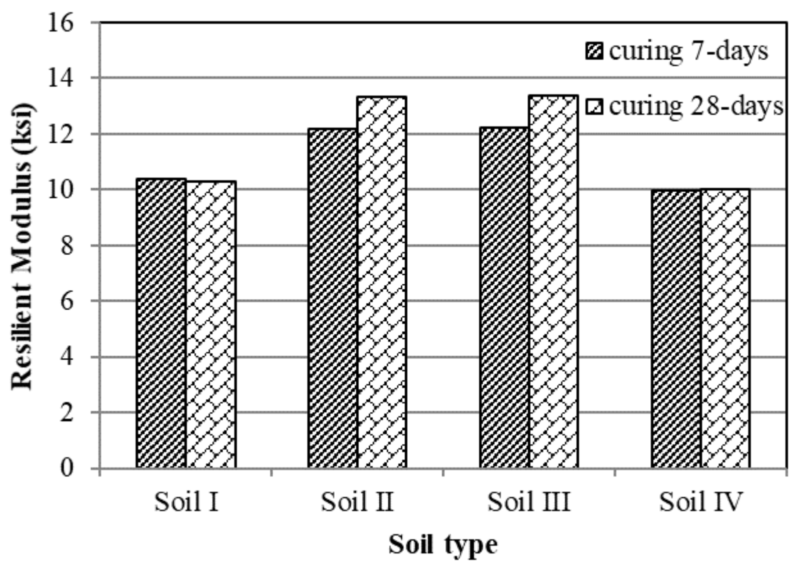


Effect of Curing Time. In order to evaluate the effect of the duration of the curing time on the resilient modulus of treated/stabilized soils, three replicate specimens from each soil type were prepared at MC2 and medium additive content were selected and tested at 7 and 28 days curing times in the MTS machine. The results are presented in Figure 61(a) and Figure 61(b) for 50 psi and 100 psi UCS values, respectively. The figures show that the resilient modulus increases with increasing the curing time for all soils except Soil I. An increase in the resilient modulus of up to 40% was observed for Soil III at 50 psi UCS after 28 days of curing time as compared to the resilient modulus obtained after 7 days of curing. For Soil I, the resilient modulus slightly decreases for 50 psi UCS and almost no change for 100 psi UCS. The values of resilient moduli were selected using a cyclic stress level of 5.4 psi and a confining stress of 2 psi.

Figure 61. Resilient modulus of treated soils at different cement content



(a) at MC_2 and UCS = 50 psi

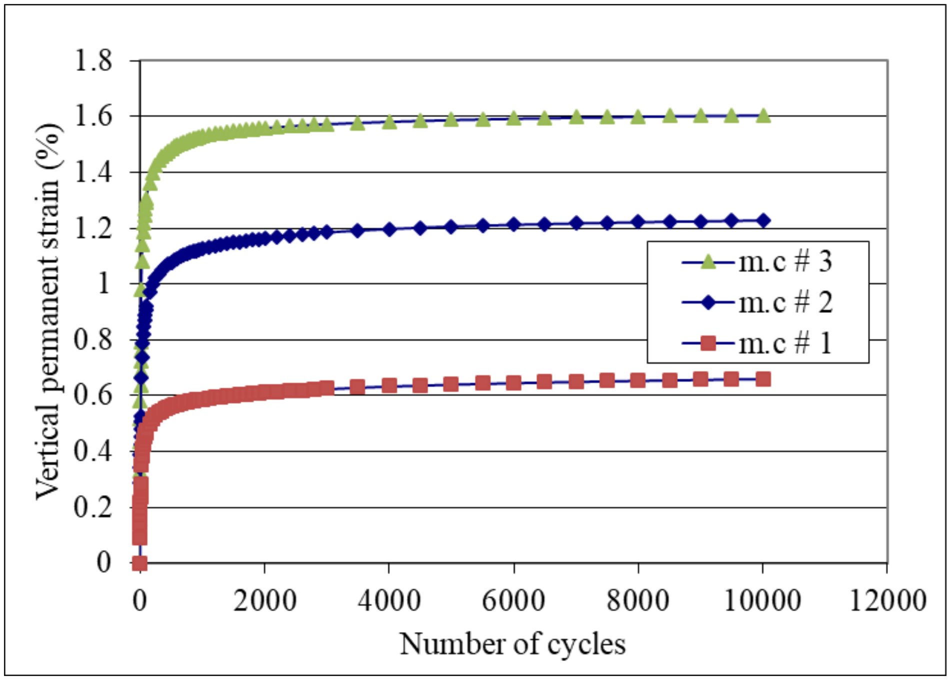


(b) at MC_2 and UCS = 100 psi

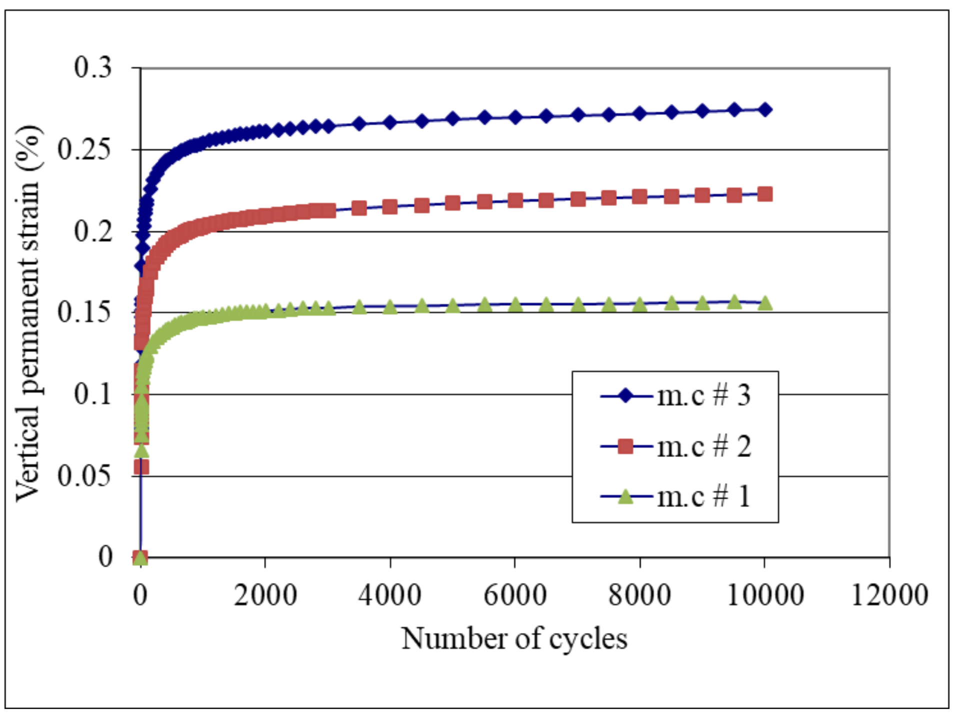
Results of Single-stage Permanent Deformation Tests

Repeated load triaxial (RLT) tests were performed to evaluate the permanent deformation (PD) behavior of different treated/stabilized subgrade soil specimens prepared at three different moisture contents (MC1, MC2, and MC3). Figure 62 and 63 present the typical curves of the average permanent (axial) strain versus the number of cycles obtained for the different RLT cases at 50 psi and 100 psi UCS, respectively. The averages here consisted of two specimens. The permanent (axial) deformation curve has two distinct stages. In the first stage (post-compaction stage), the material accumulates a significant amount of permanent deformation. This is most probably due to extra compaction and initial particle bonding breakage induced particle re-arrangement. During the second stage (secondary stage), the material accumulates permanent strain at a much lower rate and even in some cases the permanent deformation approaches a constant value. The permanent (axial) strains observed for all treated soil specimens after 10,000 cycles of loading are summarized in Table 26. The figures demonstrate that the permanent deformation is somehow negligible (permanent strain < 0.08%) that might be ignored in pavement design. This observation is consistent with the UCS test results, which show that the stress-strain curves shifted towards the left side (brittle behavior) as the percent of stabilizer content increases (Figure 52). This is also in agreement with the MEPDG, which does not consider the deformations of cement stabilized layers in pavement design.

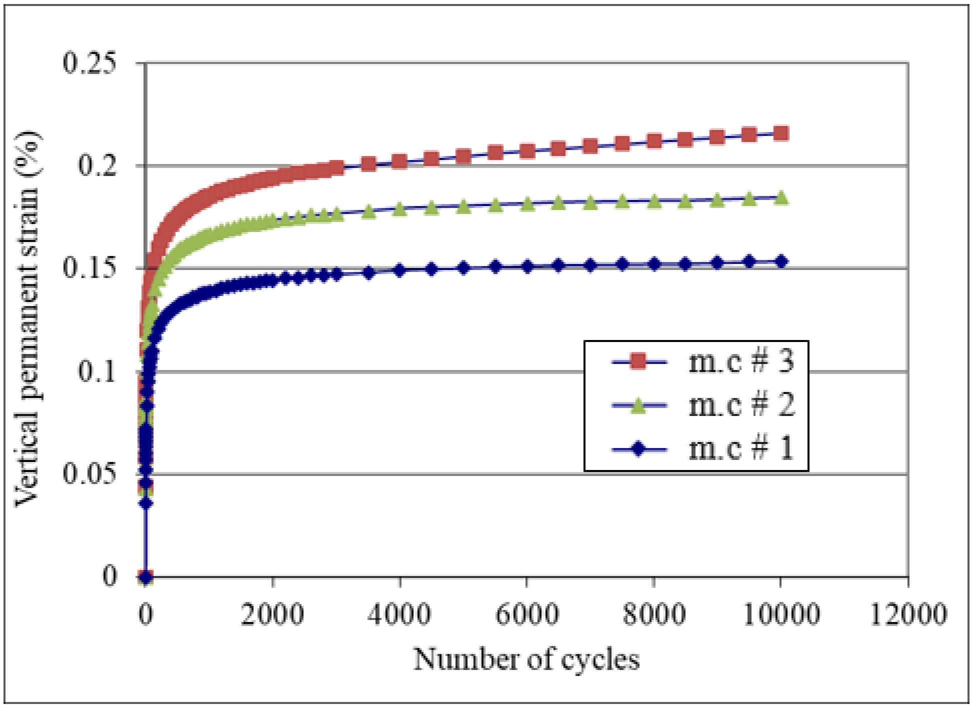
Figure 62. Permanent deformation of treated soil specimens (UCS = 50 psi)



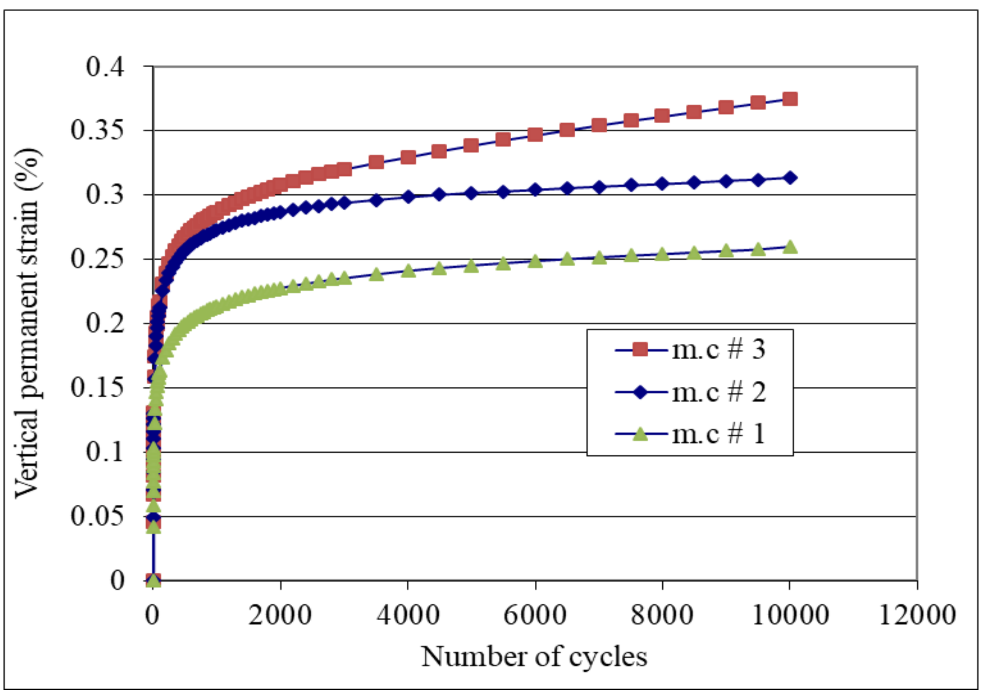
(a) Soil I



(b) Soil II

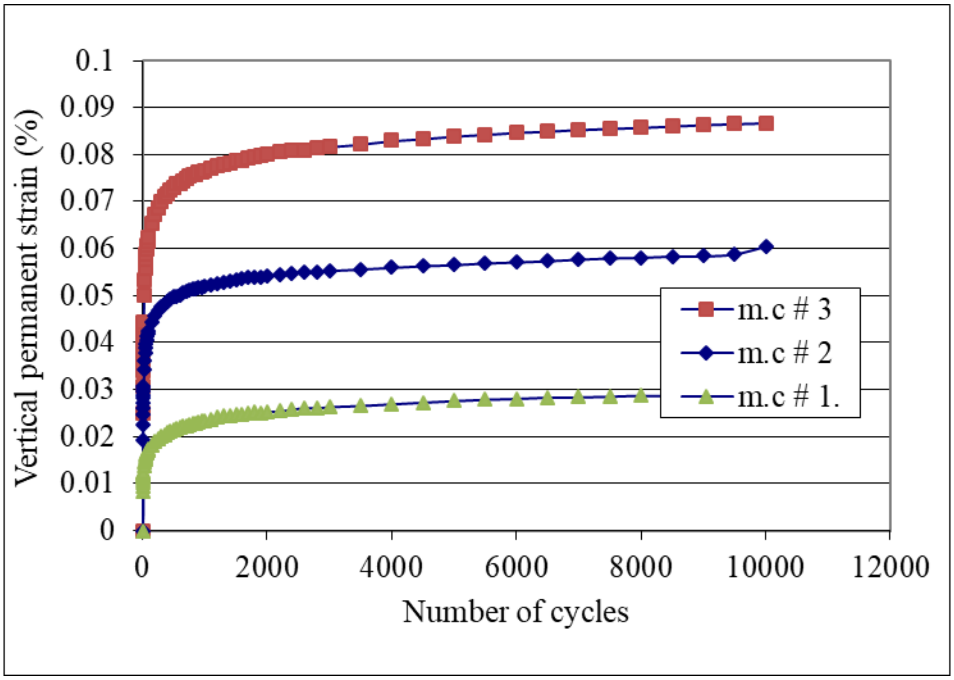


(c) Soil III

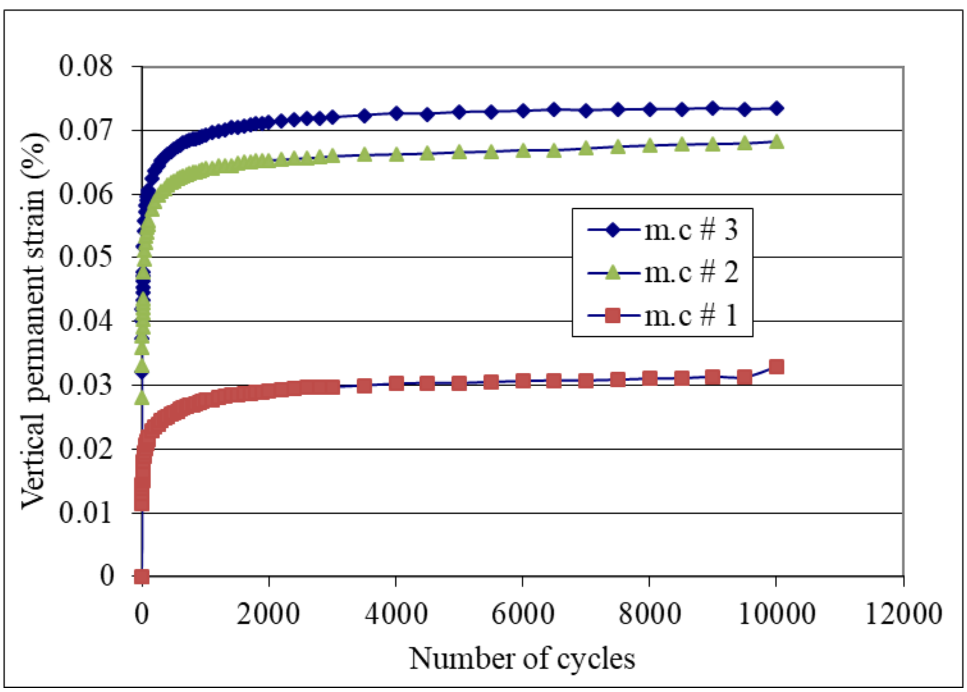


(d) Soil IV

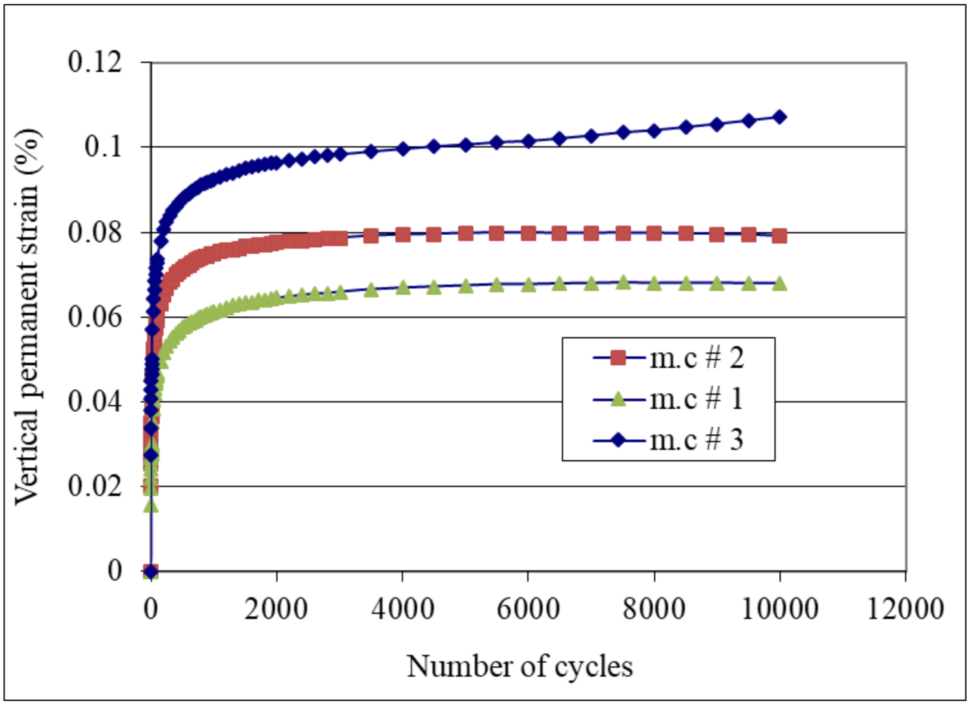
Figure 63. Permanent deformation of treated soil specimens (UCS = 100 psi)



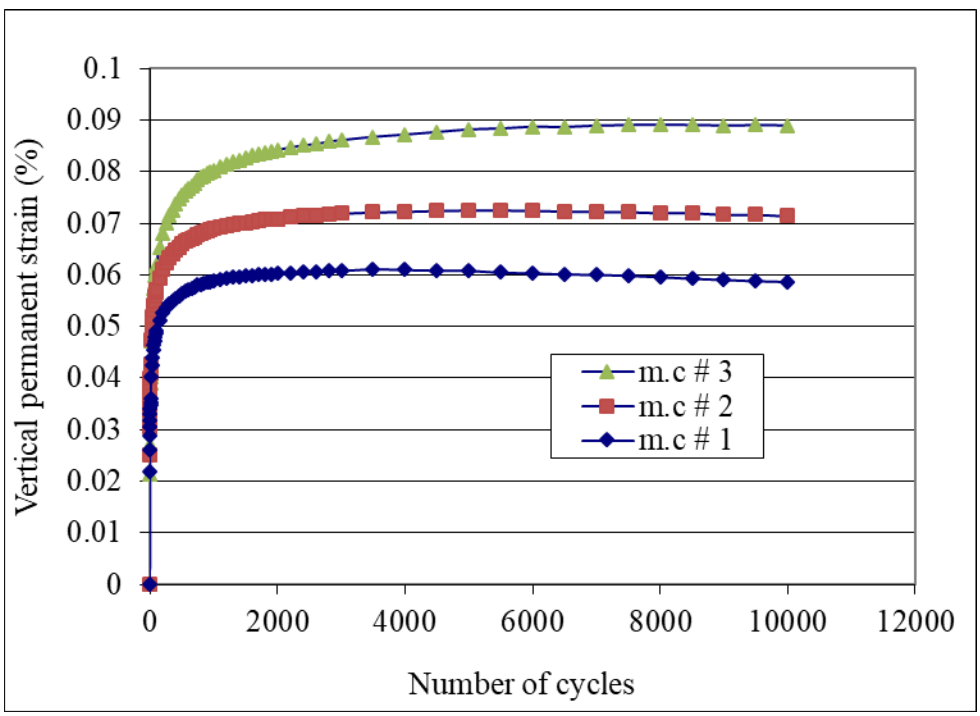
(a) Soil I



(b) Soil II



(c) Soil III



(d) Soil IV

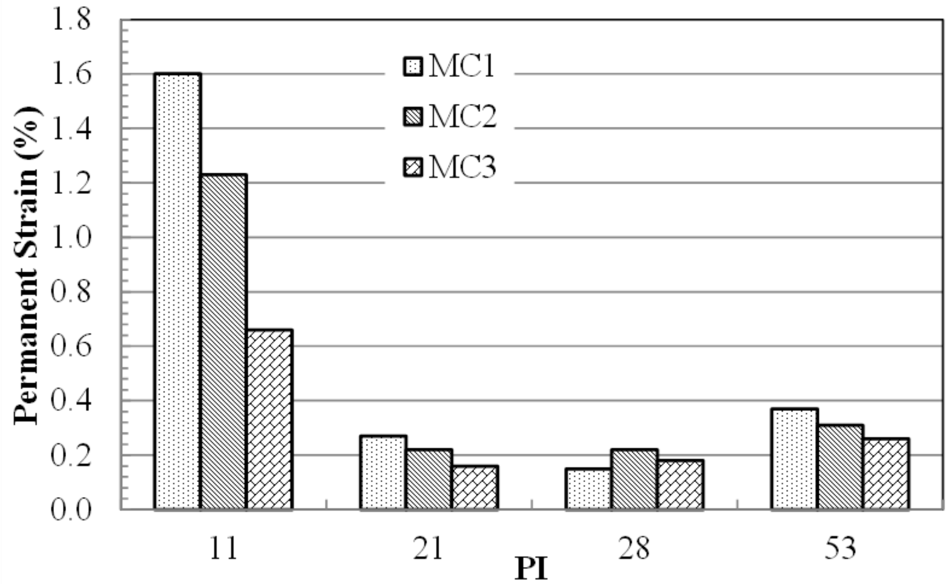
Table 26. Vertical permanent strain of specimens at the 10,000th cycle

	28 days					
	50 psi target			100 psi target		
Soil #	**MC1**	**MC2**	**MC3**	**MC1**	**MC2**	**MC3**
I	0.66	1.23	1.60	0.029	0.06	0.087
II	0.16	0.22	0.27	0.032	0.067	0.074
III	0.15	0.18	0.22	0.068	0.08	0.11
IV	0.26	0.32	0.37	0.06	0.072	0.09

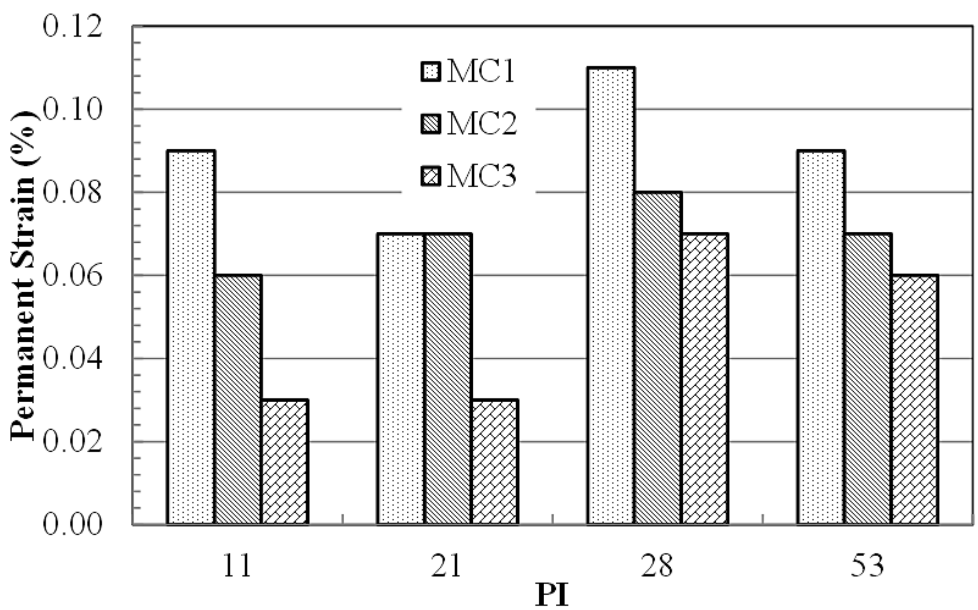
Effect of Plasticity. Figure 64(a) and 64(b) present the comparison of permanent deformation obtained for the different soil types treated/stabilized to reach the same target UCS value of 50 psi and 100 psi, respectively. The figures show no definite relation between the permanent deformation and the PI value. However, for the same soil type, the permanent deformation increases with the increase in the stabilizer content regardless of the moisture content. The figure clearly demonstrated that, even for lightly treated subgrade soil of working table application, the permanent deformation is small; while for heavily treated subgrade soil of subbase application, the permanent deformation is somehow negligible (permanent strain < 0.12%) that might be ignored in pavement design. This is in agreement with the MEPDG, which does not consider the deformations of cement or lime stabilized layers in pavement design.

Effect of Water/Additive Ratio. The effect of water/additive (cement, lime-cement, or lime-fly ash) ratio on the permanent deformation behavior is presented in Figure 65. The figure clearly demonstrates that the permanent strain of treated/stabilized soil specimens increases with increasing the water/additive ratio. For example, the behavior of cement treated soil specimens (soil I and soil II) at 100 psi target UCS [Figure 65(a)] showed that the permanent strain increases with the increase in the water/cement ratio. The behavior of lime-fly ash treated/stabilized soil specimens (soil I, soil II and soil III at 100 psi target UCS) [Figure 65(b)], and lime-cement treated/stabilized soil specimens (soil III and soil IV) [Figure 65(c)] also showed the similar trends, i.e., the permanent deformation increases with increasing the water/stabilizer ratio. This behavior may be due to the decrease in capillary pressure (suction) with the increase in water/additive ratio, as discussed earlier. It should be noted here that the decrease in water/additive ratio for the tested specimens in this study is also associated with an increase in the additive content; i.e., the permanent deformation decreases with increasing the additive content regardless of the moisture content for the same UCS.

Figure 64. Variation of permanent strain with PI of the different soils

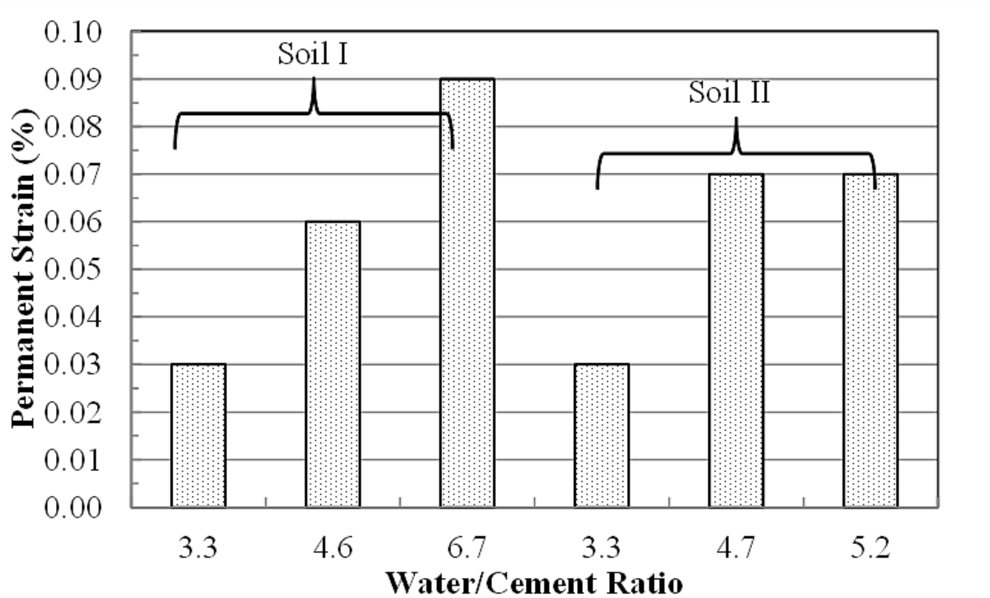


(a) UCS = 50 psi

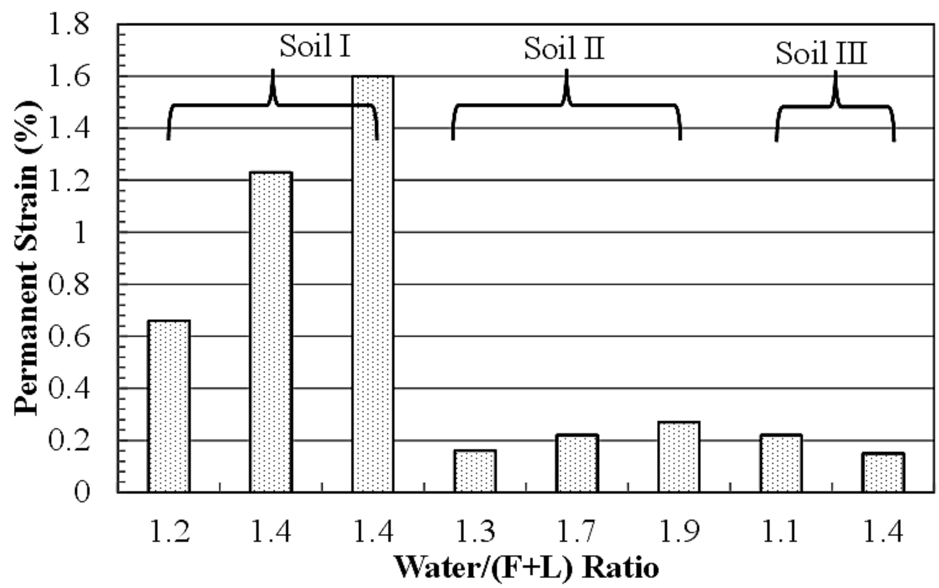


(b) UCS = 100 psi

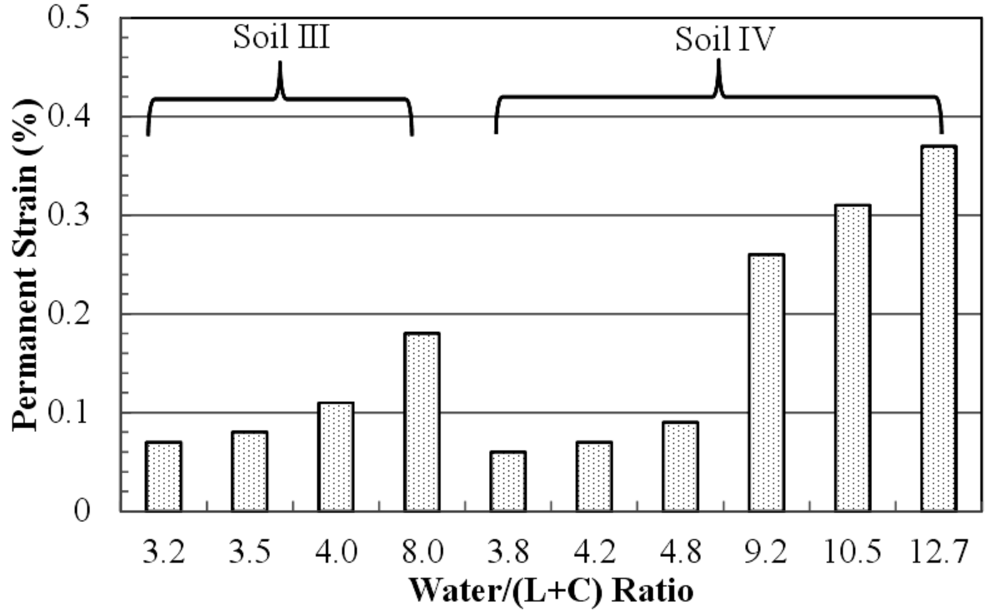
Figure 65. Permanent deformation of treated soils at different water/stabilizer ratio



(a)

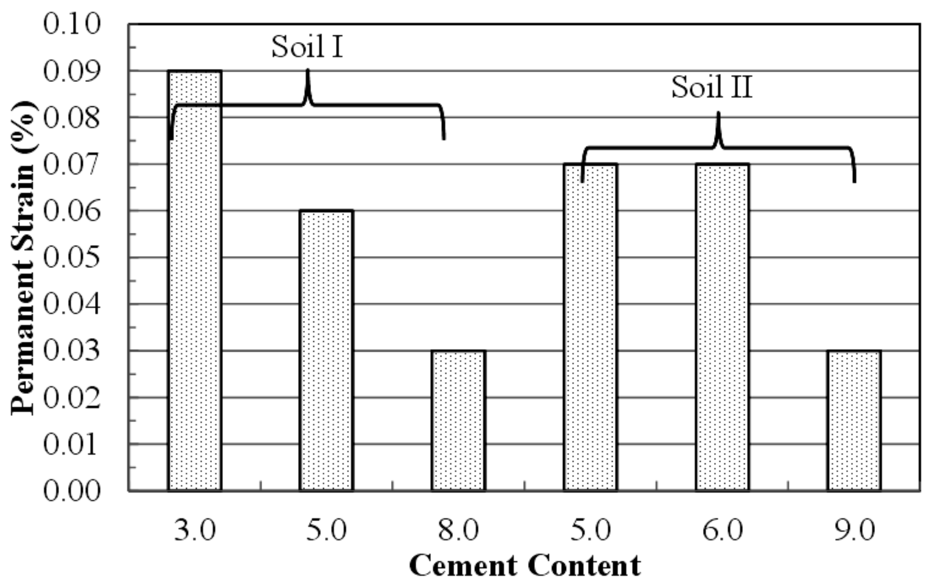


(b)



(c)

Figure 66. Permanent deformation of treated soils at different cement content

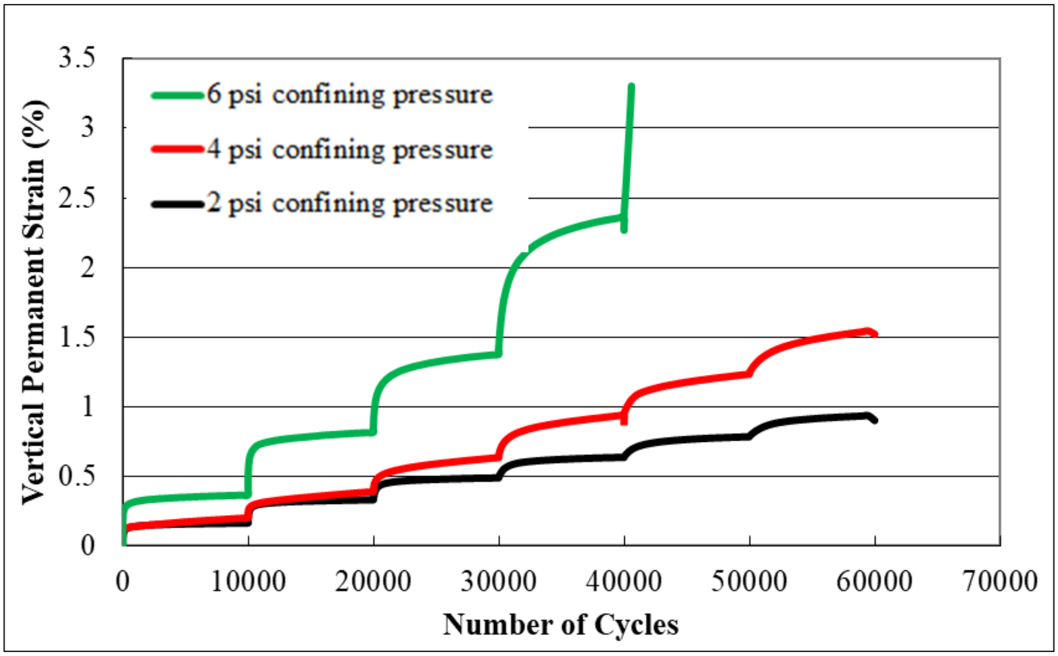


(d)

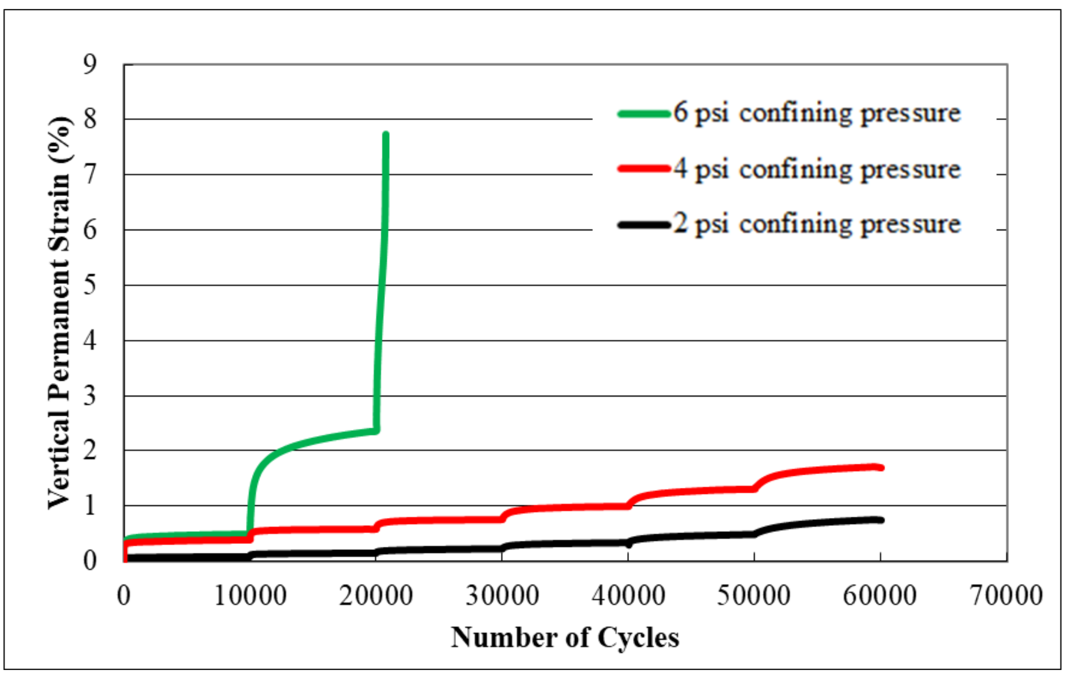
Results of Multi-stage Permanent Deformation Tests

Multi-stage permanent deformation tests were conducted to evaluate the behavior of treated/stabilized soil specimens for the case of moisture contents MC2 and under different deviatoric stresses and confining pressures (2 psi, 4 psi, and 6 psi). The plots of the measured permanent strains versus number of load cycles for the four soil type specimens at MC2 are presented in Figure 67 and Figure 68 for the 50 psi and 100 psi UCS, respectively. As expected, higher permanent strains were observed with higher stress ratio. Under an identical stress ratio, the specimens tested at higher confining stress showed higher measured permanent strains. This is because under a similar stress ratio of σ_1/σ_3, higher confining stress resulted in higher deviatoric stress. For example, at $\sigma_1/\sigma_3 = 6$, the corresponding deviatoric applied stresses are 10 psi, 20 psi, and 30 psi for confining stress of 2 psi, 4 psi, and 6 psi, respectively. Therefore, the larger measured permanent strain for high confining stress is mainly due to the application of larger deviatoric stress. In all curves presented, higher initial deformations were observed at the start of each new stress ratio, which is followed by a much lower strain accumulation rate throughout that stress ratio. Once the stress ratio increased, the deformation increased instantaneously, and the material attained a much lower strain accumulation rate after few hundred of cycle of loading. The initial higher deformation of the specimens can be attributed to the partial breakage of the bond between the stabilizer and soil caused by applying a higher deviatoric stress than the previous stage. The flat curves indicate that the specimens reached stable elastic condition at the specified deviatoric and confining stresses.

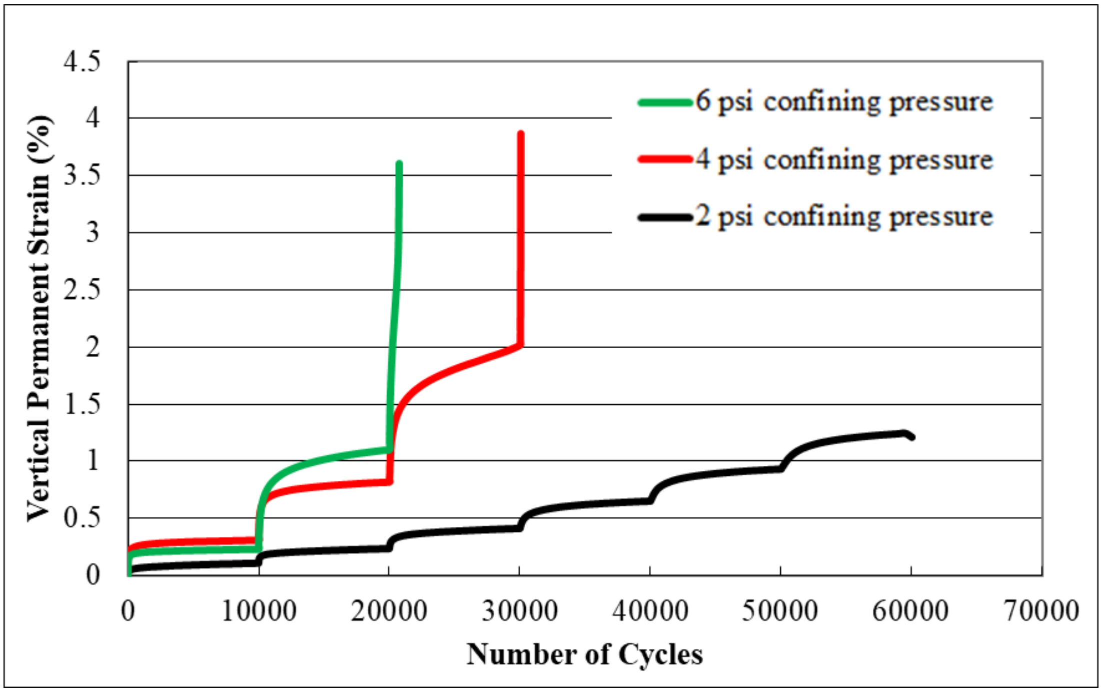
Figure 67. Multi-stage permanent deformation for treated/stabilized soil specimens at MC2 (UCS = 50 psi)



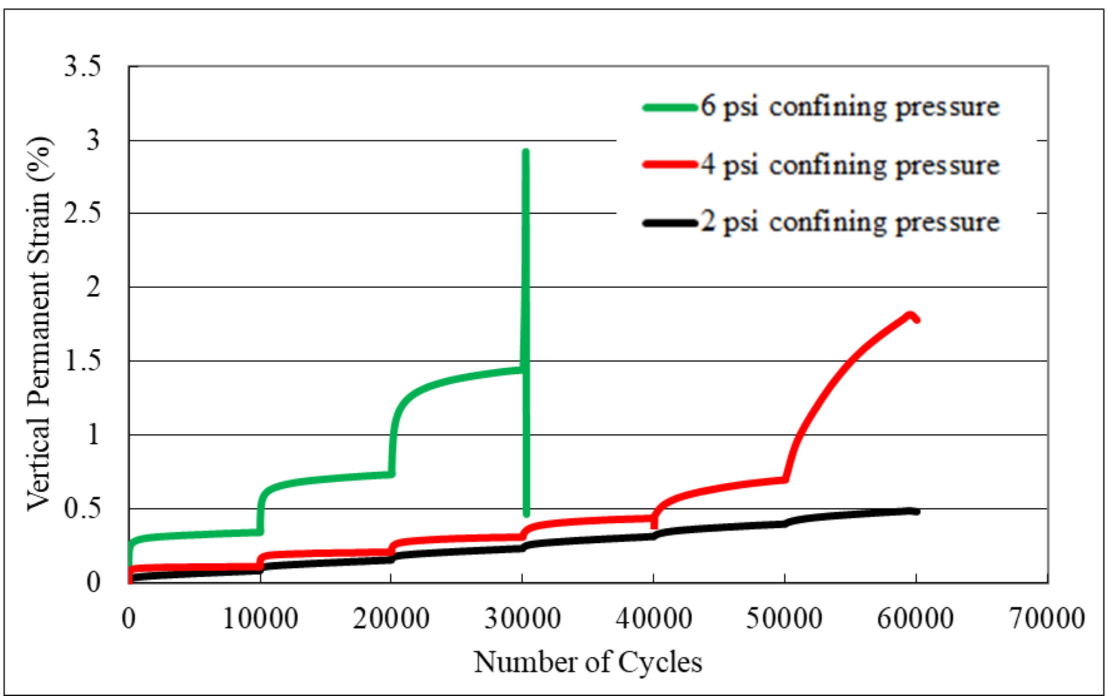
(a) Soil I



(b) Soil II

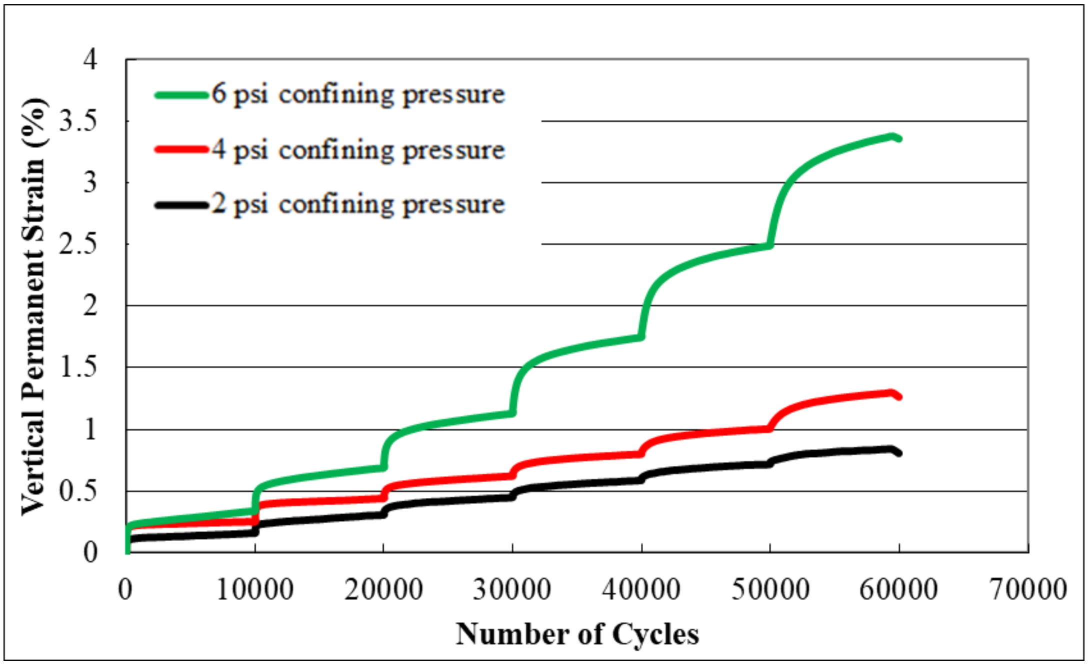


(c) Soil III

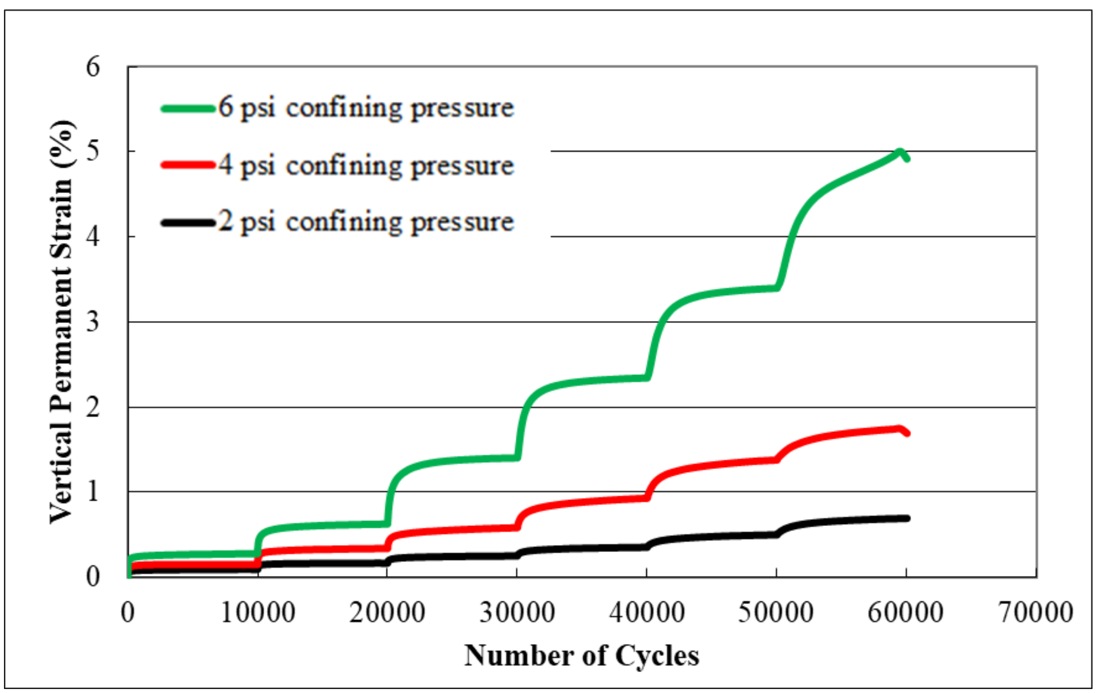


(d) Soil IV

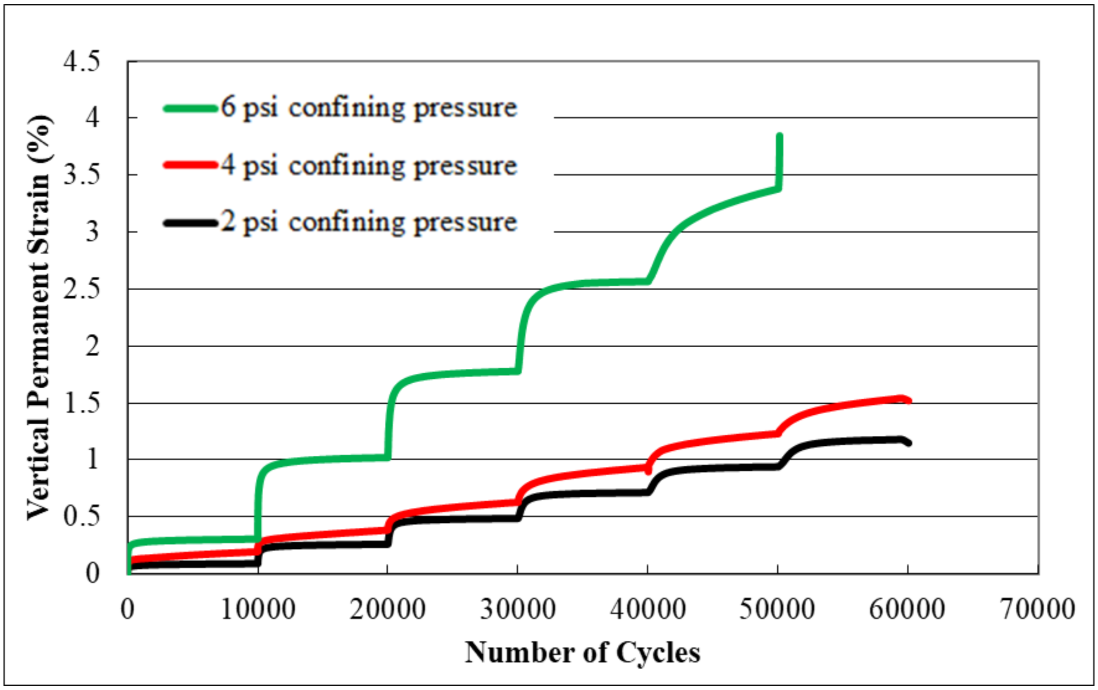
Figure 68. Multi-stage permanent deformation for treated/stabilized soil specimens at MC2 (UCS = 100 psi)



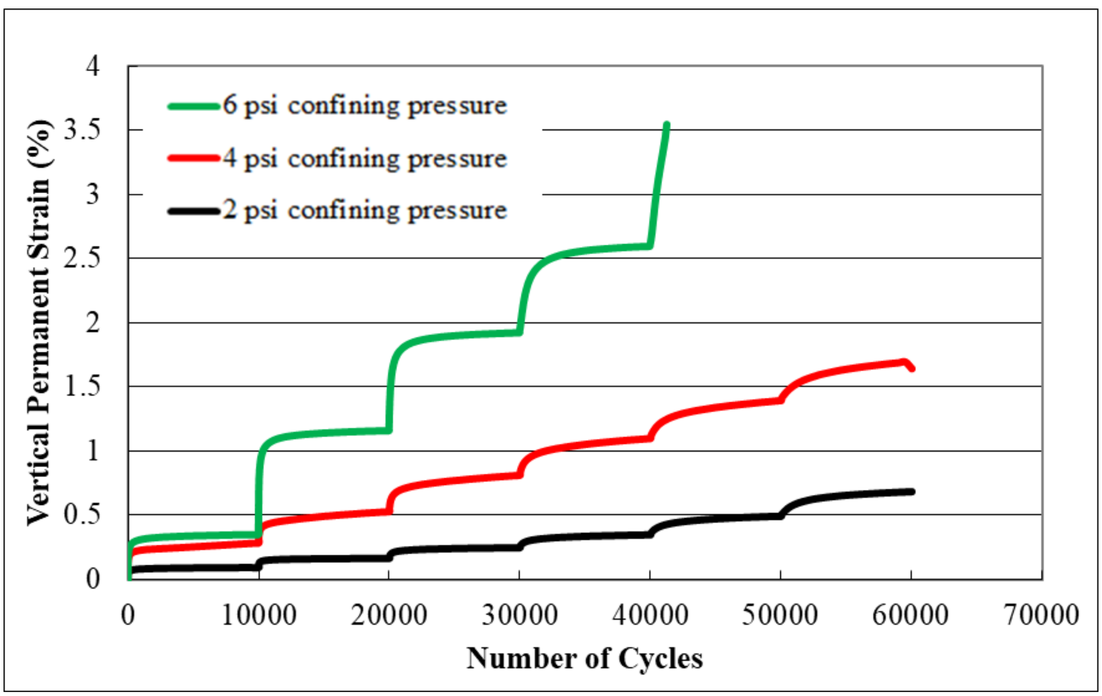
(a) Soil I



(b) Soil II



(c) Soil III



(d) Soil IV

Resilient Modulus Inputs of Flexible Pavement Systems

Results of resilient modulus obtained from laboratory testing of the treated/stabilized subgrade soils can be fed into the MEPDG analysis of flexible pavement sections. The typical pavement sections in use in low-volume, medium-volume, and high-volume roads were shown earlier in Figure 34 for the typical flexible pavement sections with additional stabilized subgrade layer. All three pavement systems consisted of four main layers: asphalt concrete top layer, an unbound aggregate or soil-cement base, treated/stabilized subgrade, and natural subgrade. The confining and deviatoric stresses for the treated/stabilized subgrade layers present in the three typical pavement structures are first calculated using KENLAYER. The resilient moduli are then estimated with k1, k2, and k3 constants in Table 25 and the corresponding values are presented in Table 27 for UCS = 50 psi and Table 28 for UCS = 100 psi.

Table 27. Recommended resilient modulus (ksi) for treated/stabilized subgrade (Phase II, UCS=50 psi)

Soil Type	Low-volume		Medium-volume		High-volume	
	Stone Base	Soil Cement Base	Stone Base	Soil Cement Base	Stone Base	Soil Cement Base
PI≤15	7	6	6	5	5	5
16≤PI≤25	9	8	8	7	8	7
26≤PI≤35	8	7	7	7	7	6
PI>35	7	6	7	6	6	5

Table 28. Recommended resilient modulus (ksi) for treated/stabilized subgrade (Phase II, UCS=100 psi)

Soil Type	Low-volume		Medium-volume		High-volume	
	Stone Base	Soil Cement Base	Stone Base	Soil Cement Base	Stone Base	Soil Cement Base
PI≤15	11	10	9	9	9	8
16≤PI≤25	12	10	10	9	9	9
26≤PI≤35	13	12	12	12	11	11
PI>35	10	9	8	8	8	7

Phase II-b

Cyclic Plate Load Tests

Cyclic plate load tests were performed on several test sections at the Accelerated Load facility (ALF) site. The cyclic plate loads were applied using an MTS hydraulic actuator, which has a force rating of 22 kips and a dynamic stroke of 6 in. Figure 69 presents a photo of the outdoor setup of the cyclic plate loading testing. The cyclic load was applied through a steel rod that fits into a concave-shaped hole on the loading plate that sits on the surface of the tested layer. The loading plate was a 1-in. thick steel plate and 12 in. diameter. The maximum applied load in tests was 12,000 lb, which resulted in a loading pressure of 106 psi, which simulated dual wheels under an equivalent 18,000 lb single axle load. As shown in Figure 70, the load pulse has a linear load increase from 500 lb to 12,000 lb in 0.3 second, followed by a 0.2-second period where the load is held constant at 12,000 lb, then followed by a linear load decrease to 500 lb over a 0.3-second period. A rest period of 0.5-second at 500 lb was then applied before the next loading cycle is applied. This load pulse results in a frequency of 0.77 Hz.

A total of ten test sections were constructed and tested at ALF site, in which, six sections were selected based on the results of Phase II study; while the other four sections were selected from a previous study on micro cracks of cement stabilized soils. The Phase II selected test sections were constructed inside a 5 ft. wide × 5 ft. long × 1 ft. deep trench in the field at ALF site. The layout of the six test sections from Phase II is presented in Figure 71(a). Table 29 provides summary for these six constructed test sections. These sections were constructed with selected cementitious materials including either low PI (PI=11) or heavy clay (PI=53) soil. The dosage of stabilizer (lime, cement, fly ash, and lime/fly ash) was determined based on the results of Task 2 (Table 24). Each section consisted of only a newly-built treated/stabilized subgrade layer over existing subgrade. All test sections were not surfaced (i.e., no asphalt surfacing) and endured any traffic loading during the study. The length of each the four cement soil sections for the micro crack project was 70 ft. long by 7 ft. width as shown in Figure 71(b). The cement soil sections were designed with a minimum 7-day UCS of 150 and 300, as shown in Table 30. The cement soil sections were constructed at the optimum moisture content with a dry density of 104 pcf. After construction, all the sections were sprayed with water and covered by a plastic sheet for a 7-day curing time. The cement-soil section 7 and section 8 were constructed using 8 percent cement maintaining 8.5-in. thickness for each section; while section 9 and section 10 were constructed using 6 percent cement maintaining 12-in. thickness. Table 30 provides summary for the four soil cement sections that were constructed at ALF for the micro crack project. An A-2-7 (AASHTO) material was used for the construction of these four sections. Figure 71(b) presents the soil cement sections that were previously constructed at ALF for the micro crack project (i.e., no micro crack, low micro crack, medium micro crack, and high micro crack).

Figure 69. Outdoor setup of cyclic plate load testing facility

Figure 70. Load pulse applied in the cyclic load test

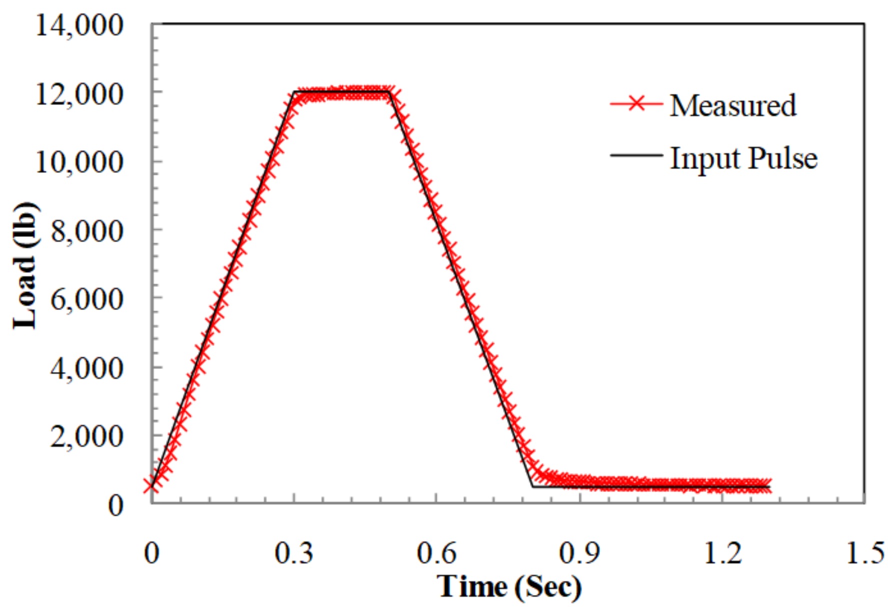


Table 29. Summary of the constructed test sections from Phase II study

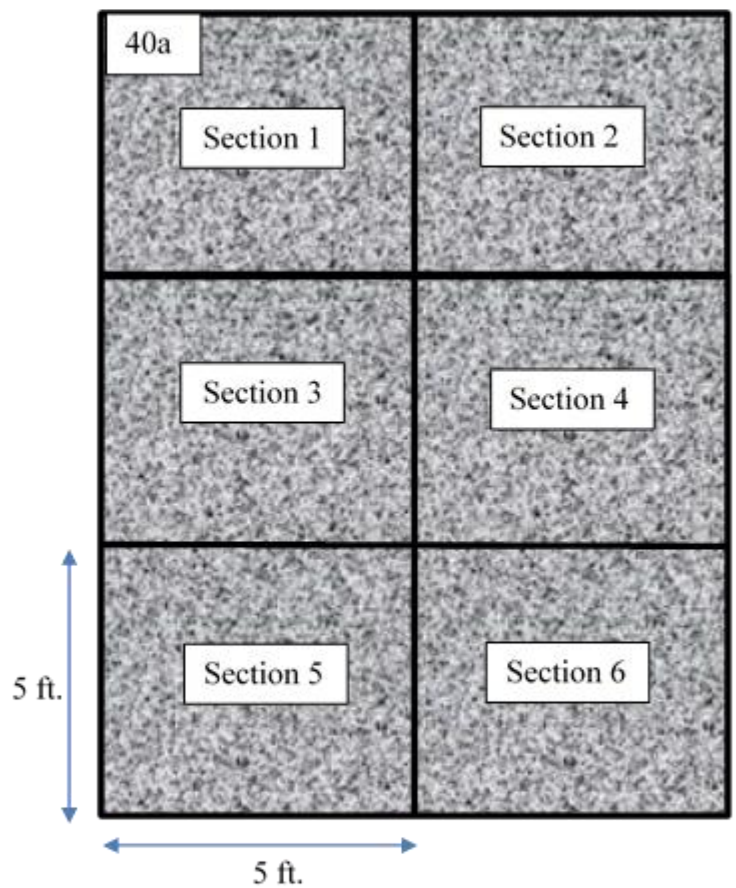
Test	Soil Type	Subgrade Soil	Stabilizer	Target 7-day UCS for Subgrade (psi)	Remarks
1	Low PI Clay	Embankment	8% Lime + 15% Fly Ash	50 for MC3	PI≤15
2	Heavy Clay	Embankment	6% Lime + 5% Cement	100 for MC3	PI≥35
3	Low PI Clay	Embankment	7% Cement	100 for MC3	PI≤15
4	Heavy Clay	Embankment	10% Lime + 15% Fly Ash	50 for MC2	PI≥35
5	Low PI Clay	Weak	5% Cement	100 for MC2	PI≤15
6	Heavy Clay	Weak	6% Lime + 4% Cement	100 for MC2	PI≥35

MC: Moisture content

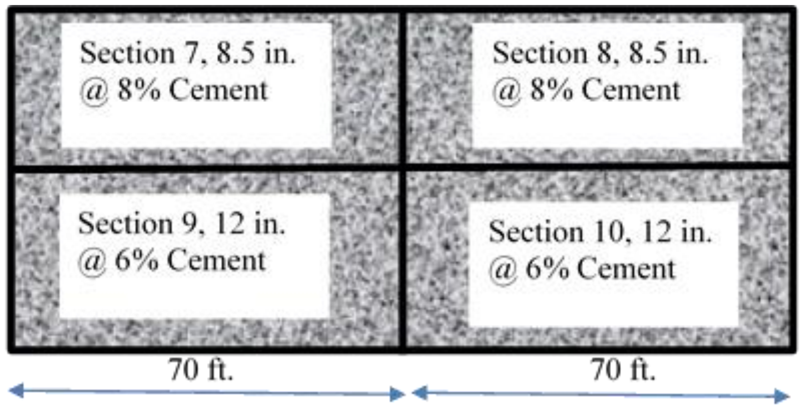
Table 30. Summary of the constructed soil cement sections for the micro-crack project

Test	Soil Type	Subgrade Soil	Thickness/Stabilizer	Target 7-day UCS for Subgrade (psi)	Remarks
7	Low PI Clay	Embankment	8.5 in. / 8% Cement (No MC)	≥300	PI≤15
8	Low PI Clay	Embankment	8.5 in. / 8% Cement (Low MC)	≥300	PI≤15
9	Low PI Clay	Embankment	12 in. / 6% Cement (No MC)	≥150	PI≤15
10	Low PI Clay	Embankment	12 in. / 6% Cement (Medium MC)	≥150	PI≤15

Figure 71. Layout of soil cement sections



(a) Six test sections from Phase II study



(b) Four test sections from micro cracks of cement stabilized soils study

Results and Analysis for Phase II-b

Cyclic plate load tests were performed to evaluate the composite resilient modulus ($M_{R\text{-}comp}$) and the permanent deformation characteristics of the different treated/stabilized sections at ALF. The tests were performed following the multi-state load testing procedure to evaluate the in-situ $M_{R\text{-}comp}$. Prior to tests, the sections were first conditioned by applying 1,000 load cycles to remove the irregularities on the top surfaces of the test sections and to suppress most of the initial stage of permanent deformation. After conditioning, multi-stage loading was conducted by applying 10,000 cycles for each deviatoric stress (total = 50,000 cycles). Selected deviatoric stresses for this test are 9, 27, 44, 62, and 80 psi (Figure 72).

Figure 73 to 76 present the results of multi-stage tests for sections 1 to 4 of Phase II that were constructed over embankment soil at ALF site. The maximum permanent deformation observed at stage 5 loading ranged from 0.11 in. to 0.17 in. The results of multi-stage tests, the elastic strains toward the end of each losing stage, were used to evaluate the composite resilient modulus ($M_{R\text{-}comp}$) for the four test sections 1 to 4 under different deviatoric stresses. The corresponding $M_{R\text{-}comp}$ for sections 1 to 4 are presented in Figure 77, which ranged from about 7.8 ksi to 12.2 ksi. The M_R values for the treated/stabilized layers will be disused and evaluated in next section. The results of multi-stage tests for sections 5 and 6 of Phase II, which were constructed over weak subgrade soil at ALF site, are presented in Figure 78 and 79, respectively. The maximum observed permanent deformations at stage 5 loading ranged from 0.70 in. to 0.75 in., much higher than the deformations observed for sections 1 to 4. This is mainly due to deformations of the weak subgrade soil during multi-stage testing. The composite resilient modulus ($M_{R\text{-}comp}$) for the test sections 5 and 6 were evaluated under different deviatoric stresses, and the results are presented in Figure 80. Values of $M_{R\text{-comp}}$ for the test sections 5 and 6 ranged from about 7.0 ksi to 12.0 ksi.

Figure 81 through 84 depict the results of multi-stage tests that were performed on the cement-stabilized sections 7 to 10 of the micro crack project, which were constructed over embankment soil at ALF site. The maximum permanent deformation observed at stage 5 loading ranged from 0.08 in. to 0.1 in. The results of multi-stage tests were used to evaluate the composite resilient modulus ($M_{R\text{-comp}}$) for the test sections 7 to 10 under different deviatoric stresses, and the results are presented in Figure 85. The corresponding values of $M_{R\text{-comp}}$ ranged from about 15.2 ksi to 27.4 ksi. The M_R values for the treated/stabilized layers of all test sections (1 to 10) will be discussed and evaluated in the next section.

Figure 72. Multi-stage loading

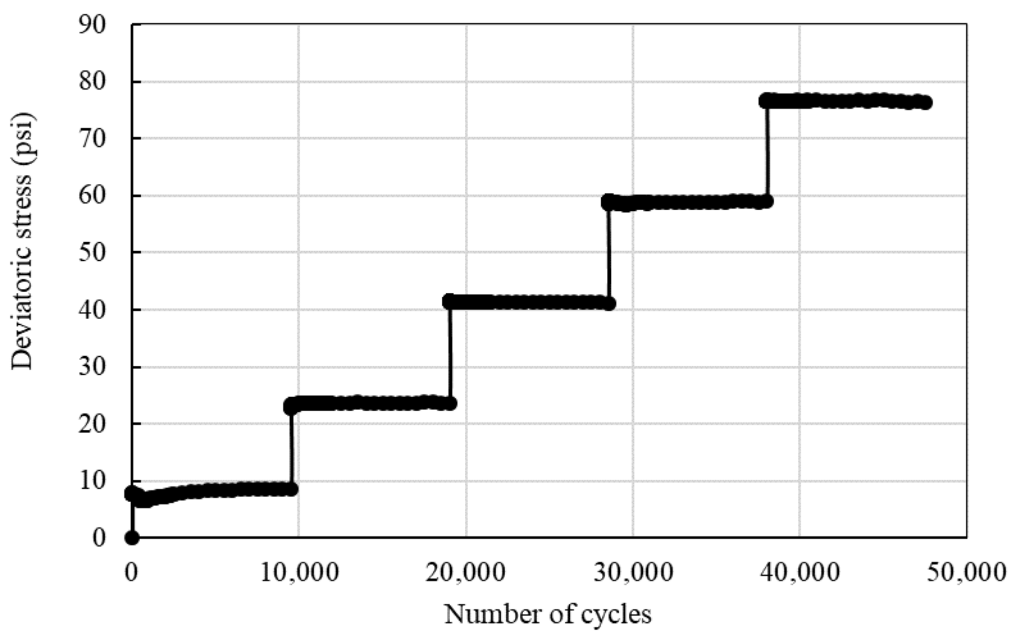


Figure 73. Multi-stage results for section 1

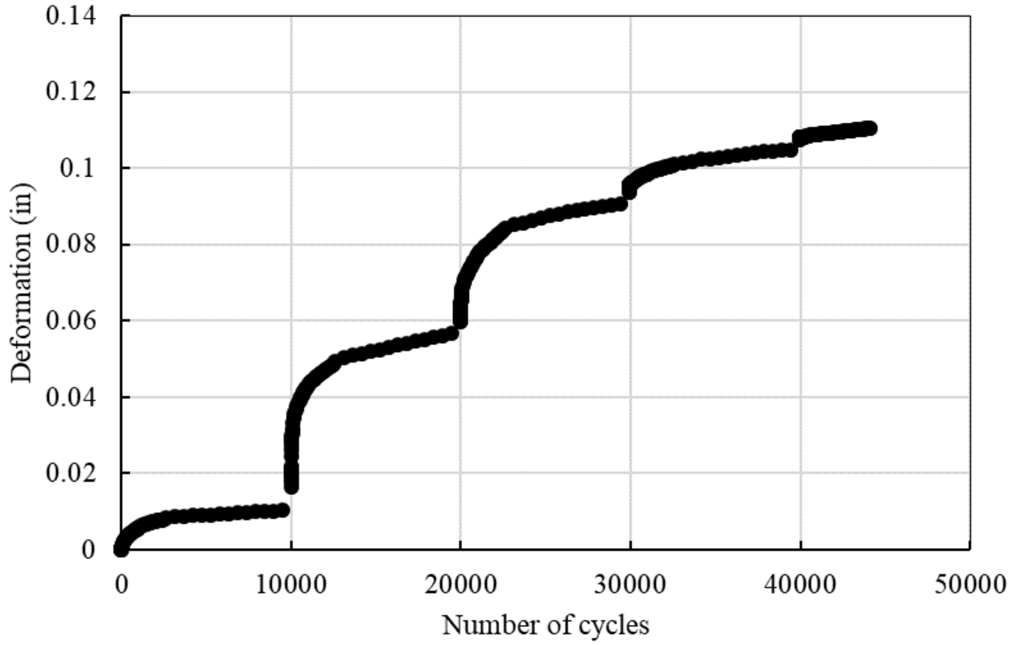


Figure 74. Multi-stage results for section 2

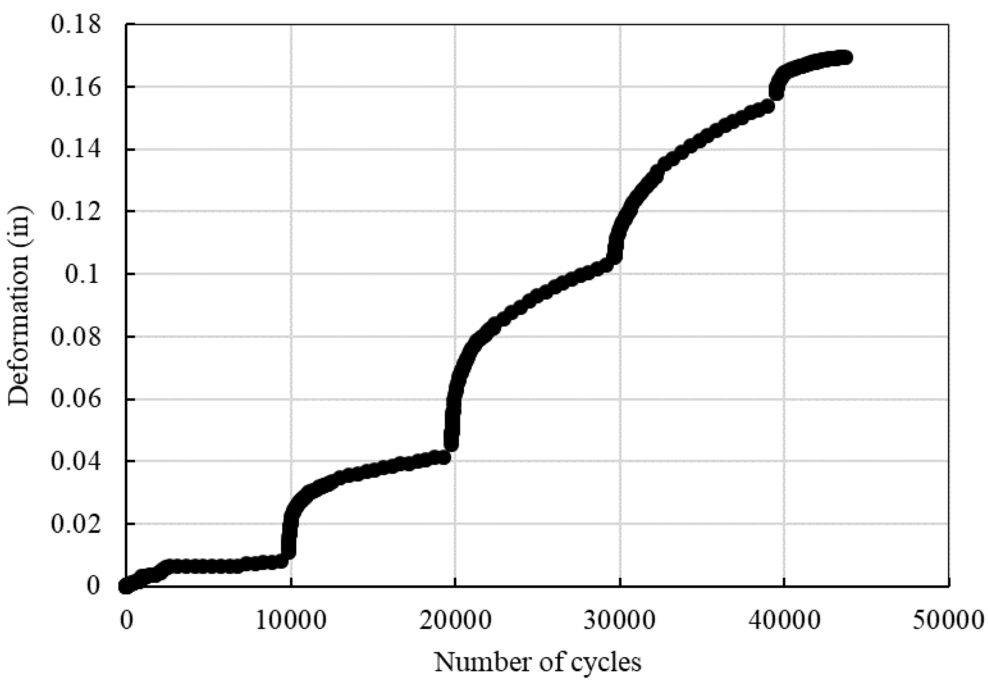


Figure 75. Multi-stage results for section 3

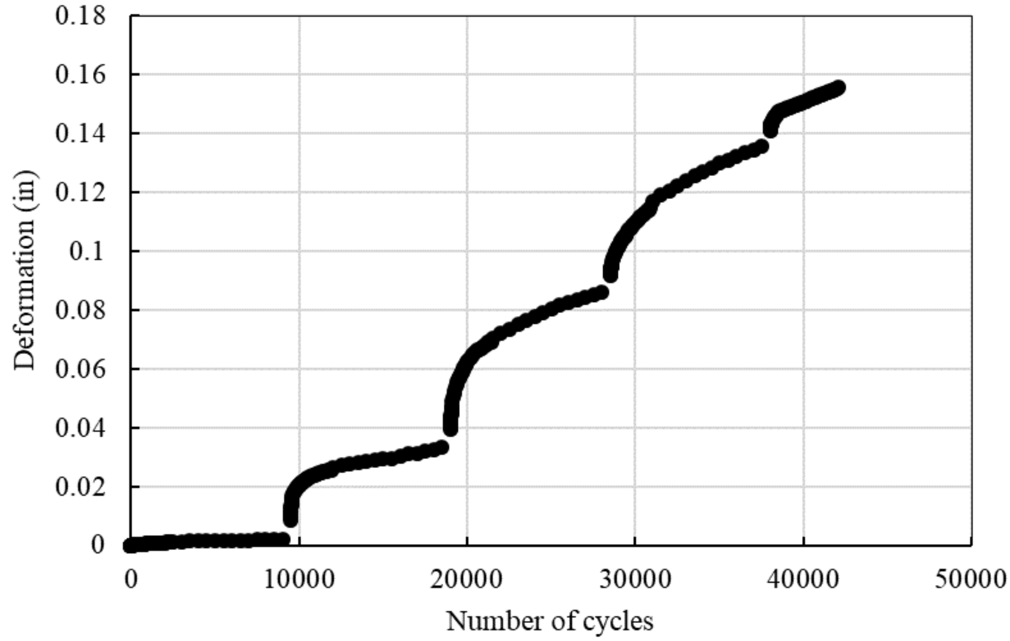


Figure 76. Multi-stage results for section 4

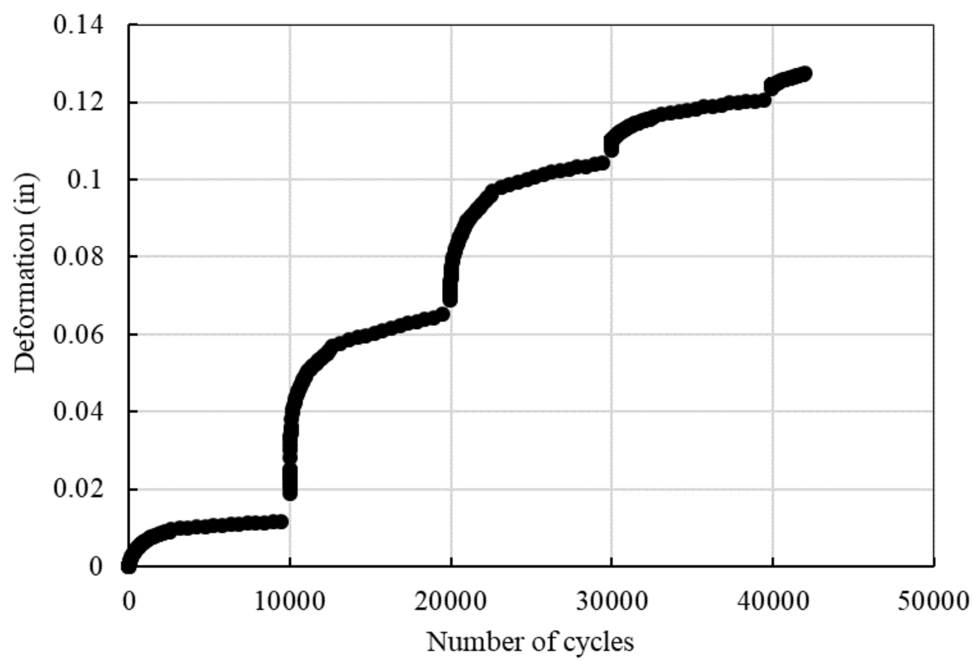


Figure 77. Results of composite resilient modulus for Phase II test sections 1 to 4

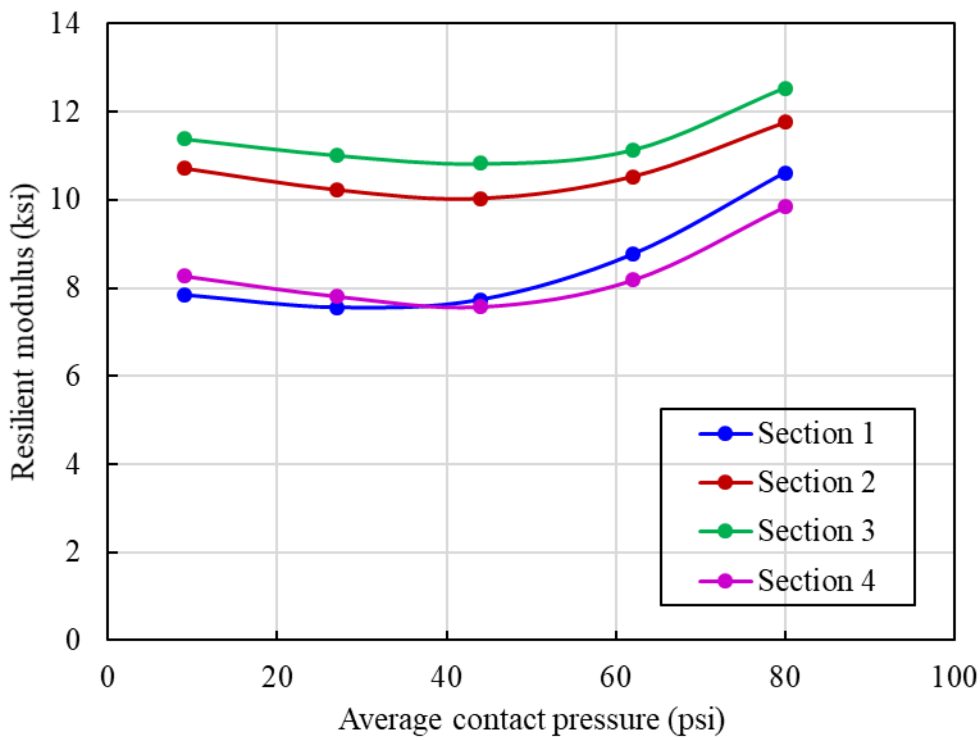


Figure 78. Multi-stage results for section 5 (weak subgrade)

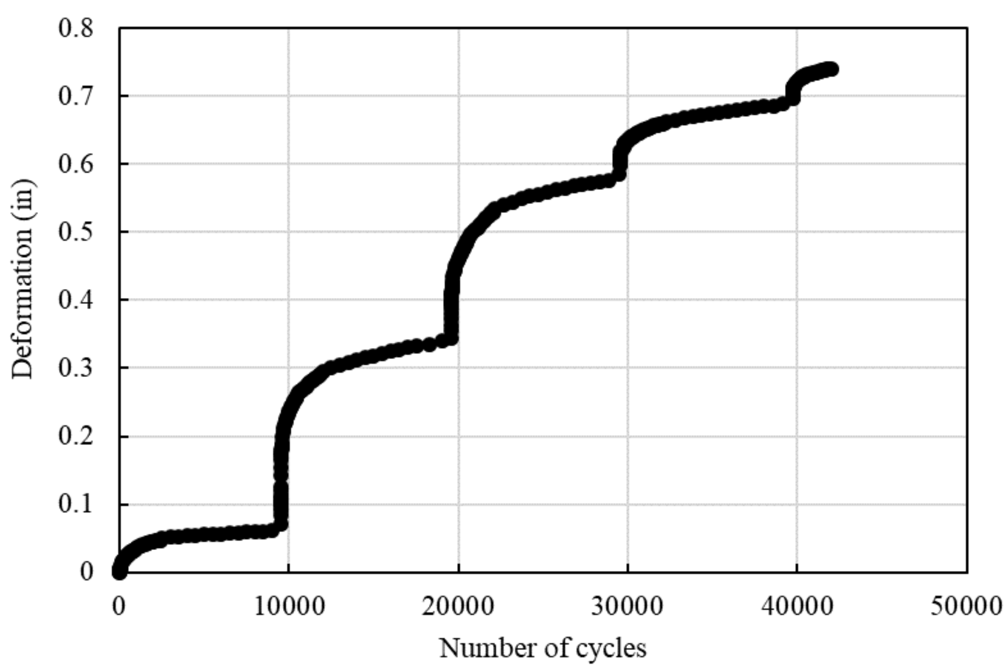


Figure 79. Multi-stage results for section 6 (weak subgrade)

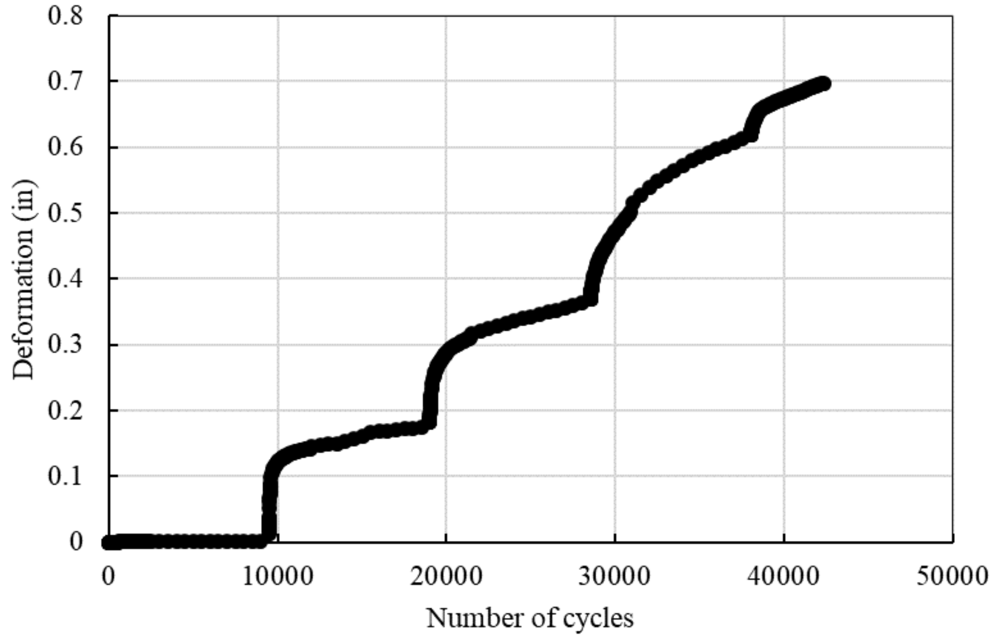


Figure 80. Results of resilient modulus for Phase II test sections 5 and 6

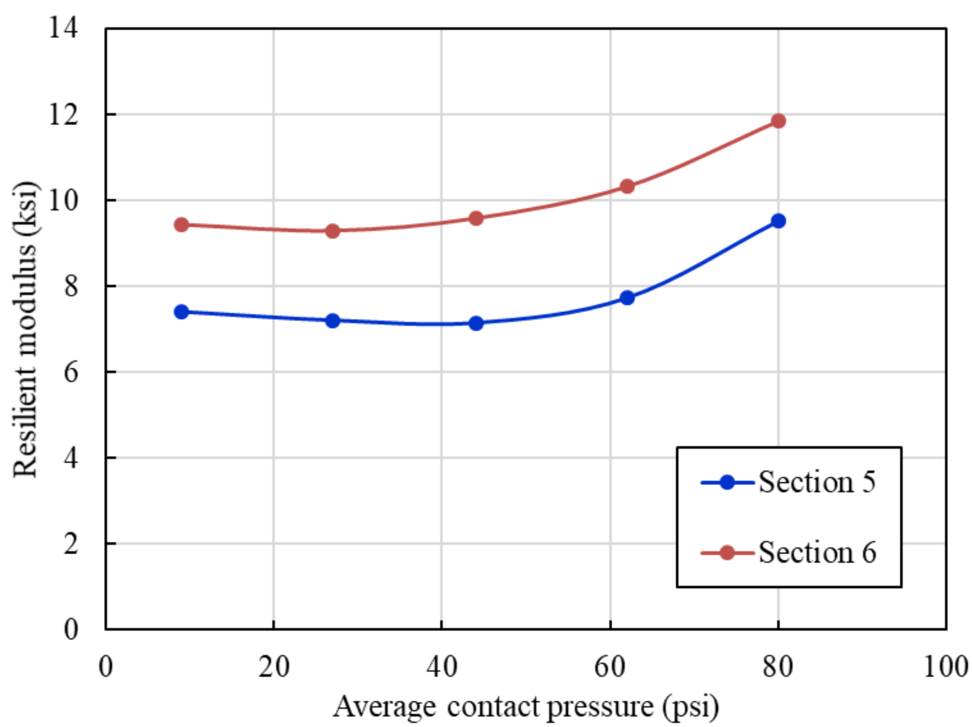


Figure 81. Multi-stage results for section 7

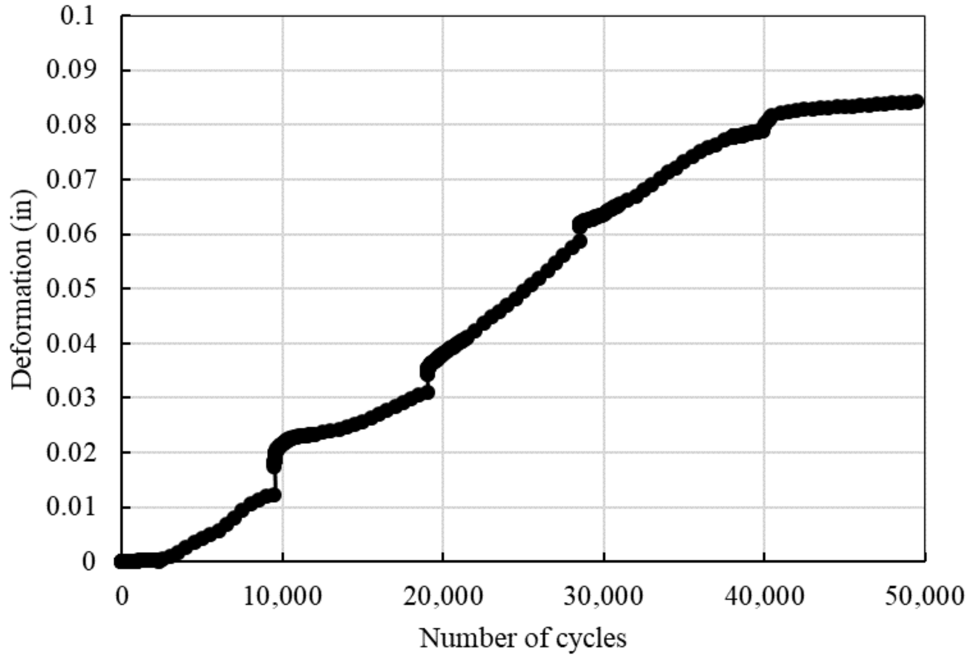


Figure 82. Multi-stage results for section 8

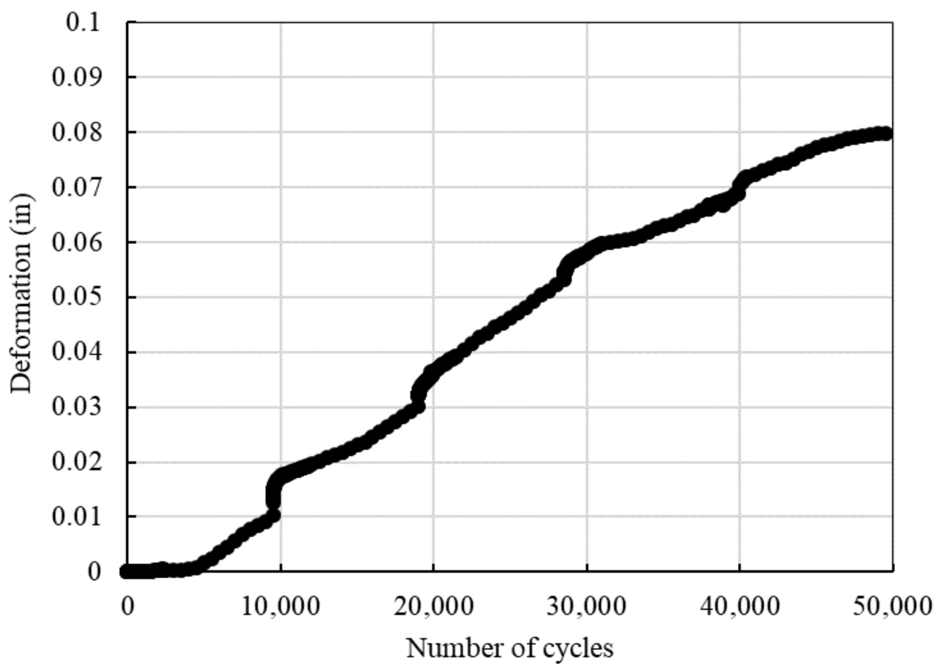


Figure 83. Multi-stage results for section 9

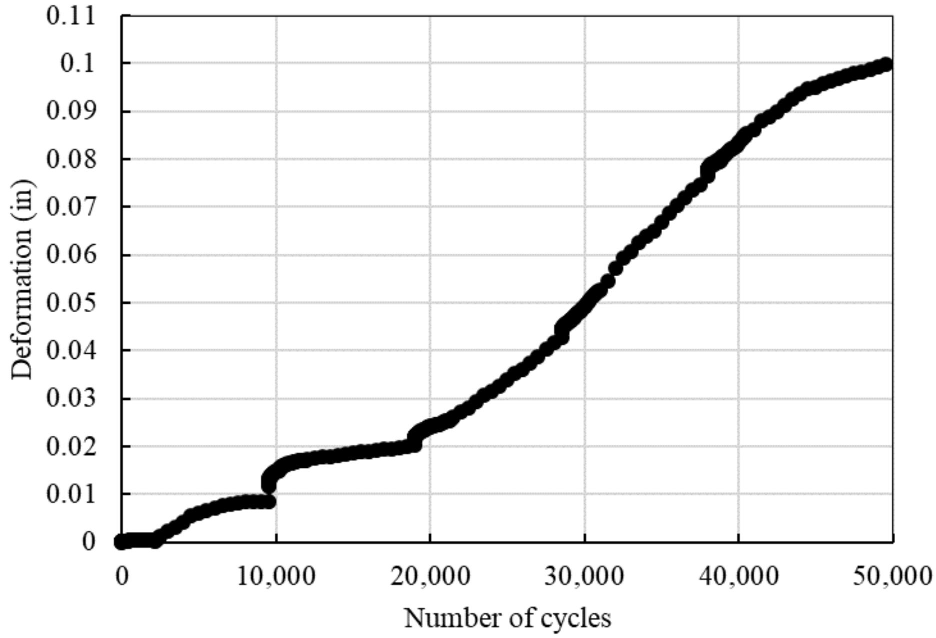


Figure 84. Multi-stage results for section 10

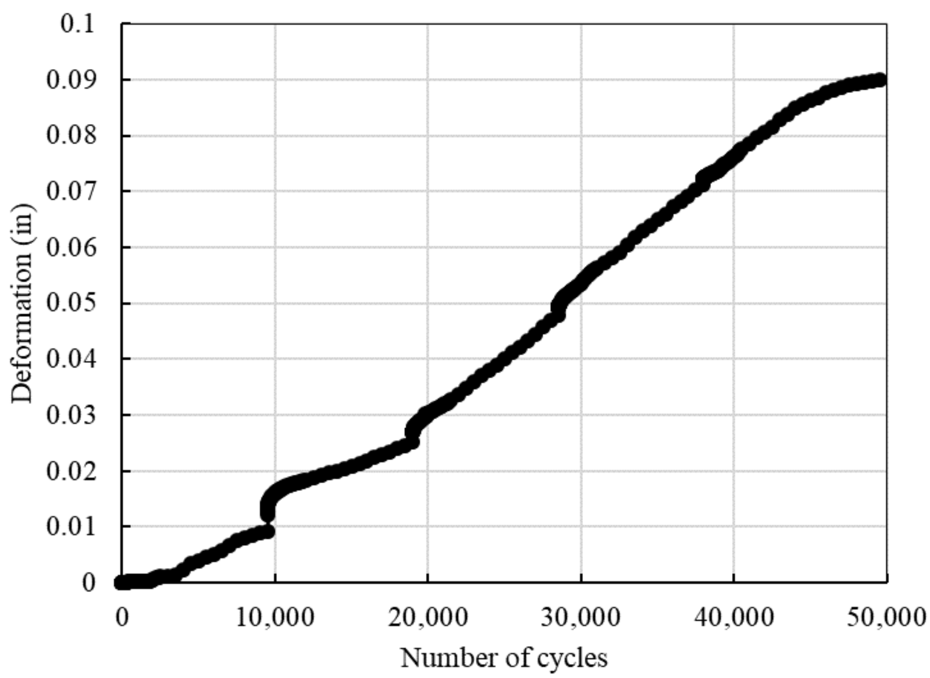
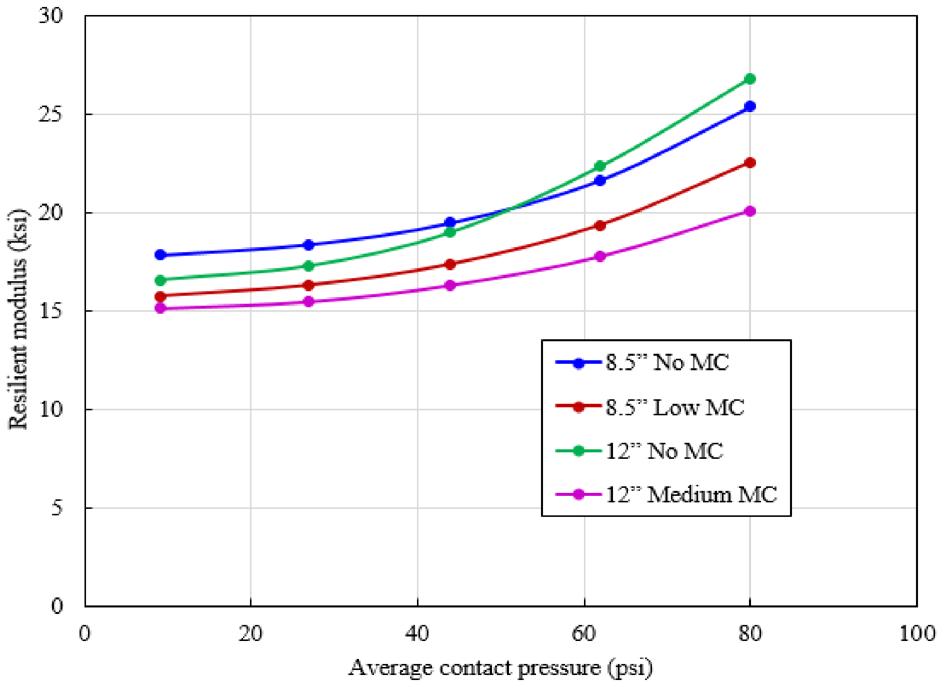


Figure 85. Results of resilient modulus for micro-crack test sections 7 to 10



Back-calculate the Resilient Modulus

Vennapusa and White [15] discussed the theoretical determination of elastic modulus based on the Bousinnesq elastic solution. The relationship between applied stress and soil displacement for a base resting on an elastic half-space geomaterial is given in Equation (27). The elastic moduli are derived from the falling weight deflectometer (FWD) data using the following equation:

$$E = \frac{(1-\nu^2)\sigma_0 a}{d_0} f \qquad [27]$$

where, E = elastic modulus (MPa); d_0 = measured displacement (mm); υ = Poisson's ratio; σ_0 = applied stress (MPa); a = radius of the loading plate (mm); and f = shape factor depending on stress distribution and geomaterial, in which f = 8/3 for granular materials and f = π/2 for non-granular materials.

The influence depth of the PLT is about two times its diameter. Since the thicknesses of the tested treated/stabilized layers in this study were ≤ 12 in., the influence zone of PLT (12 in. diameter) reached the underlying subgrade layer down to about 24 in. Therefore, the resilient modulus obtained from the cyclic PLT reflects the $M_{R\text{-comp}}$ of two-layers system rather than the true M_R of the tested treated/stabilized layer.

In this study, the Odemark method [16] (referred to as the Method of Equivalent Thickness (MET)) is used to back-calculate the M_R of the treated/stabilized layers from the measured composite $M_{R\text{-comp}}$ value obtained from the cyclic PLT modulus for the two-layer systems. According to this method, layers of different stiffnesses are first transformed to an equivalent layer of the same stiffness, such that Boussinesq's equations for homogeneous elastic half-space media can be applied to predict stresses and deflections of tested layers. For example, for a two-layers system with E_1 and E_2 are the stiffness moduli of the first (upper) and the second (lower) layer, respectively, the following equation can be used to transform the thickness of the first layer into an equivalent layer with the stiffness modulus equal E_2 [30]:

$$h_e = f h_1 \sqrt[3]{\frac{E_1}{E_2}} \qquad [28]$$

where, h_e = the equivalent thickness of the first layer; h_1 = thickness of the first layer; and f = an adjustment factor, taken to be 0.9 for the two-layers system, and 1.0 for the multi-layers system.

Von Quintus and Killingsworth [17] calculated a single equivalent M_R ($M_{R(Equivalent)}$) for an entire multilayer pavement system using equation (29) for the case when the M_R is larger for the upper layer. When the upper layer resilient modulus is smaller than the underlying layer, the design input is the smaller value.

$$M_{R(Equivalent)} = \frac{D_{S1}^3\, M_{R1} + D_{S2}^3\, M_{R2}}{(D_{S1})^3 + (D_{S2})^3} \qquad [29]$$

where, M_{R1} = resilient modulus of the upper layer; M_{R2} = resilient modulus of the lower layer; D_{S1} = thickness of the upper layer; and D_{S2} = thickness of the lower layer.

In this study, dynamic cone penetration (DCP) tests were conducted on the ten treated/stabilized test sections at ALF, which provide continuous measurements of DCP data for the two-layer system that extends enough below the influence depth of the cyclic PLT (> 24 in.). It was assumed in this study that the ratio of $(E_1/E_2)_{DCP}$ for the two-layers obtained from the DCP data is the same as the ratio of $(E_1/E_2)_{CPLT}$ obtained from the CPLT test for the two-layer system [i.e., $(E_1/E_2)_{DCP} = (E_1/E_2)_{CPLT}$].

A performance indicator, which can combine the effects of the treated/stabilized soil layer and underlying subgrade layer within the influence zone is needed to qualitatively evaluate the performance of each test section. The equivalent modulus of elasticity (or composite resilient modulus) of the treated/stabilized soil layer and the subgrade layer can be used as an indicator. However, conventionally, only the thicknesses of individual layers are taken into consideration in the evaluation of the equivalent modulus of elasticity or stiffness for a layered system, and the relative position of individual layer is ignored. Equation (30), which is adopted in the reference manual of the National Highway Institute training course: Introduction to Mechanistic-Empirical Pavement Design, is one such example [41].

$$E_{eq} = \left(\frac{E_{treated}^{1/3} h_{treated} + E_{subgrade}^{1/3} h_{subgrade}}{h_{treated} + h_{subgrade}} \right)^3 \qquad [30]$$

where, E_{eq} is the equivalent elastic (or resilient) modulus of the treated/stabilized and subgrade layers; $E_{treated}$ is the elastic (or resilient) modulus of the treated/stabilized soil layer, = $M_{R(treated)}$; $h_{treated}$ is the thickness of the treated/stabilized soil layer; $E_{subgrade}$ is the elastic (or resilient) modulus of the underlying subgrade layer, = M_R(subgrade); and subgrade is the thickness of the underlying subgrade layer within the influence zone of PLT.

Large differences can exist between as constructed and design strength values of pavement layers. As such, the dynamic cone penetrometer (DCP), light falling weight deflectometer (LFWD), and Geogauge were deployed to evaluate the in-situ strength/stiffness of field test sections at the time when the cyclic plate load tests were performed. The in-situ material properties of base and subgrade were measured prior to the cyclic loading testing. Each in situ test was conducted at a minimum of five locations along the longitudinal direction for each test section. The results of LFWD and Geogauge are presented in Table 31.

Table 31. Summary of Geogauge and LFWD elastic moduli for the ALF test sections

Test section	Geogauge		LFWD	
	Mean (ksi)	SD	Mean (ksi)	SD
Section 1	13.60	2.11	9.66	1.01
Section 2	14.22	1.78	8.63	0.80
Section 3	12.43	1.15	9.19	1.19
Section 4	10.93	2.04	9.21	1.25
Section 5	6.90	1.39	7.28	1.06
Section 6	7.89	1.12	7.58	0.52
Section 7	19.55	2.15	16.05	1.38
Section 8	16.29	1.76	14.12	2.08
Section 9	19.38	2.13	13.47	1.64
Section 10	15.86	1.49	12.87	1.13

The results of DCP tests were used to evaluate the resilient moduli of the treated/stabilized layer and subgrade, using the California bearing ratio CBR-DCPI relationship suggested by Von Quintus and Killingsworth [52] and the M_R-CBR relationship suggested by Webster et al. [53] as described in equation (31). Here, the DCPI (dynamic cone penetration index) is defined here as the penetration rate (penetration /drop) measured in in./blow (or mm/blow).

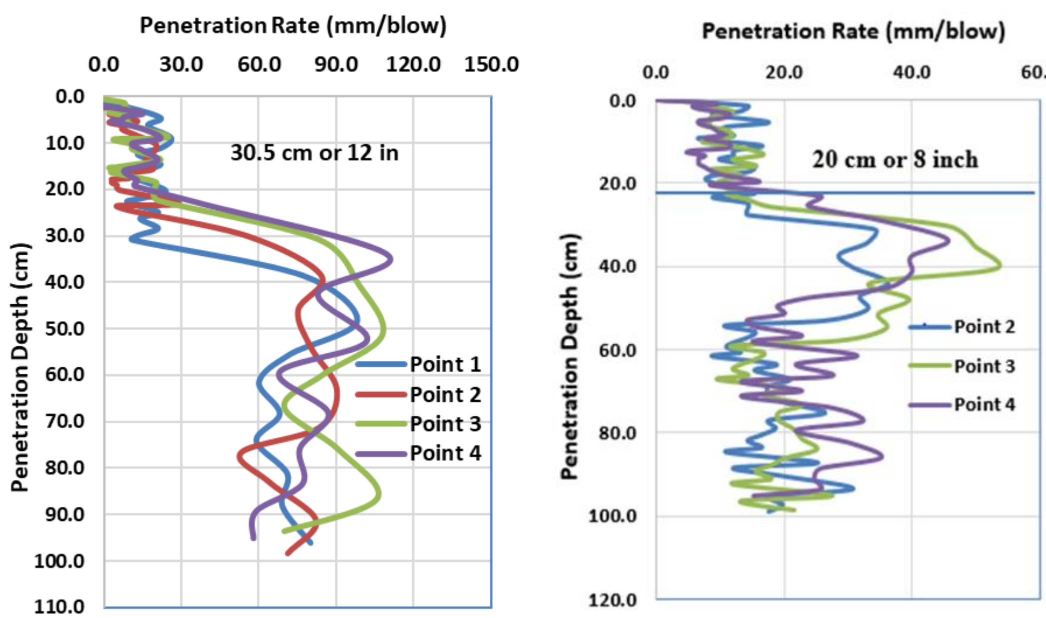
$$\left.\begin{array}{r} CBR = 292/(DCPI)^{1.12} \\ M_R(MPa) = 17.58 \times CBR^{0.64} \end{array}\right\} \xrightarrow{yields} M_R(MPa) = 665.06/(DCPI)^{0.7168} \qquad [31]$$

DCP tests were conducted on each of the treated/stabilized test sections constructed at ALF site at four different locations. Figure 86 presents two examples of the DCP tests conducted on sections 2 and 8, respectively. The average DCP index (DCPI) for each test section at ALF site were calculated and used to estimate the resilient modulus, M_R, for the subgrade and treated/stabilized layers as shown in Table 32. The composite resilient modulus ($M_{R\text{-}comp}$) for each test section was calculated using both equations (29) and (30) and presented in Table 32, which are also compared with the measured $M_{R\text{-}comp}$ (last column) from the cyclic plate load tests. The table shows that the CPLT measured and DCPI estimated $M_{R\text{-}comp}$ are generally in good agreement.

In order to back-calculate the measured resilient moduli for the treated/stabilized test sections, as stated earlier, we assumed in this study that the ratio of $(E_1/E_2)_{DCP}$ for the two-layers system obtained from the DCP data is the same as the ratio of $(E_1/E_2)_{CPLT}$

obtained from the CPLT test for the two-layer system. Based on this assumption, the measured resilient moduli for the treated/stabilized test sections from CPLTs were back-calculated using both equations (29) and (30), which are also compared with the laboratory measured M_R values for Phase II as shown in Table 33. The table shows that the back-calculated M_R values of the treated/stabilized soil layers from CPLT are consistent and in good agreement with the results of laboratory M_R tests for Phase II-a in this study.

Figure 86. Results of DCP tests for sections 2 and 8 at ALF



(a) Section 2 (b) Section 8

Table 32. Summary of calculated and measured composed resilient moduli

Test section	Subgrade		Treated/stabilized section		Calculated $M_{R\text{-comp}}$ (Eq 29) (ksi)	Calculated $M_{R\text{-comp}}$ (Eq 30) (ksi)	Measured $M_{R\text{-comp}}$ (ksi)
	DCPI (inch/blow)	M_R (ksi)	DCPI (inch/blow)	M_R (ksi)			
Section 1	1.313	7.81	1.043	9.21	8.51	8.49	7.84
Section 2	1.572	6.86	0.474	16.21	11.54	10.88	10.72
Section 3	1.374	7.56	0.495	15.71	11.64	11.14	11.46
Section 4	1.461	7.23	0.853	10.64	8.94	8.83	8.28
Section 5	3.178	4.14	0.652	12.90	8.52	7.71	7.41
Section 6	3.232	4.09	0.463	16.48	10.29	8.90	9.43
Section 7	1.063	9.09	0.372	19.28	10.53	12.13	17.84
Section 8	1.254	8.07	0.404	18.18	9.50	11.04	15.72
Section 9	1.317	7.79	0.364	19.59	13.69	12.80	16.56
Section 10	1.267	8.01	0.387	18.75	13.38	12.63	15.14

Table 33. Back-calculated measured resilient moduli for the treated/stabilized test sections

Test section	Calculated M_R from DCPI (ksi)	Back-calculated M_R using Eq 29 (ksi)	Back-calculated M_R using Eq 30 (ksi)	Laboratory Measured M_R (ksi)
Section 1	9.21	8.49	8.51	7.52
Section 2	16.21	15.06	15.98	14.35
Section 3	15.71	15.48	16.16	14.52
Section 4	10.64	9.86	9.98	7.73
Section 5	12.90	11.22	12.39	10.47
Section 6	16.48	15.11	17.47	13.18
Section 7	19.28	32.67	28.38	-
Section 8	18.18	30.08	25.88	-
Section 9	19.59	23.69	25.35	-
Section 10	18.75	21.22	22.48	-

Conclusions

Both laboratory and field testing programs were conducted in this study to evaluate the benefits and properties of cementitious treated/stabilized subgrade soils in terms of resilient modulus and permanent deformation. The laboratory testing program included two phases: Phase I aimed at evaluating the performance-related properties (e.g., the resilient modulus and permanent deformation) of the current subgrade treatment/stabilization schemes for hauled soil using cement and/or lime stabilizer as provided in DOTD standard specifications (Table 8). Three soil types with different plasticity indices prepared at OMC ± 2% were used for Phase I. Phase II focused on examining the proper treatment/stabilization schemes for very weak subgrade soils at high moisture contents, and evaluating the corresponding resilient modulus and permanent deformation. Four soil types with different plasticity indices were selected and considered in Phase II that were prepared at three different moisture contents at the wet-side of optimum to produce a raw soil strength value of 25 psi or less. Repeated load triaxial tests (RLT) tests were performed in the laboratory testing program to evaluate the resilient modulus (M_R) and permanent deformation characteristics of the treated/stabilized specimens. The soils were treated using different combinations of lime and class C fly ash (or cement) to achieve target 7-day unconfined compression strength (UCS) values of 50 psi to create a working platform and 100 psi to stabilize the subgrade soil layer for subbase application. The field testing program (Phase II-b) involved constructing and performing cyclic plate load tests (CPLT) on ten (10) test sections at the Accelerated Load facility (ALF), in which six sections were selected based on the results of Phase II-a; while the other four sections were selected from a previous study on micro cracks of cement stabilized soils. The CPLTs were performed to measure the composite resilient modulus of the ten test sections at ALF site. The measured resilient moduli of the different test sections at ALF were back-calculated based on the assumption that the ratio of $(E_1/E_2)_{DCP}$ for the two-layers obtained from the DCP data is the same as the ratio of $(E_1/E_2)_{CPLT}$ obtained from the CPLT test.

Based on the results of this study, the following conclusions can be drawn:

Phase I - Laboratory Tests

- The stress-strain curves of the treated/stabilized hauled soils with cement and/or lime shift towards the left hand side associated with significant increase in compressive strength; hence, increasing both the elastic and shear moduli. The unconfined compressive strength (UCS) of treated/stabilized hauled soils prepared at OMC ± 2% increased from 18 to 29 psi (for untreated soil) to 150 to 434 psi after 28 days of curing time.
- Results of tube suction tests showed that the maximum dielectric value (DV) for the treated/stabilized soil #1 was less than 10, which means good quality material according to TxDOT's criteria; while for the treated/stabilized soil #2 and soil #3, the maximum DVs were between 10 and 16, which means marginal material based on TxDOT's criteria.
- Results of repeated load triaxial (RLT) tests showed that the resilient moduli (M_R) of treated/stabilized soil specimens ranged from about 25 ksi for treated/stabilized soil #3 (after 7 days curing time) to about 36 ksi for treated/stabilized soil #1 (after 28 days curing time).
- Results of single-stage PD tests showed that the treated/stabilized hauled soil specimens with cement and/or lime experienced significantly lower permanent deformations compared to raw soils for the three soil types. The values of permanent deformations of treated/stabilized soils are negligible (permanent strain < 0.07%) that might be ignored in pavement design.
- Results of multi-stage PD tests demonstrated significant improvement on the behavior of treated/stabilized soils, compared to untreated soils, especially at high stress ratios. Higher permanent strains were observed for untreated soils at higher stress ratios, which is not the case for the treated/stabilized soils. However, under identical stress ratio, the specimens tested at higher confining stress showed higher measured permanent strains.

Phase II-a - Laboratory Tests

- The proper selection of additive type and content for very weak and wet subgrade soils can substantially improve their performance in terms of increasing resilient modulus and decreasing the permeant deformation for use as working platform table and subbase applications.

- The lime/class C fly ash combinations were found to achieve the strength target value of 50 psi but much less than the 100 psi target value based on the upper practical additive's percentage limit "per volume" of 30%. The cement was then used to replace the class C fly ash to achieve the 100 psi UCS target value for subbase application.
- Results of tube suction tests showed that the final dielectric values (DVs) of the treated soil specimens at the target UCS of 50 psi were all above the value of 16. This means that all treated soil samples at the target UCS of 50 psi were water susceptible. However, for the treated/stabilized soil at the target UCS of 100 psi, the maximum DV were mostly between 10 and 16, which means marginal material based on TxDOT's criterial.
- The resilient modulus of treated/stabilized soils shows a mixed behavior with respect to the increase of deviatoric/cyclic stress, such that it remains constant for UCS of 50 psi or increases for UCS of 100 psi with increasing the deviatoric/cyclic stress. Furthermore, as expected, the confining pressure has positive effects on the resilient modulus, such that M_R increases with increasing the confining stress.
- Results of repeated load triaxial (RLT) tests showed that the resilient moduli (M_R) of treated/stabilized very weak soil specimens at 50 psi UCS (working platform) ranged from about 5.5 ksi for treated/stabilized soil I at MC2 to about 11 ksi for treated/stabilized soil III at MC 3. However, for the 100 psi UCS (subbase application), the M_R of treated/stabilized very weak soil specimens ranged from about 9.5 ksi for treated/stabilized soil IV at MC 1 to about 14.5 ksi for treated/stabilized soils I and III at MC3.
- The resilient modulus and permanent deformation of the cement-treated/stabilized soils were found to be a function of the water/additive ratio, such that the resilient modulus increases and the permanent deformation decreases with the decrease in water/additive ratio.
- For treated/stabilized subgrade soils, the resulting permanent deformations are somehow negligible (permanent strain < 0.09%) that can be ignored in the pavement design.
- The resilient modulus (M_R) increase with increasing the curing time (up to 40%).
- It is also noted that there is no definite trend between the resilient modulus (M_R)/permanent deformation and the plastic index (PI).
- Results of multi-stage PD tests on treated/stabilized weak soil specimens at moisture contents MC2 showed that the permanent deformation increases with

increasing stress ratio ($\sigma_1/\ \sigma_3$), which is associated with increasing the deviatoric stress, and increases with increasing the number of load cycles. However, much lower permanent deformations were measured for specimens with 100 psi UCS as compared to specimens with 50 psi UCS.

Phase II-b - Field Tests

- The cyclic plate load test (CPLT) was used to measure the in-situ composite resilient modulus ($M_{R\text{-}comp}$) of ten treated/stabilized test sections constructed at ALF site, and the measured values were in good agreement with the calculated $M_{R\text{-}comp}$ values from results of dynamic cone penetration (DCP) test data (i.e., DCP index) using equations (29) and (30).
- The measured resilient moduli (M_R) for the treated/stabilized test sections at ALF were back-calculated from $M_{R\text{-}comp}$ values using both equations (29) and (30), and assuming that, for the two-layers system, the ratio of $(E_1/E_2)_{DCP}$ obtained from DCP test data is the same as the ratio of $(E_1/E_2)_{CPLT}$ obtained from the CPLT test. The results M_R values were in good agreement with the laboratory measured M_R for phase II.

Recommendations

Based on the results of this research study, the following recommendations are offered to DOTD engineers:

It is recommended that DOTD pavement design engineers start giving credit to the treated/stabilized hauled subgrade soil (stabilized according to DOTD selection guidelines in Table 8) through evaluating the composite resilient modulus ($M_{R\text{-comp}}$) of the treated/stabilized soil layer and the underneath untreated subgrade soil based on thickness of treated/stabilized layer, influence depth and using equations (29) or (30). The M_R for the treated/stabilized soil layer can be evaluated using the MEPDG M_R model in equation (23) and the corresponding model constants (k_1, k_2, and k_3) presented in Table 20, or directly select M_R directly from Table 22. The M_R for the untreated subgrade soil can be evaluated using the dynamic cone penetration test data (i.e., DCP index).

It is recommended that DOTD pavement design engineers to use the final selected additive combinations (lime, fly ash, and cement) and the corresponding contents by volume as presented in Table 24 for treating/stabilizing the in-situ weak and wet subgrades soils based on PI values and in-situ moisture contents.

It is recommended that DOTD pavement design engineers to start giving credit to the treated/stabilized in-situ weak and wet subgrade soil through evaluating the composite resilient modulus ($M_{R\text{-comp}}$) of the treated/stabilized soil layer and the underneath untreated subgrade soil based on thickness of treated/stabilized soil layer, influence depth and using equations (29) or (30). The M^R for the treated/stabilized soil layer can be evaluated using Table 27 for working platform (50 psi UCS) and Table 28 for subbase application (100 psi UCS). Again, M_R for the untreated weak subgrade soil can be evaluated using the dynamic cone penetration test data (i.e., DCP index).

It is recommended to evaluate the composite resilient modulus ($M_{R\text{-comp}}$) and M_R of treated/stabilized subgrade sections (hauled and weak in-situ soils) for real pavement field projects constructed at different additive combinations to verify the findings of this study.

Acronyms, Abbreviations, and Symbols

Term	Description
AASHTO	American Association of State Highway and Transportation Officials
ALF	Accelerated Load Facility
ASTM	American Society for Testing and Materials
C	Cement
CBR	California bearing ratio
CFA	class C fly ash
CH	High plasticity clay
CKD	Cement kiln dust
CL	Low plasticity clay
Cm	centimeter(s)
CPLT	Cyclic plate load tests
DCP	Dynamic cone penetrometer
DCPI	Dynamic cone penetration index
DOT	Department of transportation
DOTD	Louisiana Department of Transportation and Development
DV	Dielectric values
E	Elastic modulus
F	Fly ash
FHWA	Federal Highway Administration
ft.	foot (feet)
FWD	Falling weight deflectometer
HVS	Heavy vehicle simulator
INDOT	Indiana Department of Transportation
in.	inch(es)
L	Lime
LFWD	Light falling weight deflectometer
LL	Liquid Limit

Term	Description
LTRC	Louisiana Transportation Research Center
LVDT	Linearly variable differential transducer
lb.	Pound(s)
m	Meter(s)
MC	Moisture content
MD	Moist density
MDD	Maximum dry density
MET	Method of equivalent thickness
MH	High plasticity clay
ML	Low plasticity silt
M_R	Resilient modulus
MTS	Material Testing System
OMC	Optimum moisture content
PD	Permanent deformation
pH	Measure of the acidity or basicity (alkalinity)
PI	Plasticity index
PLT	Plate load test
RH	Relative humidity
RLT	Repeated load triaxial
SAS	Statistical analysis software
SM	Silty sand
TxDOT	Texas Department of Transportation
UCS	Unconfined compressive strength
UFC	Unified facilities criteria
UU	Unconsolidated-undrained
VA	Volcanic ash
ε	Strain
ε_p	Plastic strain
ε_r	Resilient strain
σ	Stress
σ_d	Deviatoric stress
σ_{oct}	Octahedral normal stress
τ_{oct}	Octahedral shear stress

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Appendix A
Results of Resilient Modulus

The figures below present the results of resilient modulus tests for the untreated and treated/stabilized soils prepared at the optimum moisture content ± 2% for Phase I. Each figure represents the average resilient modulus for three repeated specimens.

Figure 87. Soil #1: Resilient modulus at optimum moisture content (untreated)

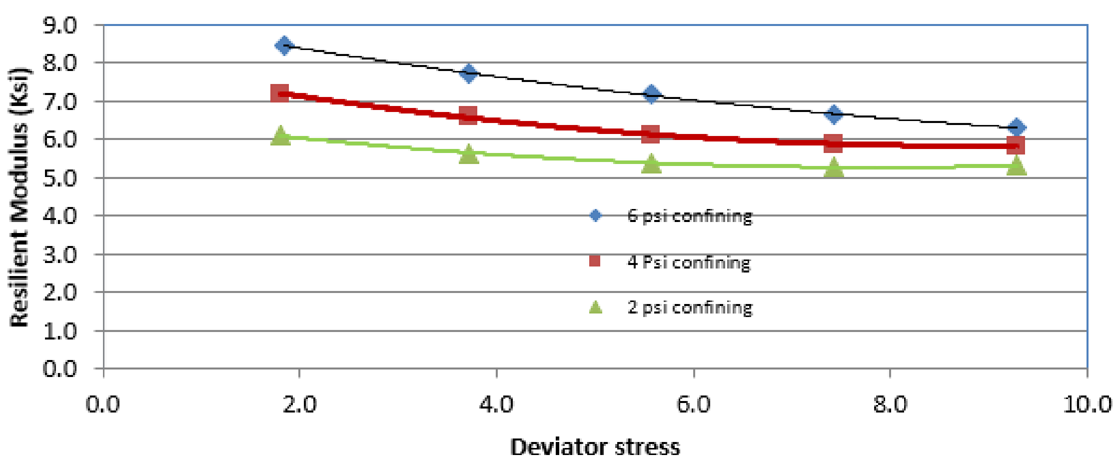


Figure 88. Soil #1: Resilient modulus at optimum moisture content + 2% (untreated)

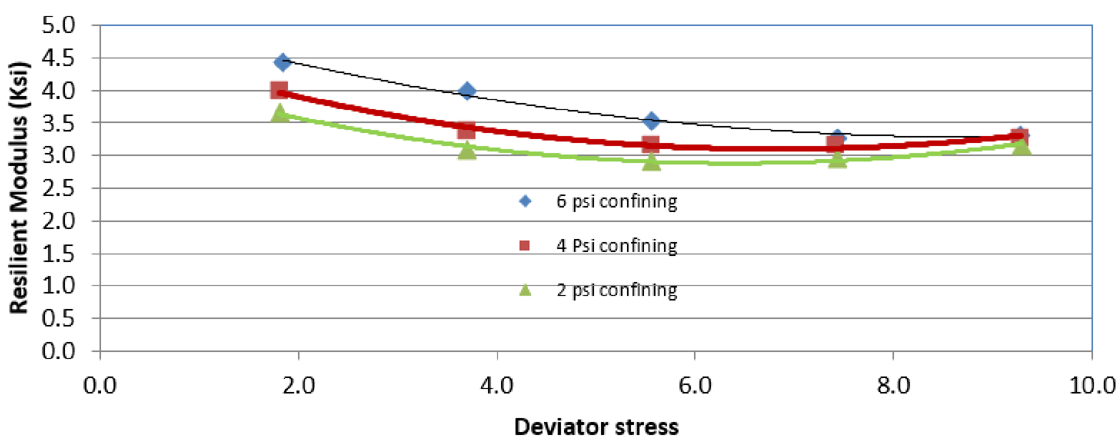


Figure 89. Soil #1: Resilient modulus at optimum moisture content - 2% (untreated)

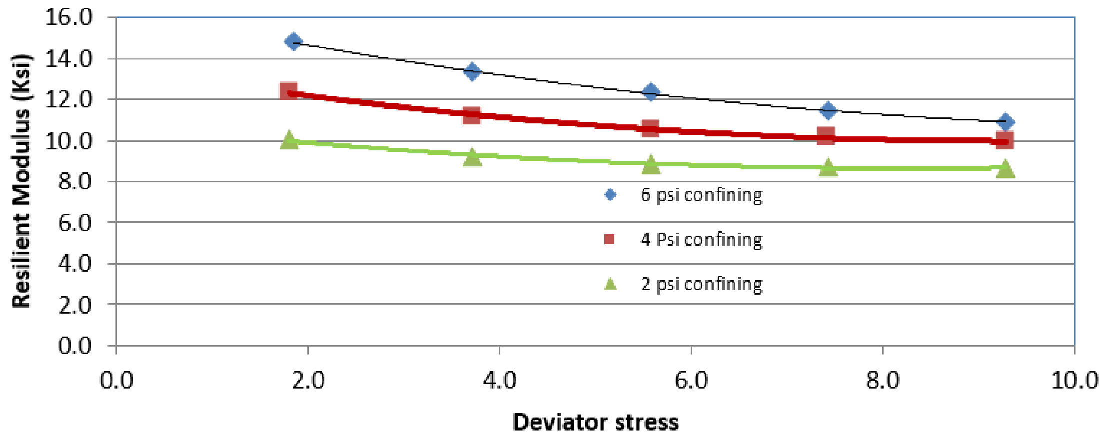


Figure 90. Soil #1: Resilient Modulus at optimum moisture content (treated 6% cement, 7 days)

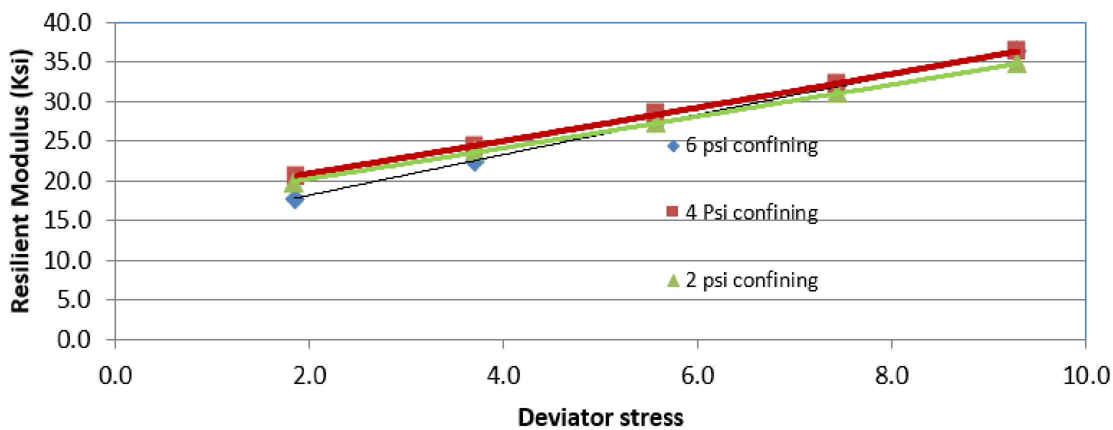


Figure 91. Soil #1: Resilient modulus at optimum moisture content (treated 6% cement, 28 days)

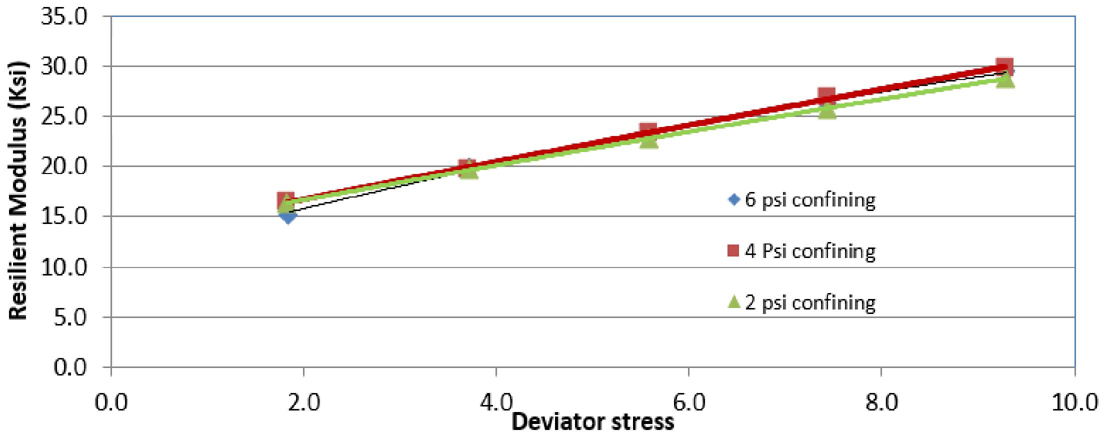


Figure 92. Soil #2: Resilient modulus at optimum moisture content (untreated)

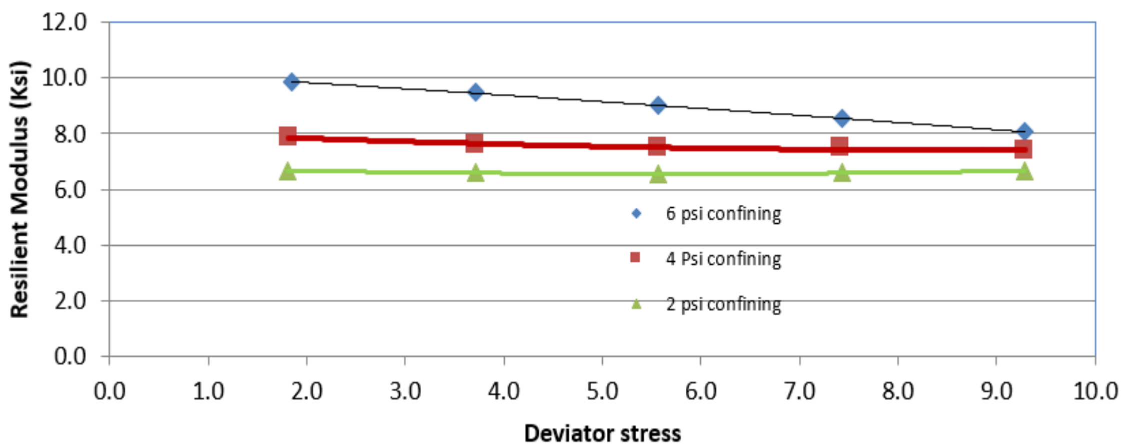


Figure 93. Soil #2: Resilient modulus at optimum moisture content + 2% (untreated)

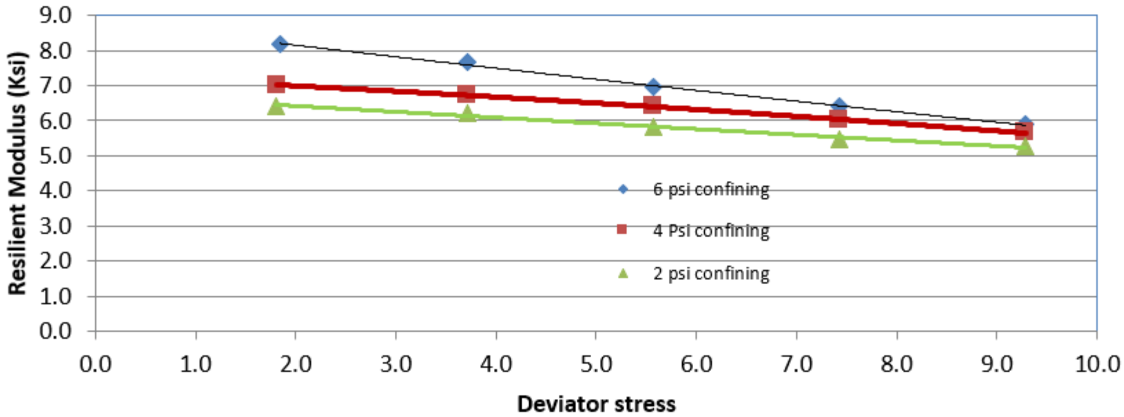


Figure 94. Soil #2: Resilient modulus at optimum moisture content - 2% (untreated)

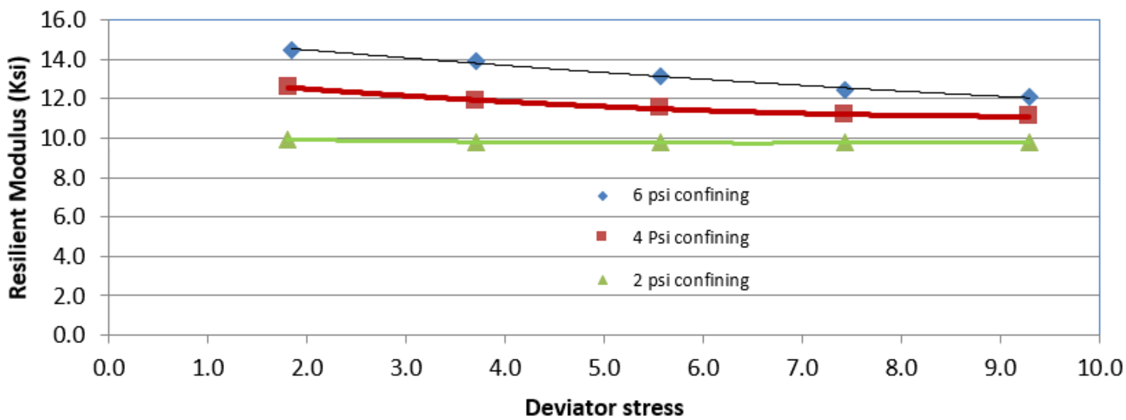


Figure 95. Soil #2: Resilient modulus at optimum moisture content (treated 6% cement, 6% lime, 7 days)

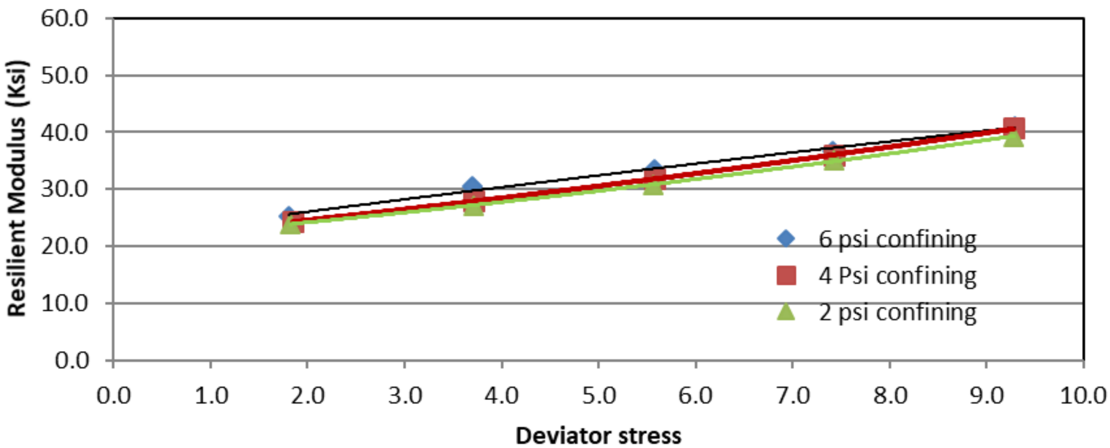


Figure 96. Soil #2: Resilient modulus at optimum moisture content + 2% (treated 6% cement, 6% lime, 7 days)

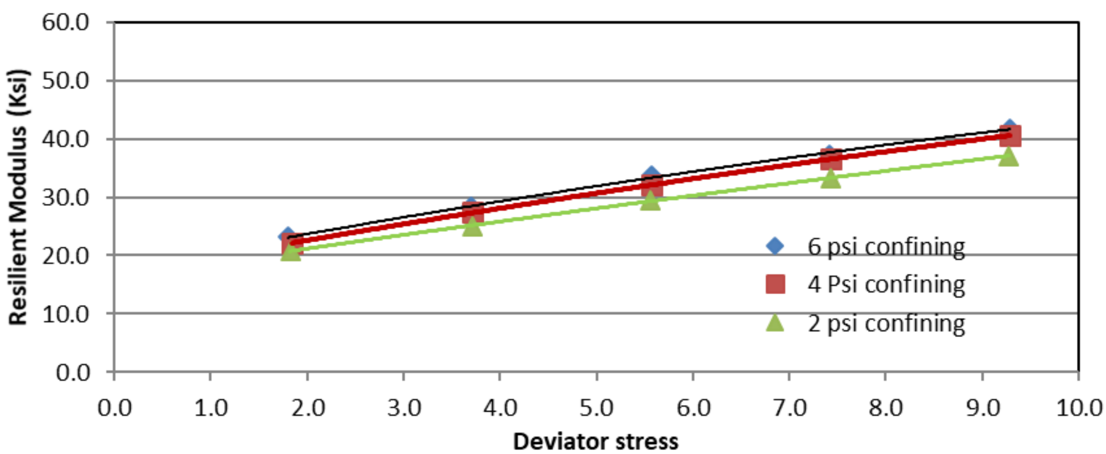


Figure 97. Soil #2: Resilient modulus at optimum moisture content - 2% (treated 6% cement, 6% lime, 7 days)

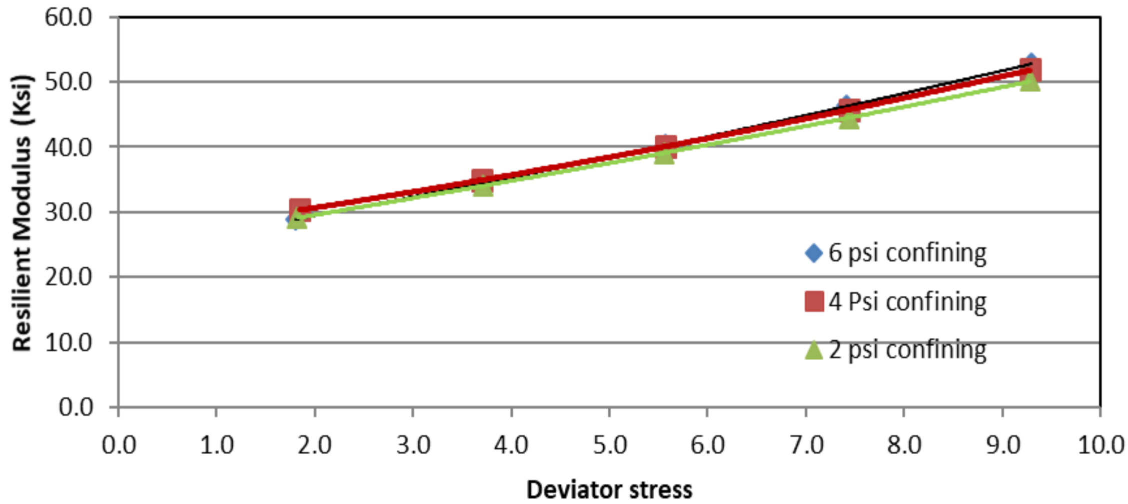


Figure 98. Soil #2: Resilient modulus at optimum moisture content (treated 6% cement, 6% lime 28)

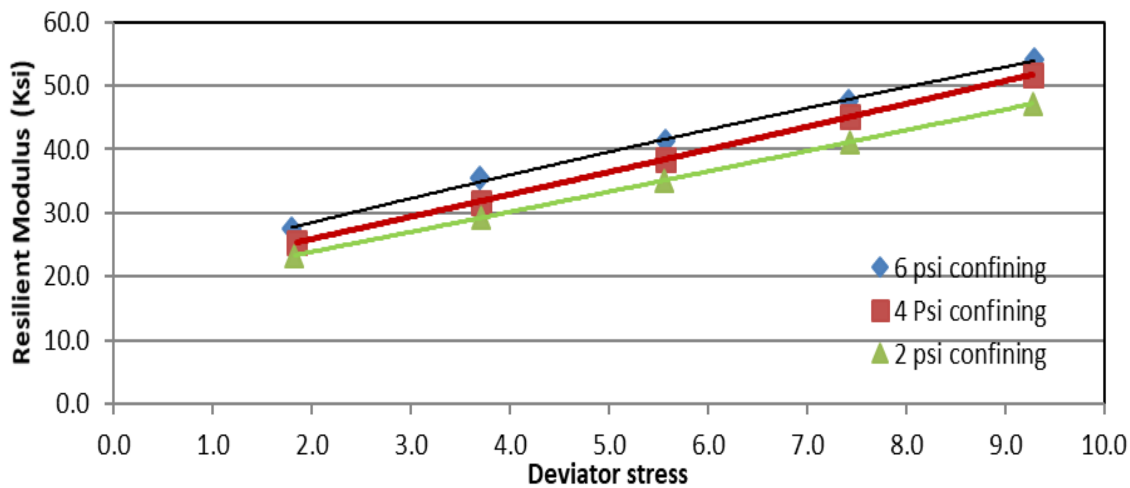


Figure 99. Soil #2: Resilient modulus at optimum moisture content + 2% (treated 6% cement, 6%lime, 28 days)

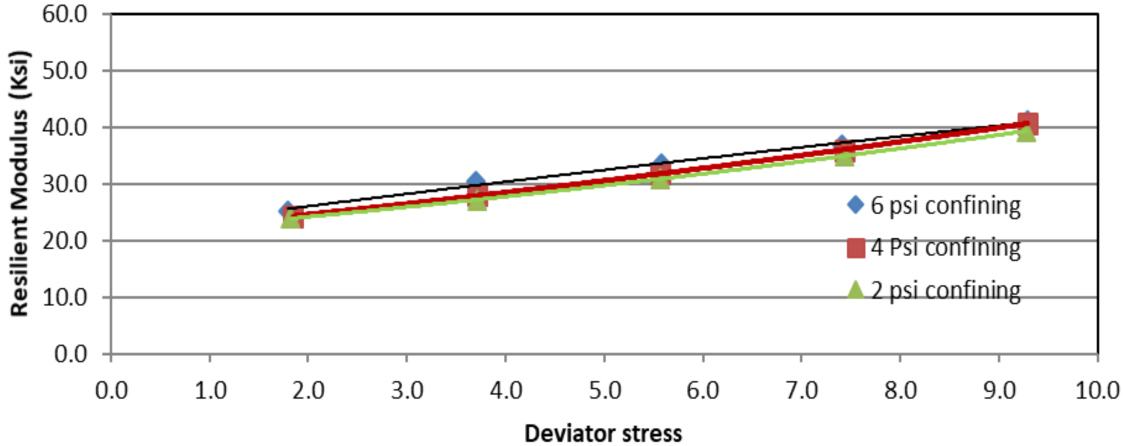


Figure 100. Soil #2: Resilient modulus at optimum moisture content - 2% (treated 6% cement, 6% lime, 28 days)

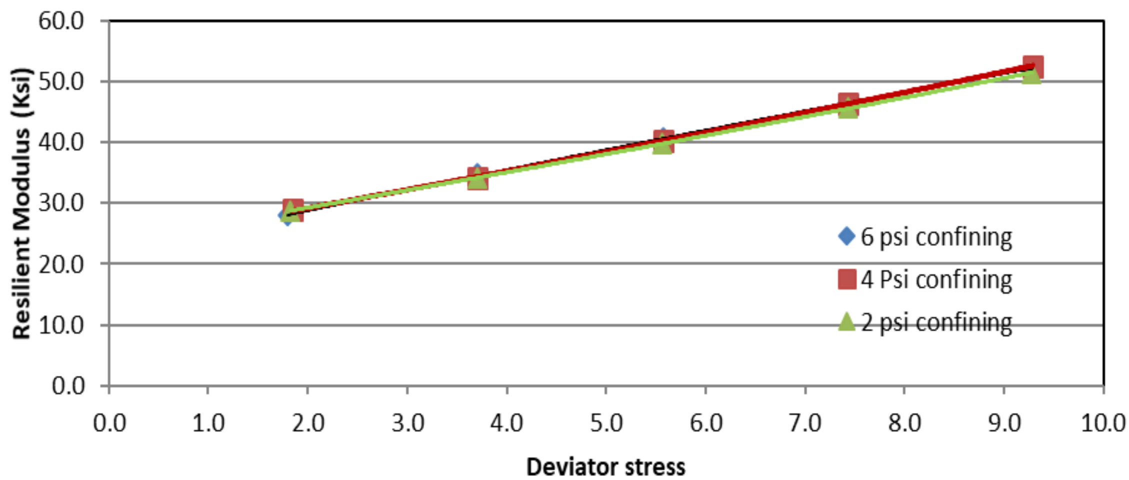


Figure 101. Soil #3: Resilient modulus at optimum moisture content (untreated)

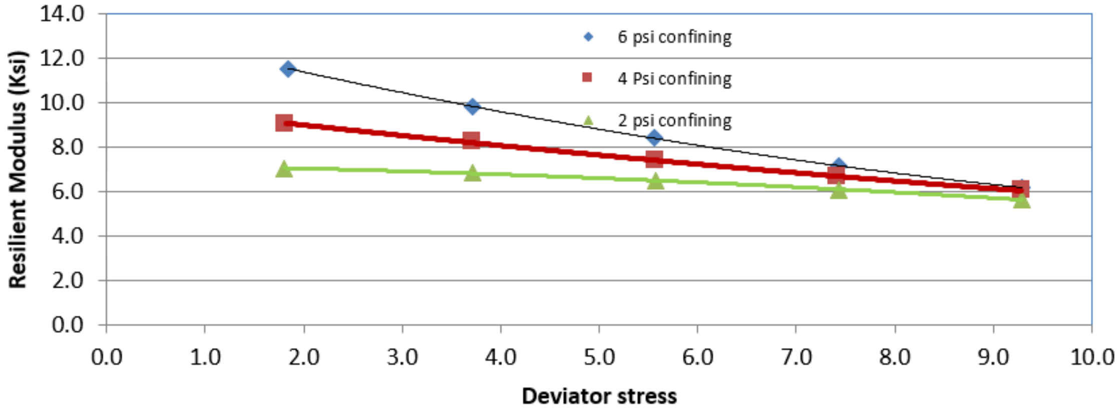


Figure 102. Soil #3: Resilient modulus at optimum moisture content + 2% (untreated)

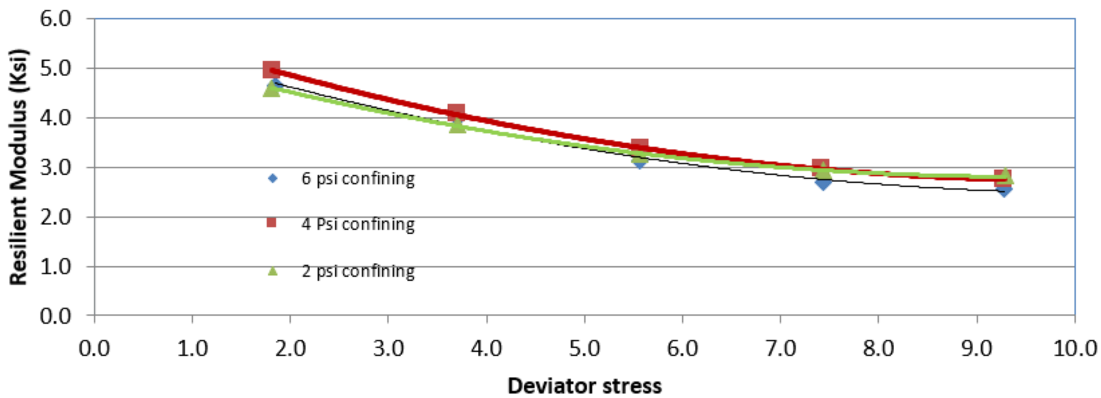


Figure 103. Soil #3: Resilient modulus at optimum moisture content - 2% (untreated)

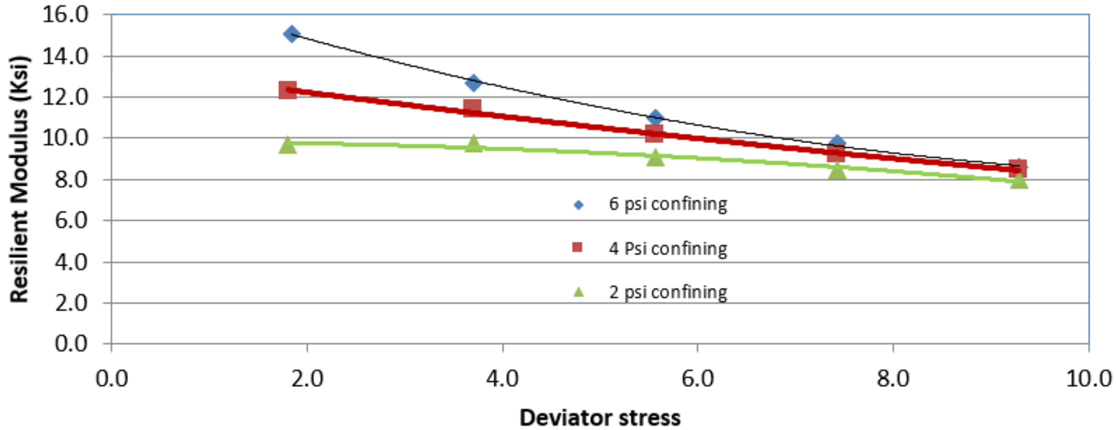


Figure 104. Soil #3: Resilient modulus at optimum moisture content (treated 6% cement, 9% lime, 7 days)

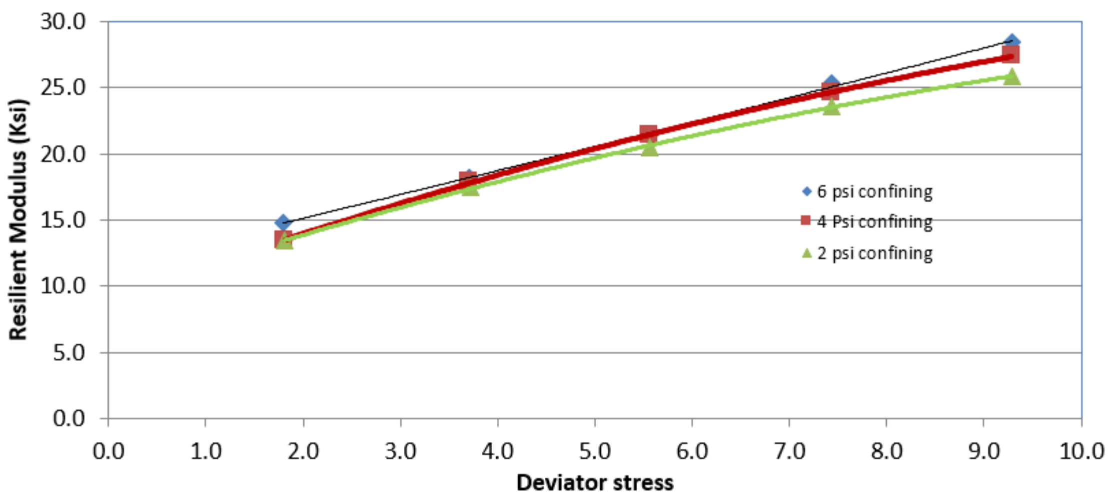


Figure 105. Soil #3: Resilient modulus at optimum moisture content + 2% (treated 6% cement, 9% lime, 7 days)

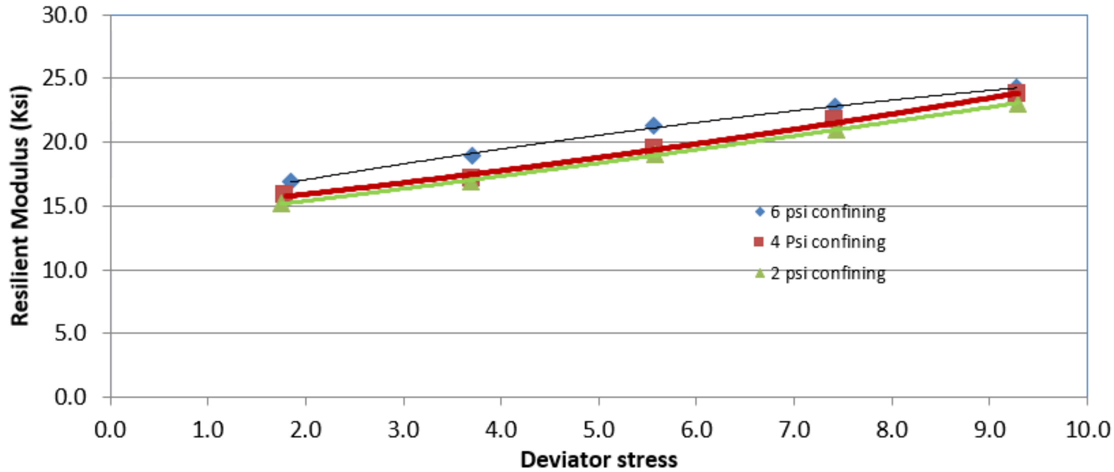


Figure 106. Soil #3: Resilient modulus at optimum moisture content - 2% (treated 6%, cement, 9% lime, 7 days)

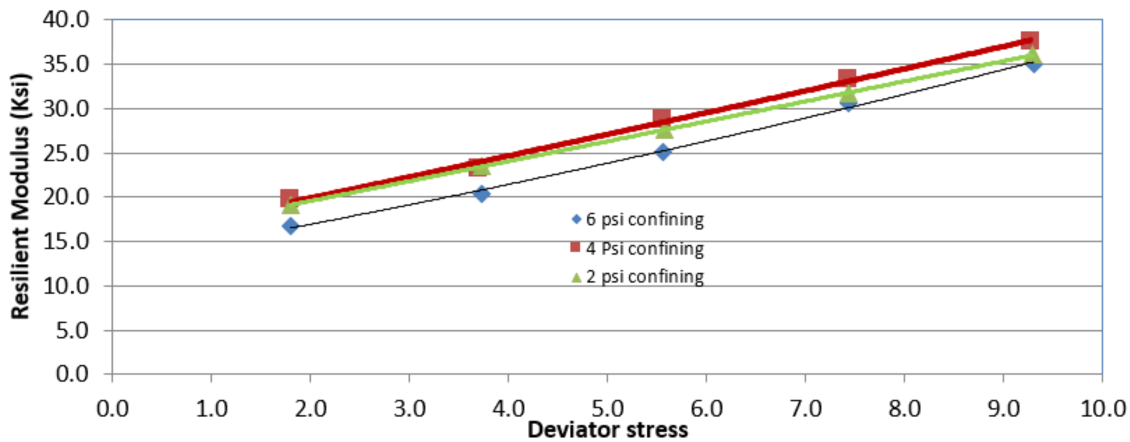
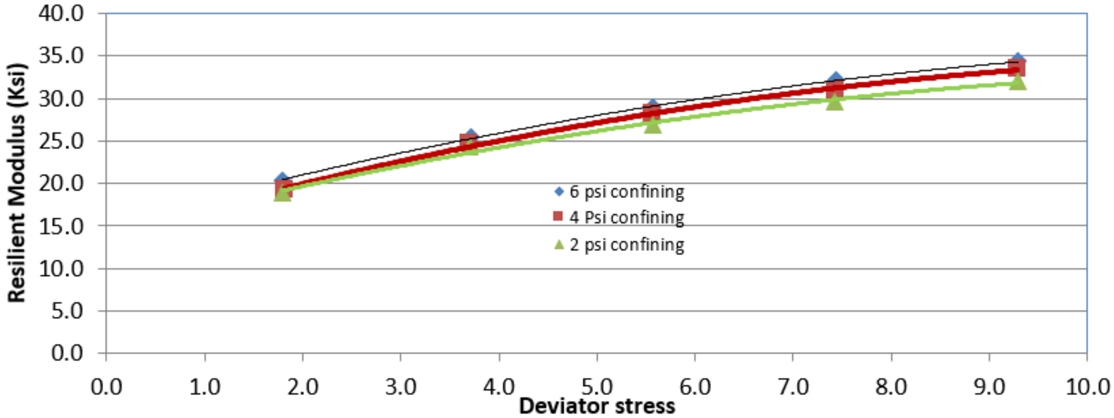


Figure 107. Soil #3: Resilient modulus at optimum moisture content (treated 6% cement, 9% lime, 28 days)



Appendix B
Results of Single-Stage Permanent Deformation Tests

The figures below show the results for single stage permanent deformation tests for the three untreated and treated/stabilized soil specimens prepared at optimum moisture content ± 2%.

Figure 108. Single stage permanent deformation results for soil #1 (treated and untreated specimens)

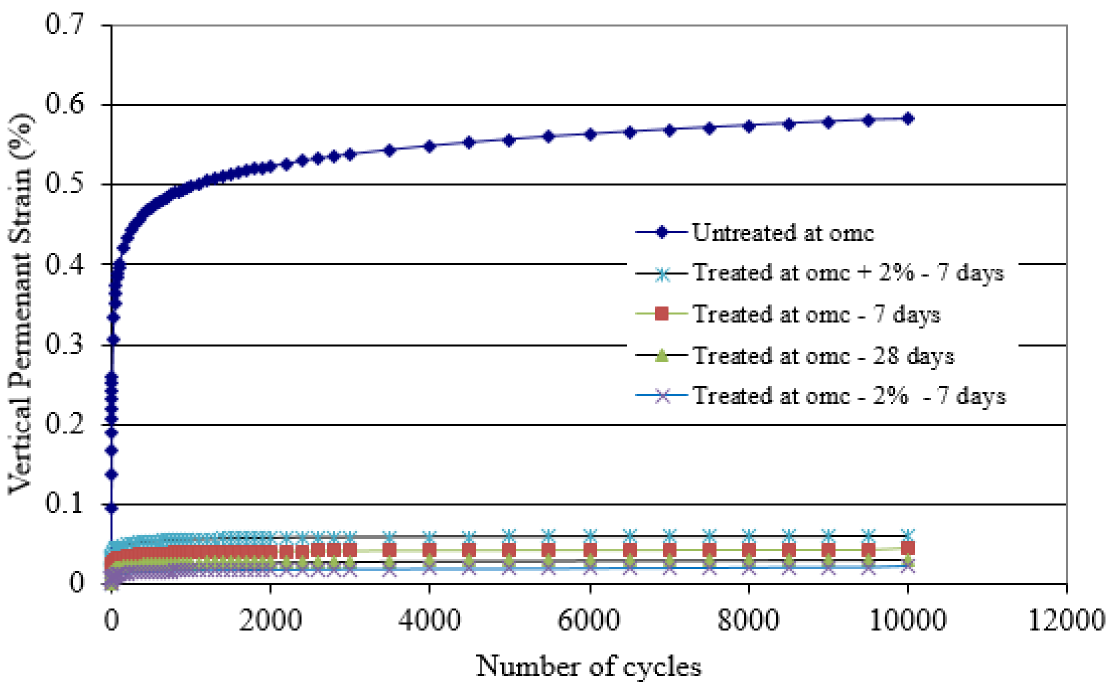


Figure 109. Single stage permanent deformation results for soil #1 (only treated specimens at different moisture content)

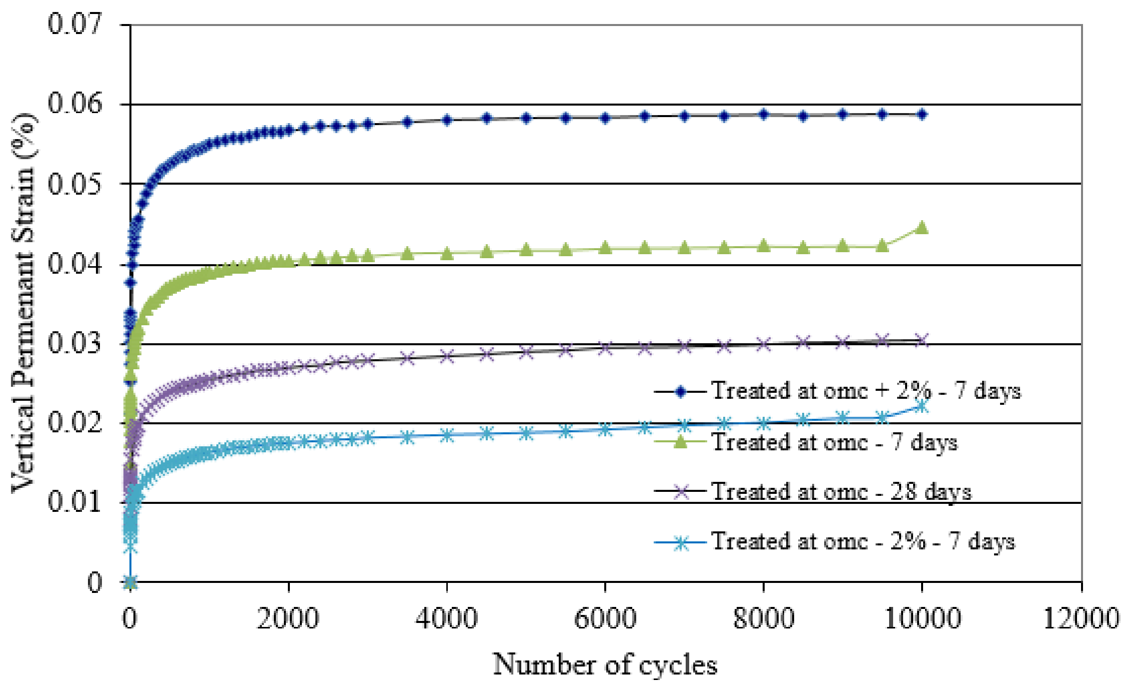


Figure 110. Single stage permanent deformation results for soil #2 (treated and untreated specimens)

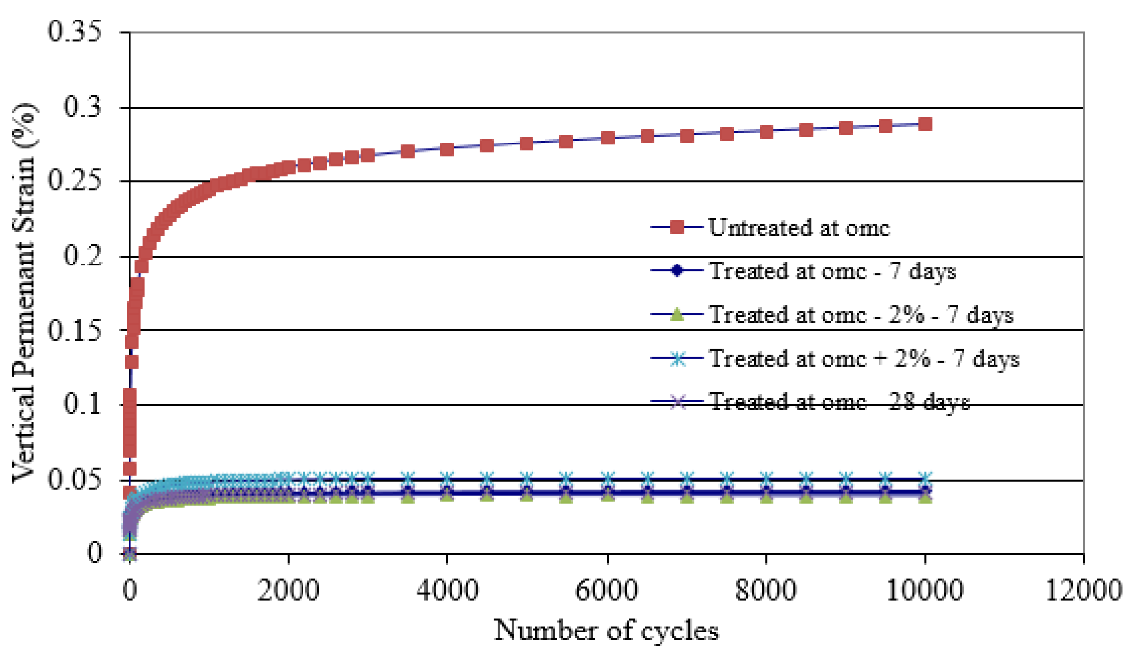


Figure 111. Single stage permanent deformation results for soil #2 (only treated specimens at different moisture content)

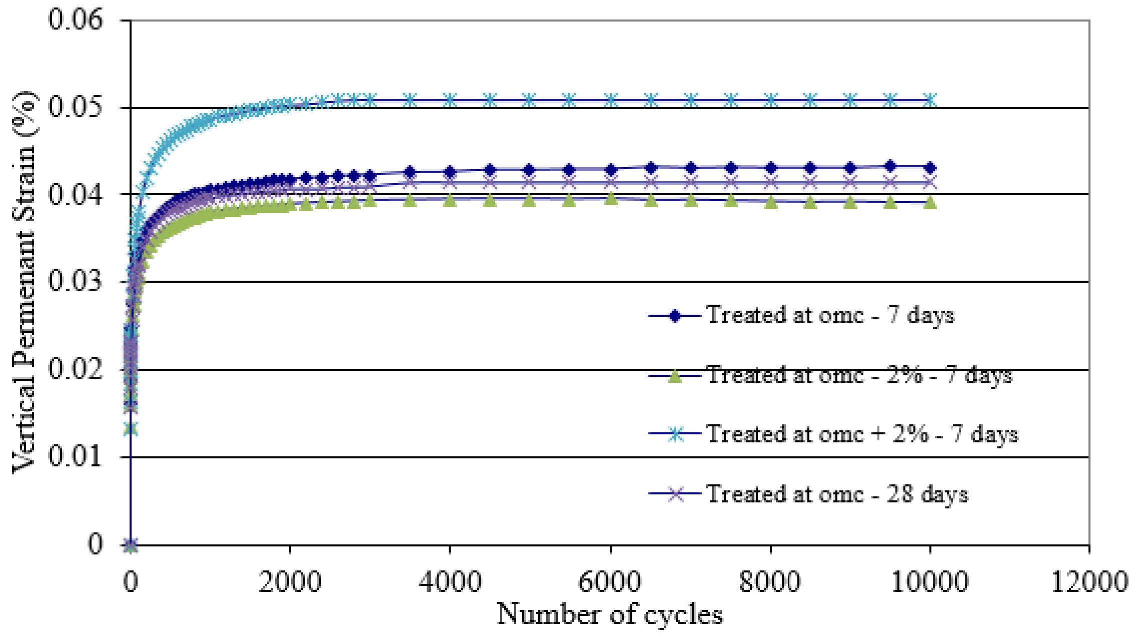


Figure 112. Single stage permanent deformation results for soil #3 (treated and untreated specimens)

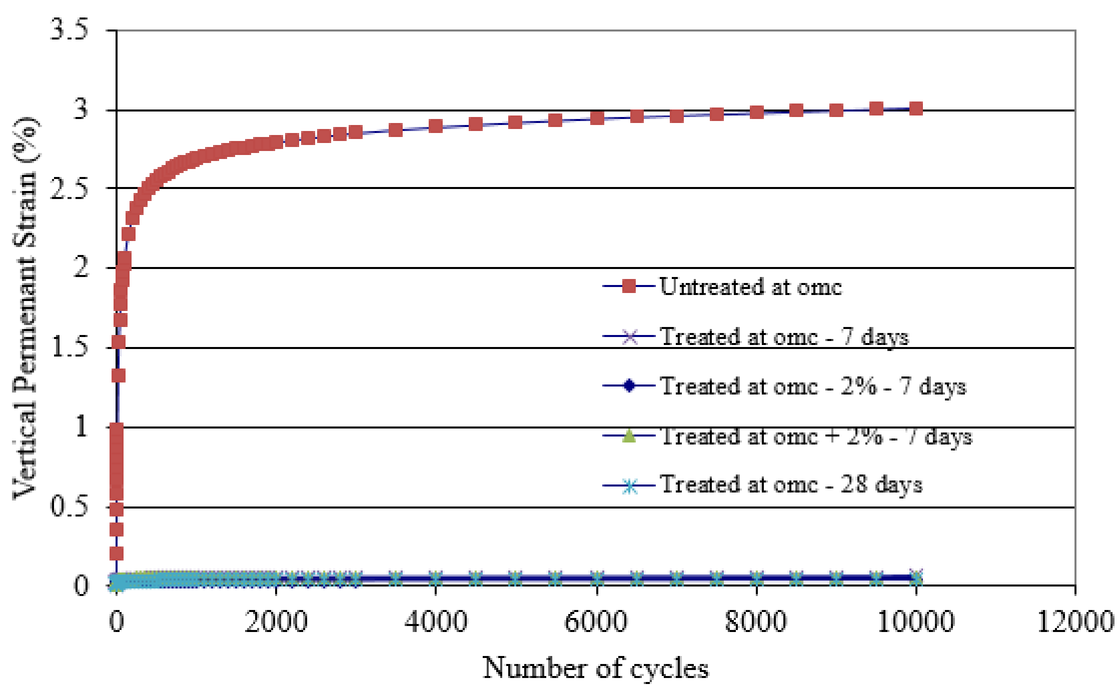
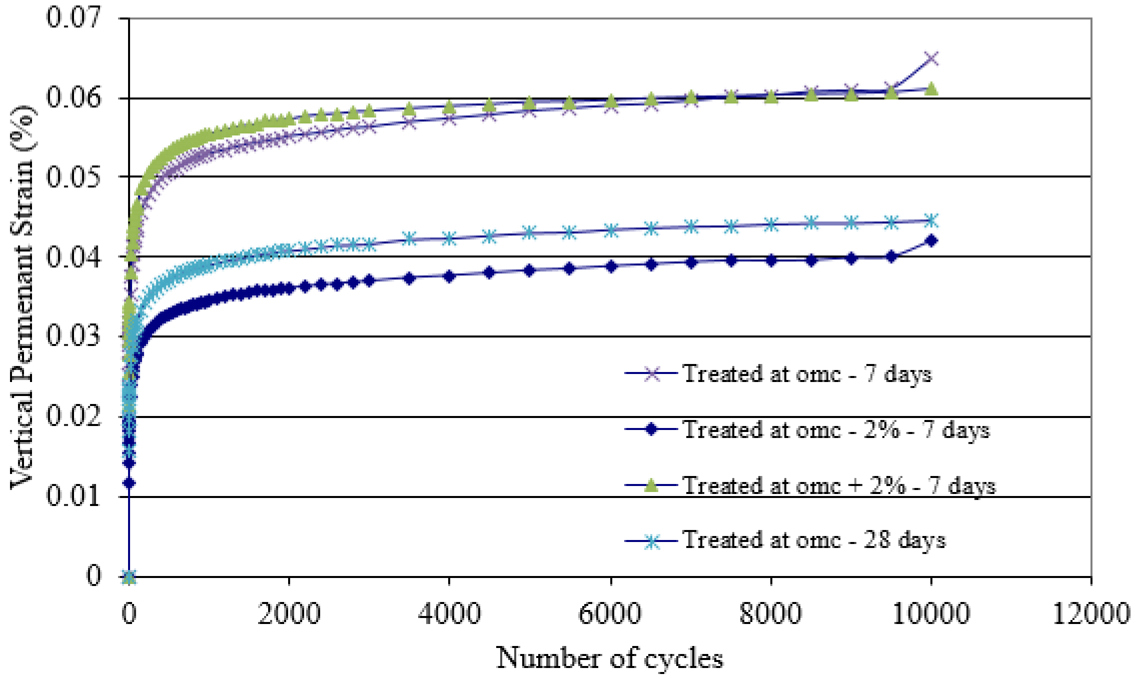


Figure 113. Single stage permanent deformation results for soil #3 (only treated specimens at different moisture content)



Appendix C
Results of Multi-Stage Permanent Deformation Tests

The figures below show the results for multi stage permanent deformation tests for the three untreated and treated/stabilized soil specimens prepared at optimum moisture content ± 2%.

Figure 114. Multi-stage permanent deformation soil #1 untreated at optimum moisture content

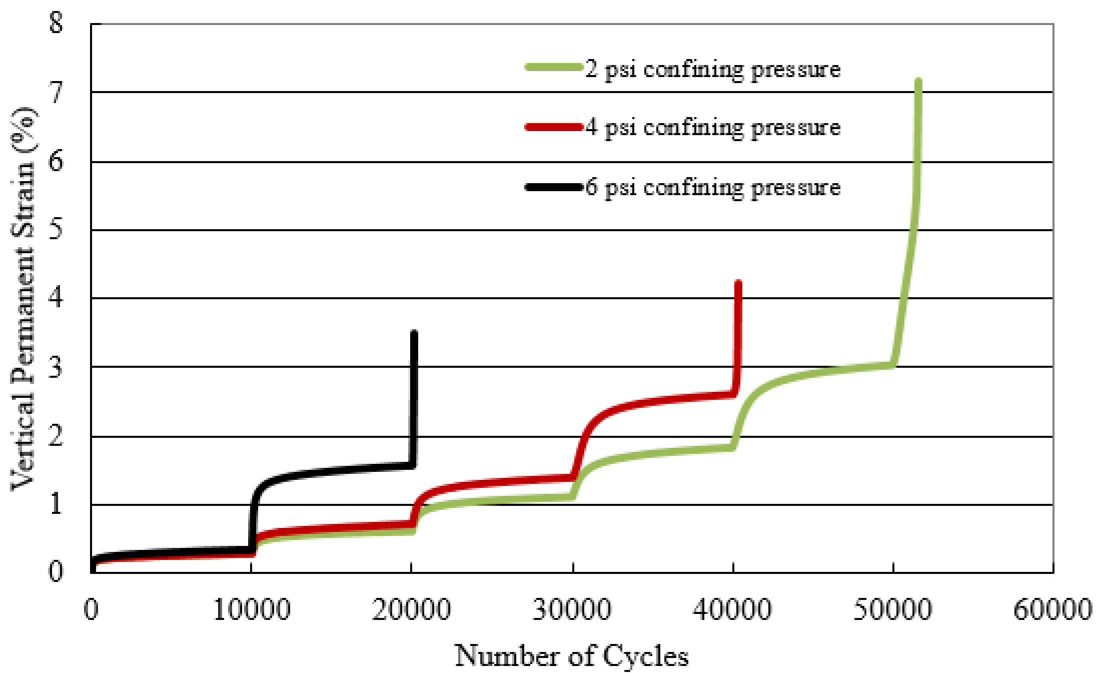


Figure 115. Multi-stage permanent deformation soil #1 treated with 6% cement at OMC (7 days)

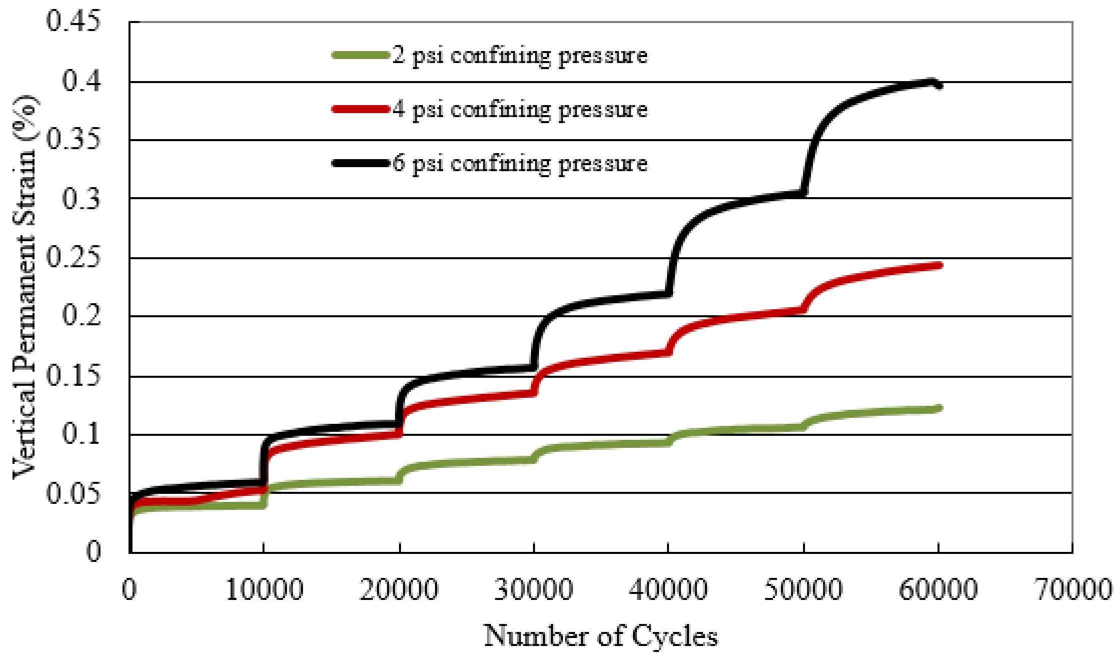


Figure 116. Multi-stage permanent deformation soil #1 treated with 6% cement at omc+2% (7 days).

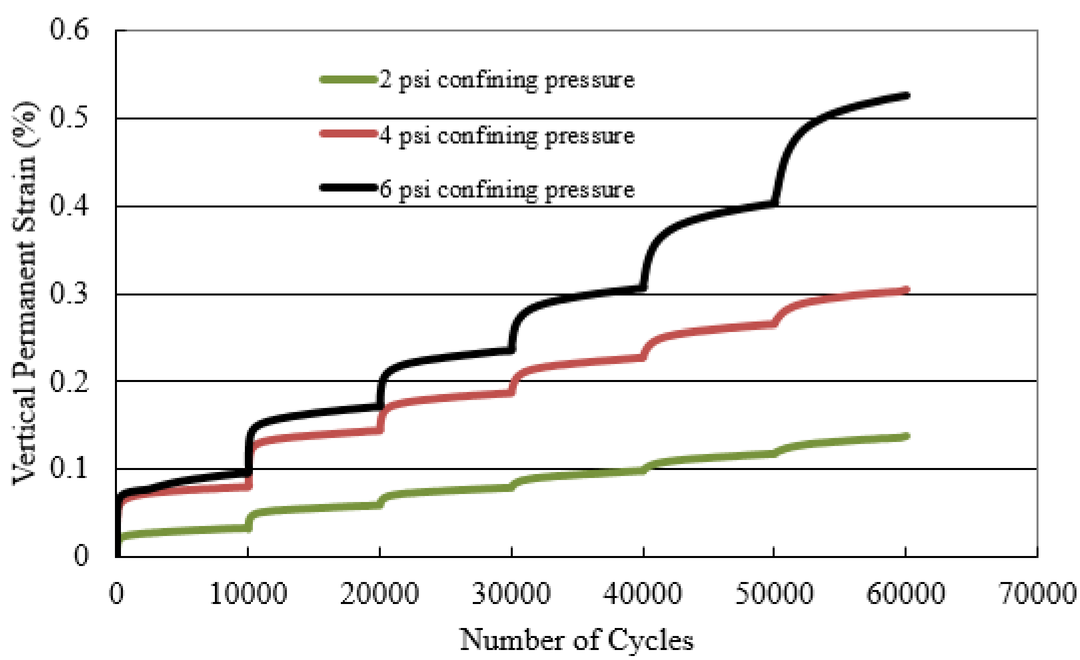


Figure 117. Multi-stage permanent deformation soil #1 treated with 6% cement at omc-2% (7 days).

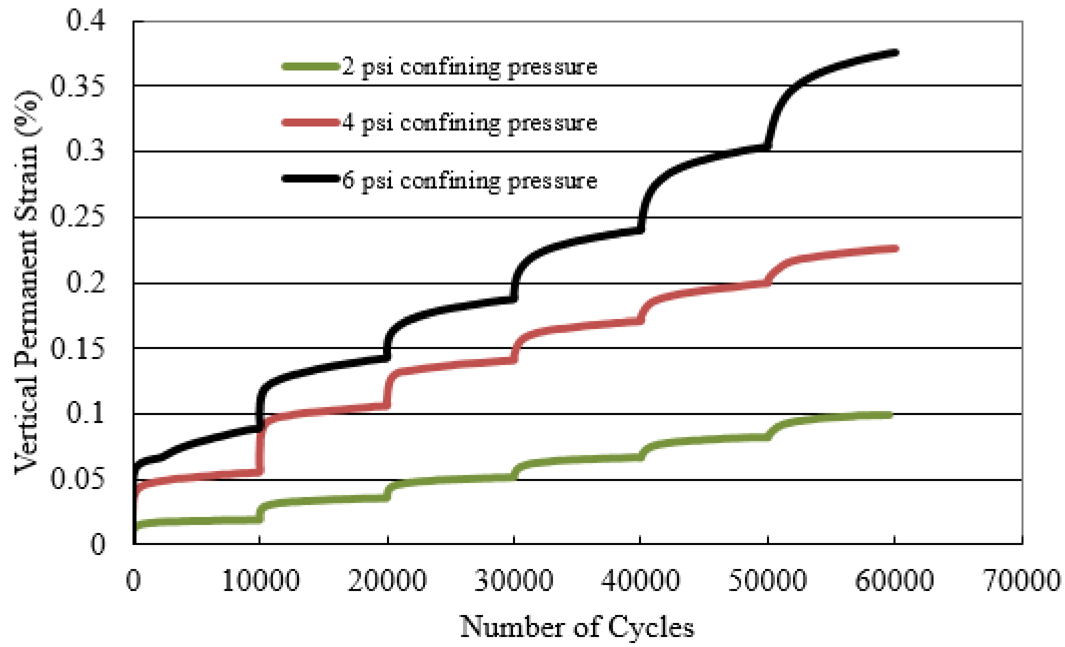


Figure 118. Multi-stage permanent deformation soil #1 treated with 6% cement at omc (28 days).

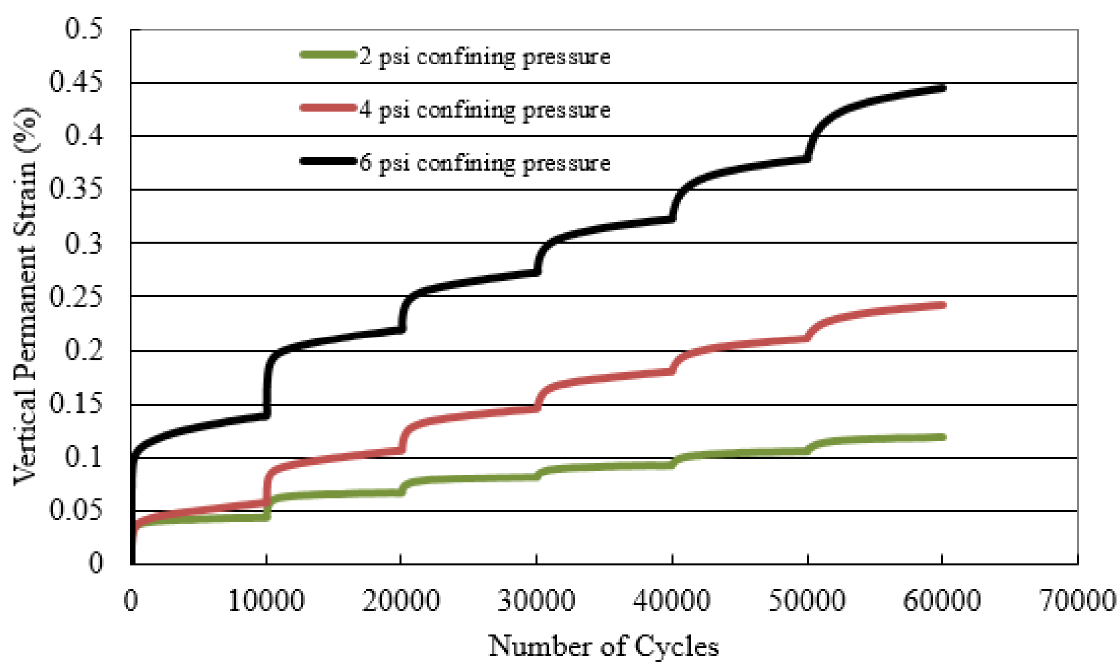


Figure 119. Multi-stage permanent deformation soil #2 untreated at optimum moisture content.

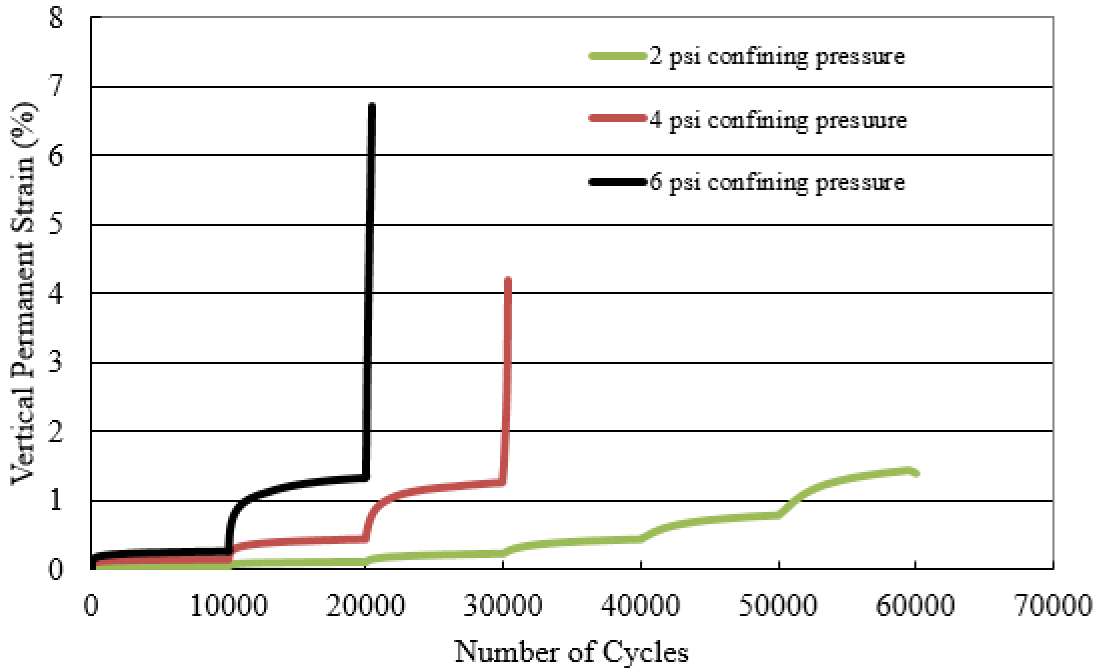


Figure 120. Multi-stage permanent deformation soil #2 treated with 6% cement, 6% lime at omc (7 days).

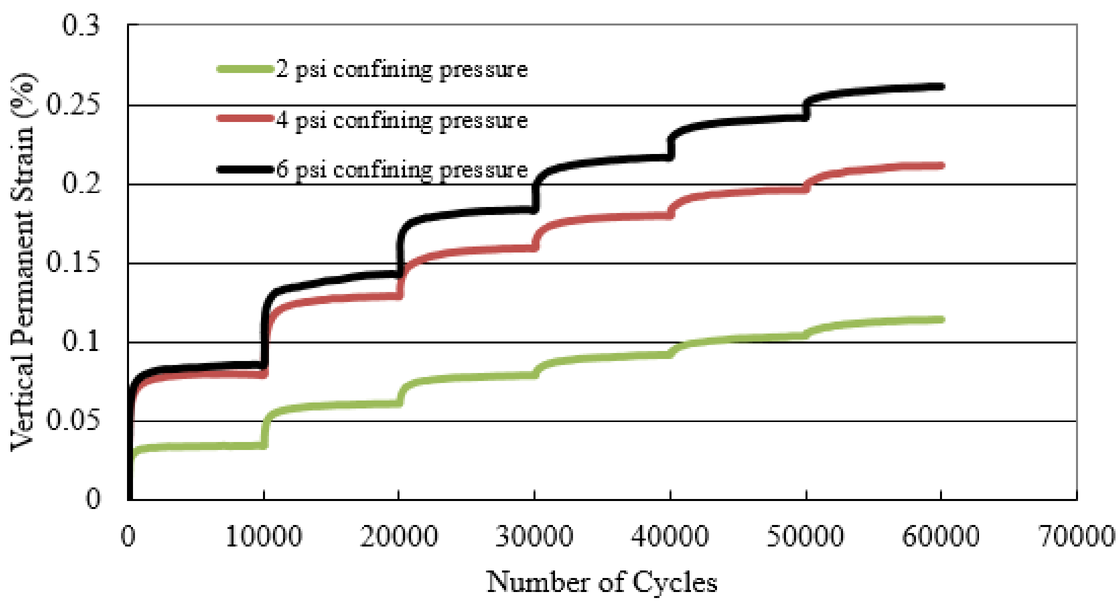


Figure 121. Multi-stage permanent deformation soil #2 treated with 6% cement, 6% lime at omc+2% (7 days).

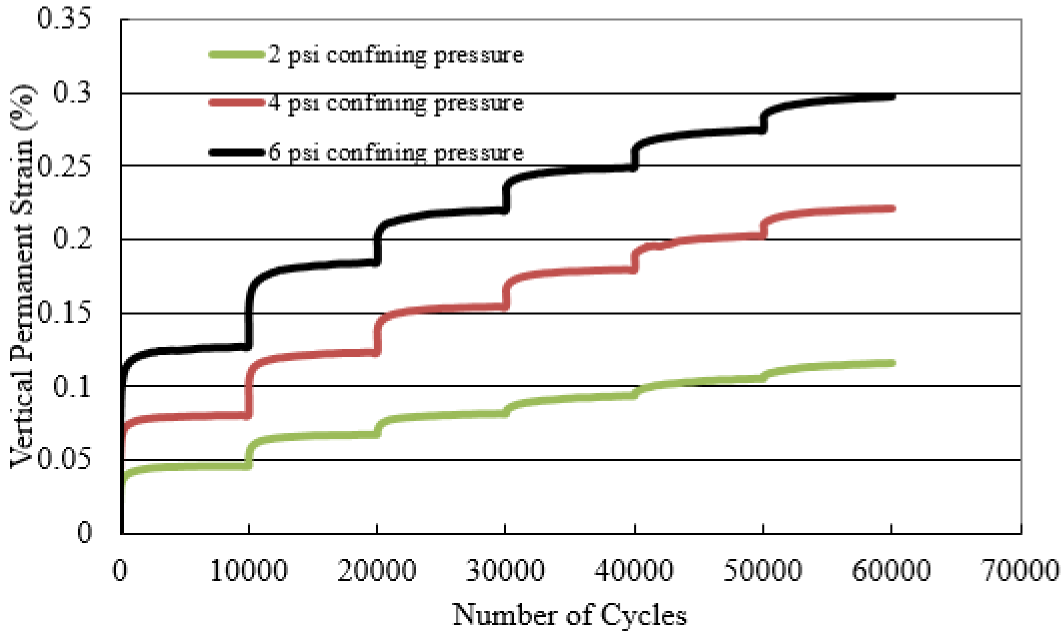


Figure 122. Multi-stage permanent deformation soil #2 treated with 6% cement, 6% lime at omc-2% (7 days).

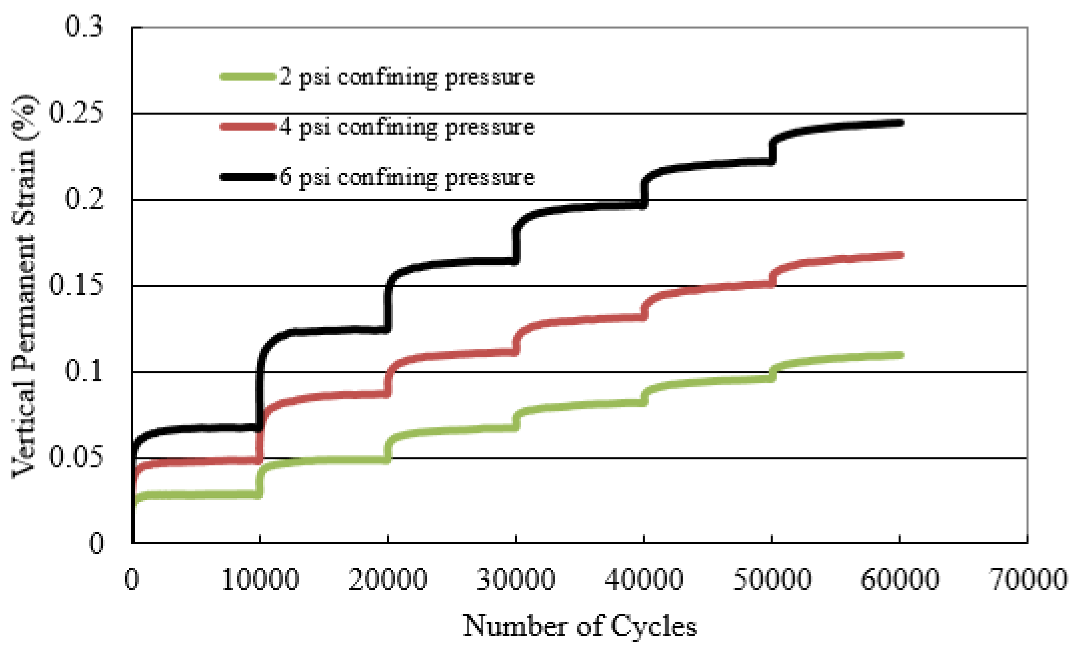


Figure 123. Multi-stage permanent deformation soil #2 treated with 6% cement, 6% lime at omc (28 days).

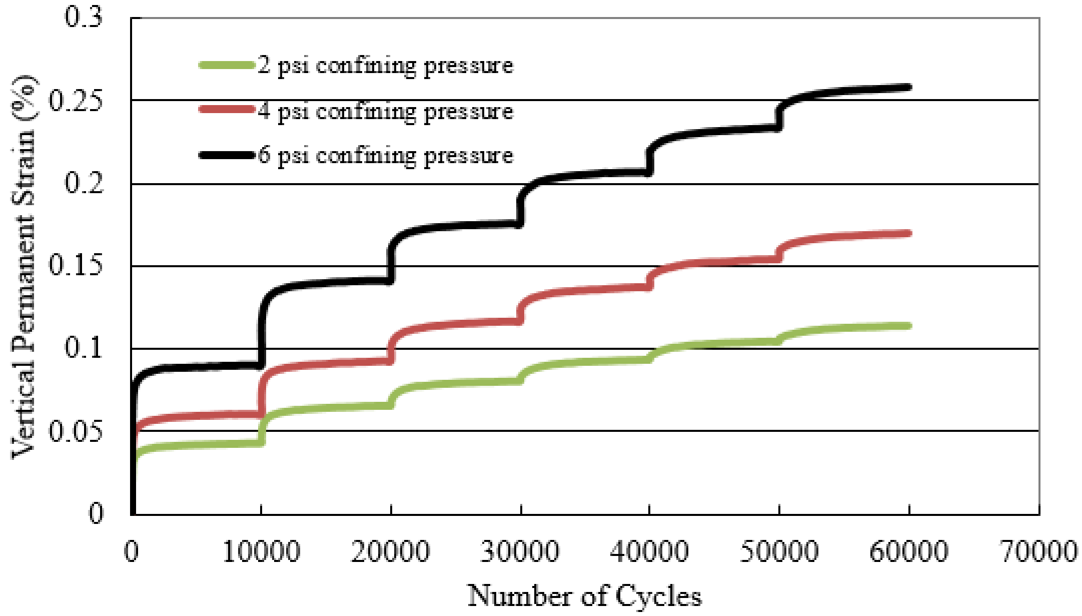


Figure 124. Multi-stage permanent deformation soil #3 untreated at optimum moisture content.

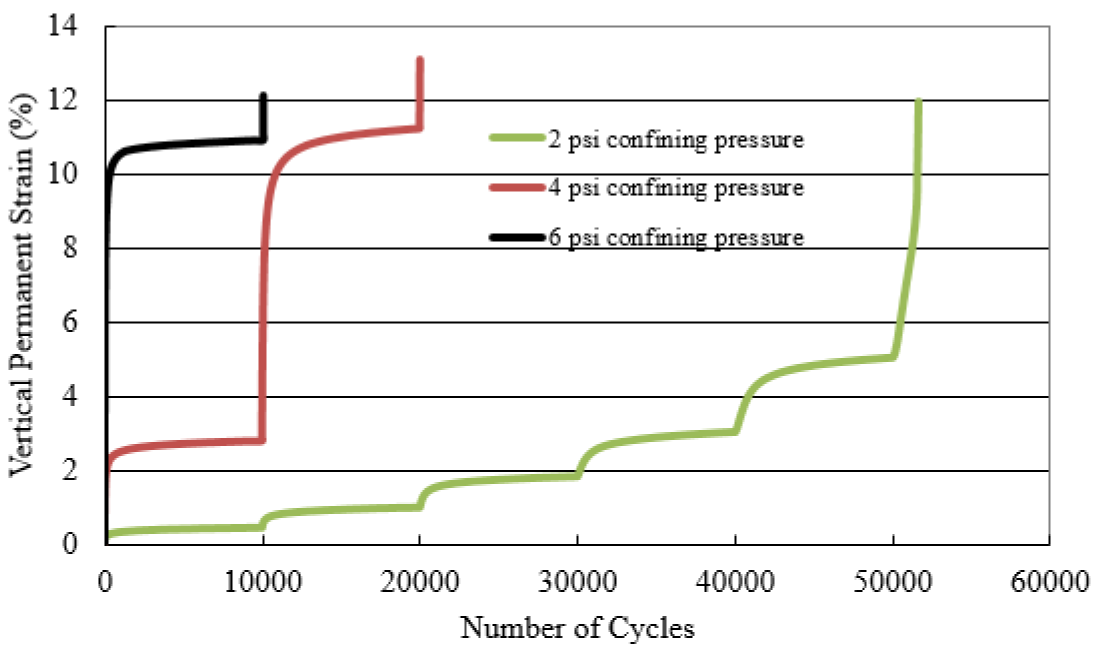


Figure 125. Multi-stage permanent deformation soil #3 treated with 6% cement, 9% lime at omc (7 days).

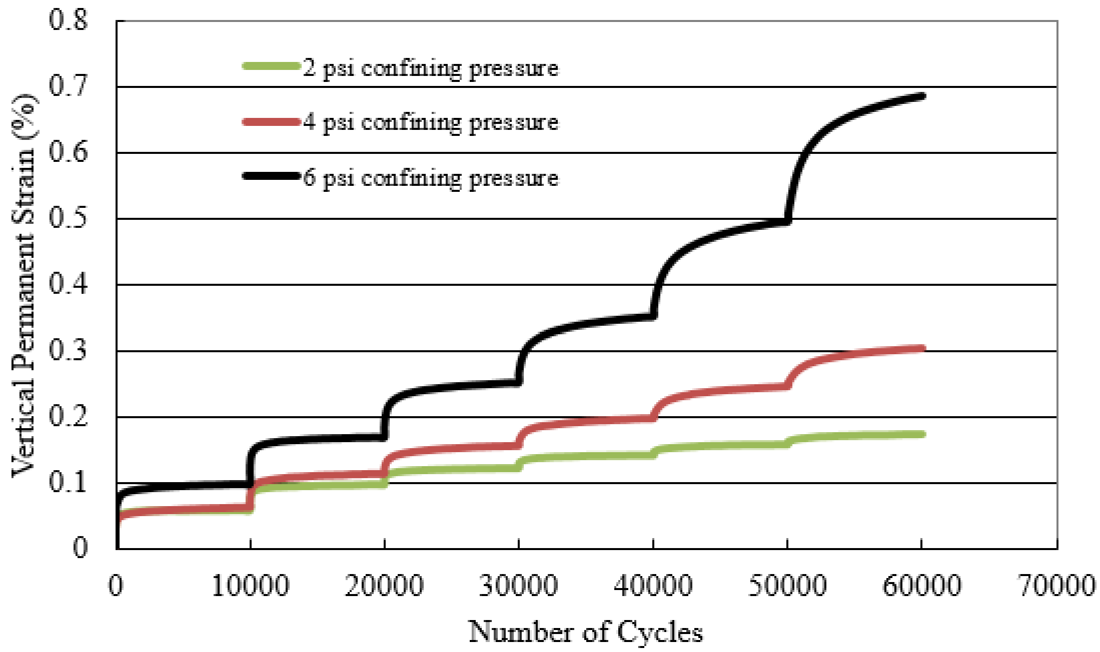


Figure 126. Multi-stage permanent deformation soil #3 treated with 6% cement, 9% lime at omc+2% (7 days).

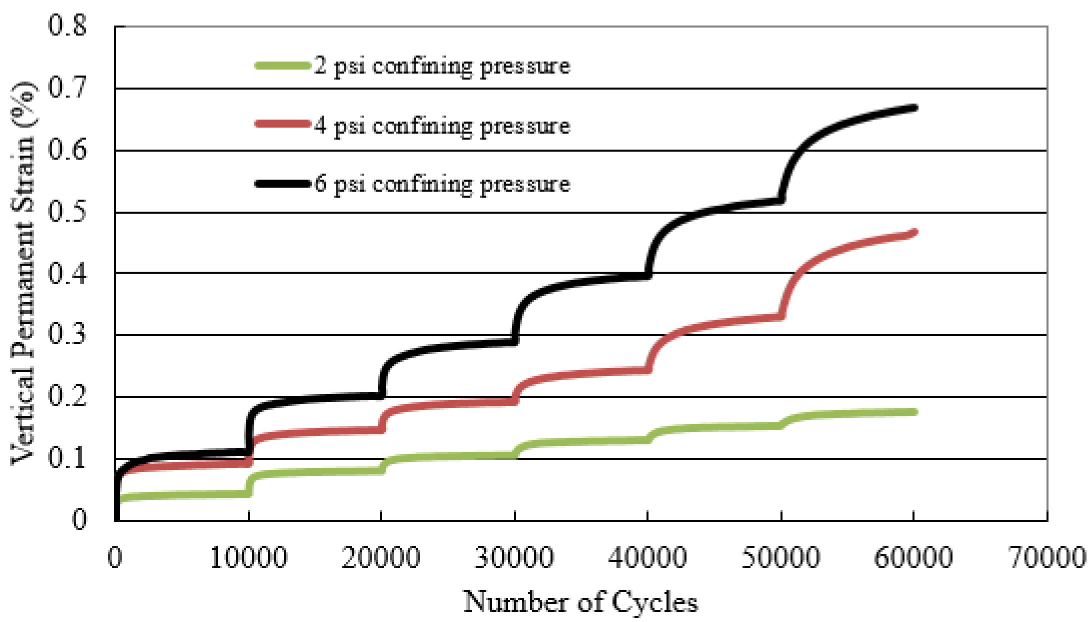


Figure 127. Multi-stage permanent deformation soil #3 treated with 6% cement, 9% lime at omc-2% (7 days).

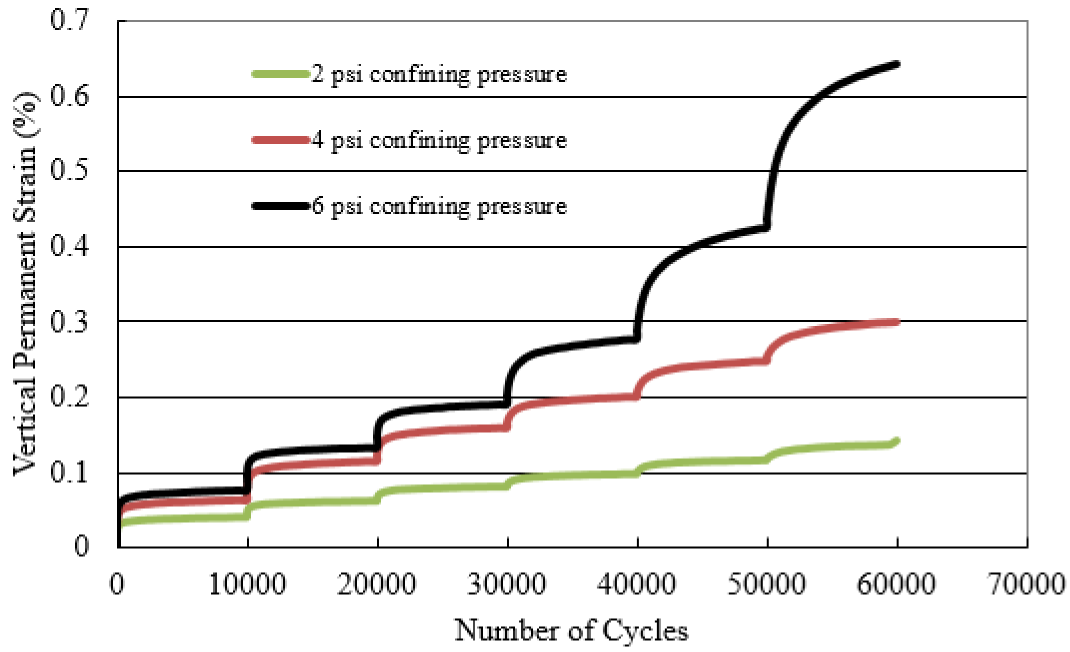
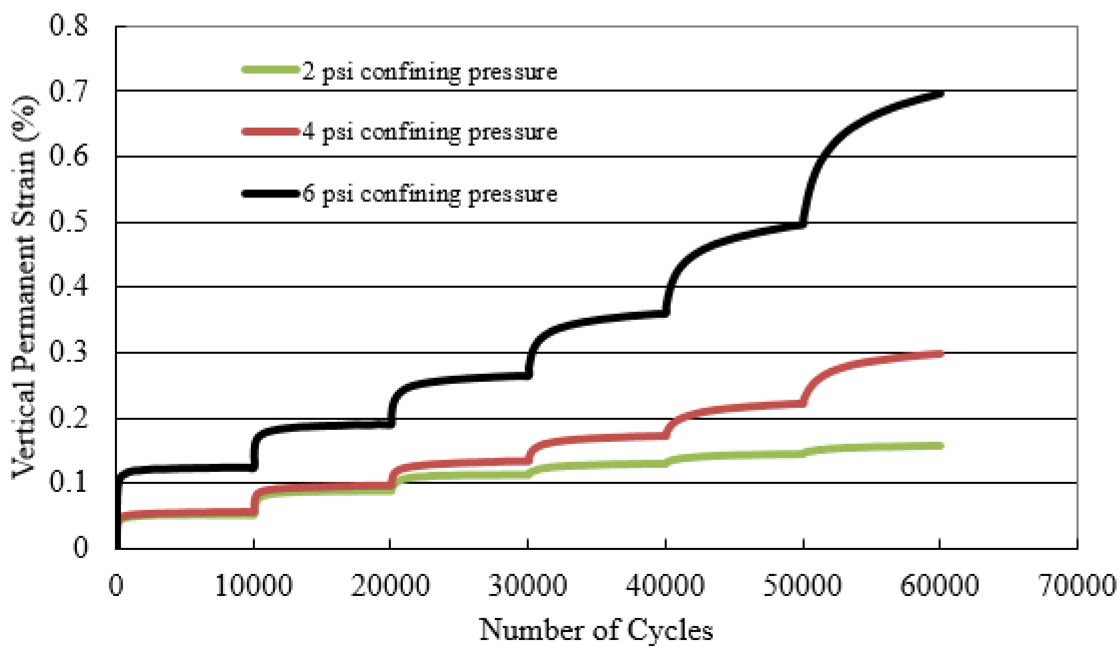


Figure 128. Multi-stage permanent deformation soil #3 treated with 6% cement, 9% lime at omc (28 days).



Appendix D
Variation of Resilient Modulus with Stress Conditions for Phase II

Figure 129. Soil I average resilient modulus for three treated specimens at MC1 (50 psi UCS)

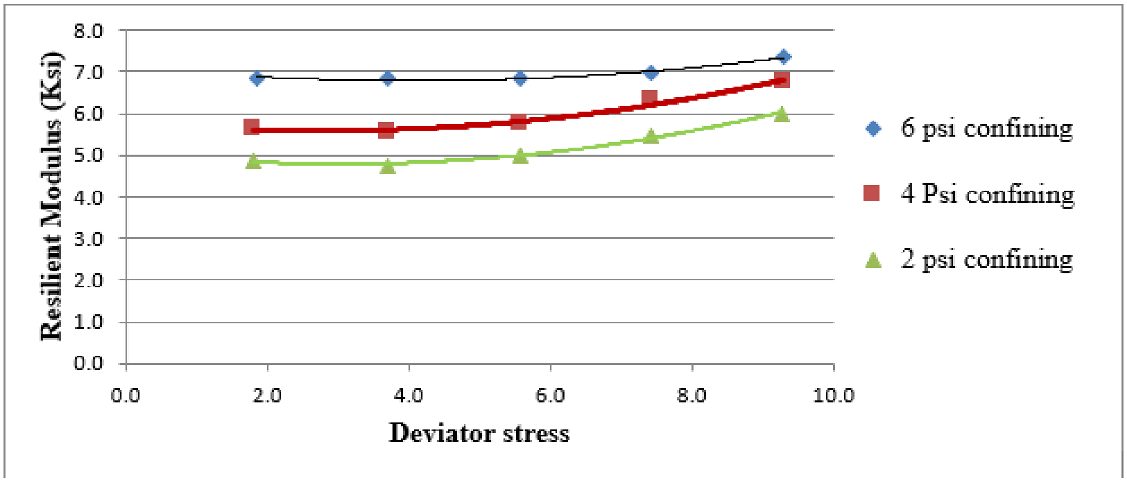


Figure 130. Soil I average resilient modulus for three treated specimens at MC2 (50 psi UCS)

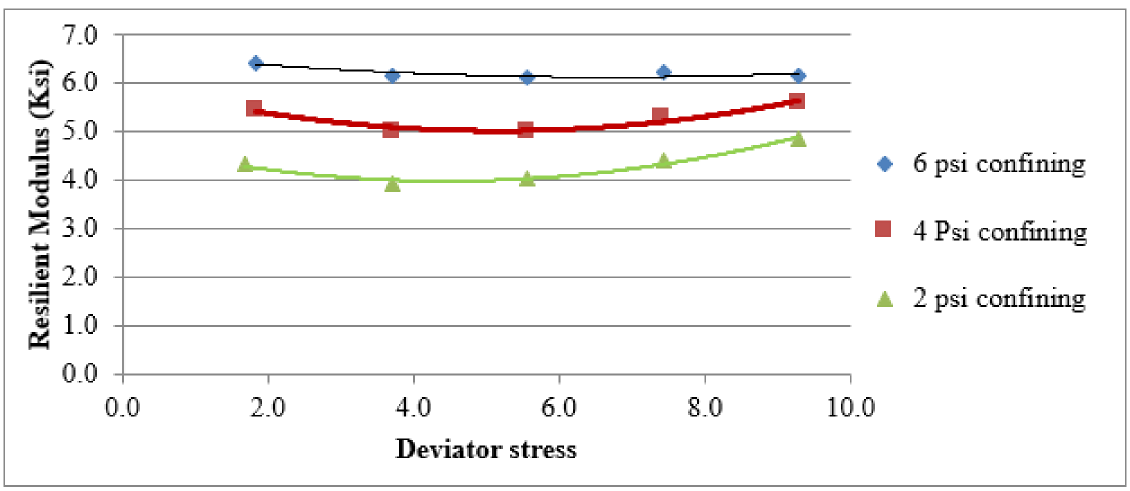


Figure 131. Soil I average resilient modulus for three treated specimens at MC3 (50 psi UCS)

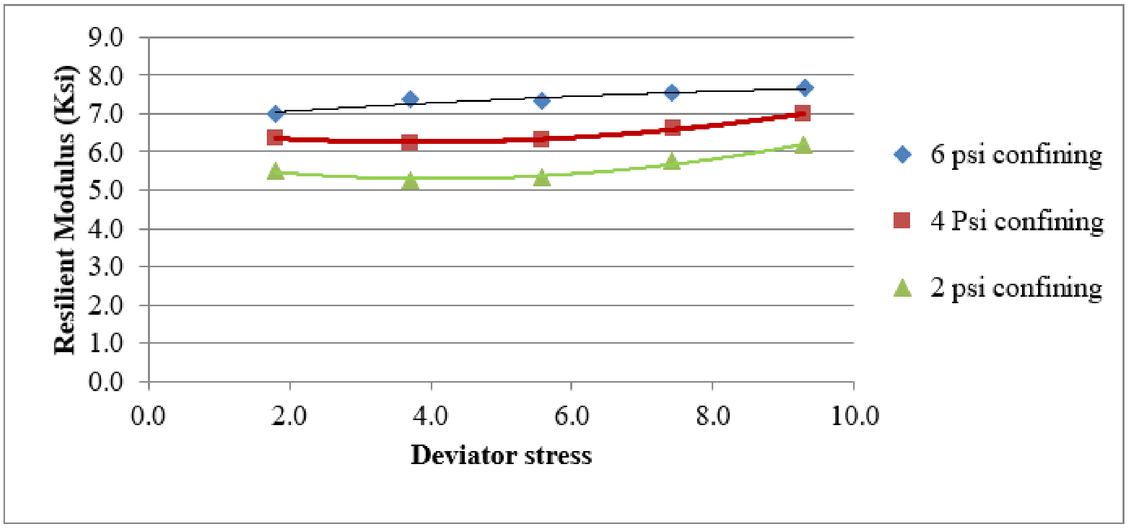


Figure 132. Soil I average resilient modulus for three treated specimens at MC1 (100 psi UCS)

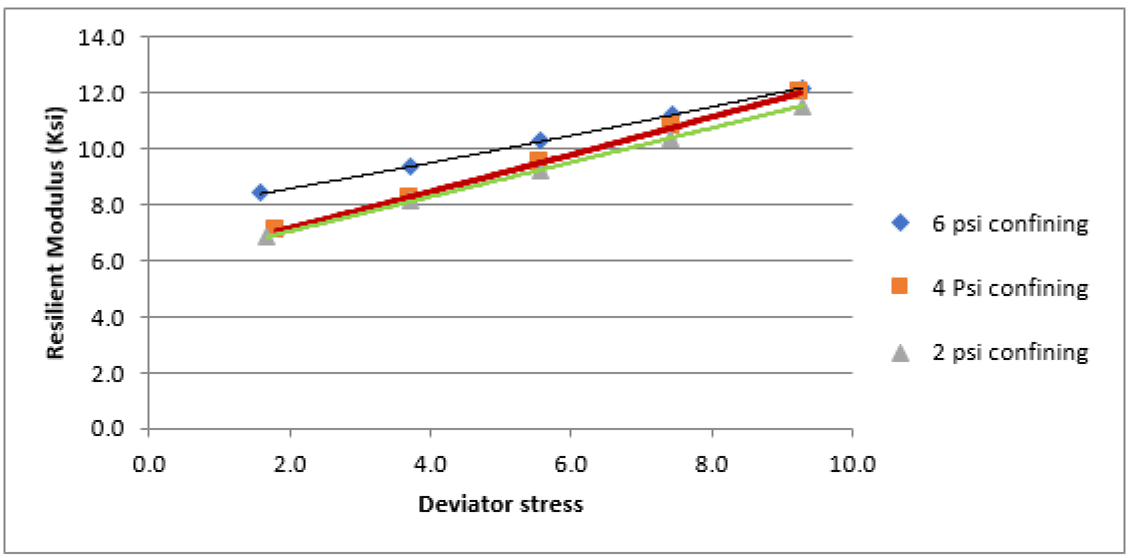


Figure 133. Soil I average resilient modulus for three treated specimens at MC2 (100 psi UCS)

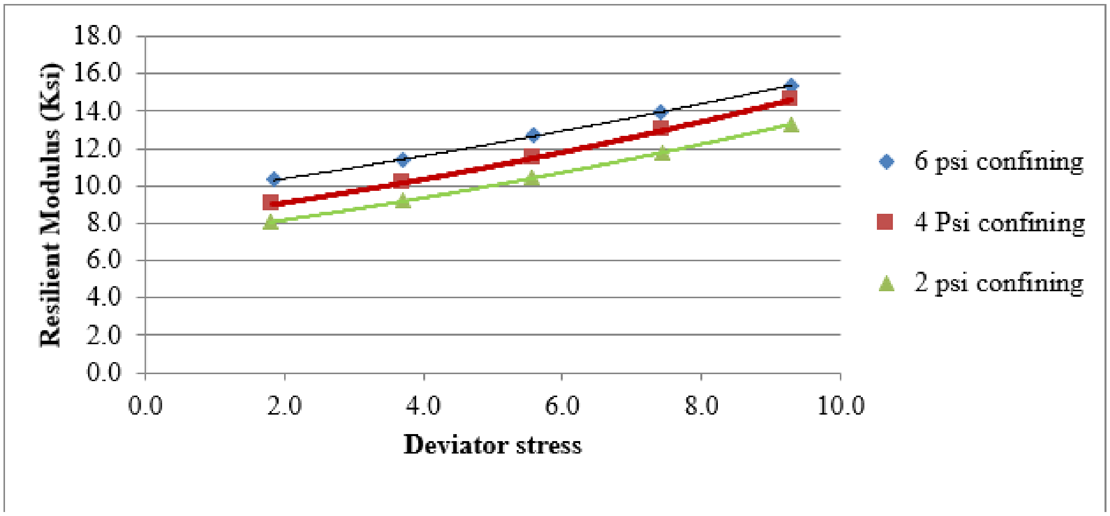


Figure 134. Soil I average resilient modulus for three treated specimens at MC3 (100 psi UCS)

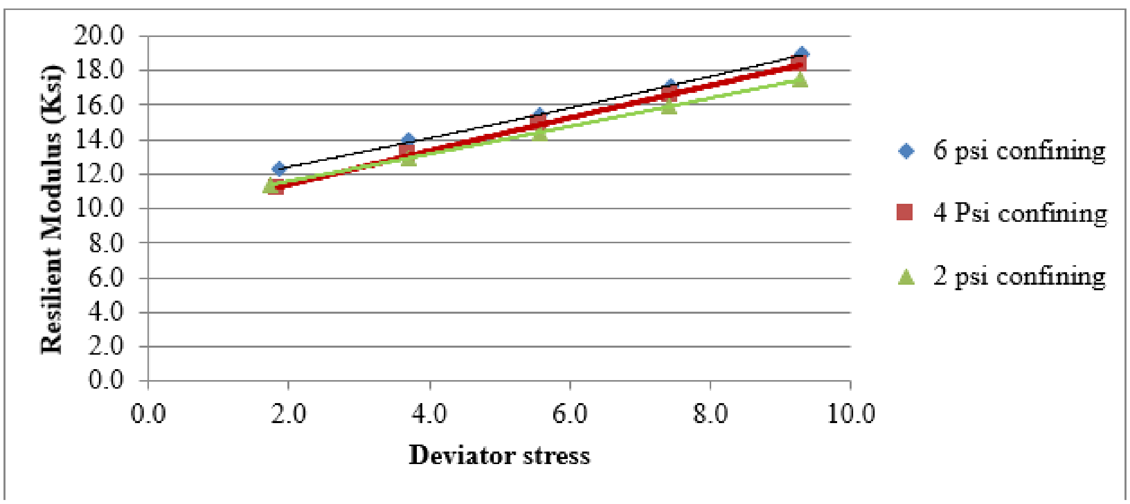


Figure 135. Soil II average resilient modulus for three treated specimens at MC1 (50 psi UCS)

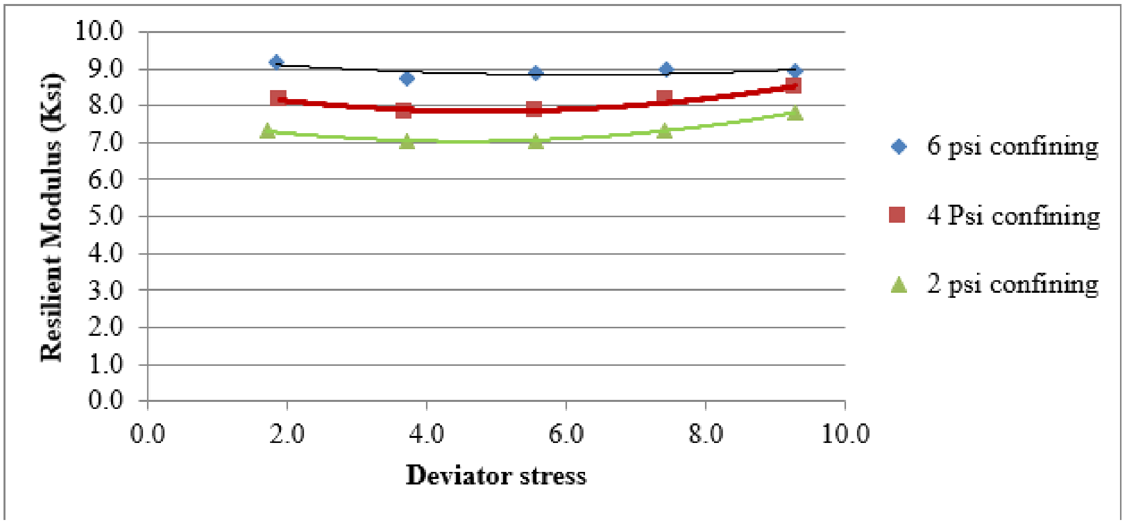


Figure 136. Soil II average resilient modulus for three treated specimens at MC2 (50 psi UCS)

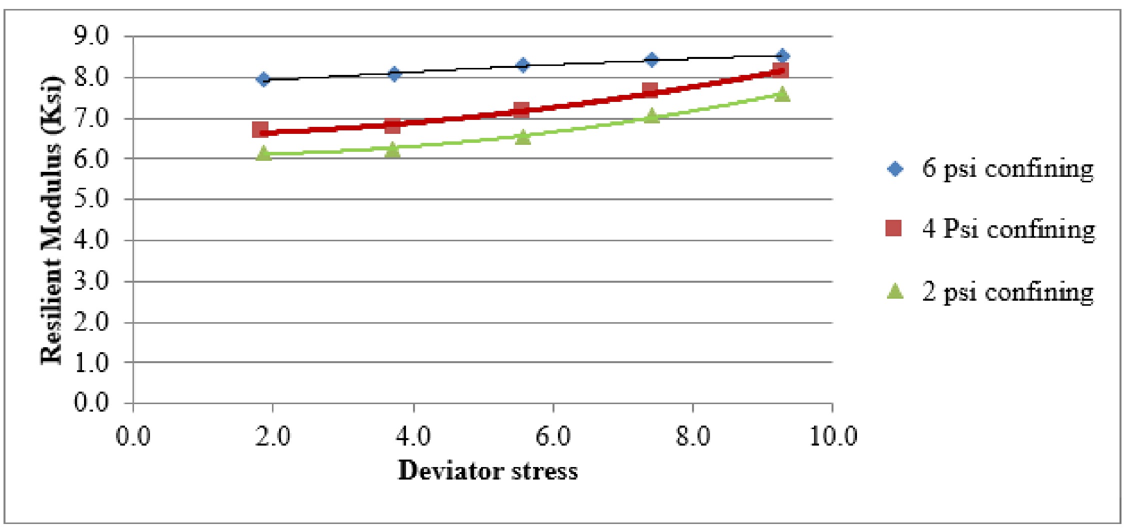


Figure 137. Soil II average resilient modulus for three treated specimens at MC3 (50 psi UCS)

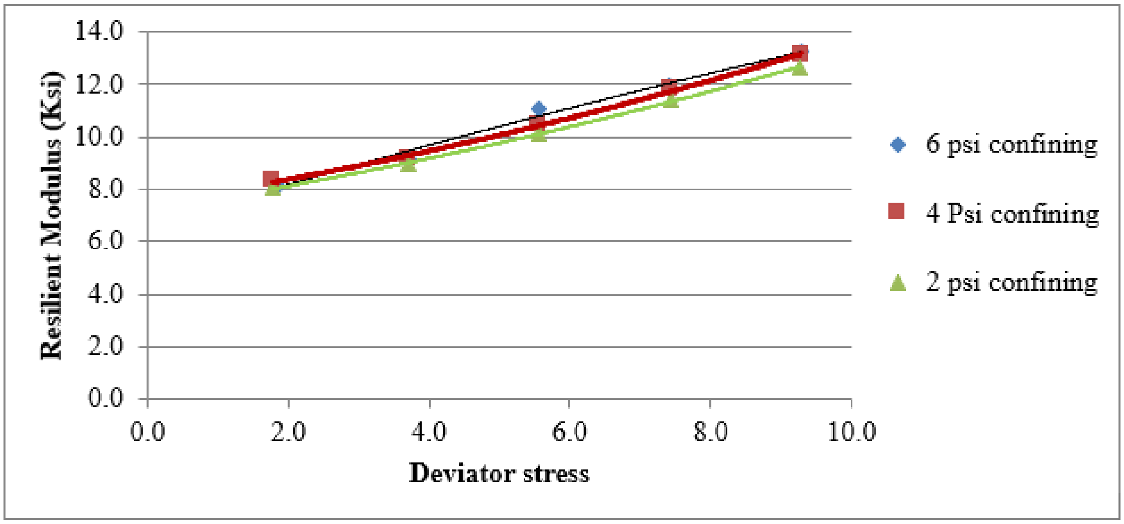


Figure 138. Soil II average resilient modulus for three treated specimens at MC1 (100 psi UCS)

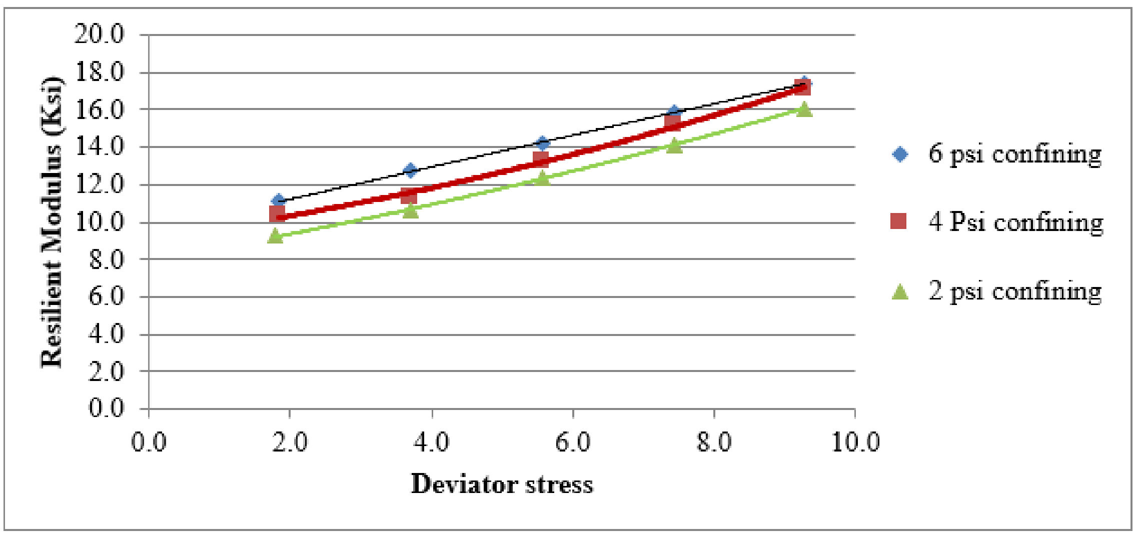


Figure 139. Soil II average resilient modulus for three treated specimens at MC2 (100 psi UCS)

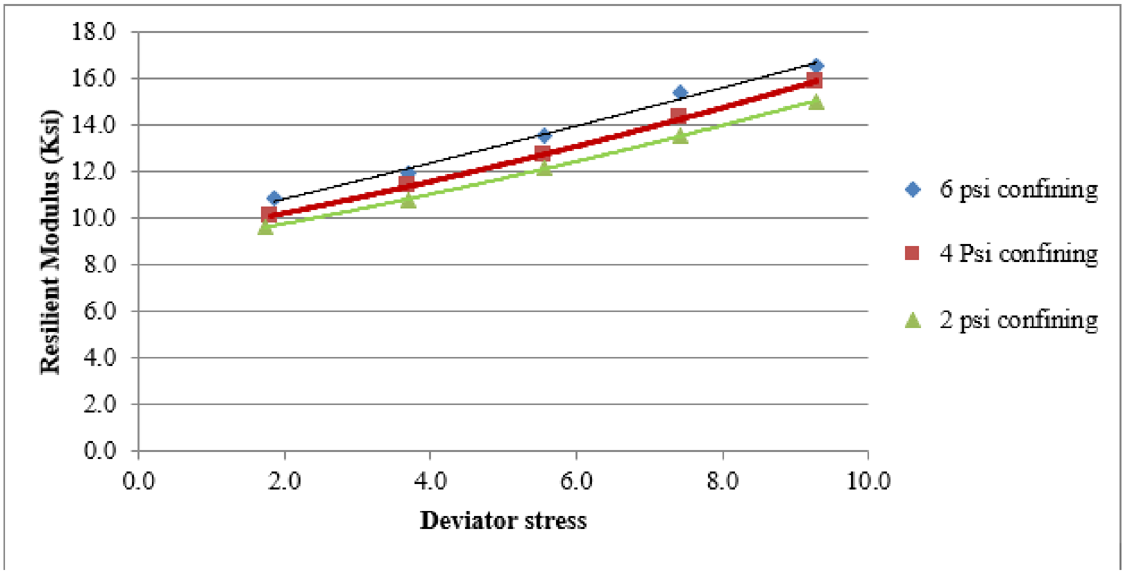


Figure 140. Soil II average resilient modulus for three treated specimens at MC3 (100 psi UCS)

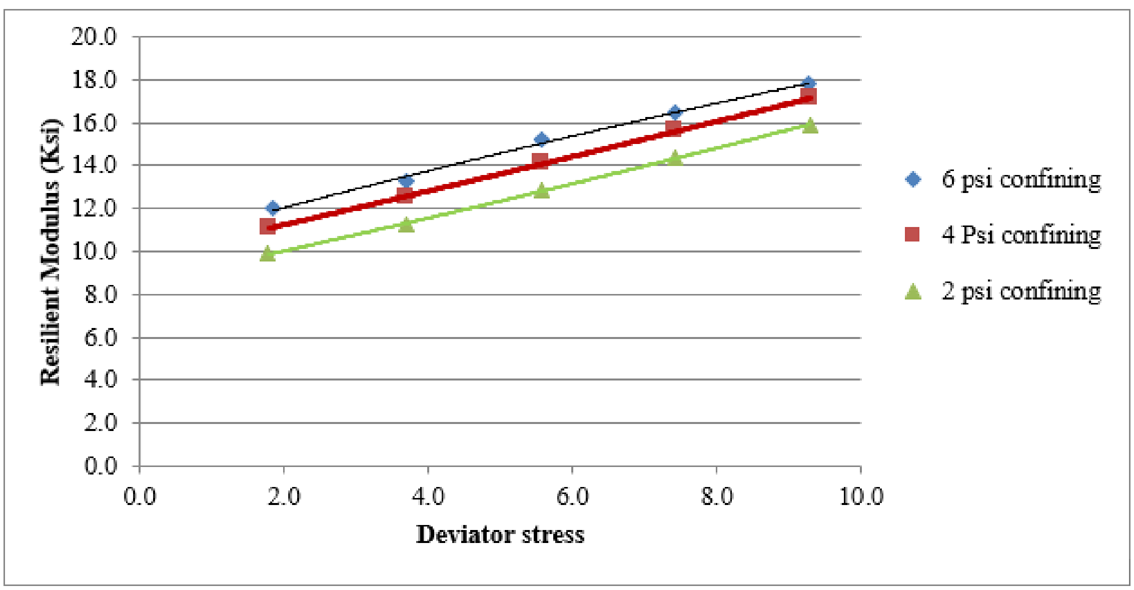


Figure 141. Soil III average resilient modulus for three treated specimens at MC1 (50 psi UCS)

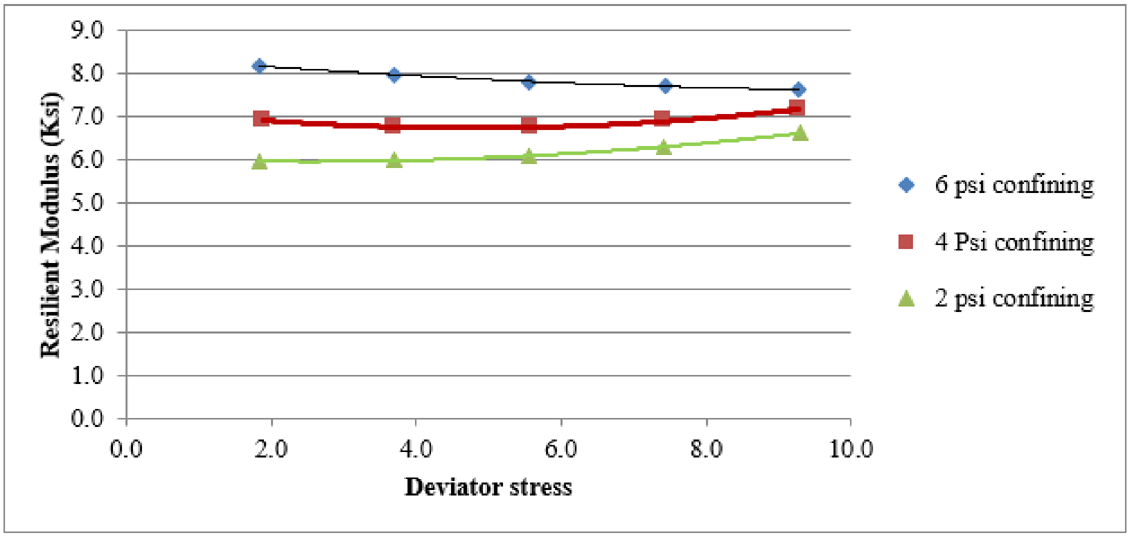


Figure 142. Soil III average resilient modulus for three treated specimens at MC2 (50 psi UCS)

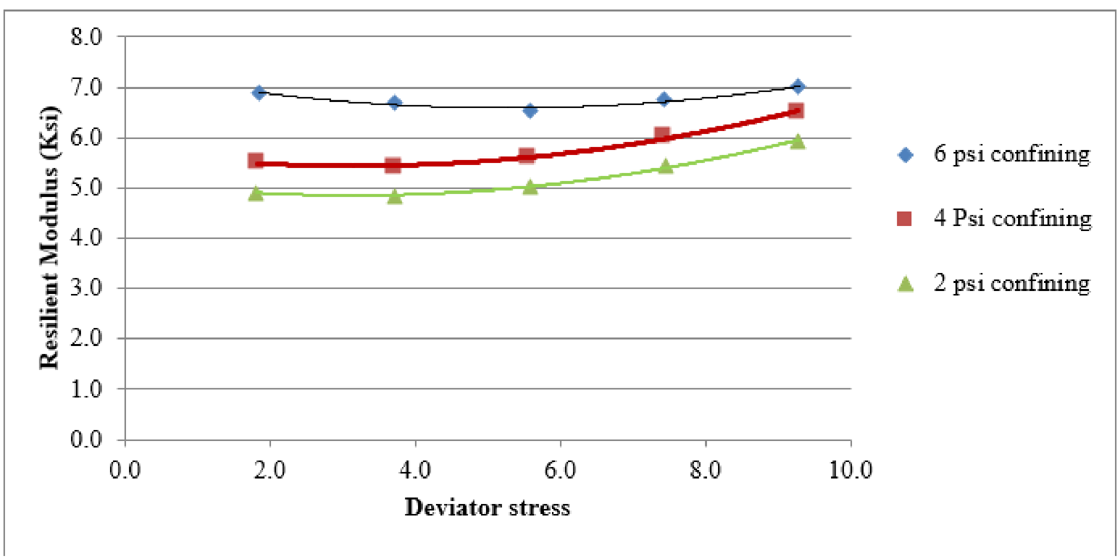


Figure 143. Soil III average resilient modulus for three treated specimens at MC3 (50 psi UCS)

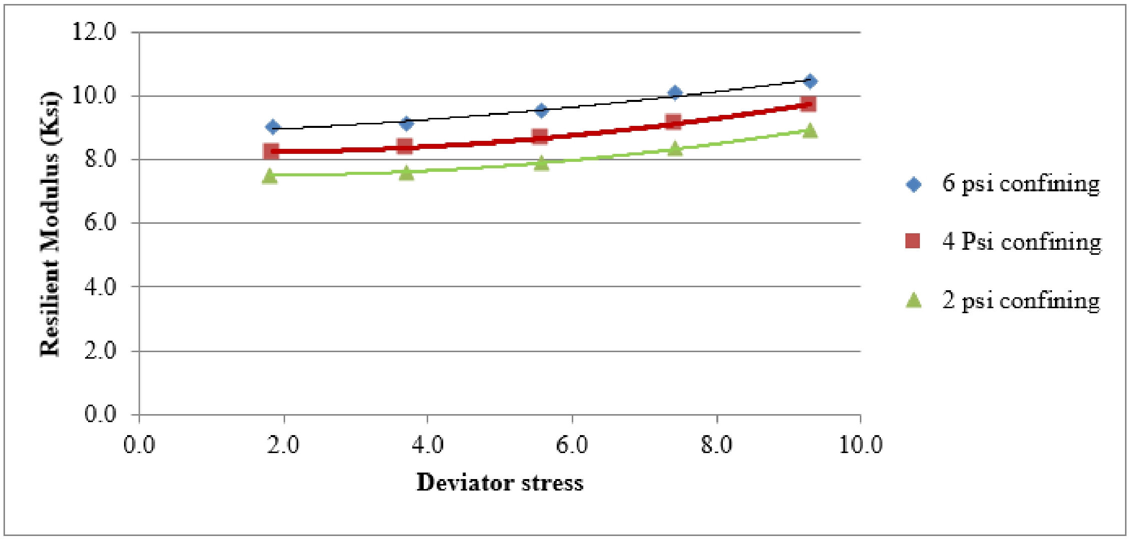


Figure 144. Soil III average resilient modulus for three treated specimens at MC1 (100 psi UCS)

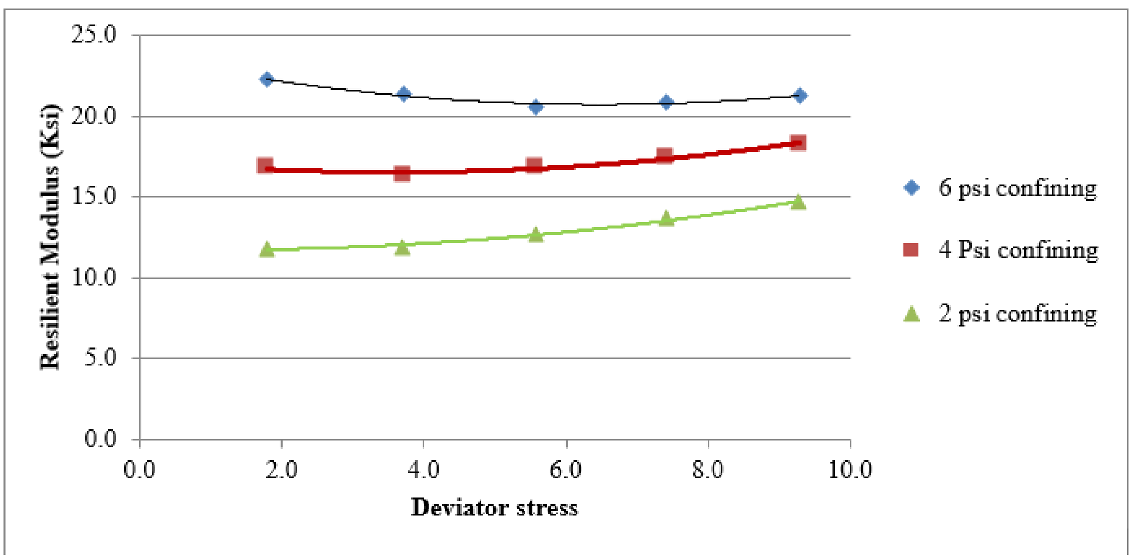


Figure 145. Soil III average resilient modulus for three treated specimens at MC2 (100 psi UCS)

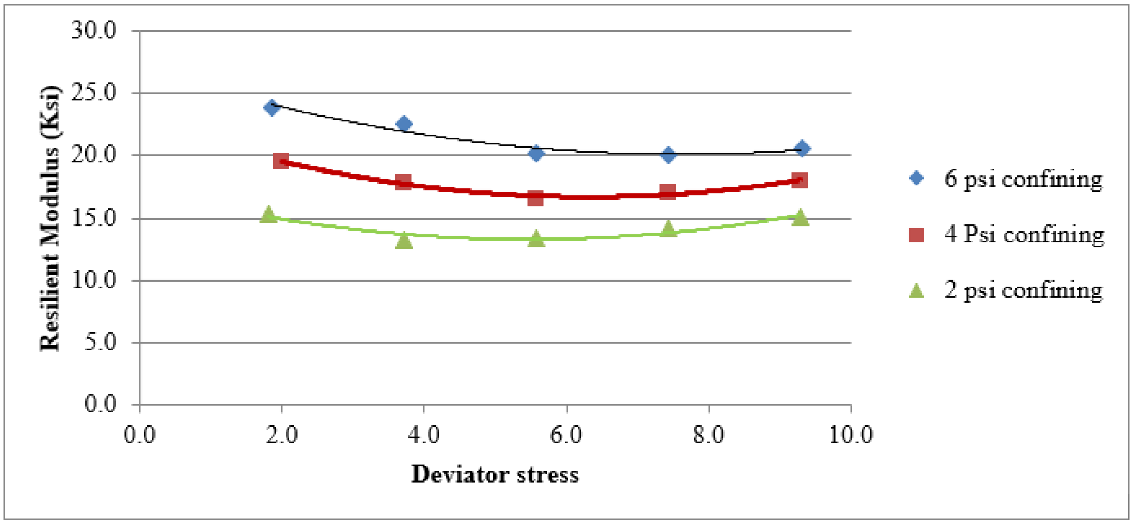


Figure 146. Soil III average resilient modulus for three treated specimens at MC3 (100 psi UCS)

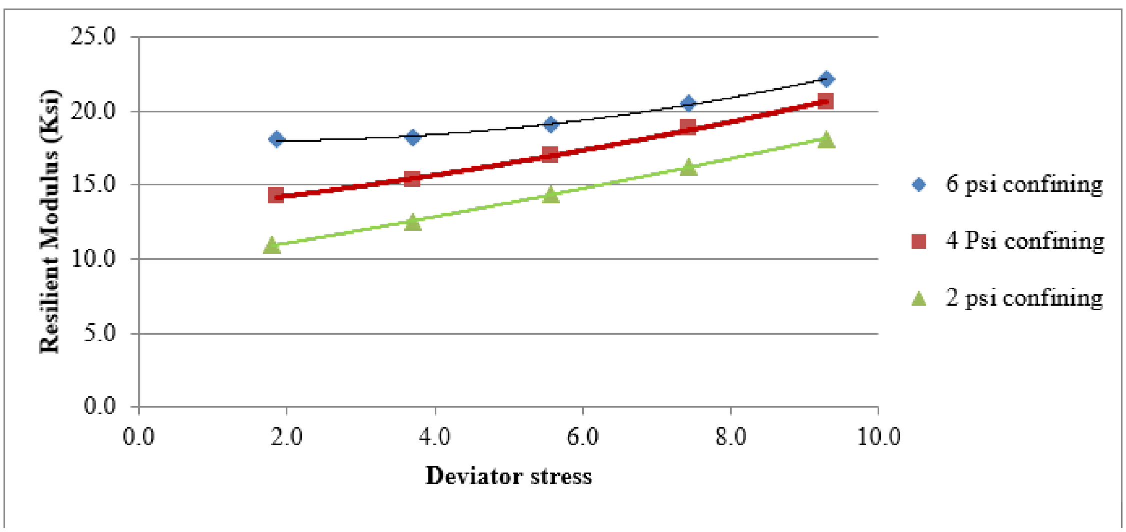


Figure 147. Soil IV average resilient modulus for three treated specimens at MC1 (50 psi UCS)

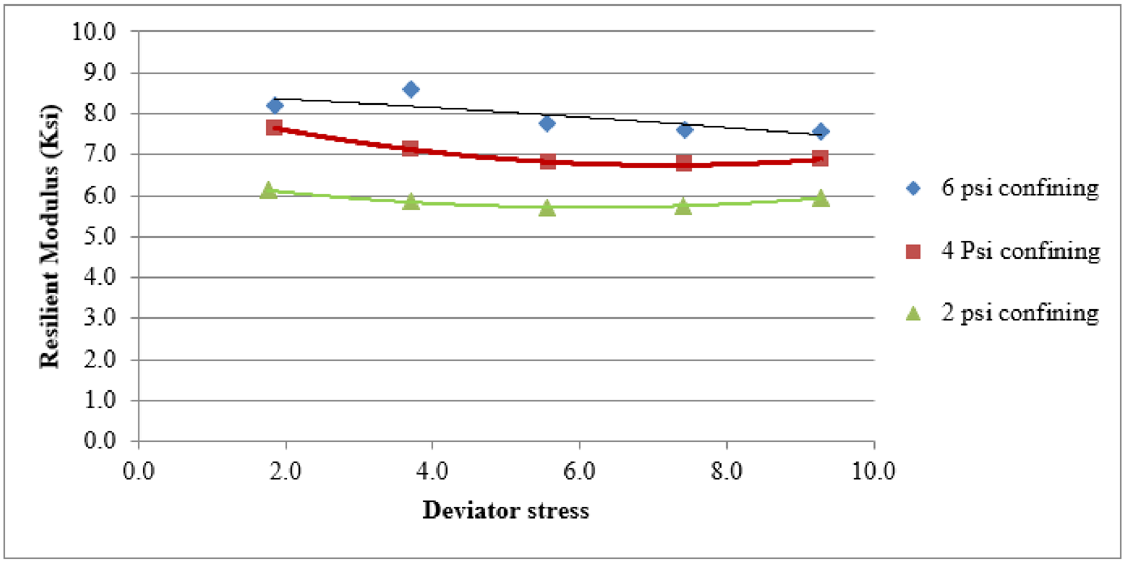


Figure 148. Soil IV average resilient modulus for three treated specimens at MC2 (50 psi UCS)

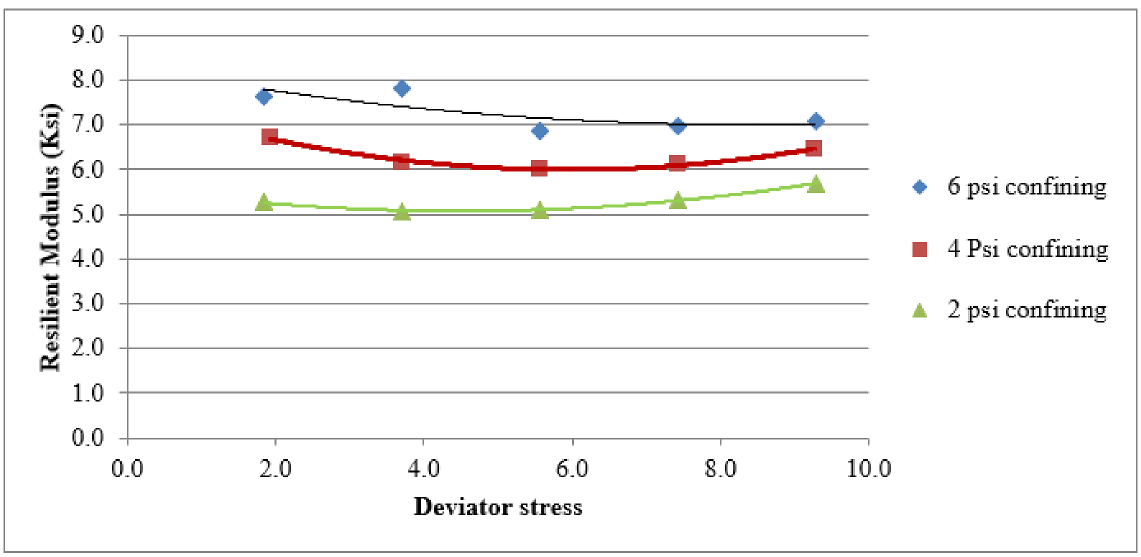


Figure 149. Soil IV average resilient modulus for three treated specimens at MC3 (50 psi UCS)

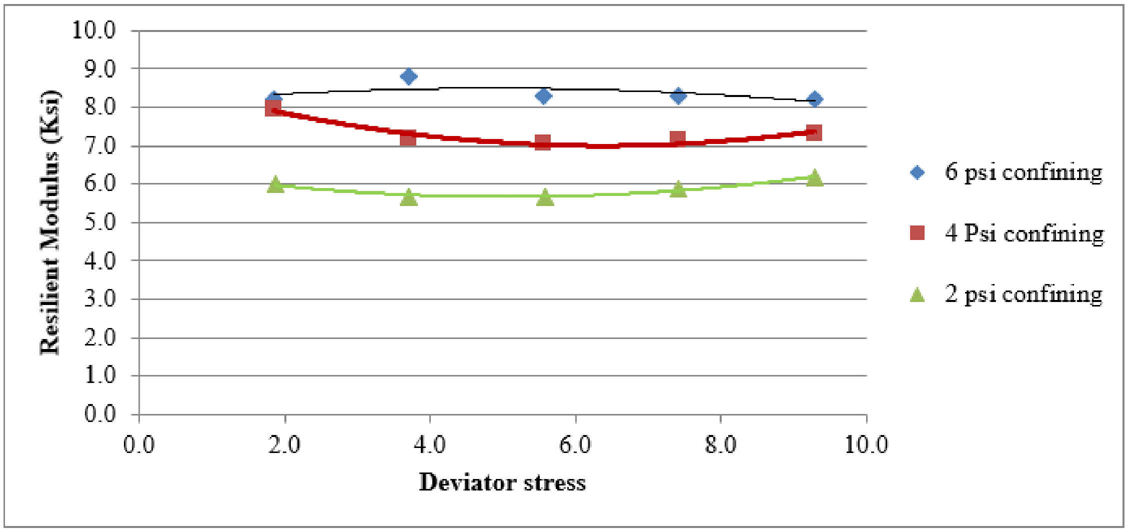


Figure 150. Soil IV average resilient modulus for three treated specimens at MC1 (100 psi UCS)

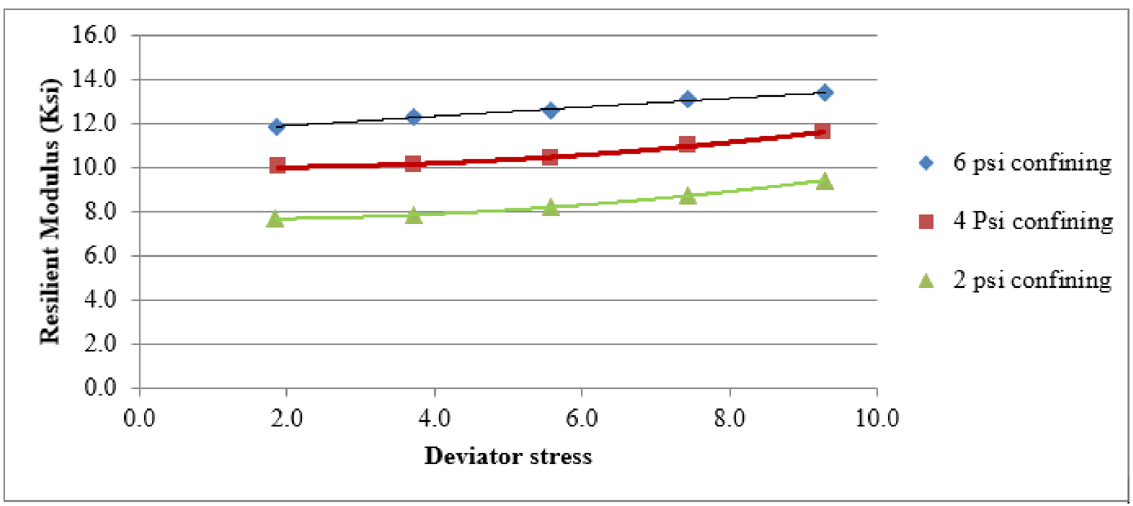


Figure 151. Soil IV average resilient modulus for three treated specimens at MC2 (100 psi UCS)

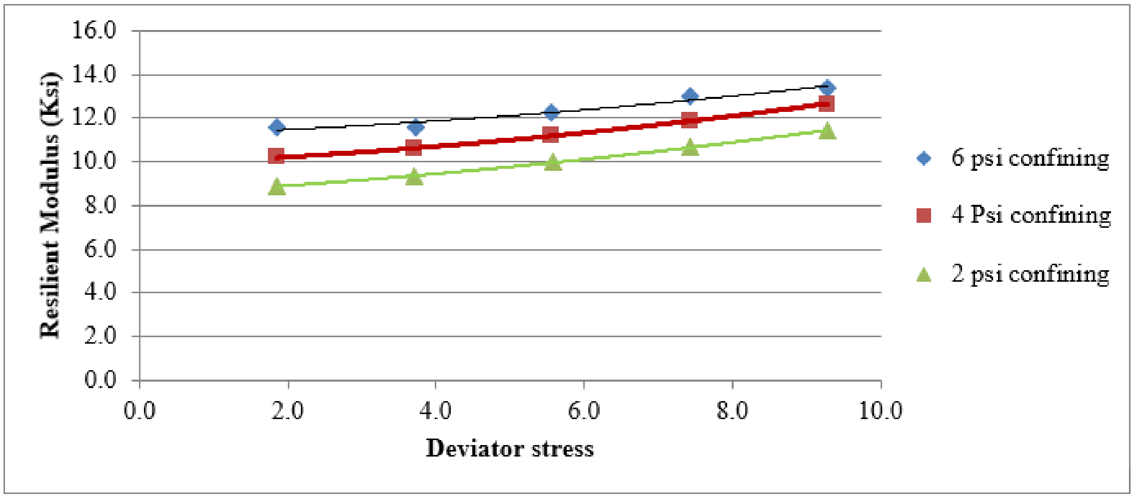
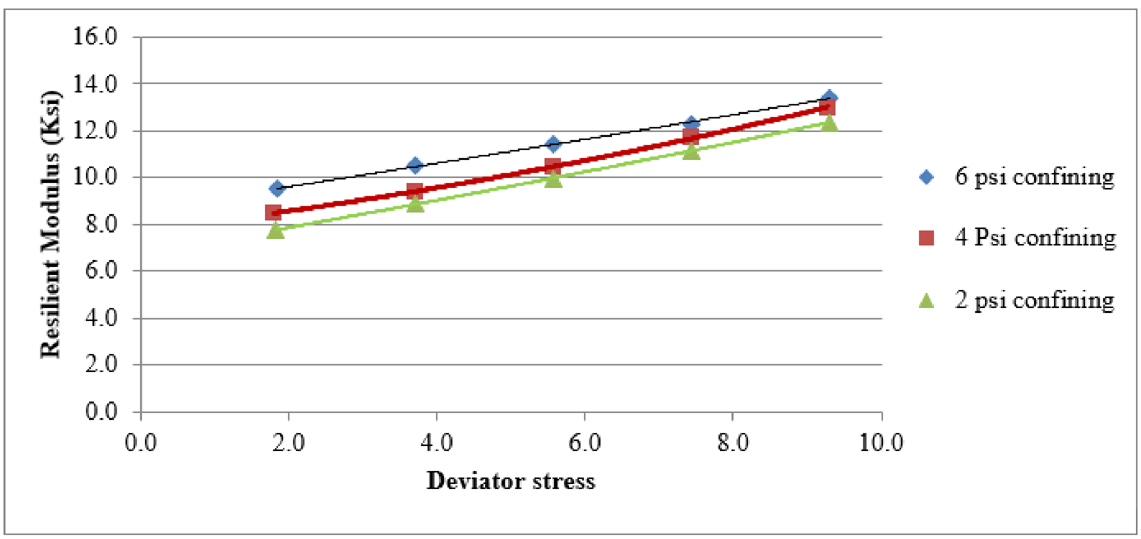


Figure 152. Soil IV average resilient modulus for three treated specimens at MC3 (100 psi UCS)



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