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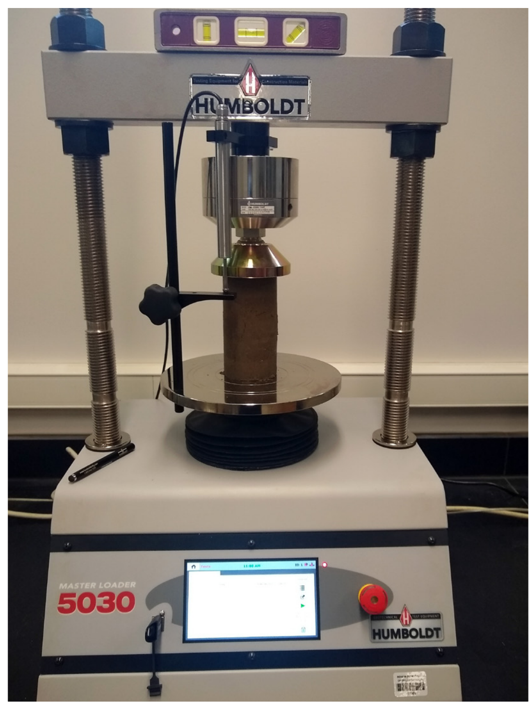
Estimating Strength from Stiffness for Chemically Treated Soils



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16. Abstract

The central theme of this study is to identify strength-stiffness correlations for chemically treated subgrade soils in Indiana. This was done by conducting Unconfined Compression (UC) Tests and Resilient Modulus Tests for soils collected at three different sites—US-31, SR-37, and I-65. At each site, soil samples were obtained from 11 locations at 30 ft spacing. The soils were treated in the laboratory with cement, using the same proportions used for construction, and cured for 7 and 28 days before testing. Results from the UC tests were compared with the resilient modulus results that were available. No direct correlation was found between resilient modulus and UCS parameters for the soils investigated in this study. A brief statistical analysis of the results was conducted, and a simple linear regression model involving the soil characteristics (plasticity index, optimum moisture content and maximum dry density) along with UCS and resilient modulus parameters was proposed.

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EXECUTIVE SUMMARY

Introduction

The quality of any pavement structure and its performance capabilities depends on the strength and stiffness properties of the subgrade soil; thus, the performance of a pavement can be improved by stabilization of available subgrade material. Cement stabilization is one of the most effective subgrade improvement methods because it increases the thermal, chemical, and mechanical resistance of the soil. Subgrade stabilization with cement-treated subgrades for INDOT projects is based on compressive strength. While UCS is a simple and commonly used laboratory test to measure the strength of treated soils, the in-situ strength determination of subgrade soils is a rather cumbersome process. However, the Falling Weight Deflectometer (FWD) Test is a non-destructive test used frequently by INDOT to determine the stiffness of subgrades and the quality of the pavement structure. However, the laboratory determination of subgrade stiffness is carried out by a Resilient Modulus Test (M_R). The current project aims to develop correlations between the compressive strength (e.g., UCS) and the stiffness (e.g., MR) of cement-treated subgrades. Three Indiana road project sites with access to untreated subgrade (US-31, SR-37, and I-65) were selected for this project. Cement-treated soil specimens were then used to perform UCS and M_R tests. The specimens were prepared based on the cement percentages used for the design of the roads. Linear regression analyses were carried out to correlate soil properties (plasticity index, OMC and MDD) and resilient modulus with UCS results.

Findings

Based on the results of the tests and analyses performed, no direct statistically significant correlation was found between resilient modulus and UCS. The absence of good correlations between the two tests was consistent with the findings reported in technical literature. However, it was observed that resilient modulus values depend on the type of soil. The range of resilient modulus values for cement-treated A-2-4 soils in this study ranged from 210 MPa to 275 MPa (30,000 psi to 40,000 psi), while cement-treated A-4 and A-6 soils fell in the range of 135 MPa to 240 MPa (20,000 psi to 35,000 psi). Linear regression analyses using normalized resilient modulus and normalized UCS showed a good correlation; however, this finding requires prior knowledge of the resilient modulus and unconfined compression strength of the soil, and the observation is based on limited data. More high quality laboratory data is needed to increase confidence in the results.

Implementation

The test results showed that the resilient modulus of A-2-4, A-4, and A-6 soils treated with cement fall within a fairly narrow range of values. These results could then be used as preliminary estimates of the expected values of the stiffness of the soil in the field, as well as the UCS values. Also, the correlation found between normalized M_R and UCS can be used; however, it requires prior knowledge of the M_R and UCS values of the soil.

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1. INTRODUCTION

Stiffness of the subgrade layer is one of the leading soil properties used for pavement design. *The Mechanistic-Empirical Pavement Design Guide* (MEPDG) relies on the resilient modulus (M_R) of the subgrade; however, M_R tests are specialized tests that require expensive equipment and are time consuming. Alternatively, the resilient modulus may be estimated from correlations with FWD after the construction of pavement for quality assurance (QA) purposes. Chemical stabilization using lime or cement is widely used by INDOT to improve the subgrade. Also, INDOT requires that mix designs for subgrade stabilization have a minimum UCS. The UCS test is easily performed in the laboratory but is rather difficult in the field requiring extensive equipment for imparting sufficient stress to induce bearing capacity failure. Given that both strength and stiffness affect construction, design, and performance of pavements, the central theme of this study is to establish correlations between strength and stiffness for subgrade soils in Indiana and, more specifically, of chemically treated subgrades.

The objectives of this research are as follows.

- Explore availability of target projects with stabilized subgrades.
- Specify target subgrades based on prevalence and urgency to INDOT.
- Conduct an extensive laboratory testing and develop performance-based correlations of compressive strength (e.g., UCS) and stiffness (e.g., M_R).
- Develop an effective procedure that can be easily adopted to other subgrades.
- Provide recommendations for specification of chemically stabilized subgrade for new pavement design.

To achieve the objectives of this study, a detailed literature review of existing correlations between resilient modulus and UCS values for untreated as well as chemically treated subgrade soils was first performed. Resilient modulus and UCS data from a recent JTRP report (Sandoval et al., 2019) were used to study initial correlations for chemically treated Indiana soils. This was followed by subgrade soil sample collection from three Indiana road construction sites: US-31, SR-37 and I-65. Disturbed subgrade soil samples collected from these sites were used to perform soil characterization, M_R tests as well as UCS tests. An attempt was then made to develop correlations between the treated UC strength and stiffness as obtained from M_R data. The highlights of statistical analysis performed to establish correlations for the resilient modulus and UCS test results are discussed in the following sections. During the course of the study, the need to look into the history of M_R test procedure was realized. The last section in this report presents a brief literature review of the history of the M_R test protocol evolution and discusses experimental changes that could improve the interpretation of laboratory test results.

2. LITERATURE REVIEW

2.1 Untreated Soils

The MEPDG, for pavement design, relies on the resilient modulus of the subgrade. M_R tests are specialized tests that require expensive equipment and are time consuming. Past research has focused on using index properties such as percentage of fines (percent passing No. 200 sieve), plasticity, compaction, and related properties such as moisture content, dry density, degree of saturation and UCS to develop empirical relationships to estimate the resilient modulus of the soil.

Hossain and Kim (2015) performed a comprehensive study of correlations between UCS and other properties derived from UCS curves for fine grained subgrade soils. Figure 2.1 displays the results of a correlation between UCS and resilient modulus (at 2 psi confining pressure and 6 psi deviatoric strength), for two different types of sample preparation methods for UCS (static and proctor compaction).

As we can see from the Figure 2.1, there is a fair correlation for the soils investigated. To improve the correlation, Hossain and Kim (2015) explored a model involving soil index properties along with UCS. They found that the addition of index properties improved the correlations for both sample preparation methods. The models along with their R^2 values are expressed in Equations 2.1 and 2.2.

$$\text{For static compaction:}$$
$$M_R = 7{,}884.2 + 99.7 \times (UCS) + 193.1 \times \mathrm{PI} - 47.9 \times P_{200};\ R^2 = 0.86 \qquad \text{(Eq. 2.1)}$$

$$\text{For proctor compaction:}$$
$$M_R = 6{,}113.0 + 95.1 \times (UCS) + 173.7 \times \mathrm{PI} - 27.8 \times P_{200};\ R^2 = 0.93 \qquad \text{(Eq. 2.2)}$$

Where, PI: plasticity index and P_{200}: percent passing No. 200 sieve.

The study also looked at correlations between initial tangent modulus (derived from the UCS stress-displacement plot) and resilient modulus but found no good correlations as the values of the initial tangent modulus were not accurately determined due to the initial seating deformations. However, the researchers were able to obtain an excellent correlation (R^2 = 0.97) between stress at 1% strain level and resilient modulus.

Lee et al. (1997) found similar results, i.e., a strong correlation between stress at 1% strain level (extracted from UCS curves) and resilient modulus, for three Indiana clayey soils: A-4–A-6 (CL), A-6 (CL), and A-7-6 (CH). The M_R test was conducted on the same sample where the UCS test was performed up to a 1% strain level. The resilient modulus values used were

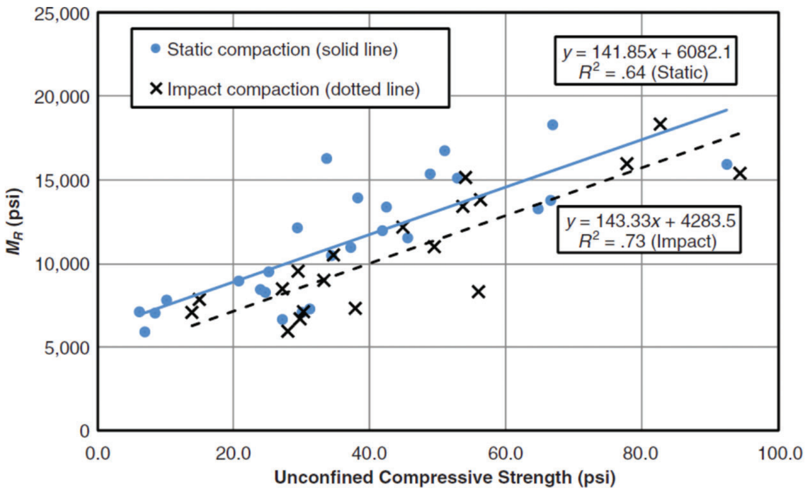


Figure 2.1 Correlation results between UCS and resilient modulus (Hossain & Kim, 2015).

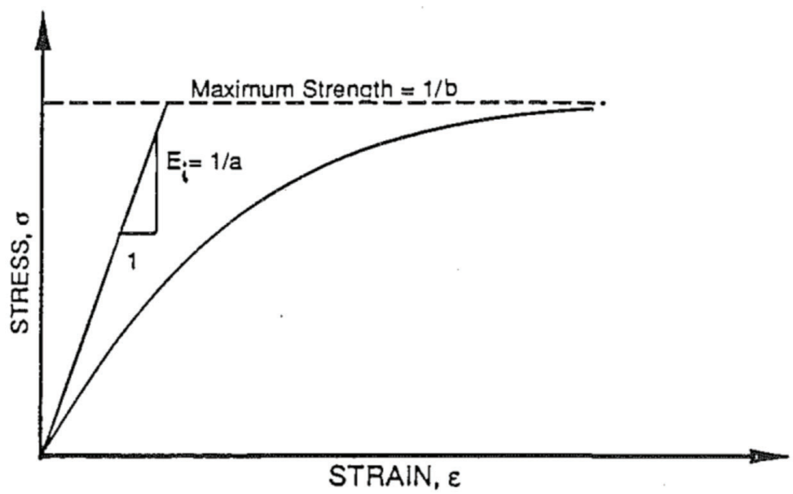


Figure 2.2 Hyperbolic representation of UCS curve, as shown in Drumm et al. (1990).

taken at 3 psi confining pressure and 6 psi deviatoric stress. The correlation found is given by Equation 2.3.

$$M_R = 695.4 \times (S_{u1\%}) - 5.93 \times (S_{u1\%}) \quad \text{(Eq. 2.3)}$$

Where, $S_{u1\%}$: stress at 1% strain level, and R^2 = 0.93.

Drumm et al. (1990) used 11 different types of fine-grained soils in Tennessee to obtain correlations between UCS, index properties, moduli obtained from UCS curves and resilient modulus. The UCS curve was assumed as hyperbolic and curve fitting parameters were used to find the initial tangent modulus or the small strain modulus. These parameters were then utilized for correlations with resilient modulus. Figure 2.2 shows the hyperbolic representation of the UCS curve.

The parameters a and b are calculated using standard curve fitting techniques, by writing the hyperbola equation in the form of Equation 2.4.

$$\sigma = \frac{\varepsilon}{a + b\varepsilon} \rightarrow \frac{\varepsilon}{\sigma} = a + b\varepsilon \quad \text{(Eq. 2.4)}$$

The correlation obtained for predicting resilient modulus, for a range of deviatoric stress (2.5 psi to 25 psi) under no confining pressure, can be expressed as Equation 2.5.

$$M_R = \frac{a' + b'\sigma_d}{\sigma_d} \quad \text{(Eq. 2.5)}$$

Where,
R^2 = 0.73,
a' = 318.2 + 0.337(q_u) + 0.73(% clay) + 2.26(PI) − 0.92 (γ_d) − 2.19(S) − 0.304(P_{200}),
b' = 2.10 + 0.00039(1/a) + 0.104(q_u) + 0.09(LL) − 0.10(P_{200}),
Where,
% clay = percentage finer than 0.002 mm,
P_{200} = percentage passing No. 200 sieve,
LL = liquid limit (%),
σ_d = deviator stress (psi),
q_u = UCS (psi), and
S = degree of saturation (%).

There are additional correlations and models in the literature for predicting resilient modulus similar to those described and based on UCS and soil index properties for untreated subgrade soils. Hossain et al. (2011) developed a relationship (Equation 2.6) between resilient modulus and UCS based on test results of 130 soil samples (A-4, A-6 and A-7-6) from Oklahoma.

$$\frac{M_r}{P_a} = 2{,}494.2 + 0.6(PI) - 8.66(P_{200}) + 16.4(GI) + 165.53(MCR) - 1{,}961(DR) + 185.29\left(\frac{q_u}{P_a}\right) \quad \text{(Eq. 2.6)}$$

Where,
$R^2 = 0.44$,
M_R = resilient modulus at deviator stress of 41.34 kPa (6 psi) and confining stress of 13.78 kPa (2 psi),
P_a = atmospheric pressure (kPa),
GI = group index,
MCR = moisture content ratio (moisture content/optimum moisture content), and
DR = density ratio (dry density/maximum dry density).

The literature reviewed shows fair correlations between UCS and resilient modulus for untreated soils. The correlations seem to improve when additional variables/parameters are included such as the index properties of the soil. Other stiffness parameters derived from the UCS curves such as the initial tangent modulus (calculated through curve fitting) have been used for establishing correlations of untreated soils.

2.2 Treated Soils

Thompson's (1966) work from the late 1960's in Illinois is one of the first studies looking into the relationship between stiffness and UCS for lime-stabilized subgrade soils. Thompson compared the shear strength (psi) and secant modulus of elasticity E (ksi), at peak stress, obtained from static, unconsolidated-undrained (UU) triaxial compression tests. The main equation of the correlation is given by Equation 2.7.

$$E(ksi) = 9.98 + 0.1235 q_u \ (psi) \quad \text{(Eq. 2.7)}$$

Where, q_u: UCS.

CTL/Thompson (1998) performed three sets of resilient moduli and UCS tests on A-7-6 soil mixed with 6% quicklime to verify the applicability of the Thompson's correlation. The M_R tests in this study were performed in accordance with AASHTO T 294 (1994). The results generally agree with Thompson's correlation for UCS values within the range of 1,000 to 1,400 kPa.

Little et al. (1995) studied subgrade soils in Texas stabilized with lime. They used both laboratory and field data and concluded that Thompson's correlation was conservative for UCS values greater than 1,000 kPa. Little et al. (1995) proposed a relationship between resilient modulus and UCS for lime-stabilized subgrades based on Thompson's correlation between UCS and flexural modulus (Thompson & Figueroa, 1980), and between UCS and resilient modulus back-calculated from FWD (Little et al., 1995). Figure 2.3 depicts

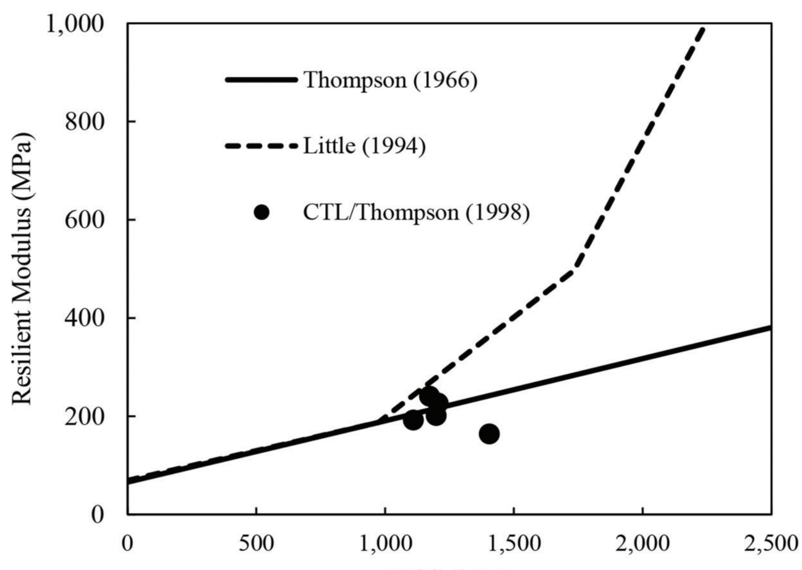


Figure 2.3 Comparison of Thompson's (1966) correlation, CTL/Thompson (1998) results, and Little et al. (1995) proposed relationship for lime stabilized soils (adapted from Toohey et al. 2013).

the comparison between Thompson's correlation and Little et al.'s (1995) proposed relationship for lime stabilized subgrade soils.

Toohey et al. (2013) tested three fine grained soils to try to reproduce the relationship between resilient modulus and UCS recommended by Thompson (1966) and Little et al. (1995) for lime-stabilized subgrade soils. Table 2.1 summarizes the soil classifications, grain size, and plasticity data for the soils used. Resilient modulus and UCS tests were performed on a total of 15 lime-treated soils (five per each soil type). Lime-treated specimens with 100 mm diameter and 200 mm height were prepared at OMC and ρ_{dmax} conditions (see Table 2.1). M_R values obtained with confining stresses at 14 kPa and 28 kPa, at a deviator stress of 41 kPa, were used in the analysis. The UCS tests were performed on the same specimens used for the M_R test, and immediately after the M_R test was completed.

Figure 2.4 (a) and (b) are plots of all M_R and UCS test results of A, B, and C soil specimens, at 14 kPa and 28 kPa confining stresses, respectively (both at 41 kPa deviatoric stress). The plots also include the Thomson's (1966) correlation and Little et al. (1995) proposed relations for lime-stabilized subgrades. As observed in Figure 2.4, there is no clear correlation between UCS and M_R values (that is, $R^2 < 0.05$ at both confining stresses).

Data from the recent JTRP project, SPR-4107: *Subgrade Stabilization Alternatives* (Sandoval et al., 2019), was used to test the correlation between resilient modulus and UCS for fine-grained Indiana subgrade soils. The project investigated soils from three locations in Indiana treated with cement, Lime Kiln Dust (LKD) and a combination of cement and LKD. Details of the soil properties (classification, index properties etc.) are summarized in Table 2.2. Table 2.3 lists the optimum amount of chemical needed, the optimum moisture content (OMC) and the maximum unit weight of the soils.

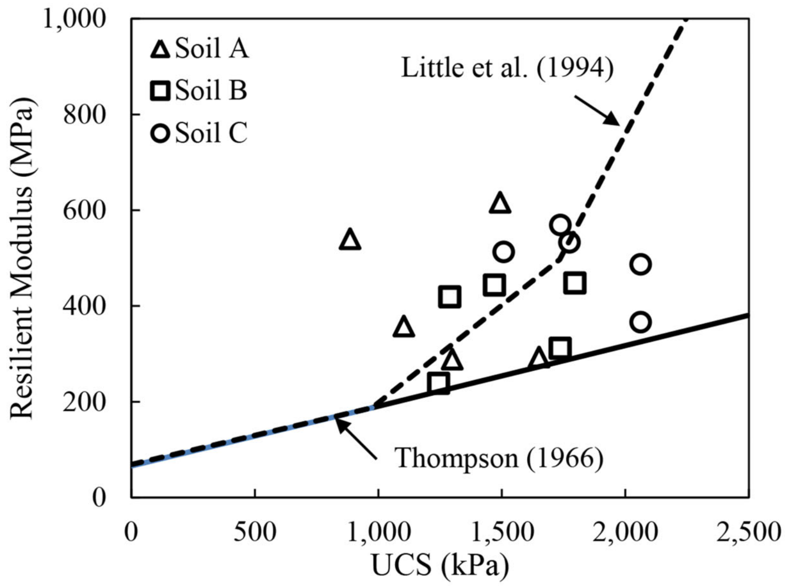
Resilient modulus and UCS tests were performed at 7-days and 28-days curing time for the treated specimens, which were prepared at OMC and at maximum unit weight, as described in Table 2.3. The results of the correlations between UCS and MR can be found in Figures 2.5 and 2.6. The resilient modulus at 2 psi confinement and 6 psi deviatoric stress was chosen for the comparisons. It is clear from the figures that there is no direct correlation between UCS and resilient modulus for the soils investigated in this study.

Becker (2021) developed a correlation between the resilient modulus and the unconfined compression strength for soils encountered on I-69 near Anderson, Indiana. The soil used was primarily fine-grained A-6 soil (CL based on USCS classification). Cement-treated soil specimens at different moisture content, relative compaction and cement content were prepared for resilient modulus and UCS testing. The study resulted in a fair correlation ($R^2 = 0.485$) between M_R and UCS for cement stabilized subgrade soils, as observed in Figure 2.7. The study proposed that since UCS correlated with M_R (it also correlated well with LWD deflection), UCS could be well-suited to relate cement-stabilized subgrade performance requirements (pavement design) and acceptance criteria (construction).

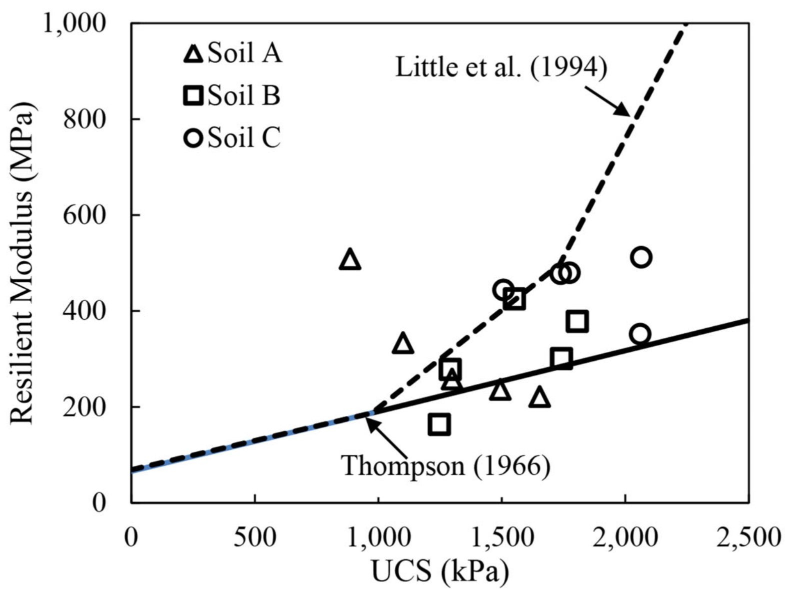
TABLE 2.1
Soil properties (untreated and treated) for soils A, B, and C (from Toohey et al., 2013)

	Untreated							Lime Treated	
Soil	**AASHTO Class**	**USCS Class**	**Clay (%)**	**Silt (%)**	**LL (%)**	**PL (%)**	**PI (%)**	**OMC (%)**	**ρ_{dmax} (kg/m^3)**
A	A-7-6	CH	29	19	55	18	37	29	1,394
B	A-6	CL	12	41	33	16	17	29	1,684
C	A-7-6	CL	15	58	43	15	29	25	1,554

Note: LL = liquid limit; PI = plasticity index; PL = plastic limit; ρ_{dmax} = maximum unit weight.



(a)



(b)

Figure 2.4 Summary of laboratory measured UCS versus (a) M_R (confining stress $\sigma_c = 14$ kPa; deviatoric stress $\sigma_d = 41$ kPa) and (b) M_R (confining stress $\sigma_c = 28$ kPa; deviatoric stress $\sigma_d = 41$ kPa) (adapted from Toohey et al., 2013).

TABLE 2.2
Site locations and soil properties from SPR-4107 project (Sandoval et al., 2019)

Site	LL (%)	PL (%)	PI (%)	Passing # 200	AASHTO Class
Hartford City	26.00	11.60	14.40	–	A-6
	37.20	14.20	23.00	88.20	A-6
Bloomington #1	41.20	17.30	23.90	88.40	A-7-6
Fort Wayne	43.00	14.10	28.90	82.00	A-7-6
Bloomington #2	66.00	20.80	45.20	93.50	A-7-6
Bloomington #3	58.60	21.00	37.60	–	A-7-6

TABLE 2.3
Optimum amount of treatment, maximum unit weight, and optimum moisture content for the four soils in SPR-4107 (Sandoval et al., 2019)

	Optimum LKD			Optimum Cement			Optimum Cement + LKD		
Site	**Amt (%)**	**γ_d (pcf)**	**OMC (%)**	**Amt (%)**	**γ_d (pcf)**	**OMC (%)**	**Amt (%)**	**γ_d (pcf)**	**OMC (%)**
Hartford City	6	115.4	16.5	3	121.1	12.3	–	–	–
Bloomington #1	6	103.6	20.8	3	107.3	19.6	2+2	106.1	20.2
Fort Wayne	5	113.6	15.6	3	117.9	14.8	–	–	–
Bloomington #2	5	98.6	26.3	5	101.1	25.7	2+2	99.8	26.4
Bloomington #3	5	101.1	23.1	5	101.8	22.7	–	–	–

Note: OMC = optimum moisture content; γ_d = maximum unit weight; Amt = amount.

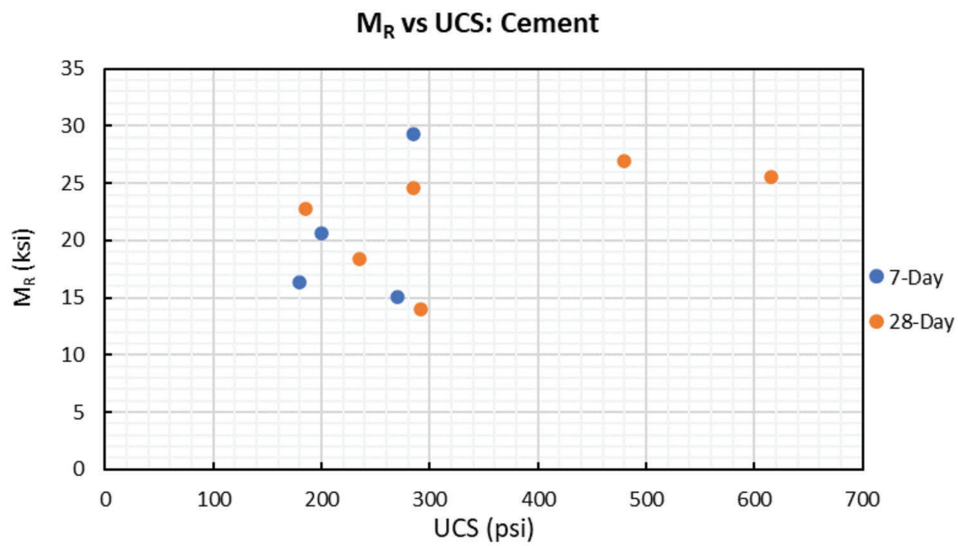


Figure 2.5 Correlation between UCS and resilient modulus for cement-treated samples (data from Sandoval et al., 2019).

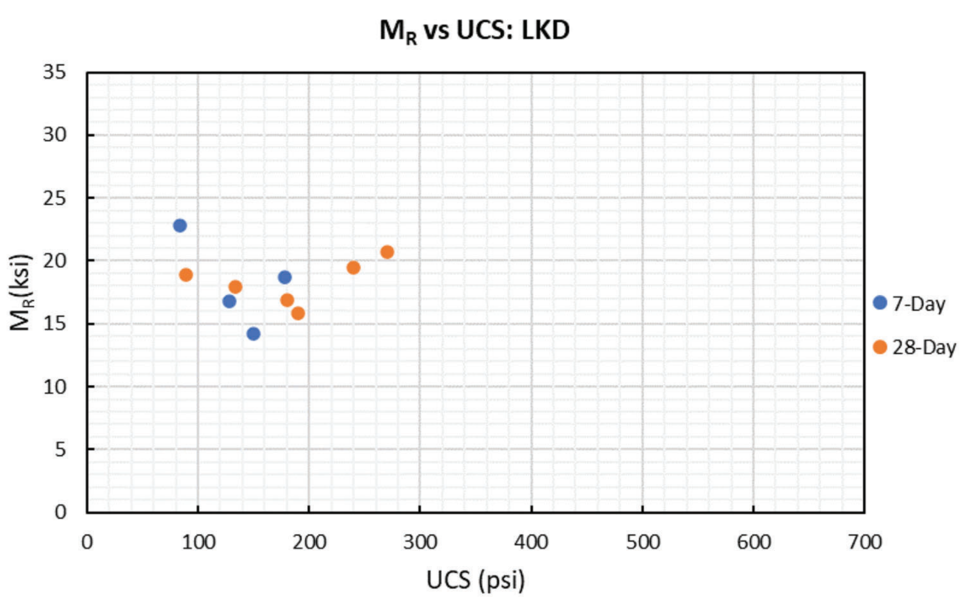
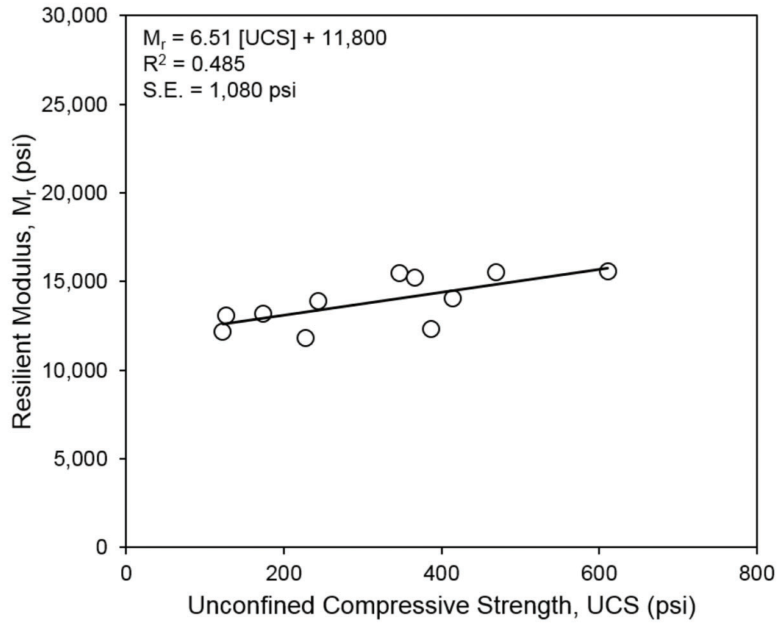


Figure 2.6 Correlation between UCS and resilient modulus for LKD-treated samples (data from Sandoval et al., 2019).



Note: S.E. = standard error.

Figure 2.7 Correlation between resilient modulus and UCS for cement stabilized A-6 soil specimens (Becker, 2021).

3. SOIL CHARACTERISTICS AND RESILIENT MODULUS

The key objective of this project is to establish a statistically significant correlation between resilient modulus (M_R) and UCS for chemically treated subgrade soils in Indiana. In order to achieve this objective, different sites in Indiana were identified to collect soil samples for laboratory testing. Site locations included treated subgrade soils ranging from A-1 to A-6. Soil samples were collected during the construction of new roads and road reconstruction projects, when the subgrade soils were accessible. At each site, a representative section of 90 m (300 ft) length was selected for sample collection (Figure 3.1). Eleven locations at 9 m (30 ft) intervals were identified at each site, where two bags of soil (approximately 25 kg each) were collected at each location.

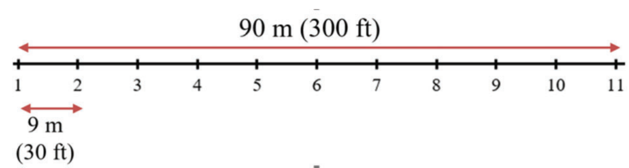


Figure 3.1 Representative section and selected points for sample collection.

3.1 Site Location

These are the sites where untreated subgrade soil was collected: US-31, SR-37, and I-65. The location of the sites is displayed in Figure 3.2.

Site 1: US-31

This site is located on US-31 in St. Joe County, near South Bend, from station 170+00 to 173+00. Untreated soil samples were collected in July 2020 during road construction under Contract No. R-41975. The site has PCCP pavement over a cement stabilized subgrade treated with 4% cement by weight. The water content of the in-situ soil was between 6.0% and 13.1% (average 8.6%).

Site 2: State Road 37

This site is located on State Road 37 (SR-37) in Martinsville. Soil samples were collected from RP 349+08 to 346+08 in July 2020, during construction under Contract No. R-33493. The road has PCCP pavement over a cement stabilized subgrade treated with 5% cement by weight. The water content of the in-situ soil was between 9.8% and 13.9%. A Sand Cone Test was performed near Station 6 and the in-situ soil unit weight was determined to be 1.98 g/cc (123.3 lb/ft^3).

Site: 3 Interstate 65

This site is located on Interstate Road I-65 in West Lafayette, Tippecanoe County. Untreated soil samples were collected in August 2020 on the south-bound section near exit 178 (RP 815+00). The site has a Portland cement concrete pavement (PCCP) over a cement stabilized subgrade (5% cement content by weight). Water content of the in-situ soil was between 6.0% and 13.1% (average 8.6%).

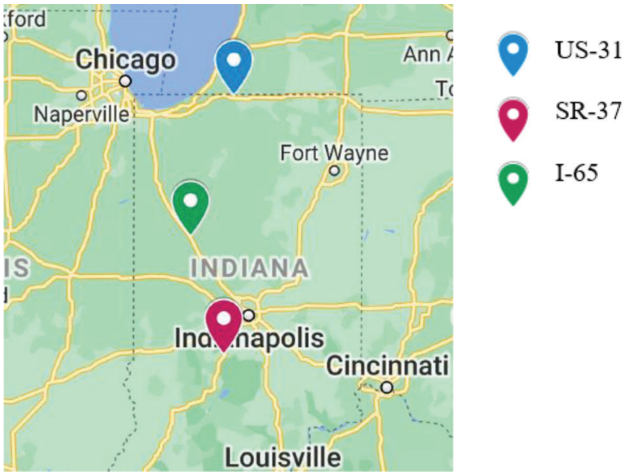


Figure 3.2 Location of the sites for sample collection.

3.2 Laboratory Testing: Soil Characterization

The soil samples collected from the selected sites were tested in the laboratory. The tests included Atterberg limits and grain size analysis, following AASHTO T-89-10 (2011), AASHTO T 90 (2020) and ASTM C 136-14 standards, respectively. The soil from each location was air dried and crushed before using it for the tests. Moist samples were washed through the US Standard No. 200 Sieve (75 Microns) prior to sieve analysis to obtain the grain size distribution. Based on the results, the soils were classified as per AASHTO M 145-91 (2012). Standard Proctor Tests, following AASHTO T 99, (2019) were performed on all the soil samples to obtain the optimum moisture content (OMC) and the maximum dry density (MDD) of the soils. All compaction tests were performed with soils passing US Standard No. Sieve 4 (4.75 mm). The results of all classification tests for the sites included in this study were obtained from Gupta (2022).

3.2.1 Site 1 (US-31) Results

The test results of the samples obtained from Site 1, US-31, indicate considerable variability. Most of the soils are coarse-grained A-1 (A-1-a and A-1-b) except for Samples 4 and 11, which are classified as A-2-4. The soils exhibited low to no plasticity with a small percentage of fines (11% ~ 22%). Figure 3.3 and Figure 3.4 present the grain size distribution and compaction curves, respectively, for all 11 locations of the site. Samples 4 and 11 have different gradation curves than the rest, which is consistent with their different classification (A-2-4). As seen from the results, the soils also exhibited a wide range of MDD values, from 1.83 to 1.99 g/cc (114 lb/ft^3 to 124 lb/ft^3). The results from classification and compaction tests on all eleven samples are summarized in Table 3.1.

3.2.2 Site 2 (SR-37) Results

All soils samples collected from this site show little variation in terms of grain size distribution and compaction. The results obtained from classification and compaction tests performed on all eleven samples are summarized in Table 3.2. All soil samples had a high percentage of sand (50%–60%), with low liquid limit and plastic limit. The gradation and compaction curves also showed uniform results (Figures 3.5 and 3.6). The average OMC and MDD values were found to be 10% and 2.02 g/cc (126.1 lb/ft^3), respectively. Based on the soil properties, the soils were classified as A-2-4 as per AASHTO classification.

3.2.3 Site 3 (I-65) Results

Soil samples collected from the third site (I-65) had considerably high percentage of fines (50%–80%) compared to the previous two sites. Figures 3.7 and 3.8 represent the grain size distribution and compaction test results, respectively, for all the samples. All the soil

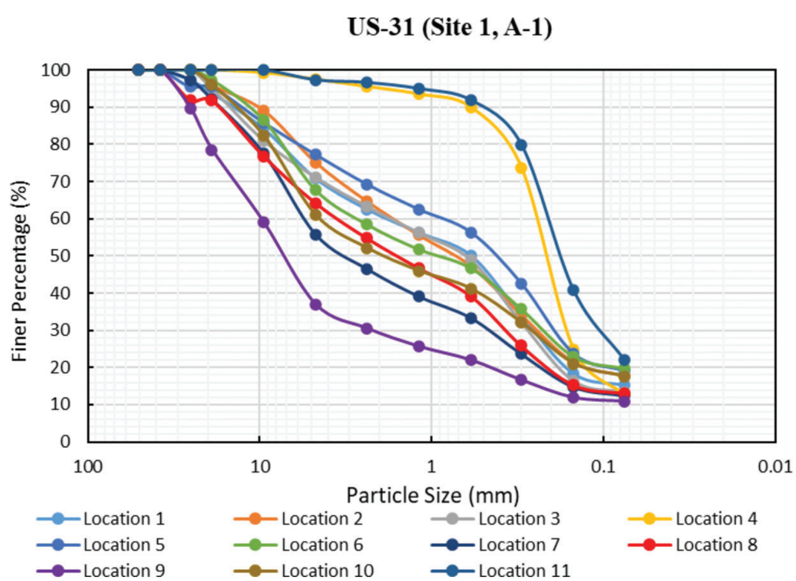


Figure 3.3 Grain size distribution curves for US-31 samples (data from Gupta, 2022).

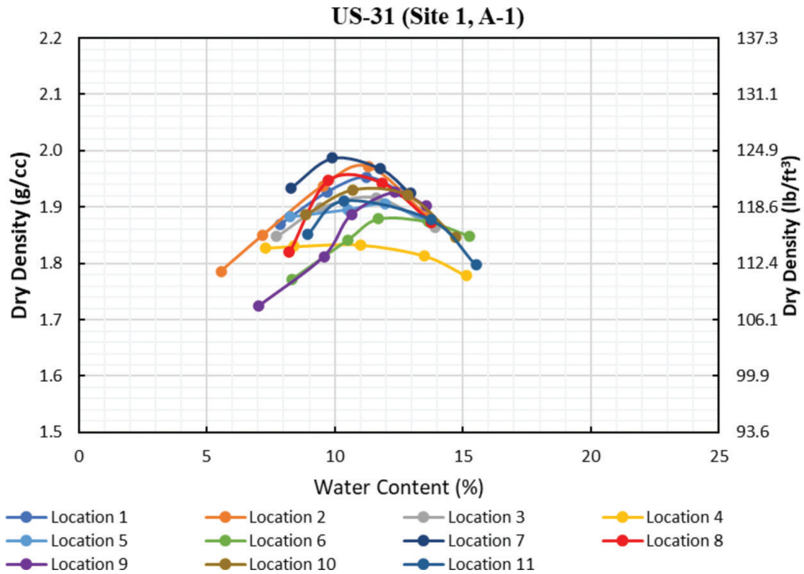


Figure 3.4 Compaction curves for US-31 samples (data from Gupta, 2022).

TABLE 3.1
Soil classification and compaction results for US-31 samples (data from Gupta, 2022)

Sample	LL %	PL %	PI %	% Fines	Classification		OMC %	MDD g/cc (lb/ft^3)
					AASHTO	USCS		
1	21.4	14.7	6.7	15.4	A-1-b	SC-SM	11.2	1.95
2	18.2	NP	NP	17.7	A-1-b	SM	11.0	1.97
3	18.5	NP	NP	13.6	A-1-b	SM	11.2	1.92
4	NP	NP	NP	12.8	A-2-4	SM	11.0	1.83
5	23.3	NP	NP	19.1	A-1-b	SM	12.0	1.91
6	23.6	17.3	6.3	19.8	A-1-b	GC-GM	12.0	1.88
7	19.1	16.0	3.1	12.4	A-1-a	GM	10.2	1.99
8	19.5	13.1	4.4	13.4	A-1-b	SC-SM	10.8	1.96
9	18.3	14.7	3.6	10.9	A-1-a	GM	12.4	1.93
10	25.9	14.4	11.5	17.4	A-1-b	SC	11.6	1.93
11	23.5	NP	NP	22.0	A-2-4	SM	10.4	1.86
	18~26	*13~17*	*3~11*	*11~22*	*A-1*	*SM*	*10~12*	*1.83~1.99*

Note: NP = non-plastic.

TABLE 3.2
Soil classification and compaction results for SR-37 samples (data from Gupta, 2022)

Sample	LL %	PL %	PI %	% Fines	Classification		OMC %	MDD g/cc
					AASHTO	USCS		
1	23.2	12.4	10.8	23.0	A-2-4	SC	10.0	2.01
2	21.4	12.6	8.8	21.5	A-2-4	SC	10.0	2.03
3	23.1	13.7	9.4	21.8	A-2-4	SC	9.7	2.03
4	21.0	12.3	8.7	25.9	A-2-4	SC	9.6	2.01
5	21.0	12.5	8.5	26.9	A-2-4	SC	9.6	2.03
6	21.5	12.7	8.8	29.6	A-2-4	SC	10.4	2.00
7	21.7	12.7	9.0	29.1	A-2-4	SC	9.8	2.01
8	20.1	13.8	6.3	23.5	A-2-4	SC	9.8	2.02
9	20.6	12.5	8.1	30.3	A-2-4	SC	9.8	2.03
10	19.9	12.7	7.2	28.8	A-2-4	SC	9.8	2.03
11	20.4	14.7	5.7	29.0	A-2-4	SC	9.6	2.01
–	*20~23*	*14~22*	*6–11*	*20–30*	*A-2-4*	*SC*	*~10.0*	*~2.02*

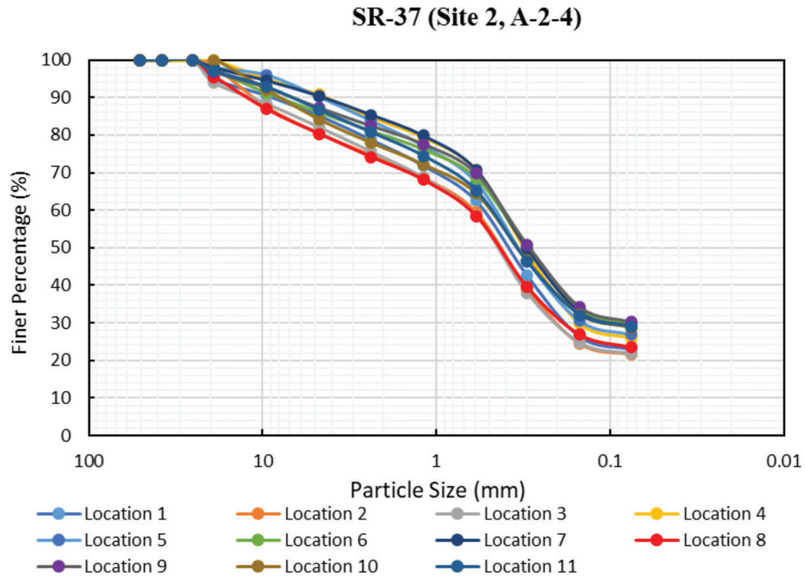


Figure 3.5 Grain size distribution curves for SR-37 samples (data from Gupta, 2022).

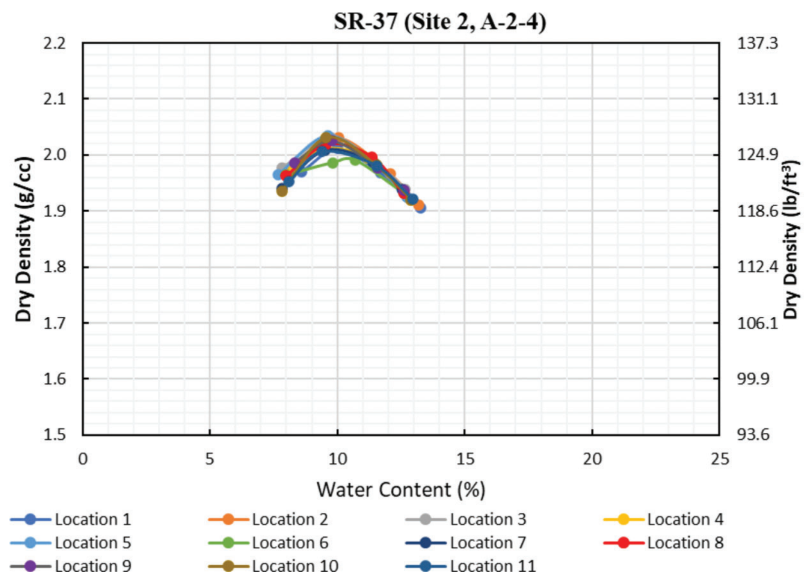


Figure 3.6 Compaction curves for SR-37 samples (data from Gupta, 2022).

samples showed high liquid limit but relatively low plasticity. Except for Sample 1, the rest of the samples had high OMC values (14%–17%). The samples also showed a wide range of MDD varying from 1.71–2.07 g/cc. The results of the classification and compaction tests are summarized in Table 3.3. The soil samples for this site are classified as A-6 (8 out of 11 samples) with three samples being A-4, according to the AASHTO classification.

3.3 Resilient Modulus Tests

The M_R tests were performed on samples at all locations, following AASHTO T 307-99 (2007). The M_R test is essentially a cyclic test designed to simulate real life traffic loading in the laboratory. It is comprised of 16 loading sequences with a combination of five deviatoric (2, 4, 6, 8, and 10 psi) and three confining stresses (2, 4, and 6 psi), including a conditioning sequence. The first sequence consists of a conditioning cycle of 750 repetitions to ensure proper contact between the specimen and the loading cap, and to remove any effects of initial loading versus reloading. All other sequences involve 100 cycles of loading and reloading. The average resilient modulus obtained for the last five cycles is reported for each sequence. Each test results in 15 resilient modulus values corresponding to each deviatoric and confining stress. For design purposes, often the resilient modulus value at 6 psi deviatoric stress and 2 psi confining stress is used, as this best represents the loading of a single axle wheel load.

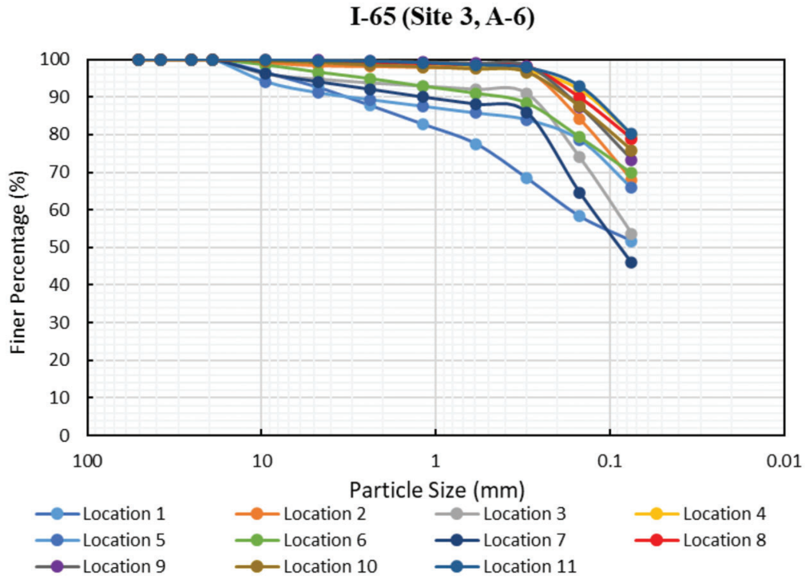


Figure 3.7 Grain size distribution curves for I-65 samples (data from Gupta, 2022).

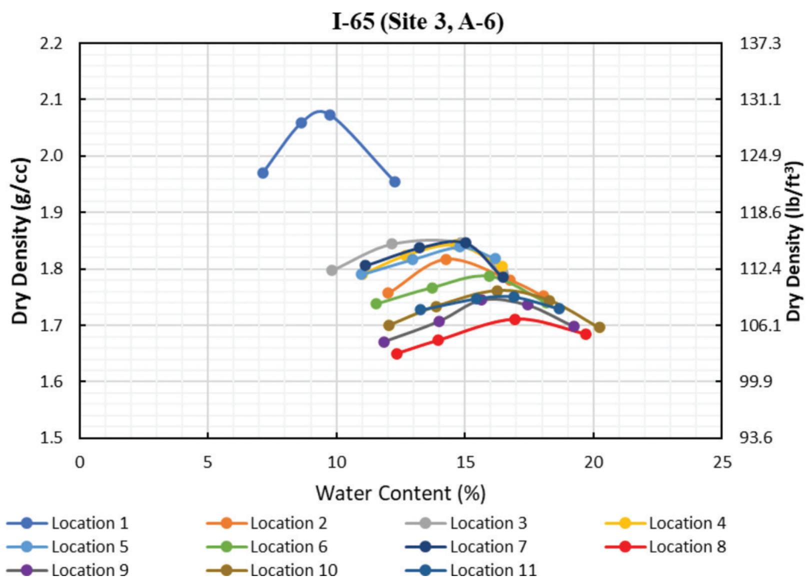


Figure 3.8 Compaction curves for I-65 samples (data from Gupta, 2022).

Specimens are prepared for the tests at each location following a modified double plunger method specified in Annex C of AASHTO T 307-99 (2007) standard for type 2 subgrade materials (fine-grained soils). The method involves a split mold, spacer disks and a hand press used for static compaction. Remolded specimens are compacted in five layers using spacers of different thickness to ensure all layers have equal volume. After compaction, three measurements of height and diameter are taken to obtain the average volume of the sample. The specimens prepared for testing are approximately 2.8 in. (71 mm) in diameter and 5.6 in. (142 mm) in height (2:1 height to diameter ratio). The mass and density of the specimens is also obtained.

For all locations, both untreated and cement-treated M_R test specimens were prepared at OMC and MDD values corresponding to the standard proctor test results of the untreated samples. The cement treated samples were cured for 28 days before testing. Curing involved carefully wrapping the samples with cling film and storing them in a cooler for the curing period to ensure minimal loss of moisture content. Control specimens of cement treated, and untreated specimens were also prepared to determine changes in water content during the curing period. The water content of the treated specimens was measured just after specimen preparation as well as after performing the M_R test. The loss of water content of the treated samples after

TABLE 3.3
Soil classification and compaction results for I-65 samples (data from Gupta, 2022)

					Classification			
Sample	**LL %**	**PL %**	**PI %**	**% Fines**	**AASHTO**	**USCS**	**OMC %**	**MDD g/cc**
1	21.5	11.5	10.1	51.7	A-4	CL+ML	9.5	2.07
2	28.5	16.0	12.5	67.9	A-6	CL	14.3	1.82
3	25.4	16.3	9.1	53.7	A-4	CL+ML	14.0	1.84
4	30.1	16.8	13.3	80.1	A-6	CL	14.7	1.84
5	31.4	16.6	14.8	65.9	A-6	CL	14.8	1.84
6	32.7	18.7	13.9	69.8	A-6	CL	16.0	1.79
7	24.1	15.1	9.0	46.0	A-4	CL+ML	15.0	1.85
8	36.2	19.7	16.5	78.9	A-6	CL	17.0	1.71
9	31.0	17.9	13.1	73.3	A-6	CL	16.0	1.75
10	35.1	17.6	17.4	75.8	A-6	CL	16.5	1.76
11	35.3	18.7	16.6	80.2	A-6	CL	16.5	1.75
–	*21~35*	*11~20*	*9–17*	*50~80*	*A-6*	*CL*	*9.5~16.5*	*1.71~2.07*

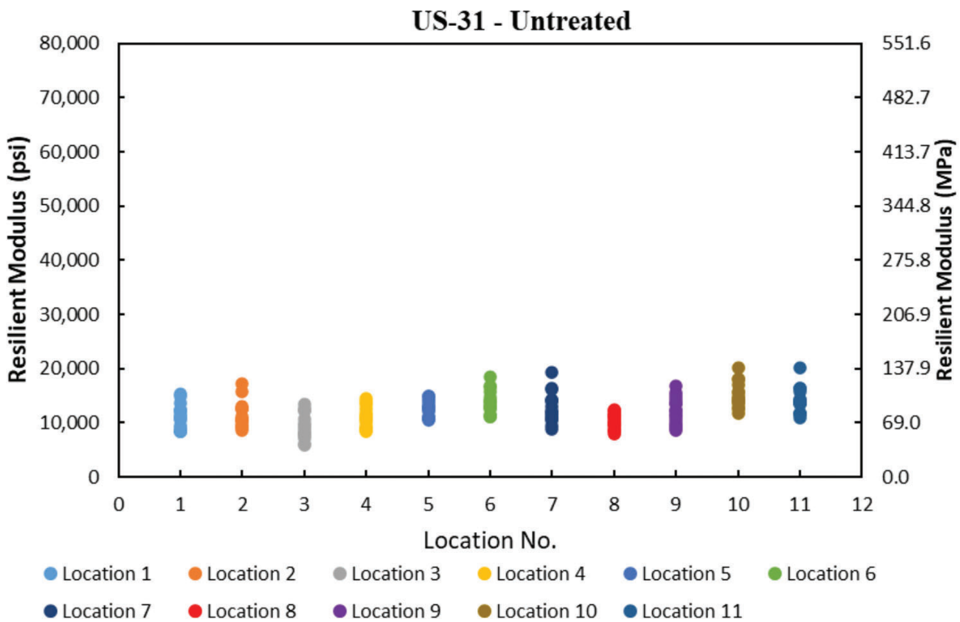


Figure 3.9 Summary of resilient modulus results for untreated samples—US-31 (data from Gupta, 2022).

curing was about 1%, while that of the untreated specimen showed negligible change in water content. The decrease in water content for the treated specimens is attributed to the chemical reaction between the soil and cement during the curing process. The data of M_R tests for all sites included in this study was obtained from Gupta (2022).

3.3.1 US-31 Results

Resilient Modulus Tests were performed on treated as well as on untreated soils at all 11 locations for Site 1 (US-31). The soil specimens were compacted at MDD values obtained from the Standard Proctor Tests on untreated samples. The relative compaction for all samples was found to be between 97% to 99%. The treated specimens were prepared with 4% cement, by weight, mixed with the natural soil and cured for 28 days. Little variations in M_R values, for untreated specimens, were found for all the soils at the site; however, treated specimens exhibited larger differences. For untreated specimens, the M_R, ranged from 40 MPa–140 MPa (5,800 psi–20,200 psi), while for treated specimens, the resilient modulus was three times higher, and ranged from 120 MPa–470 MPa (18,000 psi–68,000 psi). Figure 3.9 and Figure 3.10 provide the M_R test results for all untreated and treated samples, respectively. The variability observed in the treated specimens could be due to differences in gradation and plasticity of the soils. The resilient modulus of the treated specimens showed a slight dependency on confining stress, but little to none on deviatoric stress.

3.3.2 SR-37 Results

For the SR-37 site, M_R tests were performed on treated and untreated specimens, which were compacted at the MDD and OMC of the untreated soil. The relative compaction for all samples was found to be, on average, 98%. The treated specimens were prepared with 5% cement mixed, by weight, and cured for 28 days. The range of M_R values of the untreated specimens were between 48 MPa and 190 MPa (7,000 psi to 28,000 psi) and of the treated specimens, in the range of 170 MPa to 520 MPa (25,000 psi to 75,000 psi). Figures 3.11 and 3.12 are plots of the resilient modulus for all untreated and treated samples. The effects of confining stress were observed in the treated as well as in the untreated specimens, with treated specimens exhibiting a large decrease of resilient modulus with increasing confining stress (indicated by the larger spread of values in Figure 3.12). The resilient modulus results are fairly uniform, which indicate uniformity of the soil across all stations. This is expected due to the small differences in soil characteristics at the site (Table 3.2).

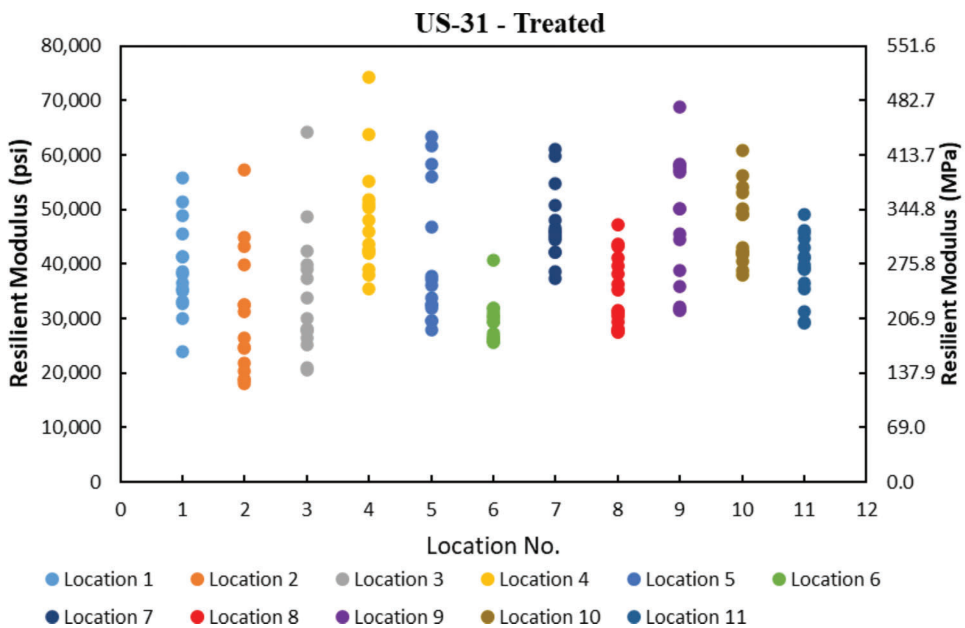


Figure 3.10 Summary of resilient modulus results for treated samples—US-31 (data from Gupta, 2022).

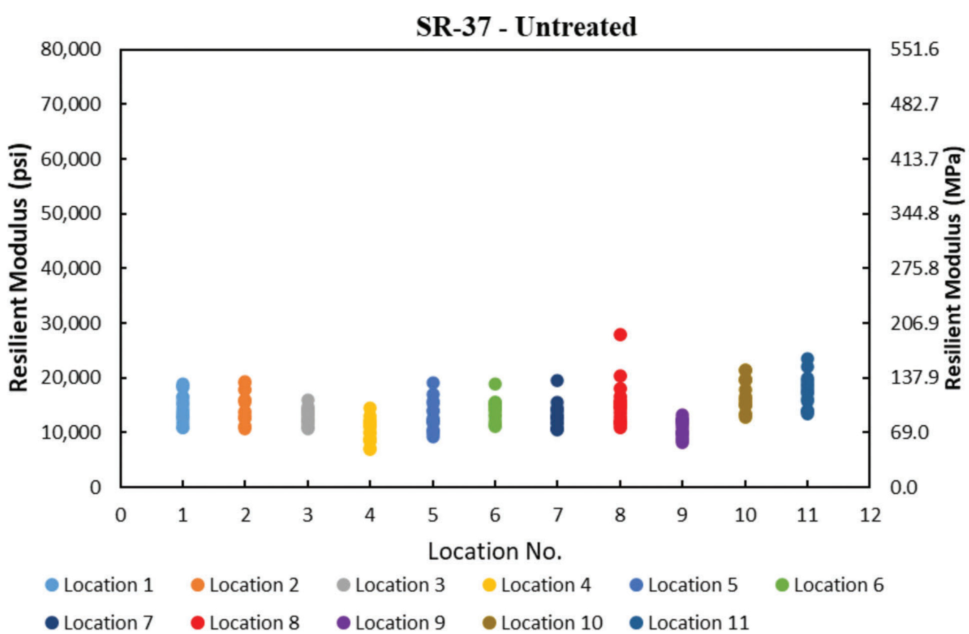


Figure 3.11 Summary of resilient modulus results for untreated samples—SR-37 (data from Gupta, 2022).

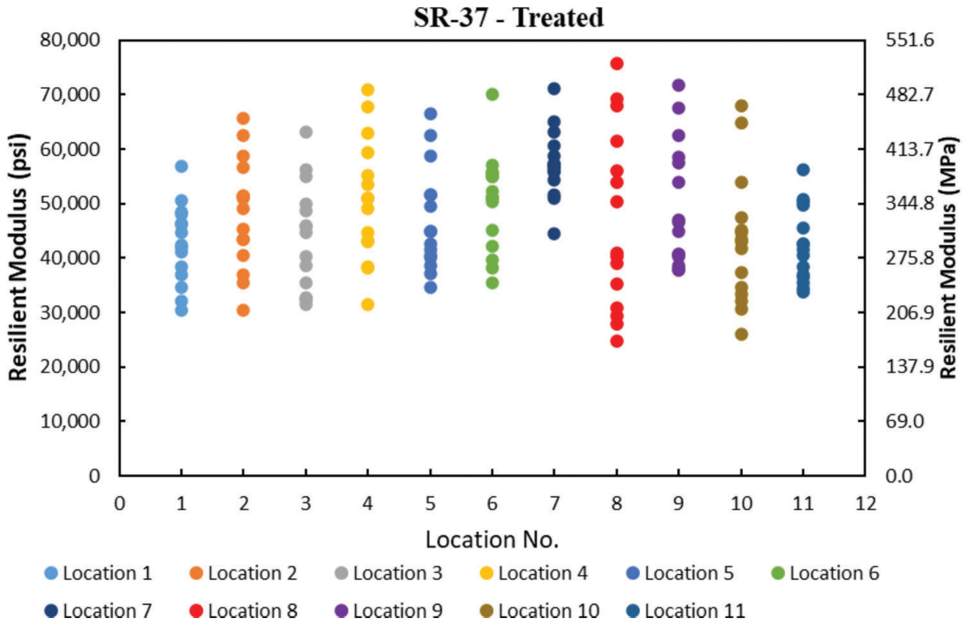


Figure 3.12 Summary of resilient modulus results for treated samples—SR-37 (data from Gupta, 2022).

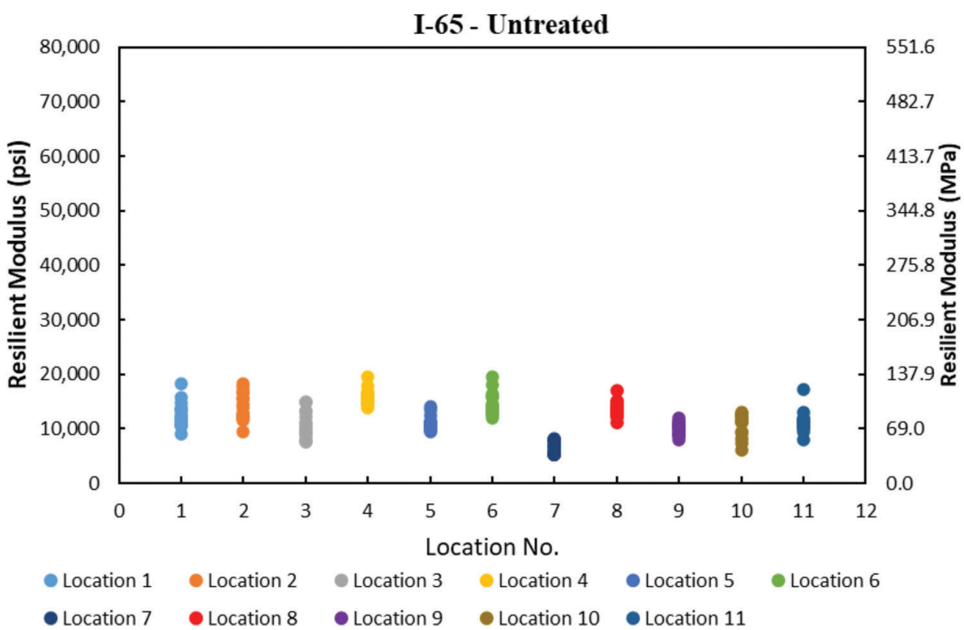


Figure 3.13 Summary of resilient modulus results for untreated samples—I-65 (data from Gupta, 2022).

3.3.3 I-65 Results

At the I-65 site, as with the other sites, M_R tests were performed on treated and untreated specimens. All specimens were compacted at the MDD, and OMC of the Standard Proctor Tests conducted on untreated soil. The relative compaction for all samples was found to be on average between 97% to 99%. The treated specimens were prepared with 5% cement mixed, by weight, with the natural soil and cured for 28 days. The M_R values of the untreated specimens were between 34 MPa and 135 MPa (5,000 psi to 20,000 psi) and of the treated specimens, 82 MPa to 530 MPa (12,000 psi to 77,000 psi). Figures 3.13 and 3.14 show the resilient modulus of all untreated and treated samples. The effect of confining stress was pronounced in the treated specimens, with values decreasing with decreasing confining. The untreated samples did not exhibit much variation with respect to confining and deviatoric stresses, as indicated by the narrow range of values displayed in Figure 3.13. The variability observed in the resilient modulus values of the treated specimens could be due the wide range of soil properties observed at the site (see Table 3.3).

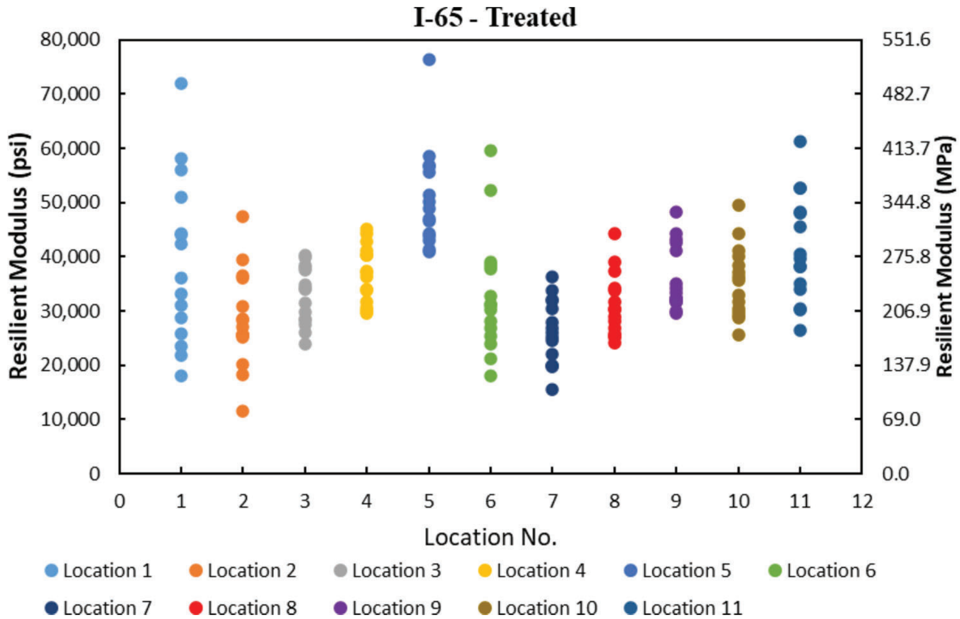


Figure 3.14 Summary of resilient modulus results for treated samples—I-65 (data from Gupta, 2022).

4. UNCONFINED COMPRESSIVE STRENGTH TESTS

The UCS test is the primary method used to determine the strength of the subgrade. UCS tests were performed on reconstituted treated samples at all locations, as per AASHTO T 208-15 (2019). Three tests were performed at each location for all sites to ensure repeatability of results. The sample preparation procedure was kept similar to that for the M_R tests (compacted in five layers of equal volume using the double plunger method) to ensure compatibility of results. The specimens prepared were approximately 2.8 in. (71 mm) in diameter and 5.6 in. (142 mm) in height (2:1 height to diameter ratio).

The cement-treated specimens were prepared at OMC and MDD values corresponding to the standard proctor test results on untreated samples. The samples were cured for 7 days before testing. Curing involved carefully wrapping the samples with cling film and storing them in a cooler for the curing period to ensure minimal loss of moisture content. The water content of the treated specimens was measured just after specimen preparation, as well as after performing the UCS test. The loss of water content of the treated samples after curing was on average about 0.8%–1% which is expected because of the chemical reaction between cement and soil.

The specimens were tested at a 1% strain rate (0.056 in./min or 0.72 mm/min) and were loaded to failure. The output of the tests was the stress-strain response from which the peak value was identified as the UCS. Additional UCS tests were done on specimens used for the M_R tests. The UCS test, in these cases, was conducted at the end of the M_R tests. Note that these tests were done on specimens 28-days old. Figure 4.1 includes photographs of representative samples loaded to failure.

4.1.1 US-31 Results

Three UCS tests were performed on treated soils at each location. The soil specimens were compacted at the OMC and MDD values obtained from Standard Proctor Tests on untreated samples. The relative compaction for all samples was found to in the range of 97% to 99%. The treated specimens were prepared with 4% cement, by weight, mixed with the natural soil and cured for 7 days. The water content was recorded at the time of sample preparation and after performing the UCS tests. The water content was found to decrease by 1%, on average, among all samples. Figure A.1 in Appendix A displays the stress-strain plots for UCS tests performed on cement-treated specimens at all

Figure 4.1 Samples post UCS tests.

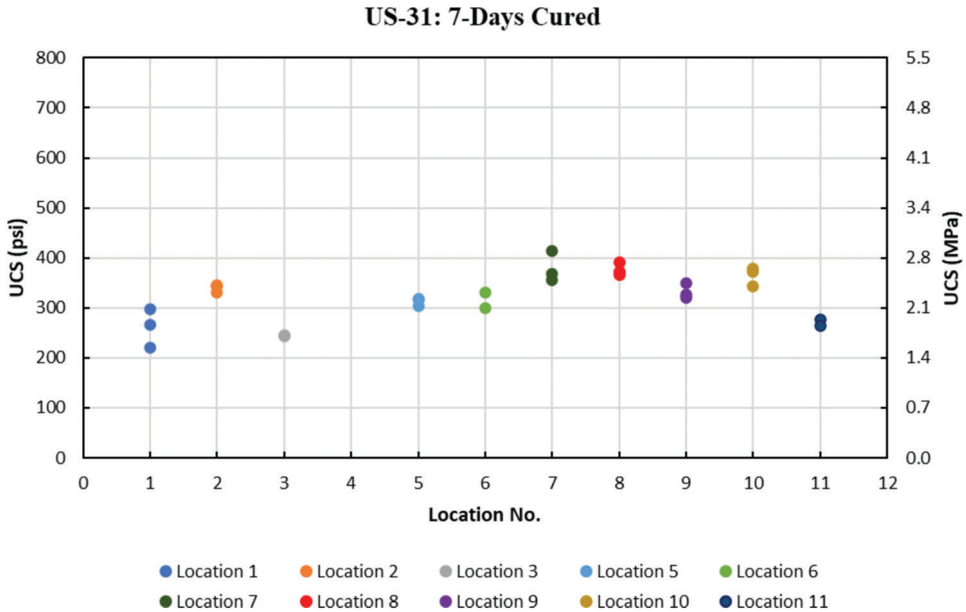


Figure 4.2 Summary of 7-days cured UCS values for all locations at US-31 site.

locations (except at Location No. 4, as the samples were found to have cracks after curing). Figure 4.2 summarizes all the UCS tests. The UCS values range from 1.5 MPa to 2.9 MPa (220 psi to 415 psi). The variability in the UCS results can be explained by the differences in gradation and plasticity in the soils found at this site (see Table 2.1). The strain at failure is also considerably lower compared to the untreated subgrade soils, as reported in the literature (Hossain et al., 2011; Lee et al., 1997; Sandoval et al., 2019) and ranges between 0.8 to 1.2%.

UCS tests were also performed on 28-days cured cement-treated samples. These tests were done on the same specimens used for M_R testing. Figure 4.3 is a plot of the UCS test results for the 28-days cured samples. The results range from 1.75 MPa to 3.2 MPa (255 psi to 470 psi) and are on average about 1.2 times larger than the UCS at 7-days.

4.1.2 SR-37 Results

Three UCS tests were performed on treated soils at each location at the SR-37 site. The soil specimens were compacted at the OMC and MDD values obtained from the Standard Proctor Tests on untreated samples. The relative compaction for all the prepared samples was found to be on average 98%. The treated specimens were prepared with 5% cement, by weight, mixed with the natural soil and cured for 7 days.

Figure A.2 in Appendix A displays the stress-strain plots of UCS tests performed on cement-treated specimens at all locations. The plots include a large number of data points because the tests were performed with the new UCS load frame (Humboldt Master Loader), capable of recording data at frequent intervals. A summary of the UCS test results is presented in Figure 4.4. The majority of UCS values lie in the range of 2.4 MPa to 3.1 MPa (350 psi to 450 psi). The similarity of the stress-strain plots and the narrow range of UCS values can be explained by the uniform nature of the soils found at this site (see Table 3.2).

UCS tests were also performed on 28-days cured cement-treated samples at all 11 locations. These tests were done on the same sample after the M_R test. Figure 4.5 displays the UCS test results for the 28-days cured samples. The UCS values range from 2.8 MPa to 4 MPa (410 psi to 575 psi) and are, on average, about 1.4 times the UCS at 7-days.

4.1.3 I-65 Test Results

Three UCS tests were performed on treated soils at each location, at the I-65 site. The soil specimens were compacted at the OMC and MDD values obtained from the Standard Proctor Tests on untreated samples. The relative compaction for the prepared samples was found to be on average 97% to 98%. The treated specimens were prepared with 5% cement, by weight, mixed with the natural soil and cured for 7 days. Figure A.3 in Appendix A shows the results of 7-days cured specimens tested at all locations of I-65 while a summary of UCS test results done at the site is presented in Figure 4.6. The majority of the UCS values for this site lie in the range of 180 psi to 290 psi (1.2 to 2 MPa) (except for Location 1). The UCS value at Location 1 is distinctly higher compared to the rest due to a considerably lower OMC value and higher MDD at this location (see Table 3.2).

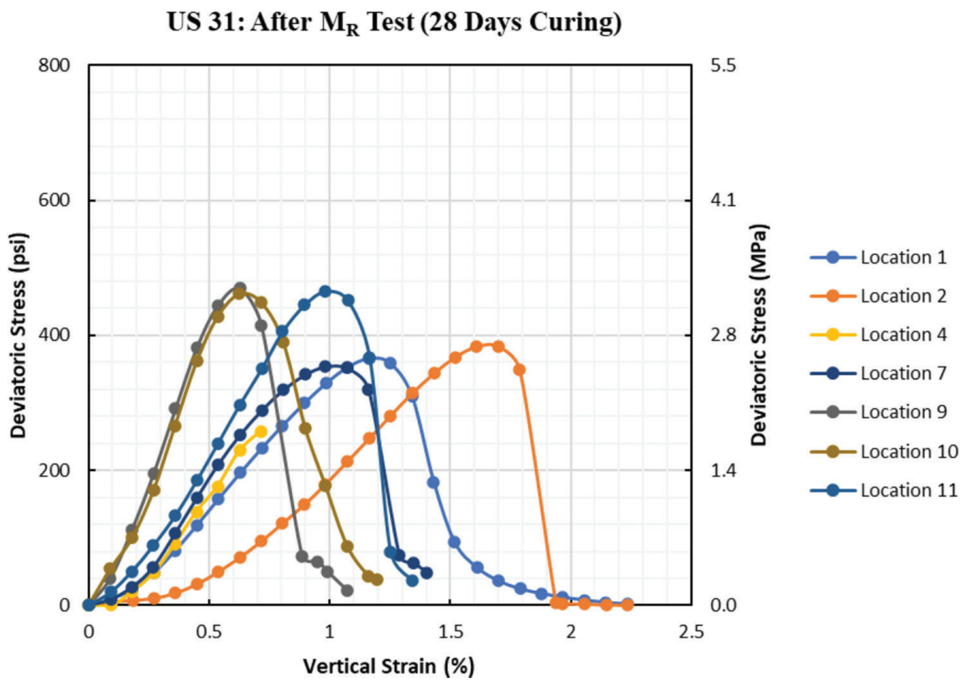


Figure 4.3 UCS test results for the 28-days cured samples performed after M_R test—US-31.

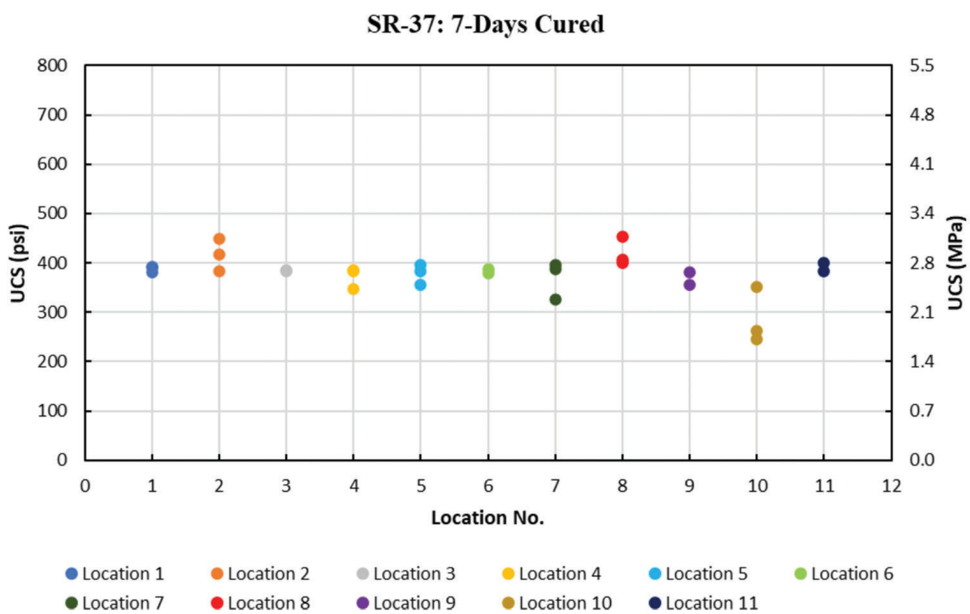


Figure 4.4 Summary of 7-days cured UCS values for all locations at SR-37 site.

Figure 4.7 includes plots of the UCS test results for the 28-days cured samples. The UCS values range from 1.7 MPa to 5.1 MPa (240 psi to 745 psi). The UCS at Location 1 is distinctly higher than the rest, which is most likely due to the high MDD and low OMC of the soil (see Table 3.3). The UCS values at 28-days curing are on average 1.4 to 1.5 times the UCS values at 7-days curing for this site, consistent with the results from the other sites.

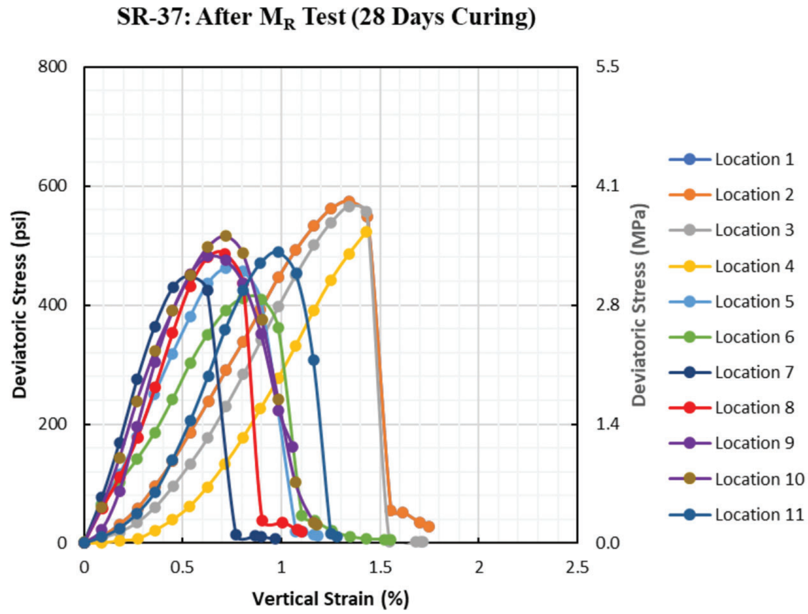


Figure 4.5 UCS test results for the 28-days cured samples performed after M_R test—SR-37.

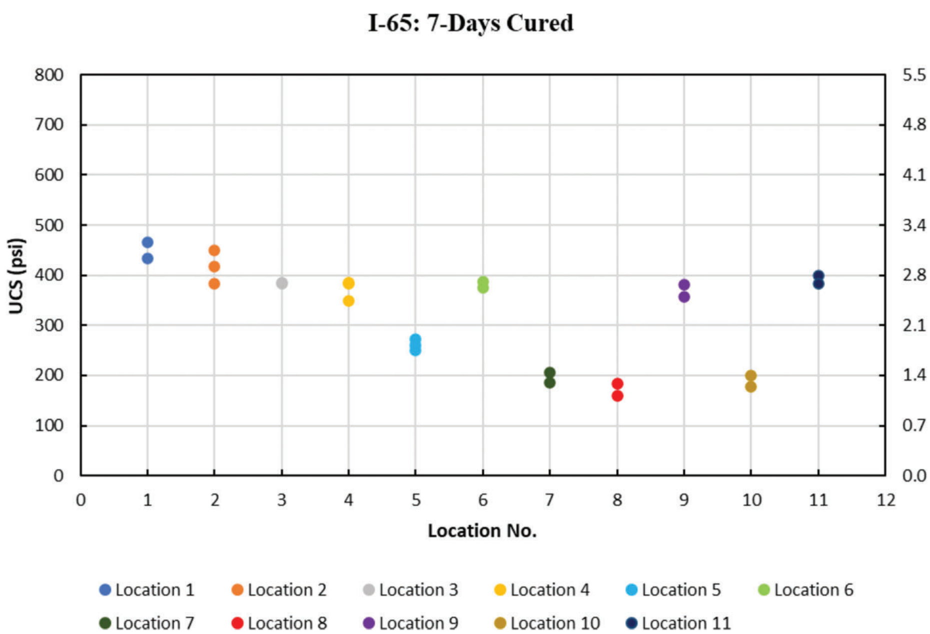


Figure 4.6 Summary of 7-days cured UCS values for all locations at I-65 site.

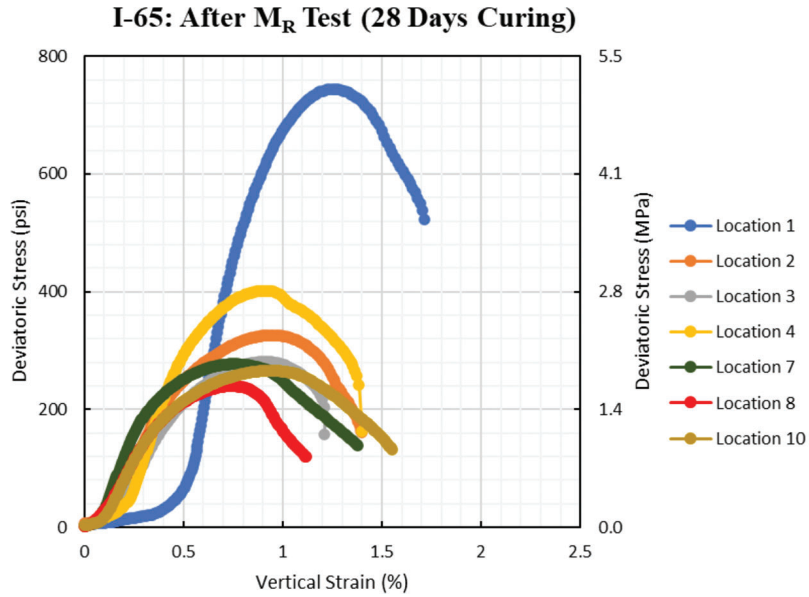


Figure 4.7 UCS test results for the 28-days cured samples performed after M_R test—I-65.

5. CORRELATIONS BETWEEN RESILIENT MODULUS AND UCS

The discussion and data presented in Chapter 2 (literature review) suggests that it is unlikely to get a statistically significant or direct correlation between the resilient modulus and UCS parameters in the case of treated soils. The approach in this study is to try to establish a range of values for the resilient modulus and UCS test results across the three sites used for this project. Data representing the resilient modulus and UCS test results were taken to represent the strength-stiffness test results. The resilient modulus values corresponding to 6 psi deviatoric stress, and 2 psi (low confinement) and 6 psi (high confinement) confining stress were selected from all the tests. For the UCS tests, the data selected were the average UCS values at the 7-days and 28-days cured samples.

Figures 5.1 and 5.2 display the plots of resilient modulus versus 7-day and 28-day cured UCS, respectively, for all the sites. As evident from the plots there is a wide range of values in terms of both resilient modulus and UCS results for the soils used in this project. The degree of scatter in the plots also indicates that there is not a direct statistically significant correlation between the resilient modulus and UCS for the treated soils considered in this study.

An alternate way to plot the resilient modulus and UCS data for all the sites is to sort them with respect to the type of soil. The MEPDG for pavement design relies on the use of the resilient modulus of the subgrade at 2 psi confinement and 6 psi deviatoric stress. A plot combining the resilient modulus, UCS and soil type data can prove useful for estimating the expected range of values of the resilient modulus. Figures 5.3 and 5.4 are plots of the resilient modulus (6 psi deviatoric stress and 2 psi confinement) versus the 7-day and 28-day UCS, respectively, including the type of soil.

From Figure 5.3 and Figure 5.4 it can be observed that the expected range of resilient modulus values for cement-treated A-2-4, A-4 and A-6 soils can be estimated from the 7-day or 28-day cured UCS values with a fair degree of confidence. The majority of resilient modulus values fall in the range of 30,000 psi to 40,000 psi (210 to 275 MPa) for cement-treated A-2-4 soils. Most of the resilient modulus values for cement-treated A-4 and A-6 soils (fine-grained/clayey soils) fall in the range of 20,000 psi to 35,000 psi (135 to 240 MPa). The resilient modulus values of A-1 soils exhibit a large degree of scatter. The reason may be the non-uniform properties of the soils at US-31, i.e., different Atterberg limits, OMC and MDD; see Table 3.1. This calls for including all soil properties, in addition to OMC and MDD, to improve estimates of resilient modulus values.

A statistical analysis was conducted to build a correlation model for the dataset. A multi-variate linear regression was performed that included the following parameters: resilient modulus, UCS (7-days and 28-days cured), percentage of fines, plasticity index, optimum moisture content (OMC) and maximum dry density (MDD). The values of the resilient modulus and UCS were normalized for the analysis. The normalization utilized is expressed by Equation 5.1(a) and Equation 5.1(b).

$$Normalized\ M_R = \frac{M_R - \min(M_R)}{\max(M_R) - \min(M_R)} \quad \text{(Eq. 5.1a)}$$

$$Normalized\ UCS = \frac{UCS - \min(UCS)}{\max(UCS) - \min(UCS)} \quad \text{(Eq. 5.1b)}$$

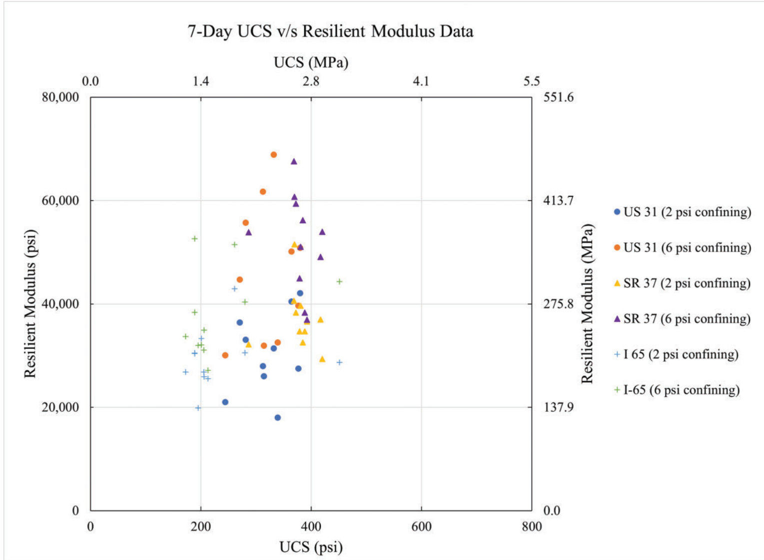


Figure 5.1 Resilient modulus versus 7-days cured UCS results for all sites.

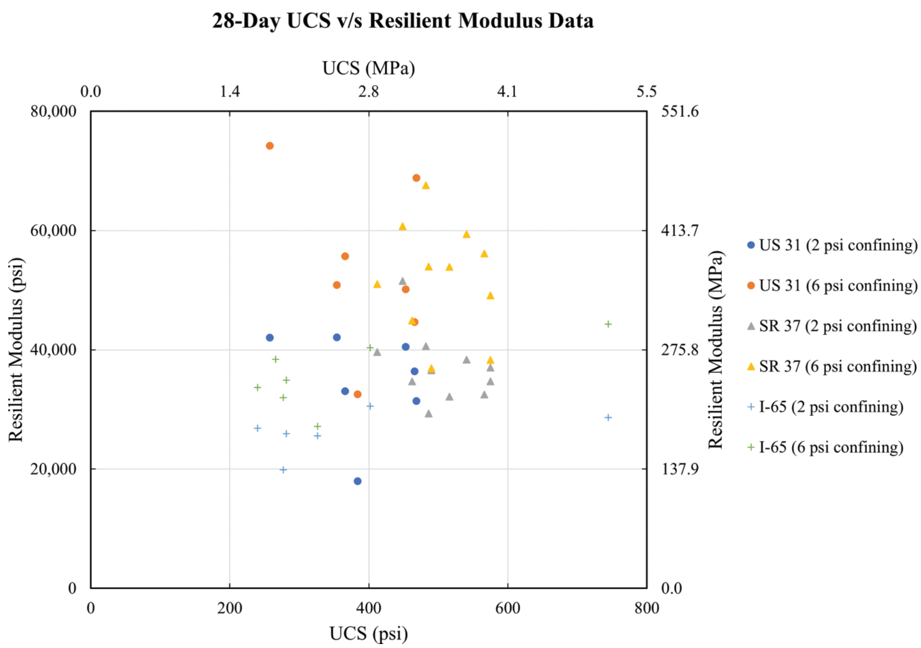


Figure 5.2 Resilient modulus versus 28-days cured UCS results for all sites.

Where,

Min M_R, Max M_R: minimum and maximum resilient modulus values across all sites, respectively.

Min UCS, Max UCS: minimum and maximum UCS values across all sites, respectively.

The generalized model utilized for the analysis is presented by Equation 5.2.

$$Normalized\ M_R = X_1 * (\%\ of\ fines) + X_2 * (PI) + X_3 * (OMC) + X_4 * (MDD) + X_5 * (Normalized\ UCS) \quad \text{(Eq. 5.2)}$$

Where,

X1 – X5: coefficients (estimated from the linear regression analysis),

PI: plasticity index (%),

OMC: optimal moisture content (%), and

MDD: maximum dry density (g/cc).

The coefficients X1-X5 for the 28-days cured UCS values are presented in Table 5.1.

The p-value represents the significance of the variable with respect to the regression analysis. If the p-value is less than a certain significance level, then

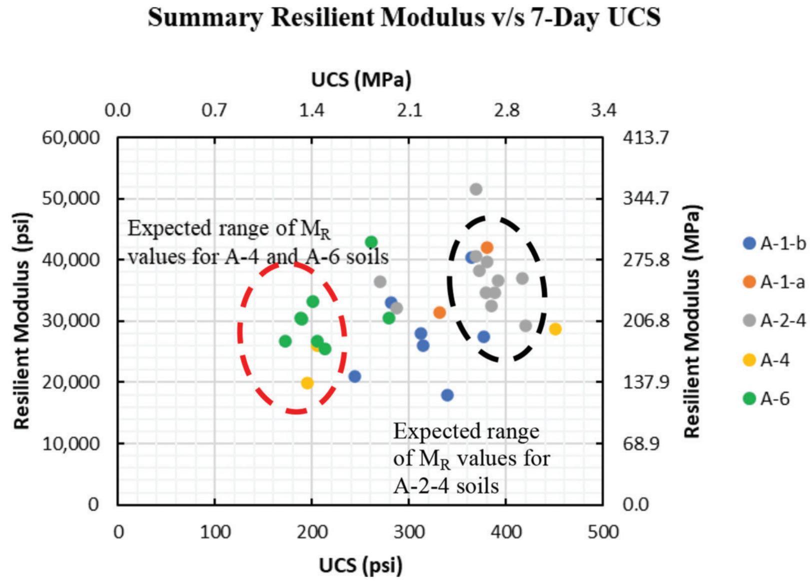


Figure 5.3 Resilient modulus versus 7-days cured UCS results with soil type.

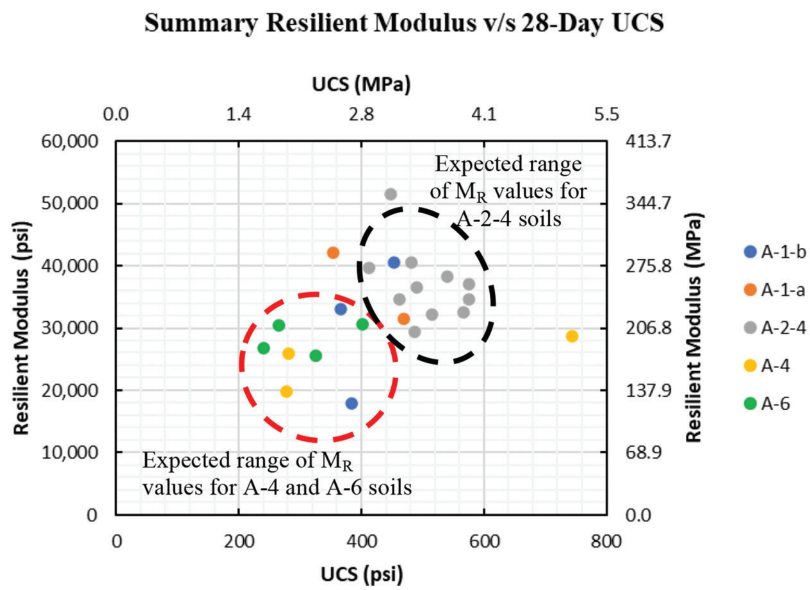


Figure 5.4 Resilient modulus versus 28-days cured UCS results with soil type.

the parameter has a statistically significant relationship with the response variable (in our case the resilient modulus). The results in Table 5.1 show that the p-value of the percentage of fines is the highest, meaning it is the least statistically significant parameter in the model (text in red). Because of that, a new trial was attempted without the percentage of fines. The new model used for analysis is expressed by Equation 5.3.

$$Normalized\ M_R = X_1 * (PI) + X_2 * (OMC) + X_3 * (MDD) + X_4 * (Normalized\ UCS) \quad \text{(Eq. 5.3)}$$

The results of the model, using the 28-days cured UCS values, are presented in Table 5.2. The table shows that the MDD and OMC parameters are the most statistically significant (lowest p – values are in green).

The predicted resilient modulus values, using the regression analysis with the values in Table 5.2, are then compared with the measured resilient modulus values, to verify the accuracy of the model. Figure 5.5 plots the predicted and the measured resilient modulus values for the all the soils tested. The model does a fairly good job of predicting the resilient modulus with a multi-variate $R^2 = 0.85$.

Table 5.3 presents the results of the statistical analysis following the same process discussed, but for the 7-days cured UCS values. The table shows that the PI and OMC are the most statistically significant parameters (lowest p – values) for the model.

TABLE 5.1
Linear regression analysis results for correlation between resilient modulus and 28-days cured UCS including all five parameters

Parameter	Coefficients	Estimate	Std. Error	t-value	p-value
% of Fines	X_1	-0.00236	0.003849	-0.612	0.54774
PI	X_2	0.038135	0.020857	1.828	0.08323
OMC	X_3	-0.0756	0.037209	-2.032	0.0564
MDD	X_4	0.632981	0.219603	2.882	0.00954
28-Day Normalized UCS	X_5	-0.44307	0.323901	-1.368	0.1873

Note: Red numbers signify least significant parameter.

TABLE 5.2
Linear regression analysis results for correlation between resilient modulus and 28-days cured UCS

Parameter	Coefficients	Estimate	Std. Error	t-value	p-value
PI	X_1	0.03159	0.01763	1.792	0.08824
OMC	X_2	-0.0867	0.03198	-2.711	0.01345
MDD	X_3	0.69546	0.19137	3.634	0.00165
28-Day Normalized UCS	X_4	-0.48531	0.31148	-1.558	0.1349

Note: Green numbers signify most significant parameter.

TABLE 5.3
Linear regression analysis results for correlation between resilient modulus and 7-days cured UCS

Parameter	Coefficients	Estimate	Std. Error	t-value	p-value
PI	X_1	-0.03987	0.01305	-3.056	0.00489
OMC	X_2	0.06429	0.02967	2.167	0.0389
MDD	X_3	0.05832	0.10203	0.572	0.57215
7-Day Normalized UCS	X_4	0.12174	0.23005	0.529	0.60083

Note: Green numbers signify most significant parameter.

Similar to what was done with the 28 days cured UCS values in Figure 5.5, Figure 5.6 plots the predicted resilient modulus using the 7-days cured UCS and provides a comparison with the measured resilient modulus. The model for the 7-days does perform reasonably well, with a multi-variate $R^2 = 0.91$.

It should be noted that although these simple linear regression models do a reasonable job relating stiffness with strength, the dataset used, albeit of high quality, is limited. Thus, the applicability of the model should be limited to the range of soils used in the study. The accuracy of the model should be tested with a large number of cases taken from different sites.

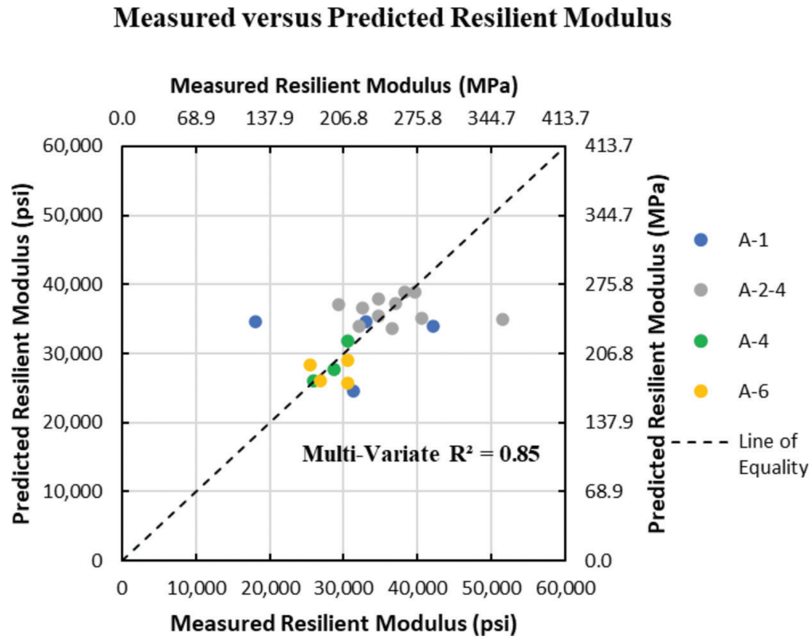


Figure 5.5 Measured versus predicted resilient modulus from 28-days cured UCS.

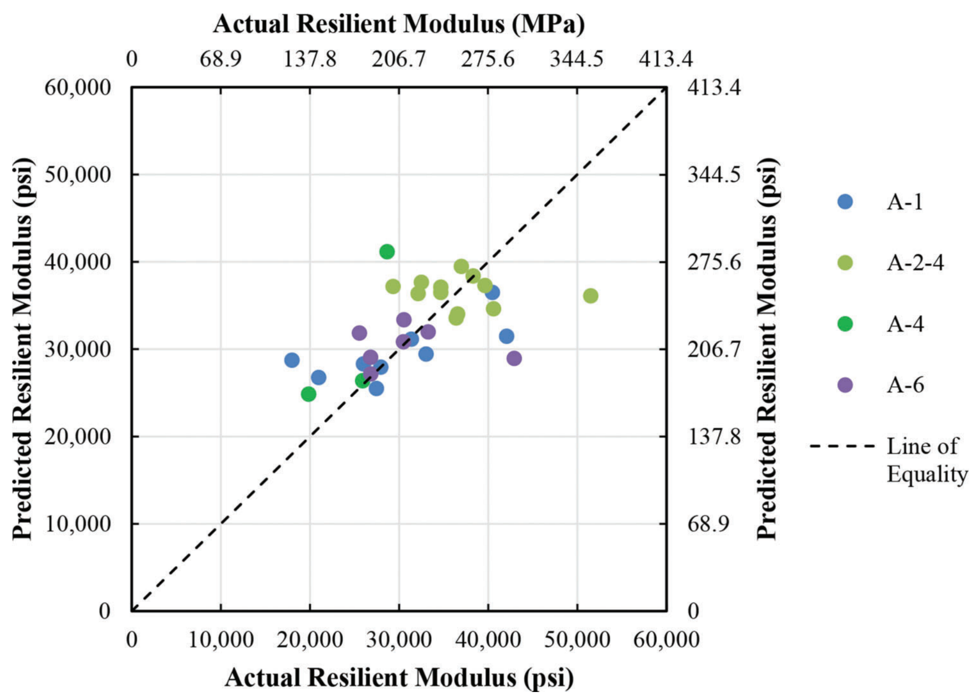


Figure 5.6 Measured versus predicted resilient modulus from 7-days cured UCS.

6. HISTORY OF RESILIENT MODULUS TEST

Early pavement thickness design was based on experience, on the type of subgrade soil, and the plastic response of pavement materials to static load. California Bearing Ratio (CBR) and Resistance Value (R value) Tests were used to assess the quality of subgrade materials. Hveem (1955) realized that one of the main causes for failure in flexible pavements was excessive rutting or cracking due to fatigue and that there was a need to develop a testing procedure that better simulated the traffic loading conditions. He also coined the term *resilience* to define the recoverable deformations observed in soils subjected to repeated load during testing. The equipment used by Hveem, a resiliometer, could measure the compression and rebound of soil specimens by measuring volumetric displacement under repeated dynamic loads. However, it provided limited information in terms of stiffness of the soil under cyclic loading. Figure 6.1 gives one example of resiliometer test results performed on an expansive soil, which shows an increase in resilience of the soils with increasing moisture content.

Pavement materials are subjected to cyclic stresses of varying magnitude and duration depending on the axle load, speed, and frequency of passing vehicles. Following the work by Hveem (1955), a very extensive testing program was carried out by Seed and coworkers (Seed & Chan, 1958; Seed & Fead, 1960; Seed & McNeil, 1956; Seed et al., 1955; Seed et al., 1958; Seed et al., 1960; Seed et al., 1962; Seed et al., 1967) to study the strength and deformation response of soils under repeated loading conditions. Seed and Chan (1958) and Seed et al. (1958) performed repeated load tests on partially saturated silty clay soil specimens and studied the effect of stress change on deformation by subjecting specimens to a large number of deviatoric stress cycles. They observed that axial strains developed in soil were a function of stress history and a specimen subjected first to a series of low deviatoric stresses followed by higher stress cycles had lower axial strains, when compared to the same specimen subjected only to high deviatoric stress cycles (Figure 6.2). This suggested that the stress history produced a stiffening effect on the

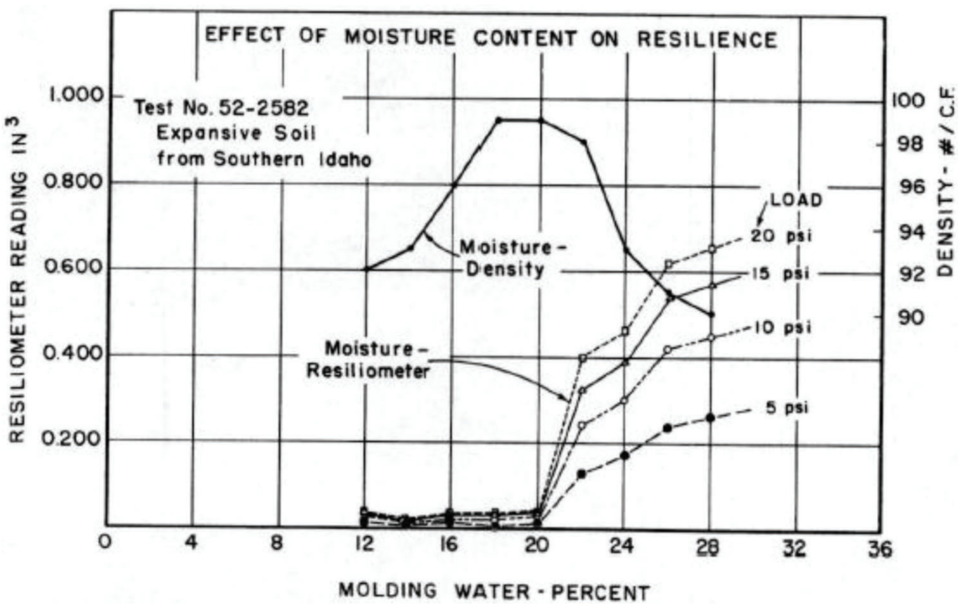


Figure 6.1 Effect of moisture content on resilience (from Hveem, 1955).

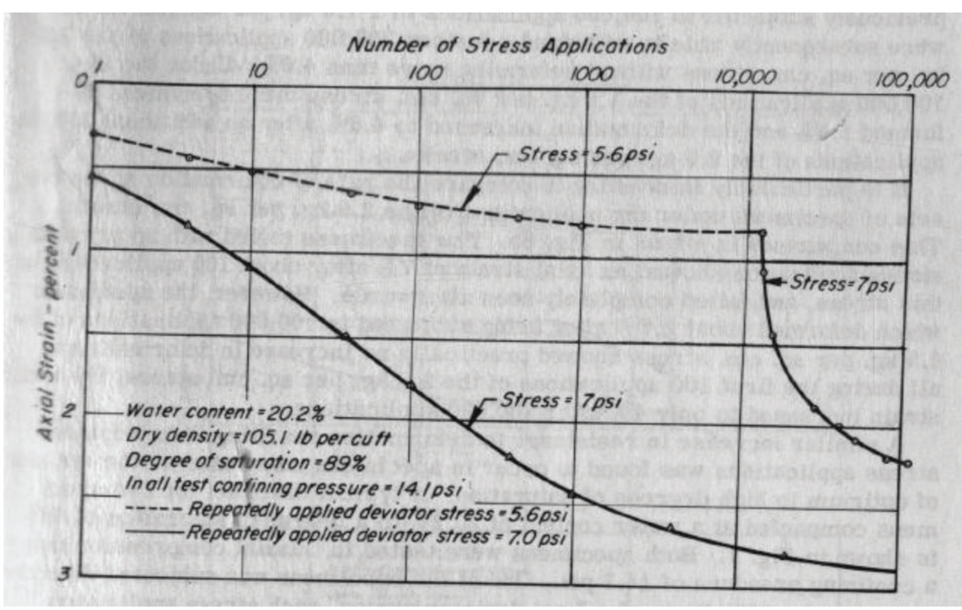


Figure 6.2 Effect of change in strain during repeated loading (from Seed et al., 1958).

specimens and thus when simulating a traffic load test the influence of densification should be considered. Seed et al. (1955) and Seed and Fead (1960) proposed the triaxial testing equipment needed for the changes in stress intensity, duration and frequency of stress application used for M_R tests. The effect of thixotropy, stress application, stress intensity, loading frequency, degree of saturation and method of compaction was discussed in detail in Seed et al. (1962, 1967). The research work paved the way for the use of resilient modulus in pavement design. NCHRP Report 128 (Van Til, 1972) first featured a procedure to perform the test and also included a correlation chart for resilient modulus and soil support value, which could be used for design.

The test procedure followed by researchers prior to 1982 varied quite a lot in terms of the number of loading repetitions, stress intensity, load waveform, etc. Larew and Leonards (1962) performed repeated load tests with a semicircular load waveform at a frequency of 20–22 cycles per minute. They reported that there existed a critical deviatoric stress beyond which, a specimen subjected to cyclic loading continued to exhibit plastic deformations until failure. In other words, at high deviatoric stresses, a resilient state may not be achieved (Figure 6.3). Dunlap (1963) applied cyclic deviatoric stresses on granular specimens with a load period of 0.2 seconds and measured deformations up to 100,000 cycles. He used two sets of tests at different deviatoric stresses (51.8 psi and 34.5 psi) and studied the influence of varying confining pressures (5, 10, 15, 20, 25, 30 psi). He reported that the dynamic modulus, defined as the ratio between the repeated deviatoric stress and the recoverable strain, had a linear relationship with confining pressure and was dependent on the applied repetitions.

Young and Baladi (1977) discussed the number of load applications, deviator stress, load wave form, load frequency and duration, and confining pressures that had to be considered to simulate traffic loading, and suggested to have 10,000 to 100,000 cycles, 1 psi to 70 psi deviatoric stress, square or sinusoidal waveform, 10 to 30 per minute frequency with a load duration varying from 0.04 to 0.25 seconds at confining pressures ranging from 0 psi to 25 psi. Brown and Hyde (1975) studied the resilient behavior of well graded crushed stone by measuring vertical as well as lateral strains under constant and varying confining pressures, at a cyclic deviatoric stress of 0 to 200 kPa. They reported that the resilient modulus and permanent strains were similar for tests performed at variable confining pressure (VCP) and constant confining pressure (CCP) equal to the mean of the cyclic confining pressures (Figures 6.4a and 6.4b). The lateral strain measurements were used to calculate the Poisson's ratio. They observed that unlike resilient modulus, Poisson's ratio was affected by the application of a constant or variable confining pressure. The effect was visible on not just the range of Poisson's ratio values obtained for the two tests but also on its variation with an increasing ratio of deviatoric stress to mean confining stress; while the Poisson's ratio increased with increasing ratio of deviatoric stress to mean confining stress for CCP it reduced in the case of VCP (Figure 6.4c). This behavior was further explained by looking at the stress strain response under the two test conditions. It was found that constant confining pressure tests were accompanied with a low range of volumetric strains and high range of shear strains. This showed that unlike resilient modulus, Poisson's ratio was affected by the variation in mean normal stresses.

Repeated load tests were found to better simulate real traffic conditions experienced by a pavement. The evolution of M_R test protocols is provided in Tables 6.1 and 6.2. Table 6.1 discusses the general test requirements such as material classification criteria, specimen size and compaction methods, used by the different test as well as the position of the LVDT and load cell, while Table 6.2 tabulates the conditioning and testing sequences adopted by each standard. The first standard

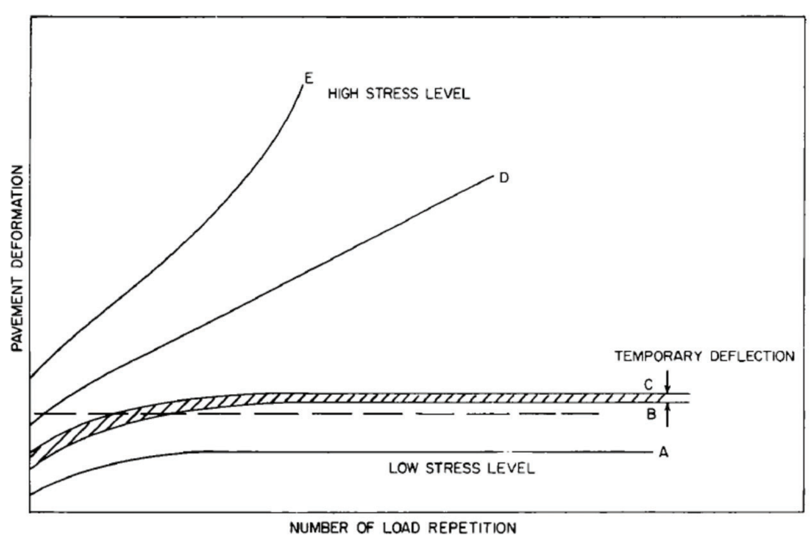


Figure 6.3 Effect of repeated loads on specimen deformation (from Larew & Leonards, 1962).

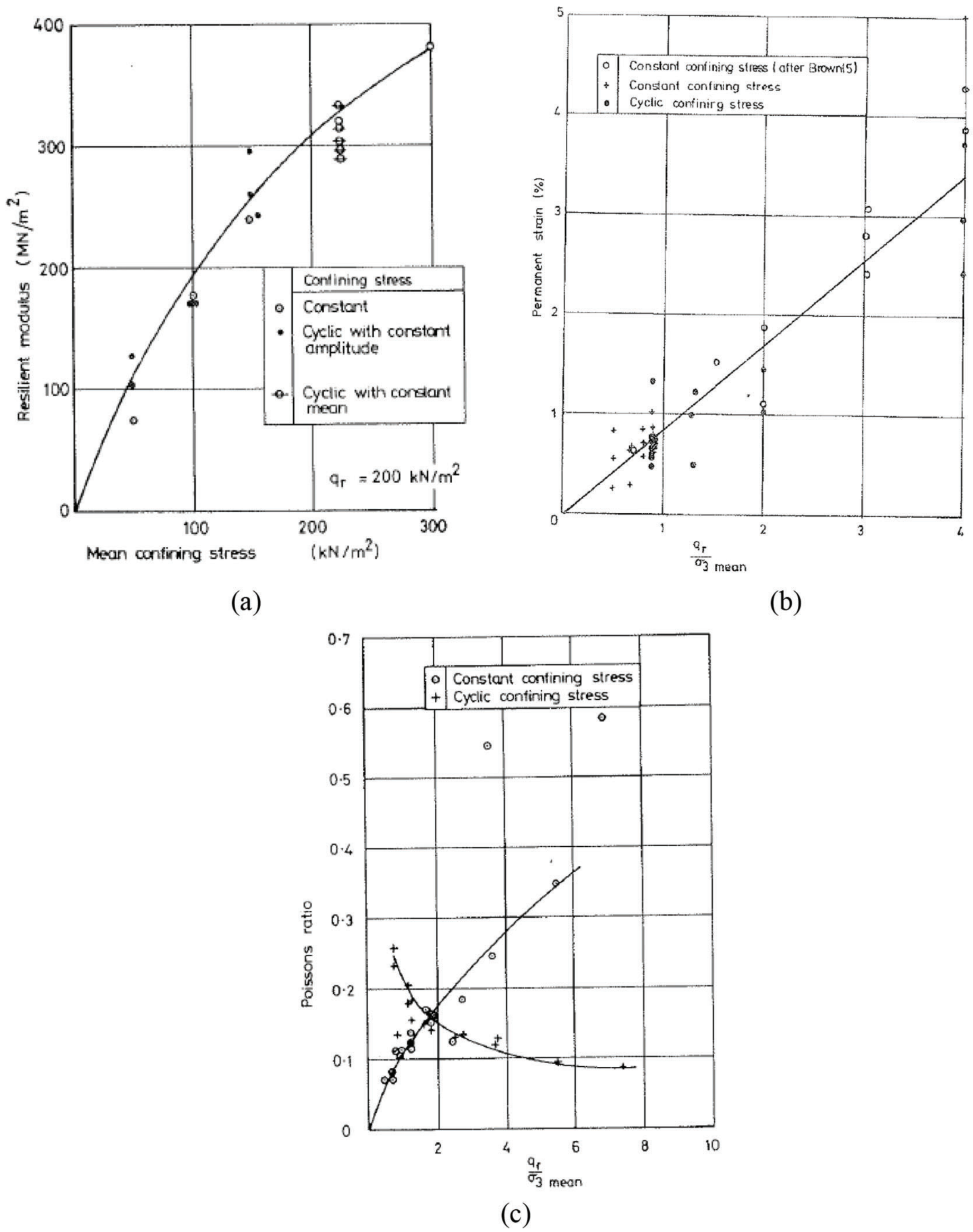


Figure 6.4 Results for constant and variable confining pressure: (a) resilient modulus vs. mean confining stress; (b) permanent strain vs. ratio of deviatoric stress to mean confining stress; and (c) poisson's ratio vs. ratio of deviatoric stress to mean confining stress.

method for the resilient modulus of subgrade soils was AASHTO T 274 (Oregon State University, 1990). This standard faced criticism due to its complicated testing sequence and lack of details in terms of selection of load wave form, specimen size, compaction technique, etc. In addition to this, the test stresses were deemed to be too severe, resulting in specimen failure. This test procedure was featured in the 1986 AASHTO design guide, which adopted the use of subgrade resilient modulus to design flexible pavements. Although a standard test procedure was available during the time, testing agencies worked with a slightly modified test procedure based on experience while researchers at the time focused on test procedure development and identification of factors affecting the resilient behavior of cohesive and granular soils. Elliott and Thornton (1988) proposed a simplified version of the test which could drastically reduce the testing time. They proposed using a single value of confining pressure and deviatoric stress based on subgrade conditions and a reduced

TABLE 6.1
Comparison of AASHTO test protocols for the Resilient Modulus Test of unbound subgrade soil materials: General test specifications

	AASHTO T 274 (1982)	AASHTO T 292 (1991)	AASHTO T 294 (1992)	AASHTO T P46 (1996)	AASHTO \ T 307 (1999)	NCHRP 1-28 (2004)	NCHRP 1-28 A (2004)
Type of Material Classification	Cohesive and cohesionless	Cohesive and cohesionless	*Type 1* (70% passing #10 and 20% passing #200) and *Type 2* (All else)	*Type 1* (70% passing #10 and 20% passing #200 and PI ≤ 10) and *Type 2* (All else)	Same as AASHTO T P46	Same as T P46 *Type 1a:* 100% passing 1.5 in. sieve *Type 1b:* 100% passing 1.0 in. sieve	Type 1 Type 2 Type 3 *Type 4*–2.8 in. undisturbed sample
Specimen Size (in.)	2.8 4 8	2.8 4 6	*Type 1:* 4, 6 *Type 2:* 2.8	*Type 1:* 6 *Type 2:* 2.8	Same as AASHTO T P46	*Type 1a:* 6 *Type 1b:* 4, 6 *Type 2:* 2.8, 4	*Type 1:* 4, 6 *Type 2:* 4 *Type 3:* 4
Compaction Method	*Cohesive:* Vibratory Kneading Static *Cohesionless:* Vibratory	Vibratory	*Type 1:* Vibratory *Type 2:* Static	Same as AASHTO T294	*Type 1:* Vibratory *Type 2:* Vibratory Kneading Static	*Type 1* Vibratory *Type 2* Vibratory Kneading Static	*Type 1:* Vibratory, Impact *Type 2:* Vibratory *Type 3:* Kneading
Load Cell Position	Inside Chamber	Inside Chamber	External	External	Inside	Inside	Inside
LVDT Position	2 LVDTs attached directly to the specimen	2 LVDTs clamped to piston rod outside chamber	2 LVDTs clamped to piston rod outside chamber	2 LVDTs clamped to piston rod outside chamber	2 LVDTs clamped to piston rod outside chamber	2 LVDTs attached directly to the specimen	2 LVDTs attached directly to the specimen
Load Shape	Sine, haversine, rectangle, triangle	Haversine, rectangle, triangle	Haversine	Haversine	Haversine	Haversine	Haversine
Load/Cycle Duration	0.1 s/ 1–3 s	0.05-0.1 s/1–3 s	0.1 s/ 1 s	0.1 s/ 1 s	0.1 s/ 1–3 s	0.1 s/ 1 s	0.1 s/ 1 s

number of stress cycles to obtain resilient modulus values that can be used in design. Seim (1990) discusses a modified version of the AASHTO T 274 (Oregon State University, 1990) procedure developed by the Soil Mechanics Bureau of New York State Transportation Department. The proposed procedure comprised of changes to load frequency and duration as well as additional deviatoric stress cycles and a phased conditioning sequence for cohesive soils. The modifications were made to overcome equipment limitations as well as reduce specimen failure associated with the AASHTO procedure. Jackson (1989) further discusses the complexities associated with the AASHTO T 274 (Oregon State University, 1990) procedure and the modified testing adopted by the Washington Transportation Department to carry out M_R testing. The modified testing program was composed of a single confining pressure and deviatoric stress (6 psi and 8 psi, respectively) for the conditioning phase followed by a set of increasing confining pressures and deviatoric stress cycles which covered the range of stresses found in their pavement sections. Ho (1989) reported that the AASHTO procedure comprised of confining pressures which were unrealistically high in comparison to typically observed values for subgrade soils in Florida. They also reported specimen failure due to the severe conditioning and high stress sequence followed by the AASHTO protocol and proposed a modified method with fewer confining and deviatoric sequences; five sequences at 1, 2, and 5 psi confinement and 2, 4, and 5 psi deviatoric stresses, accompanied with a large (10,000) number repetition. The test results also suggested that the subgrade materials tested near optimum moisture conditions are independent of number of repetitions.

Following the criticism, the testing procedure was reviewed, and a new protocol was published in 1991, AASHTO T 292. AASHTO T 292 (1991) included a slightly less complex testing sequence with different stress sequences for granular and cohesive materials, but did not provide specifications for specimen size, load wave form and duration. A revised version of the test standard was released in 1992, AASHTO T 294, which provided a more detailed testing procedure, which followed a reversed sequence of stress applications for granular subgrade soil materials to account for the stiffening of specimens during the test. However, the test protocol that came next, AASHTO T P46, as part of the Strategic Highway Research Program (SHRP), changed the sequence for granular materials back to a decreasing order of confining pressure application, making it consistent with all subgrade soil materials. However, different procedures were defined for preparation of granular and cohesive soils samples.

TABLE 6.2
Comparison of AASHTO test protocols for the Resilient Modulus Test of unbound subgrade soil materials: Stress sequence

	AASHTO T 274 (1982)	AASHTO T 292 (1991)	AASHTO T 294 (1994)	AASHTO T P46 (FHWA, 1996)	AASHTO T 307 (2007)	NCHRP 1-28 (Witczak, 2003)	NCHRP 1-28 A (Witczak, 2003)
Type of Material	Cohesive	Cohesive subgrade	Type 2 soils	Subgrade type 2	Subgrade type 2	Fine grained subgrade	Fine grained subgrade
Conditioning CS/DS	6/1, 2, 4, 8, 10	3/3	6/4	6/4	6/4	0/5	4/8
# Cycles	200 × 5	1,000	1,000	500–1,000	500–1,000	200	1,000
Test CS/DS	6/1, 2, 4, 8, 10 3/1, 2, 4, 8, 10 0/1, 2, 4, 8, 10	3/3, 5, 7, 10, 15	6/2, 4, 6, 8, 10 3/2, 4, 6, 8, 10 0/2, 4, 6, 8, 10	6/2, 4, 6, 8, 10 4/2, 4, 6, 8, 10 2/2, 4, 6, 8, 10	6/2, 4, 6, 8, 10 4/2, 4, 6, 8, 10 2/2, 4, 6, 8, 10	0/3, 5, 7, 9, 11	8, 6, 4, 2/4 8, 6, 4, 2/7 8, 6, 4, 2/10 8, 6, 4, 2/14
# Cycles per Sequence	200	50	100	100	100	50	100
# Sequences	15	5	15	15	15	5	15
Type of Material	Cohesionless	Granular subgrade	Type 1 soils	Subgrade type 1	Subgrade type 1	Granular and low cohesion subgrade	Granular subgrade
Conditioning CS/DS	5/5, 10 10/10, 15 15/15, 20	20/15	15/15	6/4	6/4	6/9.2	4/7
# Cycles	200 for each sequence	1,000	1,000	500–1,000	500–1,000	500	1,000
Test CS/DS	20/1, 2, 5, 10, 15, 20 15/1, 2, 5, 10, 15, 20 10/1, 2, 5, 10, 15 5/1, 2, 5, 10, 15 1/1, 2, 5, 7.5, 10	20/10, 20, 30, 40 15/10, 20, 30, 40 10/5, 10, 20, 30 5/5, 10, 15 3/5, 7, 9	3/3, 6, 9 5/5, 10, 15 10/10, 20, 30 15/10, 15, 30 20/15, 20, 40	6/2, 4, 6, 8, 10 4/2, 4, 6, 8, 10 2/2, 4, 6	6/2, 4, 6, 8, 10 4/2, 4, 6, 8, 10 2/2, 4, 6	2/2.4, 3.4, 4.4 3/3.6, 4.4, 6.6 4/4.8, 6.8, 8.8 6/5.2, 7.2, 9.2 8/7.6, 9.6, 11.6	CS–2, 4, 6, 8, 12 DS–1, 2, 3, 4, 6 2, 4, 6, 8, 12 4, 8, 12, 16, 24 6, 12, 18, 24, 36
# Cycles per Sequence	200	50	100	100	100	100	100
# Sequences	27	18	15	13	13	15	20

The current protocol, AASHTO T 307-99 (2007) is similar to AASHTO T P46. It describes the criterion for selecting the specimen size based on soil classification and provides a testing procedure comprising a conditioning load sequence and subsequent load sequences with different confining and maximum deviatoric stresses. The recent methods proposed by the NCHRP, NCHRP 1-28, and 1-28A recommend a significantly different testing sequence. NCHRP 1-28 supports an increasing sequence of confining pressure for granular materials while NCHRP 1-28A features a stress sequence based on increasing stress ratio.

To somewhat address the issue of how representative the M_R test is of actual conditions in the field, researchers such as Kim and Kim (2007), Ng et al. (2013) and Zhang et al. (2019) proposed a simplified testing procedure. Kim and Kim (2007) proposed a test under confining stress of 13.8 kPa (2 psi) and deviatoric stresses of 13.8, 27.6, 41.4, 55.2, and 69 kPa (2, 4, 6, 8, and 10 psi). The 13.8 kPa stress is the confinement induced by an 80 kN Equivalent Single Axle Load (ESAL). The researchers also reduced the number of cycles to 250 for the conditioning phase and 50 repetitions for the other sequences. A comparison with the normal testing protocol used in AASHTO T 307 (16 sequences) was done and it was observed that the M_R values with confining stress of 13.8 kPa (2 psi) compared well with those using the standard protocol. Ng et al. (2013) studied the soil water characteristic curve (SWCC) of the soil in conjunction with the repeated load triaxial (RLT) test to account for seasonal variation in moisture content and the effect of soil suction on resilient modulus. Soil suction was controlled by applying predefined pore pressures at the top and bottom of the specimen. The specimen was then left to rest for a few days until the pore pressure attained a state of equilibrium throughout the specimen. The RLT test protocol was followed as per AASHTO T 307-99 (2007) without the initial conditioning to remove the effects of overconsolidation. They observed that the stress-strain response of cohesive soils was highly nonlinear and the resilient modulus values decreased with increasing cyclic stress. Han et al. (2018) also considered the soil water characteristic curve to study the variation of the resilient modulus as a function of not only the stress state but also of moisture content. Cyclic triaxial tests were performed as per AASHTO T 307-99 (2007) for three confining stresses (2, 4, 6 psi) and four deviatoric stresses (2, 4, 6, and 8 psi). The higher deviatoric stress sequence (10 psi) was eliminated due to the long equilibrium period needed to control suction in the soil. They also modified the conventional unconfined compression (UC) test by allowing an unloading-reloading loop at 1% axial strain. They found that the deviatoric stress ($S_{u1\%}$) and reloading elastic modulus ($E_{1\%}$) at 1% strain obtained from the modified UC tests provided better correlations for estimating resilient modulus.

Due to the availability of so many different testing protocols for a single test with many variations in specifications and testing sequences, the accuracy of any correlation between soil properties/classification and resilient modulus is doubtful. For example, the cyclic stress-strain response of soils is often characterized by confining stress, applied deviatoric stress, void ratio, overconsolidation ratio (OCR), (Hardin & Drnevich, 1972; Seed & Idriss, 1970). Equations 6.1 and 6.2 were proposed by Hardin and Drnevich (1972) to evaluate the shear modulus (G).

$$G_{max} = 1{,}230 \frac{(2.973 - e)^2}{1 + e} OCR^a \sigma'^{1/2}_m \qquad \text{(Eq. 6.1)}$$

$$G = \frac{G_{max}}{1 + \frac{\gamma}{\gamma_r}} \qquad \text{(Eq. 6.2)}$$

Where,
G_{max} = maximum shear modulus in psi,
e = void ratio,
OCR = overconsolidation ratio,
a = parameter that depends on the plasticity index of the soil,
σ'_m = mean principal effective stress in psi,
γ = strain amplitude, and
γ_r = reference strain which depends on the small strain shear modulus and shear stress at failure.

It can be seen from the equations that the shear modulus of a soil increases with confinement, decreases with shear strain and is affected by the stress history. The lack of agreement among the different correlations that exist for resilient modulus could be explained by accounting for the effects of OCR of the soil during the M_R test. Because of all this, it may be informative to study the stress-strain response, stress path and plastic deformations of the soil with changes of stresses and increasing cycles, to better understand the soil response.

7. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

7.1 Summary

The study presents data from three different sites in Indiana, identified with the help of INDOT, for sample collection and laboratory testing of subgrade soils. The sites selected, namely US-31, SR-37 and I-65 were all projects involving stabilization of subgrade soils with cement. Soil characterization tests indicated that the soils collected from site US-31 were primarily A-1 (SM) soils, those collected from site SR-37 were A-2-4 (SC) soils and the soils collected from the I-65 site were A-6 (CL+CM) soils and A-4 (CL) soils. The Atterberg limits and the compaction tests indicated that the soils found in the US-31 and I-65 sites were highly variable in terms of plasticity index, optimum moisture content (OMC) and maximum dry density (MDD), across the 11 locations where the samples were collected. The soils from site SR-37 were uniform across the 11 locations.

Resilient Modulus Tests were performed on untreated and cement-treated samples for all locations in all the three sites. The samples from site US-31 were treated with 4% cement by weight and the samples from sites SR-37 and I-65 with 5% cement by weight. All samples were prepared at OMC and MDD and cured for 28 days prior to testing. The resilient modulus values for cement treated samples was found to be more sensitive to the deviatoric stress and confining stress compared to untreated samples across all three sites.

UCS testing was also performed on the cement treated samples. Similar to what was done for the resilient modulus, the samples were prepared at OMC and MDD and cured for 7 days prior to testing. The sample preparation and curing process was kept identical to that of the M_R tests to ensure compatibility of results. The UCS results ranged from 1.2 MPa to 3.1 MPa (170 psi to 450 psi) for all the soils studied in this project. The considerable range in the UCS test results is attributed to soil variability and different soil types across the sites. Additional tests were also performed on the 28 day cured samples after the M_R test was completed. The UCS values of the 28-days cured samples was found to be 1.2 to 1.5 times the 7-day cured UCS values. The average UCS values for the 7- and 28-days cured specimens were used for the correlations.

7.2 Conclusions

The major conclusion of the study is that there is not a direct statistically significant correlation between the resilient modulus and UCS parameters. Such correlation has not been found in the technical literature, nor from the tests performed in this study. However, it seems that the resilient modulus falls within a certain range of values depending on the soil type. The majority of the resilient modulus of cement-treated A-2-4 soils investigated in this study ranged from 210 MPa to 275 MPa (30,000 psi to 40,000 psi). The majority of the resilient modulus values for cement-treated A-4 and A-6 soils fell in the range of 135 MPa to 240 MPa (20,000 psi to 35,000 psi). These range of values can prove useful to get a sense of expected stiffnesses of cement stabilized subgrades for typical Indiana soils. A linear regression analysis involving the soil properties (plasticity index, OMC and MDD) along with resilient modulus and UCS results was also performed, to establish a simple model for predicting the resilient modulus values for the soils investigated. The linear regression models perform at R^2 values of 0.85 and 0.91 using 28-days cured and 7-days cured UCS data, respectively. The drawback of these models is that they are derived from and tested on limited high-quality laboratory data. Thus, the applicability of the model is limited to the range of soils used in the study. A review of the history of M_R test suggests that the test procedure used to evaluate M_R has undergone significant changes over the years and is still very complicated. The lack of statistically significant correlations between resilient modulus and UCS in the literature as well as in the current study could be explained by further looking into the M_R test procedure and improving our interpretation of the test.

7.3 Recommendations

The laboratory tests and analyses performed in this study, as well as those found in the literature, indicate limited potential for accurate correlations between resilient modulus and UCS for treated subgrade soils. One possible explanation for this is that the current M_R testing sequence leads to initial overconsolidation of the sample (as it goes from high confining to low confining pressure). Other possible explanation is that, because of the disparity of the tests, i.e., UCS tests strength while the M_R tests stiffness, such correlations do not exist. However, given the complexity of the M_R test and that it is a test widely used and its results employed for pavement design, further investigation on the test seems warranted. Future work could focus on a more in-depth study of the M_R testing procedure and exploring a possible modification of the test sequence to see the effects on correlations, both with UCS and with FWD tests. Additional testing could be performed to find correlations based on seismic modulus or small-strain modulus derived from shear wave velocity, as an alternative approach for estimating resilient modulus of treated subgrade soils.

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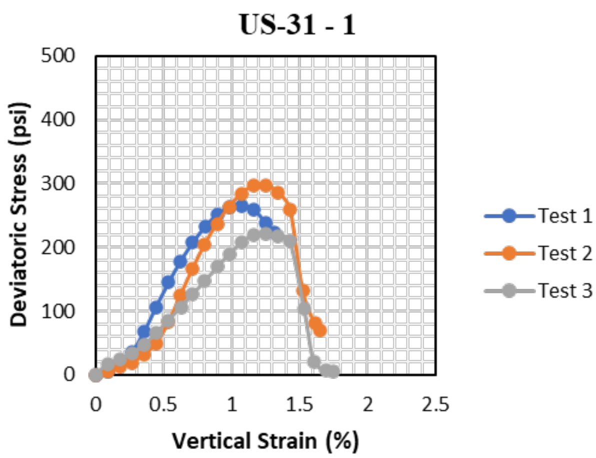
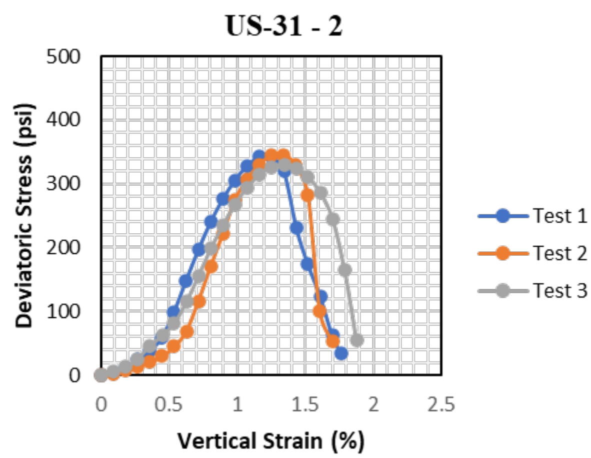
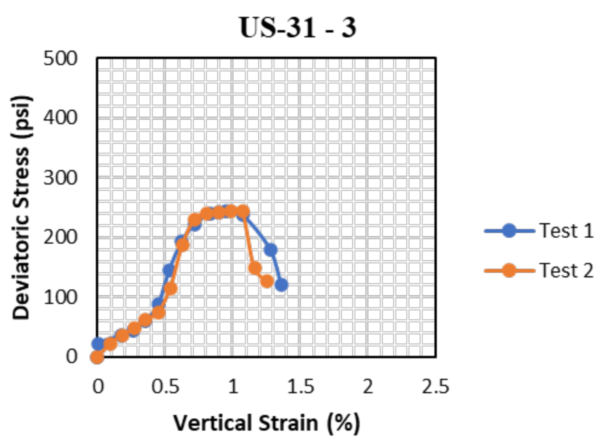
APPENDICES

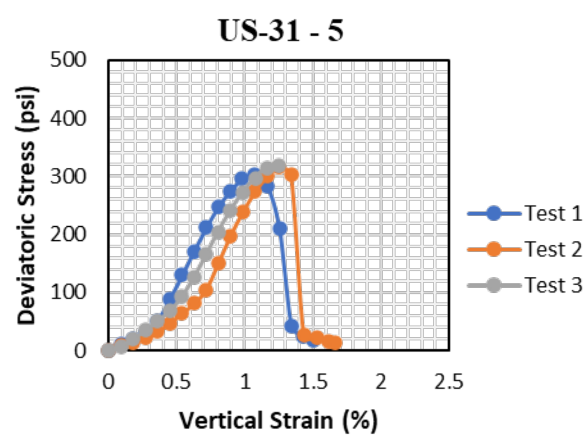
Appendix A. Stress-Strain Plots

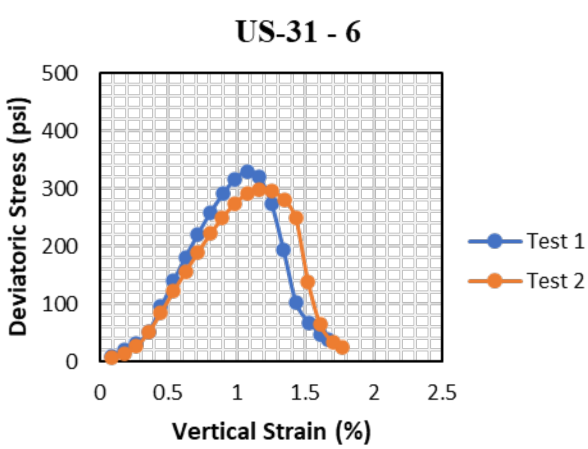
APPENDIX A. STRESS-STRAIN PLOTS

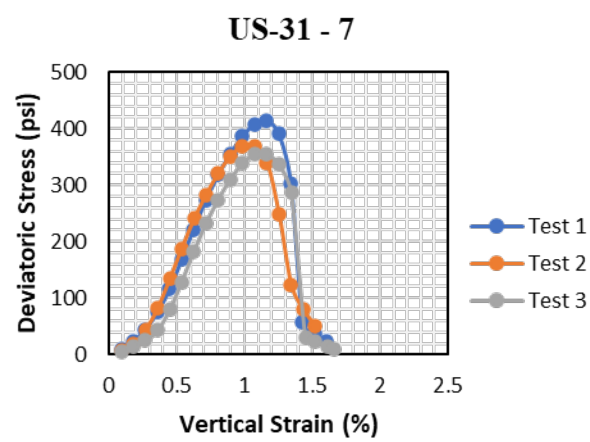












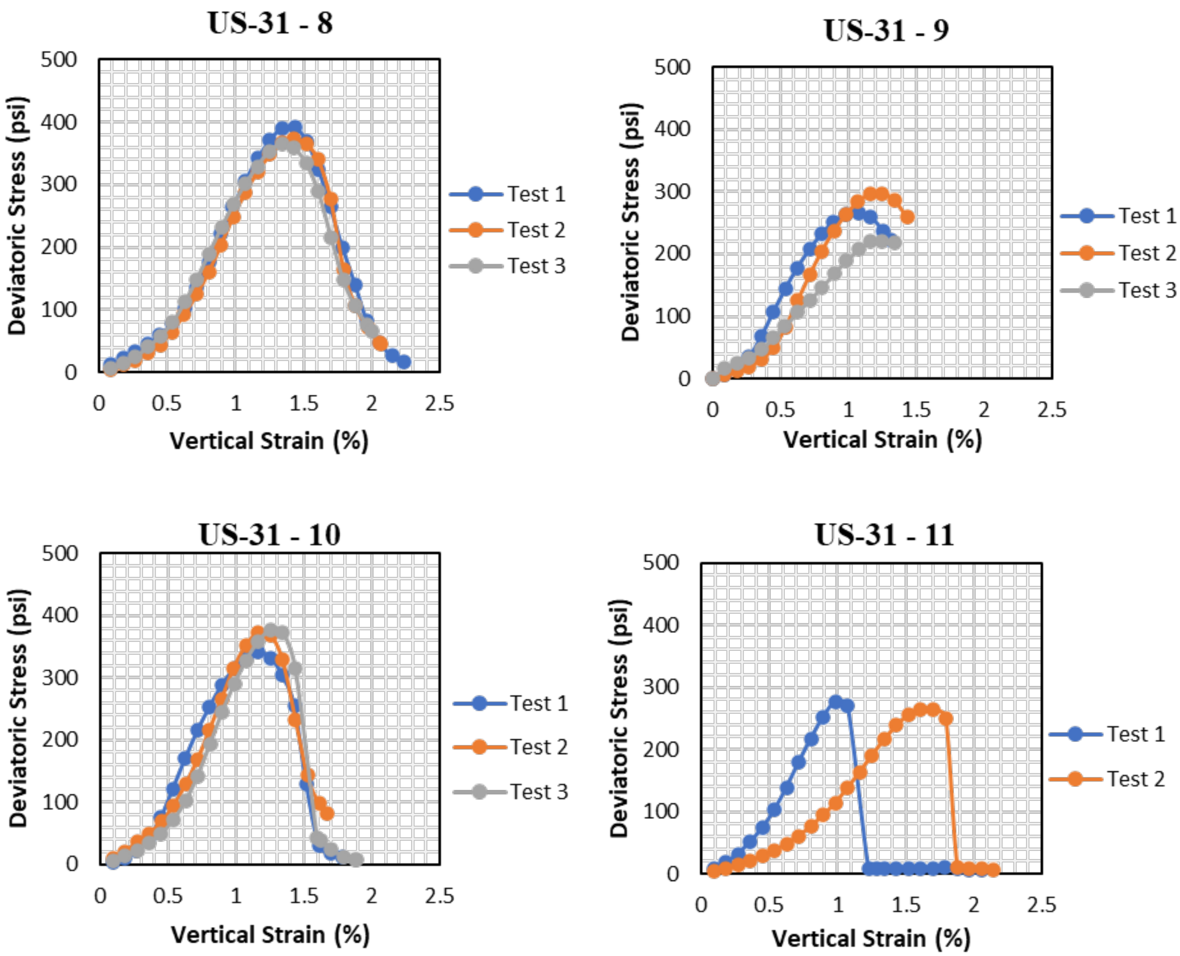
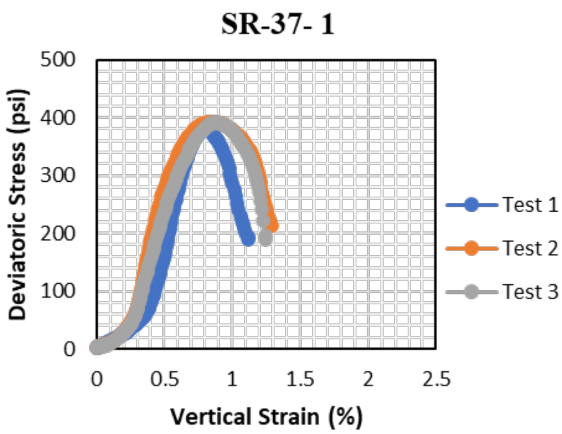
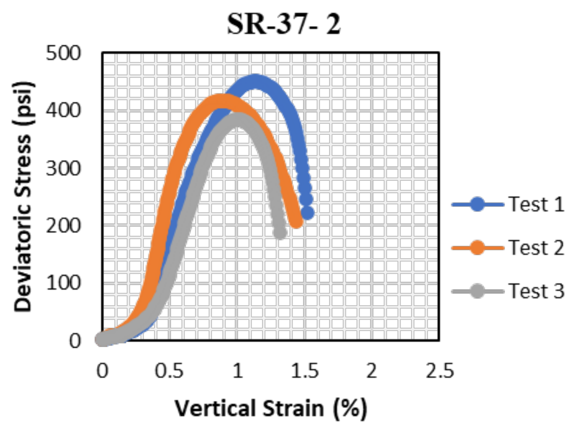
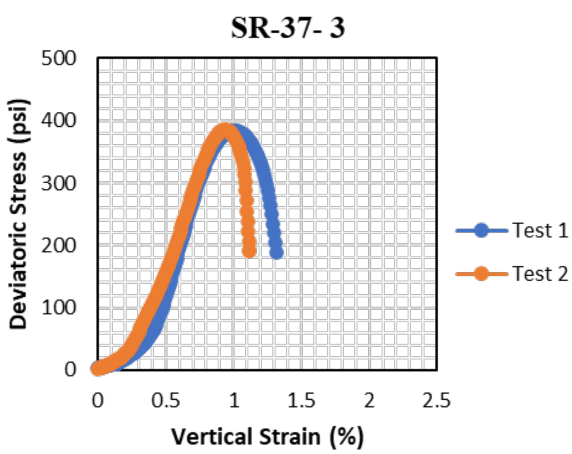
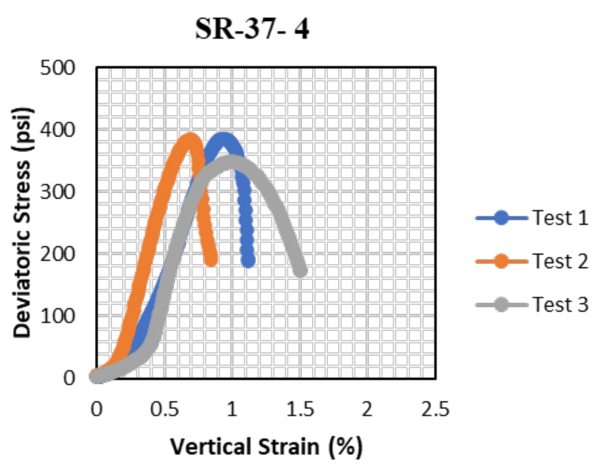


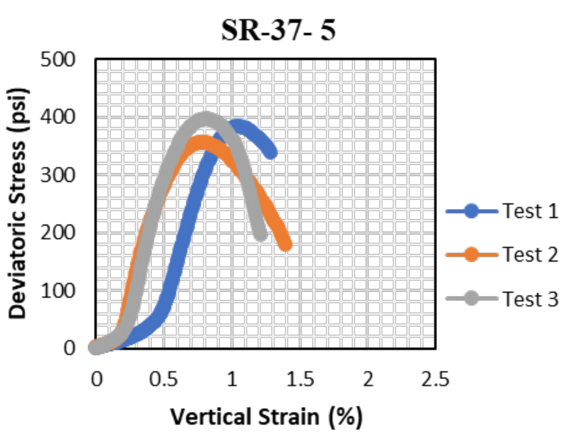
Figure A.1 Stress-Strain plots of 7-day UCS tests for site US-31.

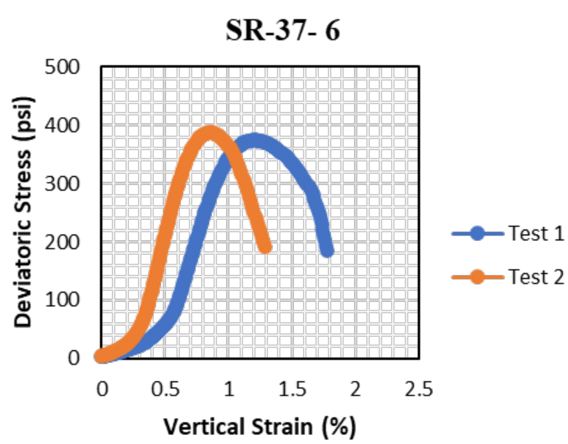

SR-37- 1
Deviatoric Stress (psi)
Vertical Strain (%)
Test 1
Test 2
Test 3


SR-37- 2
Deviatoric Stress (psi)
Vertical Strain (%)
Test 1
Test 2
Test 3


SR-37- 3
Deviatoric Stress (psi)
Vertical Strain (%)
Test 1
Test 2


SR-37- 4
Deviatoric Stress (psi)
Vertical Strain (%)
Test 1
Test 2
Test 3


SR-37- 5
Deviatoric Stress (psi)
Vertical Strain (%)
Test 1
Test 2
Test 3


SR-37- 6
Deviatoric Stress (psi)
Vertical Strain (%)
Test 1
Test 2

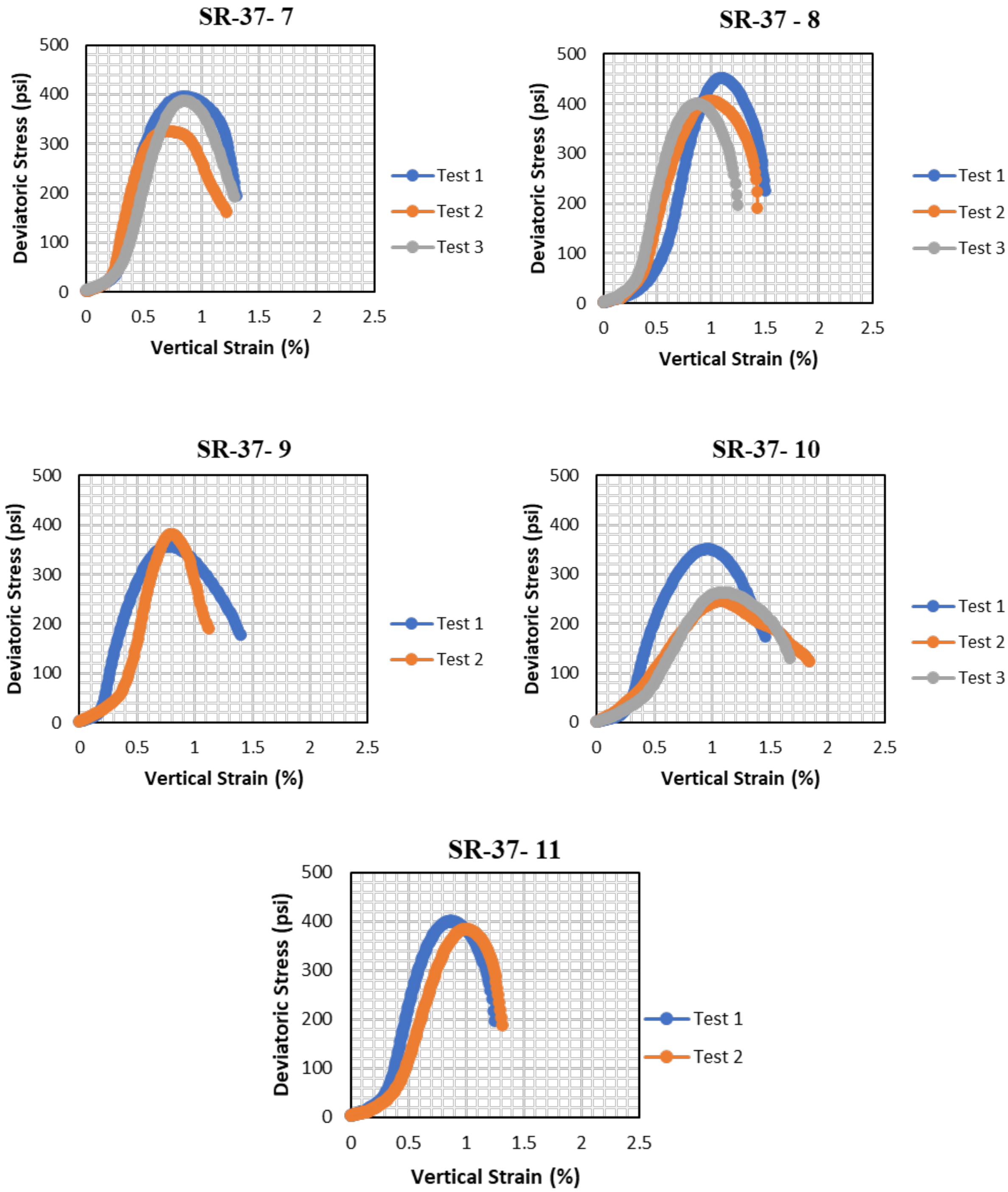
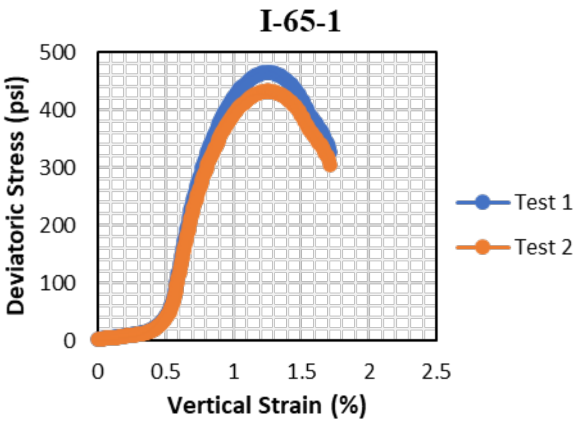
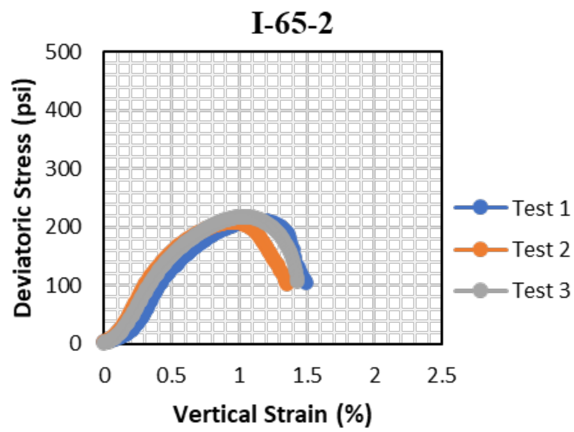
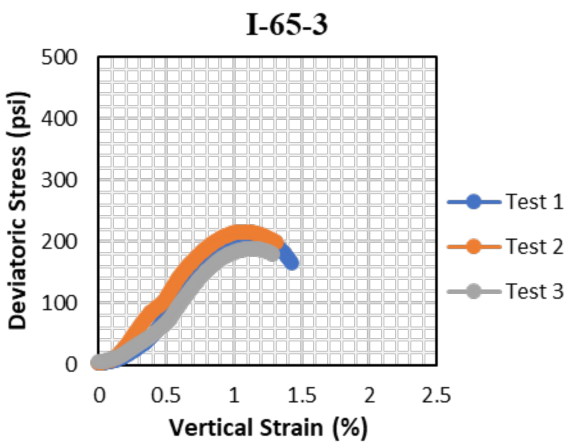
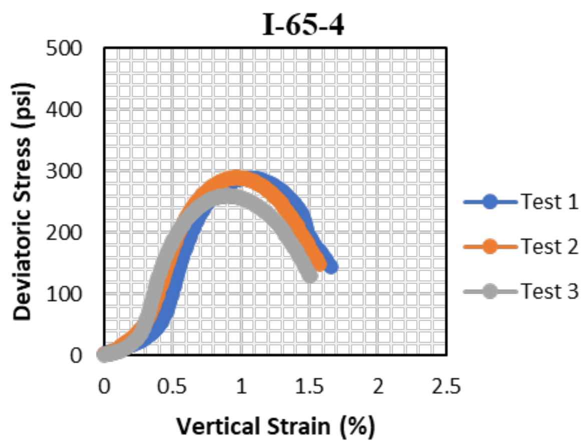


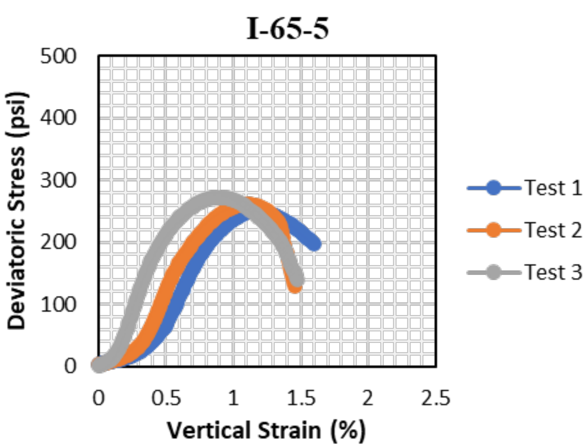
Figure A.2 Stress-strain plots of 7-day UCS tests for site SR-37.

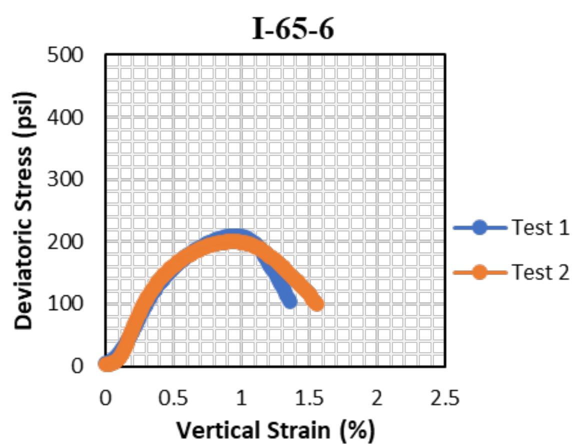

I-65-1
Deviatoric Stress (psi)
0
100
200
300
400
500
0
0.5
1
1.5
2
2.5
Vertical Strain (%)
Test 1
Test 2


I-65-2
Deviatoric Stress (psi)
0
100
200
300
400
500
0
0.5
1
1.5
2
2.5
Vertical Strain (%)
Test 1
Test 2
Test 3


I-65-3
Deviatoric Stress (psi)
0
100
200
300
400
500
0
0.5
1
1.5
2
2.5
Vertical Strain (%)
Test 1
Test 2
Test 3


I-65-4
Deviatoric Stress (psi)
0
100
200
300
400
500
0
0.5
1
1.5
2
2.5
Vertical Strain (%)
Test 1
Test 2
Test 3


I-65-5
Deviatoric Stress (psi)
0
100
200
300
400
500
0
0.5
1
1.5
2
2.5
Vertical Strain (%)
Test 1
Test 2
Test 3


I-65-6
Deviatoric Stress (psi)
0
100
200
300
400
500
0
0.5
1
1.5
2
2.5
Vertical Strain (%)
Test 1
Test 2

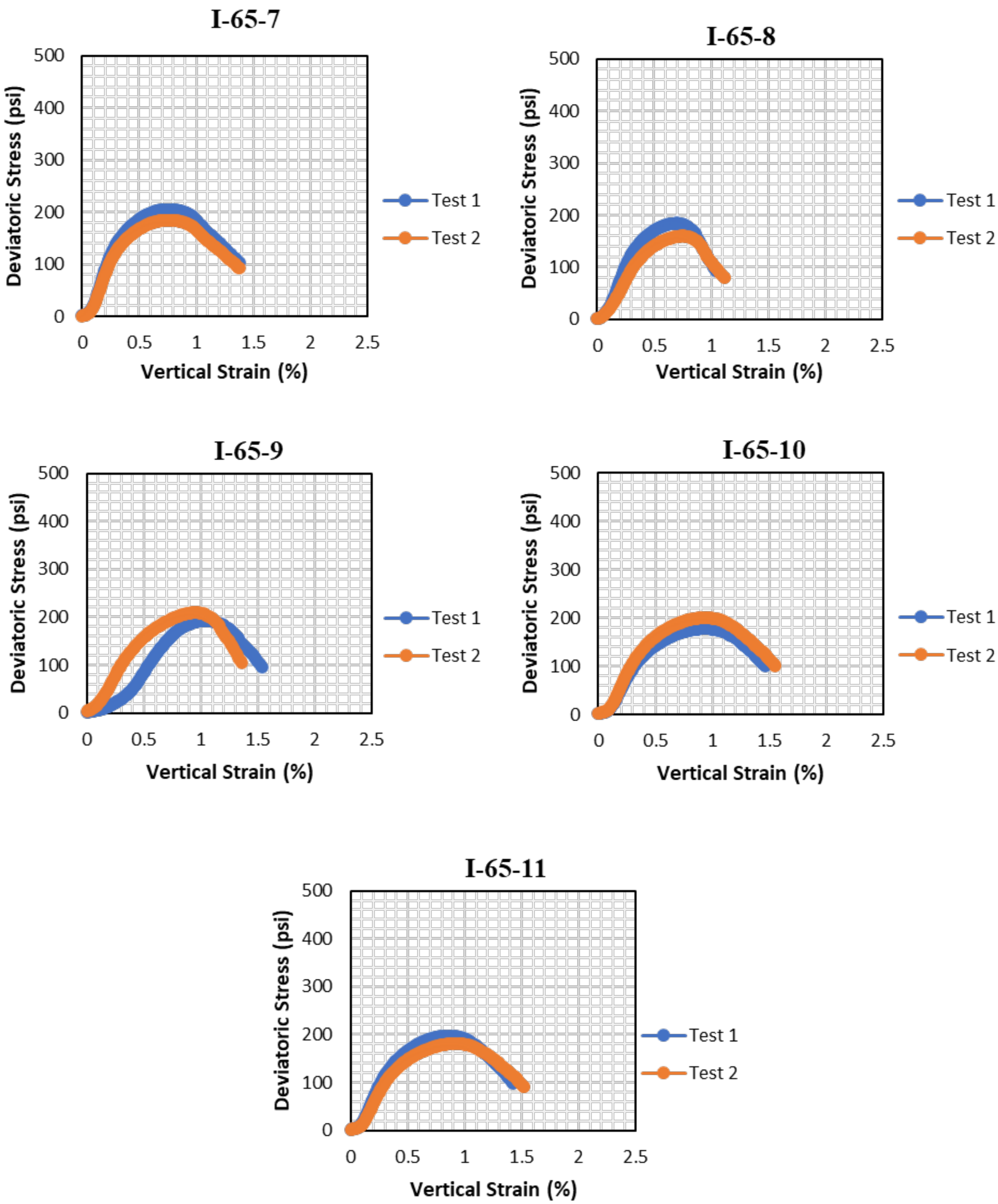


Figure A.3 Stress-strain plots of 7-day UCS tests for site I-65.

About the Joint Transportation Research Program (JTRP)

On March 11, 1937, the Indiana Legislature passed an act which authorized the Indiana State Highway Commission to cooperate with and assist Purdue University in developing the best methods of improving and maintaining the highways of the state and the respective counties thereof. That collaborative effort was called the Joint Highway Research Project (JHRP). In 1997 the collaborative venture was renamed as the Joint Transportation Research Program (JTRP) to reflect the state and national efforts to integrate the management and operation of various transportation modes.

The first studies of JHRP were concerned with Test Road No. 1—evaluation of the weathering characteristics of stabilized materials. After World War II, the JHRP program grew substantially and was regularly producing technical reports. Over 1,600 technical reports are now available, published as part of the JHRP and subsequently JTRP collaborative venture between Purdue University and what is now the Indiana Department of Transportation.

Free online access to all reports is provided through a unique collaboration between JTRP and Purdue Libraries. These are available at http://docs.lib.purdue.edu/jtrp.

Further information about JTRP and its current research program is available at http://www.purdue.edu/jtrp.

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