EFFECT OF SUBGRADE CONDITIONS ON PAVEMENT ANALYSIS AND PERFORMANCE PREDICTION: A STUDY FOR IDAHO CONDITIONS

by

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DEDICATION

Dedicated to my parents for their unconditional love and support.

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ABSTRACT

The Mechanistic-Empirical (M-E) pavement design approach detailed in the Mechanistic-Empirical Pavement Design Guide (MEPDG), and subsequently implemented through AASHTOWare® Pavement ME Design relies extensively on detailed material properties that ultimately govern the analysis and performance prediction results. For unbound materials like soils and aggregates, Resilient Modulus (M_R) is the most critical input parameter affecting layer response under vehicular and environmental loading. Representing a material's ability to 'recover' after loading, resilient modulus is determined in the laboratory through repeated load triaxial testing. Although the original test protocol to measure the resilient modulus value of a soil or aggregate was developed back in the 1980's, this test is still not widely used by state highway agencies because it is cumbersome, and requires significant investments towards equipment and personnel training. Accordingly, most agencies rely on correlation equations to predict the resilient modulus values for soils and aggregates from other easy-to-determine material properties. However, these correlation equations are mostly region specific, and therefore, do not produce adequate results across different geographic regions. This has led several state highway agencies to undertake local calibration efforts for improved prediction of material properties.

Over the past decade, the Idaho Transportation Department (ITD) has invested significant resources to facilitate state-wide implementation of mechanistic-empirical pavement design practices. A research study was recently undertaken by ITD to develop a

database of resilient modulus properties for different soils and aggregates commonly used in the state of Idaho for pavement applications. Another objective of the study was to assess the adequacy of different correlation equations currently available to predict soil and aggregate resilient modulus from easy-to-determine material (strength and index) properties. This Master's thesis is based on tasks carried out under the scope of the above-mentioned project, and focuses on laboratory characterization and analysis of representative subgrade soil types collected from across Idaho. An extensive laboratory test matrix was developed involving commonly used mechanical and index tests, repeated load triaxial tests for resilient modulus determination, as well as tests to study the soil permanent deformation (plastic strain) behavior. Effect of moisture variation on soil strength, modulus, and permanent deformation properties was also studied by testing soil specimens at three different moisture contents. The test results were thoroughly analyzed to evaluate the feasibility of predicting resilient modulus from other material properties.

Findings from this research effort have been documented in the form of two journal manuscripts. The first manuscript highlights the importance of using adequate subgrade resilient modulus values during pavement design. Eight different soil types were randomly selected from a total of sixteen soil types, and the corresponding laboratory test results were used to highlight the limitations of ITD's current approach with assumed resilient modulus values. The second manuscript focuses on highlighting the importance of unbound material permanent deformation characterization during pavement design, and how small changes in moisture content can lead to significant differences in the rutting behavior of subgrade soils. First, a new permanent deformation testing protocol was developed to simulate typical stress states experienced by subgrade layers under vehicular

loading. Subsequently, permanent deformation tests were carried out on subgrade soil types collected from two distinctly different regions in Idaho as far as annual precipitation is concerned. Tests were conducted at three different moisture contents to highlight how the rutting potential of the subgrade may change significantly based on site precipitation and drainage characteristics. Finally, recommendations were made regarding how state highway agencies can accurately represent resilient modulus properties of soils during pavement analysis and performance prediction using AASHTOWare® Pavement ME Design.

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CHAPTER ONE: INTRODUCTION AND BACKGROUND

Background and Problem Statement

The subgrade constitutes the weakest layer in a pavement system, and can lead to significant pavement damage if not protected from excessive stress levels during the pavement service life. The design and construction of a well-performing, economical pavement system requires thorough knowledge of the subgrade properties. Since the release of its 1986 edition (AASHTO 1986), the American Association of State Highway and Transportation Officials (AASHTO) pavement design method has considered 'Resilient Modulus' (M_R) to be a key soil input during the pavement design process (Taylor & Timm 2009). The recently released Mechanistic Empirical (M-E) Pavement Design approach implemented through AASHTOWare® Pavement ME Design uses the soil resilient modulus as a critical input parameter for analysis and performance predication under vehicular as well as environmental loading. Resilient modulus properties of a soil can be determined in the laboratory through repeated load triaxial testing using specifications such as AASHTO T 307 (AASHTO T307 1999), or more recently, NCHRP 1-28A (Barksdale et al. 1997). However, these tests are quite complex in nature, and require significant investments towards equipment procurement and personnel training. This has led to several state highway agencies avoiding the performance of repeated load triaxial tests, and using correlation equations to predict soil M_R values from other easy-to-measure mechanical and index properties. Although researchers have developed several such correlation equations over the years (Robnett and Thompson 1973; Jones & Witczak

1977; Elliott et al. 1988; Drumm et al. 1990; Woolstrum 1990), these equations are not able to consistently predict M_R from soil index properties (Mokwa & Akin 2009). Accordingly, M_R values predicted using such correlation equations can often under-predict or over-predict the soil modulus, thus leading to erroneous pavement design. Certain studies in the past have shown that the correlation equations work with reasonable accuracy on a regional basis. In other words, certain correlation equations may work fairly well for soils from a particular region, while not working well when applied to soils from a different region. This has led several state highway agencies to undertake regional correlation development efforts to predict soil resilient modulus from simple mechanical and/or index properties.

Objectives and Research Tasks

Over the past decade, the Idaho Transportation Department (ITD) has committed significant resources towards facilitating statewide implementation of M-E pavement design practices. The design method currently used by ITD relies on the soil Resistance Value (R-value) to calculate required pavement thicknesses; this method has been known to result in significant over designs. As a part of ITD's current M-E implementation efforts, a research study was recently initiated to characterize typical soils and aggregates commonly used in Idaho for resilient modulus characteristics. The objective was to build a database of test results that would subsequently be used by ITD engineers during M-E pavement design. Moreover, the project also aimed at assessing the accuracy of different correlation equations developed to predict the resilient modulus values for soils and aggregates from easy-to-measure mechanical and index properties.

The research work reported in the current master's thesis was based on tasks performed under the scope of the above-mentioned ITD-sponsored research project, and focused primarily on the subgrade soils collected from different parts of Idaho. Besides testing the soils for database development, this master's thesis aimed at highlighting shortcomings associated with ITD's current practice of assuming standardized resilient modulus values based on soil types (based on the Unified or AASHTO soil classification systems). Several example pavement sections were analyzed using AASHTOWare® Pavement ME Design to highlight how the predicted pavement performance is affected by varying resilient modulus values. Soil specimens were tested in the laboratory under three different moisture contents, and the corresponding resilient modulus values were used during the pavement analysis and performance prediction step to highlight the importance of testing the soils at accurate moisture contents.

Finally, this master's thesis effort also focused on studying the permanent deformation (plastic strain) characteristics of the soils under repeated loading. A new permanent deformation testing protocol was developed, and test results were used to highlight why it is important to study both the resilient and plastic strain properties of subgrade soils for design and construction of economical, well-performing flexible pavement sections.

Research Question and Hypothesis

The research performed towards completion of the current master's thesis was aimed at answering the following research questions:

1. Will results from pavement analysis and performance prediction using AASHTOWare® Pavement ME Design change significantly if assumed

standardized soil properties are replaced by those specifically determined in the laboratory?

2. Can a simplified permanent deformation testing protocol be developed in the laboratory to correlate with rutting patterns predicted using AASHTOWare® Pavement ME Design?

The overarching research hypothesis for this master's thesis was: *"better pavement performance prediction can be achieved by using locally calibrated soil properties"*.

Details of the tasks carried out to answer the above-listed research questions, and the corresponding results have been reported in this master's thesis in the form of two independent manuscripts that constitute chapters 2 and 3 of this thesis. As the last subsection of the current chapter, a brief overview of resilient behavior of unbound materials under loading has been presented. This discussion will present the background necessary for the two manuscripts included as chapters 2 and 3.

Resilient Behavior of Unbound Materials – A Brief Overview

A detailed understanding of unbound material behavior under loading is required to facilitate the implementation of mechanistic-empirical pavement design practices. Inadequate design and construction of unbound layers such as aggregate base/subbase layers as well as the subgrade layer can adversely affect pavement performance. Pavement materials experience a stress pulse which has normal, horizontal and shear stress components due to moving nature of the wheel load (Lekarp et al. 2000). Figure 1-1 shows the stress states applied to a typical soil element within the pavement structure upon the passage of a wheel load (Lekarp et al. 2000). The unbound layers experience positive vertical and horizontal stresses whereas the shear stress turns negative from positive as the load passes over the element. This reversal nature of shear stress leads to complex load deformation behavior because of principal axis rotation.

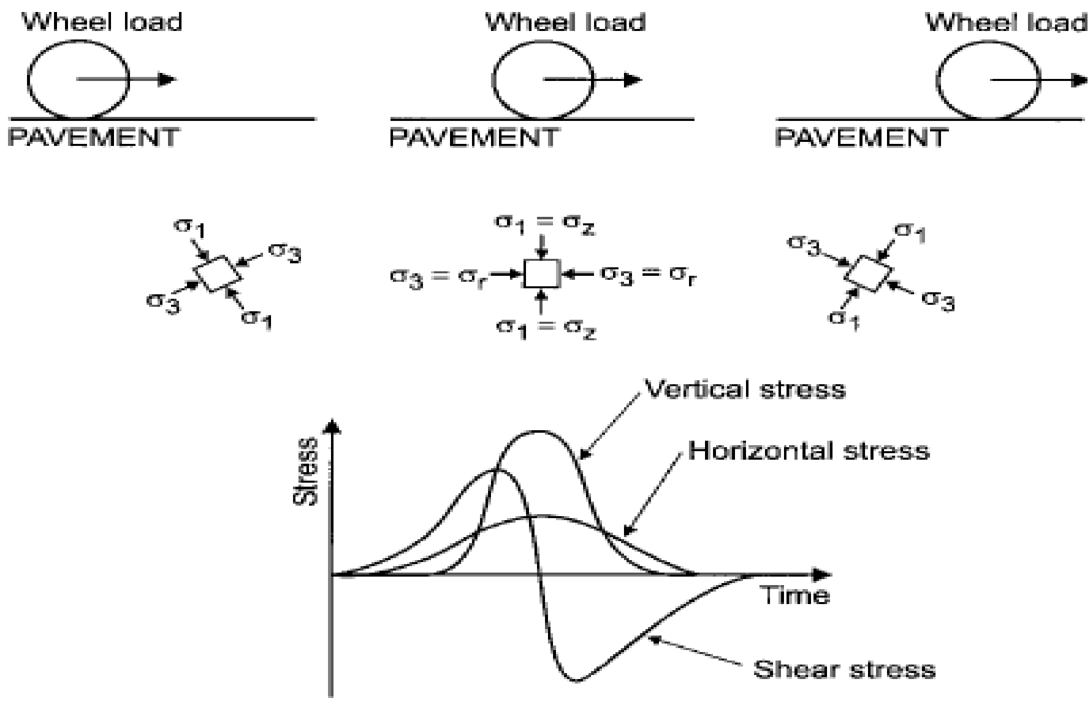

Figure 1-1 Schematic of moving wheel loads induced stress state in pavement material (Lekarp et al., 2000)

Unbound materials subjected to traffic-induced stress pulses experience both elastic and plastic deformations which are commonly known as ‘resilient’ and ‘permanent’ deformations in pavement applications, respectively. The response of a typical unbound material after being subjected to one cycle of pulse loading is shown in Figure 1-2 (Mishra 2012). These resilient and plastic components of total strain depend on several factors such as number and magnitude of traffic load applications, speed of traffic operation, thickness of undelaying and overlaying layers, quality of materials etc. The accumulation of plastic strain gradually decreases with the number of load applications for typical unbound materials during the pavement service life. Generally, well-compacted unbound layers become stable during the construction phase, and all subsequent loadings lead to deformations that are elastic in nature.

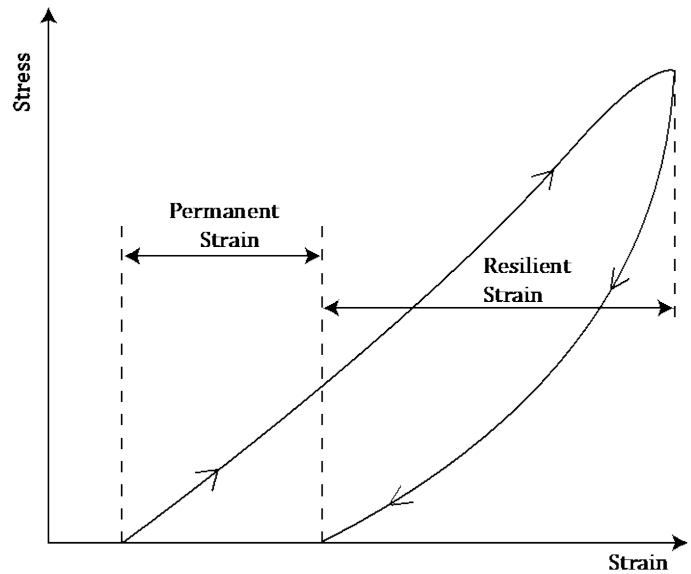

Figure 1-2 Stress strain response of unbound material under one cycle of loading (Mishra, 2012)

Ideally, a well-constructed pavement layer does not accumulate any permanent deformation during repeated traffic loadings which is a key assumption inherent to mechanistic empirical pavement design. Pavement ME considers only elastic response of materials for predicting critical pavement response parameters. Resilient Modulus is a fundamental material property which is defined as the ratio of peak deviatoric stress to recoverable strain under repeated loading. Figure 1-3 shows a typical stress-strain plot to define the concept of resilient modulus. Soil characterization under repeated loading helps to understand material behavior under different stress fields, and facilitates accurate, mechanics-based pavement analysis for pavement response calculation.

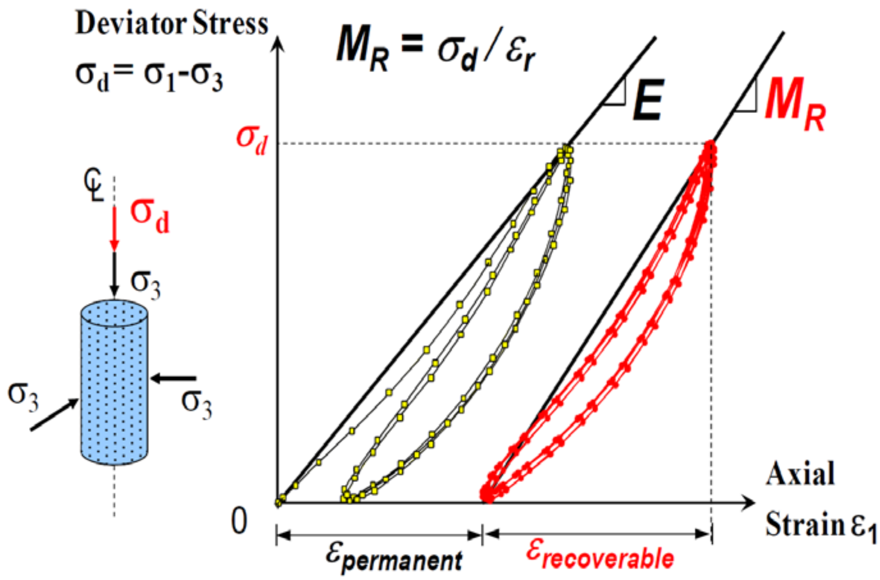

Figure 1-3 Graphical representation for defining Resilient Modulus of unbound materials (Tutumluer, 2013)

Subgrade soils are stress-dependent materials, and the resilient modulus value of a soil depends on the applied stress states. Resilient modulus characterization of soils will help understand how the subgrade layer will behave due to traffic-induced stress pulses. It is also necessary to understand how moisture in subgrade affects resilient properties of soils and the corresponding pavement performances. In the absence of detailed laboratory testing data, state highway agencies often adopt national-level default values to represent the subgrade soil properties during M-E pavement design. Such assumptions may lead to design of pavement structures that are inadequate to sustain the design loads. Considering these issues, a research effort was undertaken as a part of this master's thesis work to accurately characterize subgrade soils in the laboratory to facilitate the implementation of M-E pavement design practices.

Organization of the Thesis

This Master's thesis document contains four chapters. Chapter 2 includes all findings from the first manuscript. The title of the manuscript is, "Laboratory

Characterization of Fine-Grained Soils for Pavement ME Design Implementation in Idaho". Chapter 3 comprises findings reported in second manuscript, titled "Moisture Sensitivity of Typical Idaho Subgrade Soils and their Implication on ME Pavement Design". Summary of results, conclusion and recommendations for future research have been presented in Chapter- 4.

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CHAPTER TWO: MANUSCRIPT ONE - LABORATORY CHARACTERIZATION OF FINE-GRAINED SOILS FOR PAVEMENT ME DESIGN IMPLEMENTATION IN IDAHO

Abstract

The subgrade layer often represents the weakest component of a pavement system and can significantly affect pavement response and performance under loading. Adequate characterization of subgrade properties is critical to the design and construction of long-lasting, economical pavement systems. The mechanistic-empirical pavement design protocol implemented through AASHTOWare® Pavement ME Design requires resilient modulus of the subgrade soil as one of the primary input parameters. However, performing repeated load triaxial tests to establish the resilient modulus properties of soils is a cumbersome task, and is rarely performed by state transportation agencies on a regular basis. In such cases, generic representative values are used by state transportation agencies to specify the subgrade type during the pavement design process. Such simplifications and gross estimations often fail to represent the actual soil types encountered in the field during pavement construction. The Idaho Transportation Department (ITD) has recently invested significant amounts of time and resources to facilitate state-wide implementation of Mechanistic-Empirical (ME) pavement design practices. Under the umbrella of these efforts, a research project was recently undertaken to characterize typical subgrade soil types commonly encountered in the state of Idaho during pavement construction. Soil samples were collected from different zones of Idaho, and extensive laboratory

characterization of the soils was carried out including repeated load triaxial testing for determining the resilient modulus values. One of the objectives was to establish a database of typical soil properties that ITD engineers can use locally during pavement design and performance prediction without having to rely on state-wide generic values. Soil properties established through the laboratory testing effort were subsequently used as input in Pavement ME Design to design typical flexible pavement sections, and to predict their performances under traffic and environmental loading. The primary objective was to quantify how laboratory characterization and establishing actual soil parameters without using state-wide standard values could improve the overall pavement design practice. This chapter, drafted in the form of a stand-alone manuscript, details findings from this project and highlights the importance of such state-wide testing efforts to ensure effective implementation of ME pavement design practices.

Introduction

The subgrade represents the weakest layer in a pavement structure, and is often predominantly made of fine-grained soils. The response of these fine-grained soils under traffic loading needs to be well-understood to facilitate the development and implementation of effective pavement design methods. Subgrade soils, in particular weak subgrades, can undergo significant deformations under repeated loading, thereby causing significant damage to the overlying pavement structures. Plastic deformation accumulations in the subgrade may contribute more than 40% of total rutting in a pavement structure (Majidzadeh et al. 1978) Detailed characterization of the subgrade soil can help in the design and construction of pavement foundations that perform well under traffic and environmental loading. The Resilient Modulus (M_R) is a critical property that governs unbound material behavior under loading, and is a critical input parameter during M-E design of pavements. Resilient modulus represents the "elastic" stiffness of a material under repeated loading once the initial plastic strain deformations have stabilized. This manuscript focuses on resilient modulus characterization of eight different subgrade soil types collected from across the state of Idaho, and evaluates how errors associated with the quantification of subgrade resilient modulus can adversely affect pavement response and performance prediction. Laboratory-established soil properties were used in AASHTOWare Pavement ME design to predict the performances of pavement sections constructed over these tested subgrade soil types. Pavement performance predicted through such analyses were compared against those when state-wide standard values were used instead as subgrade soil inputs.

Factors Affecting Resilient Response of Fine-Grained Soils

Accurate resilient modulus characterization is necessary to model the performance and life span of a given pavement structure (Taylor 2008). Subgrade resilient modulus primarily depends on stress states, soil physical properties and existing moisture conditions (Rahim & George 2004; Oh et al. 2012; Nazzal et al. 2008). Nazzal et al. (2008) studied resilient modulus for four groups of soils at four different moisture levels considering both dry and wet side of optimum moisture contents and observed that resilient modulus decreased by 50% to 70% with increasing moisture content. George (2004) found that moisture content was the most prominent variables for predicting resilient modulus after conducting sensitivity analyses on several models. Yau and Von Quintus (2002) performed statistical analyses to identify soil properties that have significant effects on the resilient modulus of soils. Their analysis showed that optimum moisture content was the most significant variable among other soil physical properties. Mishra et al. (2010) tested aggregate sample at optimum moisture, dry of optimum moisture and wet of optimum to study the effect of moisture contents. The overall effect of moisture on resilient modulus was statistically insignificant at low fine contents. Recent investigation of subgrade soils in Manitoba, Canada found high sensitivity in M_R values because of moisture changes. Those soils showed more than 5% permanent deformation when tested at wet side of optimum moisture content (Soliman & Shalaby, 2010).

Pavement ME Design Implementation and Need for Correlation Development

The current flexible pavement design method adopted by the Idaho Transportation Department (ITD) is an empirical procedure based on the subgrade R-value (Resistance Value); this procedure grossly overestimates the required pavement layer thicknesses (El-Badawy et al. 2011). Over the past few years, ITD has invested significant amounts of time and resources towards state-wide implementation of Mechanistic-Empirical (ME) pavement design practices. This has included projects focused on establishing relevant input parameters for Hot Mix Asphalt (HMA) and Portland Cement Concrete (PCC) materials. As the final component of this material characterization effort, a research study was recently undertaken to characterize representative unbound materials (soils and aggregates) commonly used in the state of Idaho for pavement applications. The primary objective of this study was to generate a database of representative soil and aggregate properties to be used as inputs during M-E pavement design. Another objective of the study was to explore the adequacy of establishing correlations to accurately predict the resilient modulus of subgrade soils from other easy-to-measure index/mechanical properties. This is particularly important because repeated load triaxial tests (required to determine the resilient properties of unbound materials) are complex and time consuming in nature, and it is often not feasible for state highway agencies to perform these tests in the laboratory on a regular basis.

Objective of Study

The overall objective of this chapter is to establish resilient modulus characteristics for different subgrade types commonly found across the state of Idaho, and quantify how variations in resilient modulus properties affects pavement response and performance prediction using AASHTOWare® Pavement ME Design. A supplementary objective was to assess whether statistically significant correlations could be developed to predict the resilient modulus of soils from other easy-to-measure index and mechanical properties. The following tasks were performed towards accomplishing the overall project objectives:

- Perform repeated load triaxial tests following AASHTO T 307 specifications to establish the resilient modulus characteristics of collected subgrade soil types
- Quantify the moisture sensitivity of subgrade soils by measuring the effect of moisture content variations on resilient modulus, California Bearing Ratio (CBR), and Unconfined Compressive Strength (UCS).
- Explore the feasibility of employing soil index properties and other easy-to-measure mechanical properties to accurately predict soils resilient modulus.
- Study the effects of resilient modulus variations on pavement response and performance predictions using AASHTOWare® Pavement ME Design.
- Study the effects of assumed M_R of Idaho soils on critical pavement response parameters, which were subsequently used in pre-establshed damage models to predict flexible pavement life.

Materials and Methods

This manuscript focuses on laboratory test results for eight different subgrade soil materials (termed S_1 through S_8) that were tested in the laboratory. These materials represent subgrade soils collected from different parts of Idaho. Note that although a total of sixteen soil types were tested under the scope of the current research study, this manuscript focuses on only eight soil types in an effort to reduce duplications among similar soil types. The following sub-sections include results from the following laboratory tests performed on the eight soil types: (1) grain size distribution and soil classification; (2) compaction (moisture-density) characteristics; (3) California Bearing Ratio (CBR); (4) Unconfined Compressive Strength (UCS); and (5) Resilient Modulus (M_R) .

Grain Size Distribution

The first task in the laboratory test matrix involved establishing the particle size distribution through sieving and hydrometer analysis. Table 2-1 lists percentage of gravel, sand, silt and clay found in each soil sample from sieve and hydrometer analysis.

Table 2-1 Percentage Gravel, Sand, Silt and Clay Content in Different Types of Soils

Properties	Soil Types							
	S_1	S_2	S_3	S_4	S_5	S_6	S_7	S_8
% gravel (76.2-4.75 mm)	N/A	36.7	N/A	55.3	42.6	N/A	12	3
% sand (4.75-0.075 mm)	79.5	59.9	51.2	32.2	51.7	46.1	56.5	58.9
% fines (<0.075 mm)	20.5	3.4	48.8	12.5	5.7	53.9	31.5	38.1
% silt	17.8	3.1	43.7	10.7	5.1	47.2	27.0	34.9
% clay	2.7	0.3	5.1	1.8	0.6	6.7	4.5	3.2

As seen from the table, soil types S_3 and S_6 have approximately 50% fine particles in their natural particle distributions and soil type S_2 has the lowest amount of fine particles among the eight soil types. Figure 2-1 shows the actual grain size distributions of soils.

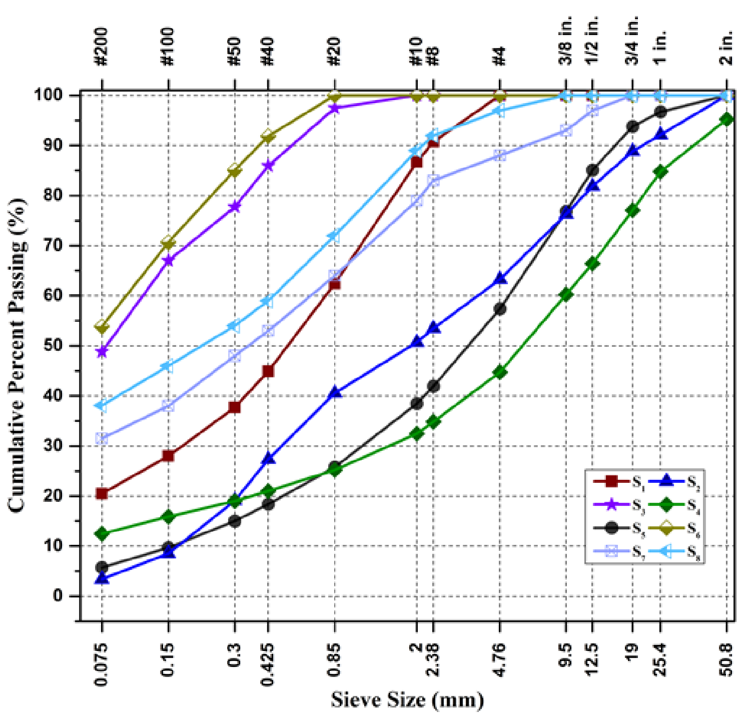


Figure 2-1 Grain size distribution of different soil types

Although the soil samples were collected from the subgrade layers for existing pavement sections, some of the collected soil samples comprised significant amounts of coarse particles; for example, soil types S_2, S_4 and S_5. Note that the primary focus of this component of the study was to characterize the fine-grained subgrade soils encountered across the state of Idaho. Therefore, the research team, after discussions with ITD engineers, decided to extract the fine fractions for the collected soil samples; all testing was carried out on the fine fraction only.

Compaction Characteristics and Soil Classification

Moisture density characteristics for these subgrade soils were established in accordance with the AASHTO T 99 test procedure, and the Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) values were established. The material fraction passing through the 4.75 mm sieve was dried overnight in an oven conducting the moisture-density tests. Commonly used soil index properties such as Atterberg's limits (Liquid Limit, Plastic Limit, and Plasticity Index) were also established. The soil classifications were done according to the Unified Soil Classification System (USCS) and AASHTO methods. Table 2-2 lists the liquid limit, plasticity index, MDD, and OMC values for each of the eight soil types. Additionally, the USCS as well as AASHTO classifications for each soil have also been listed in Table 2-2.

Table 2-2 Soil Index Properties and Compaction Characteristics

Material	Liquid Limit LL	Plasticity Index PI	MDD $\gamma_{d,max}$	OMC ω_{opt}	Soil Classification	
	%	%	kN/m^3	%	USCS	AASHTO
S_1	29	13	17.8	14	SC	A-2-6
S_2	N/A	NP	20.1	8.5	SW	A-1-b
S_3	30	3	16.8	16.5	ML	A-4
S_4	32	3	17.3	15	GM	A-1-a
S_5	19	NP	21.1	8	SW	A-1-a
S_6	26	5	17.2	16.5	CL-ML	A-4
S_7	21	NP	19.3	11	SM	A-2-4
S_8	20	NP	18.5	12	SM	A-4

Unsoaked California Bearing Ratio (CBR) Testing

CBR is an index property that can be linked to the shear strength of unbound materials and was originally developed by the California Division of Highways to quantify the shear strength of unbound materials (Croney & Croney 1991). It is a measure of the resistance that a material presents in response to penetration of a standardized plunger. Although the test is empirical in nature and does not necessarily represent any mechanistic similarity with how the pavement layers are loaded, several states still actively use CBR in their pavement design process. One common way is to use the CBR value to predict the resilient modulus of an unbound material using empirically established equations. Two of the most commonly used correlation equations are:

$$M_R \text{ (psi)} = 1500 \times CBR \text{ for CBR} \leq 10 \text{ (Heukelom and Klomp, 1962)}$$
$$M_R \text{ (psi)} = 2555 \times CBR^{0.64} \text{ (NCHRP 1-37A, 2004)}$$

CBR tests were performed on unsoaked soil specimens following the AASHTO T193 test protocol. Specimens were prepared at three different moisture contents to observe the effects of moisture content on bearing capacity of soil. Specimens compacted at OMC were used as the control specimens. Additionally, two specimens, one at 90% OMC and the other at 110% OMC, were compacted and tested, simulating dry and wet conditions, respectively. The test results have been tabulated in Table 2-3. As seen from the table, all materials exhibited the highest CBR values under dry conditions; a reduction in CBR value was observed when the moisture content increased to OMC, and subsequently to 110% OMC.

Table 2-3 Unsoaked CBR Values for the Eight Soil Materials at Different Moisture Contents

Conditions	California Bearing Ratio (%)							
	S_1	S_2	S_3	S_4	S_5	S_6	S_7	S_8
0.9*OMC	27.5	61.4	23.7	36.1	18.1	16.9	60.4	36.8
OMC	6.7	55.8	5.2	12	12.3	3.4	38.1	14.3
1.10*OMC	3.5	10.5	1.6	2.0	3.3	2.1	9.2	3.2

<u>Resilient Modulus Testing</u>

Repeated Load Triaxial Tests (RLT) were conducted on the different soil types to establish resilient modulus properties; the test procedure specified in AASHTO T307 was followed. Haversine load pulses with 0.1s loading period and 0.9s rest period were applied on top of cylindrical (100 mm diameter x 200 mm height) specimens. Six specimens were prepared and tested for each material, constituting two replicates at the three different moisture contents. For materials where significant differences in the test results were observed between the two replicates, a third specimen was tested, and results from all three specimens were statistically analyzed before reporting an average value. The specimens were prepared in three layers by applying 25 blows of the standard compactive effort hammer. While preparing the specimens for resilient modulus testing, the target density values for compaction were obtained from the moisture-density curves for the respective materials. Vertical deformation of the specimen during repeated load triaxial testing was measured using two externally mounted Linear Variable Differential Transformers (LVDTs). The average value of resilient modulus from the last last five loading cycles within each stress state was reported as the resilient modulus values for that particular stress state. Table 2-4 lists resilient modulus values for the eight soil types under OMC conditions. As already mentioned, two replicates were tested for each soil type. Therefore, the values listed in Table 2-4 represent average of the values obtained from the two replicates.

Table 2-4 Resilient Modulus of Soils at Different Stress States Tested at OMC

Seq.	Confining	Axial	Resilient Modulus of different Materials							
No.	Pressure	Stress	S1	S2	S3	S4	S5	S6	S7	S8
	kPa	kPa	MPa	MPa	MPa	MPa	MPa	MPa	MPa	MPa
1	41.4	12.4	79.6	71.7	39.5	103.5	55.6	45.5	31.9	24.5
2	41.4	24.8	71.2	72.7	35.9	88.5	62.7	41.6	39.2	29.7
3	41.4	37.3	61.4	70.6	32.7	78.4	65.9	35.3	42.1	32.6
4	41.4	49.7	51.7	71.2	31.5	72.3	68.7	29.2	46.4	36.1
5	41.4	62.0	46.5	75.1	34.1	71.0	74.4	27.6	52.7	40.0
6	27.6	12.4	61.7	46.6	21.9	65.1	38.9	29.8	24.2	19.1
7	27.6	24.8	50.9	49.2	20.8	53.2	49.3	24.6	30.3	23.6
8	27.6	37.3	45.2	56.8	23.8	52.7	60.1	23.1	38.4	29.6
9	27.6	49.7	43.2	64.9	27.8	55.4	67.7	23.5	46.6	35.5
10	27.6	62.0	42.8	71.3	32.0	58.2	73.5	25.1	53.5	40.1
11	13.8	12.4	53.9	35.6	16.7	38.3	35.6	25.1	22.1	17.3
12	13.8	27.6	43.8	42.4	18.5	36.3	48.1	20.6	28.9	22.6
13	13.8	41.4	39.2	52.1	21.2	40.3	60.1	19.9	37.2	28.9
14	13.8	55.2	38.1	61.1	26.7	45.6	68.1	21.2	45.9	35.3
15	13.8	68.9	38.5	67.9	28.1	50.3	73.8	23.4	53.3	40.4

Unconfined Compressive Strength

Unconfined compressive strength tests were performed on the eight soil types in an effort to identify whether statistically significant correlations existed between resilient modulus and UCS values. The same specimen used for resilient modulus testing was later tested for UCS. A total of 48 specimens (8 soil types x 3 moisture contents x 2 replicates) were tested for UCS under controlled strain conditions at a rate of 1 mm/minute (as specified in AASHTO T208). The peak axial stress attained for each specimen was recorded, and the average value from the two replicates was reported for a given soil type and moisture condition combination, the results have been tabulated in Table 2-5. As seen from the table, all soils exhibited significant moisture sensitivity with UCS values rapidly decreasing with increasing moisture content.

Table 2-5 Peak Compressive Strength of Soils at Different Moisture Contents

Moisture Conditions	Unconfined Compressive Strength (kPa)							
	S_1	S_2	S_3	S_4	S_5	S_6	S_8	S_9
0.9*OMC	248.9	113.8	202.7	178.6	134.4	220.6	172.4	165.5
OMC	162.7	104.8	155.1	114.5	96.5	166.2	117.2	144.8
1.10*OMC	75.8	82.7	120.7	113.8	48.3	110.3	103.4	51.7

As already mentioned, one of the objectives behind running the UCS tests was to try and establish statistically significant correlations between UCS and M_R test results. Lee et al. (1997), Drumm et al. (1990), and Hossain et al. (2011) reported good correlations between UCS and M_R, and successfully developed correlation equations to predict M_R from UCS test results.

Analysis of Test Results

Resilient Modulus Model Parameters

MEPDG input level-1 requires constitutive model parameters to calculate material responses. The constitutive model used in ME pavement design is widely known as universal model or MEPDG model (AASHTO, 2008). This model can incorporate the effect of both deviatoric and volumetric stress. The required values from repeated load triaxial testing were used in Equation (1) and multiple linear regression analysis was performed to determine model parameters.

$$M_R = k_1 \times p_a \times \left(\frac{\theta}{p_a}\right)^{k_2} \times \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3} \tag{1}$$

where

θ is known as bulk stress or the first stress invariant $(\theta = \sigma_1 + \sigma_2 + \sigma_3)$

σ_1 is the major principal stress

σ_2 is the intermediate principal stress

σ_3 is the minor principal stress

For a cylindrical triaxial test set-up

$\sigma_2 = \sigma_3$

$\sigma_1 = \sigma_3 + \sigma_d$

p_a is atmospheric pressure, and equals 101.3 kPa

The p_a term is used for normalization in the equation

τ_{oct} represents the octahedral shear stress

$$\tau_{oct} = \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}$$

For a cylindrical triaxial test specimen,

$$\tau_{oct} = \frac{\sqrt{2}}{3}(\sigma_1 - \sigma_3) = \frac{\sqrt{2}}{3} \times \sigma_d$$

In Equation (1), k_1 values are always positive, and can be thought to be representative of the Young's modulus of the material. Positive values for the k_2 parameter would mean that increase in the bulk stress leads to stiffening of the material. Similarly, negative values for the k_3 parameter would imply that increase in the shear stress leads to softening of the material. Table 2-6 lists parameters for the eight different soil types after the universal model was fitted to the resilient modulus test results.

Table 2-6 MEPDG Model Parameters for the Eight Soil Materials Corresponding to Three Different Moisture Contents

Material ID	0.9*OMC				OMC				1.10*OMC			
	k_1	k_2	k_3	R^2	k_1	k_2	k_3	R^2	k_1	k_2	k_3	R^2
S_1	1346.5	0.34	-1.93	0.95	767.2	0.52	-3.08	0.90	–	–	–	–
S_2	1182.4	0.69	-1.74	0.95	486.9	0.55	0.53	0.84	283.6	0.26	2.17	0.89
S_3	768.4	0.67	-2.61	0.81	260.6	0.76	-0.72	0.77	–	–	–	–
S_4	1001.4	0.84	-2.72	0.91	744.8	0.99	-2.47	0.89	263.5	0.81	0.07	0.78
S_5	410.2	0.23	2.35	0.92	397.6	0.26	1.94	0.85	–	–	–	–
S_6	555.9	0.57	-2.66	0.86	377.2	0.68	-2.68	0.76	–	–	–	–
S_7	327.4	0.17	1.58	0.94	223.6	0.25	2.78	0.94	190.1	0.18	3.51	0.91
S_8	273.9	0.24	1.25	0.89	175.2	0.24	2.70	0.94	–	–	–	–

The first thing to be noticed from the above table involves the missing model parameter values for several soils when tested under the wet conditions (110% OMC). This was primarily because the specimens were too weak at the high moisture content, and could not sustain the repeated loading during resilient modulus testing. Next, it should be noted that the k_1 and k_2 parameter values were positive for all soil materials under all three moisture contents. The k_1 parameter is always expected to have a positive value because a negative value would indicate a negative modulus for the material, which has no physical meaning. Positive values for the k_2 parameter indicate that all soil materials at all moisture content attained higher modulus values as the bulk stress value was increased. The k_3 model parameter on the other hand, was positive for some soils and negative for some. Although fine-grained soils are expected to exhibit softening behavior under increasing shear stress levels, the variation in the sign of the parameter can be attributed to error minimization techniques during multiple linear regression. Note that the model fitting was carried out with the primary objective of minimizing the error between observed and predicted resilient modulus values, and no physical meaning to the model parameters were assigned during the regression process. No mechanistic justification can be provided for the positive k_3 values obtained for some of the soils.

<u>Effect of Moisture Content and Deviatoric Stress on Soil Resilient Modulus</u>

The effect of changing moisture content on M_R was thoroughly investigated in this study. To aid the discussion of these test results, the eight soils have been classified into two categories: (1) plastic soils; and (2) nonplastic soils. Figure 2-2 (a)-(d) show the variation in resilient modulus for soils S_1, S_3, S_4, and S_6, respectively, with applied deviatoric stress levels. As can be seen from the Atterberg's limit tests presented earlier in this chapter (Table 2-2), these soils exhibited non-zero Plasticity Index (PI) values, thereby being classified as "Plastic". On the other hand, soils S_2, S_5, S_7, and S_8 had zero PI values, thus being classified as "nonplastic"; results for these soils have been presented in Figure 2-3 (a)-(d). Note that the different lines in the plots correspond to different confining pressures, and also to different moisture conditions. Remember that AASHTO T 307 protocol requires each specimen to be subjected to five different deviator stress levels at three different confining pressures, thus resulting in 15 different stress states.

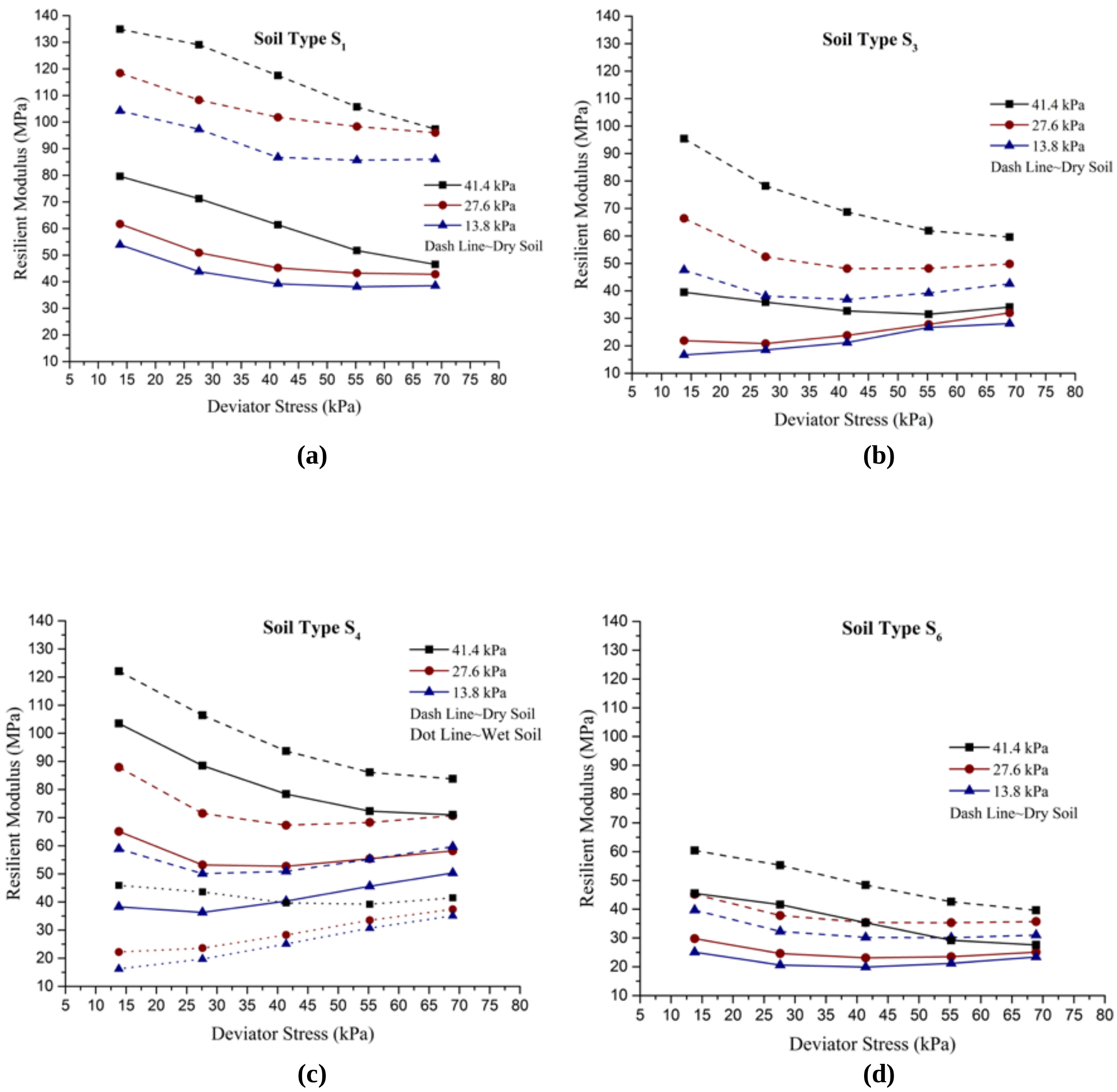


Figure 2-2 Variation of M_R with different moisture content for plastic soils

Careful inspection of the above figure clearly highlights that that moisture content had a significant impact on the M_R values for the soils with non-zero PI values; significant reduction in M_R was observed as the moisture content increased. This trend was consistent across all confining pressure levels. It can also be seen from the plots that as the deviator stress magnitude increased, the M_R values mostly decreased, implying stress-softening behavior. This trend was reversed in certain cases, for example, the wet condition for Soil

S_4 (Figure 2-2c). Increasing M_R with increasing deviator stress implies stress-hardening behavior, and should lead to a positive k_3 parameter for this particular soils. This is confirmed from Table 2-6 (soil S_4 has a positive k_3 parameter under wet conditions). It should also be noted that three of the four plastic soils failed during resilient modulus testing when the specimen was compacted at 110% OMC. This finding agrees with the trend observed by Mishra (2012), where he reported that unbound aggregate specimens with plastic fines exhibited significantly higher moisture sensitivity compared to those with nonplastic fines.

Figures 2-3a through 2-3d present similar results for the remaining four soil types, that were grouped under the ‘nonplastic’ category. As seen from these plots, the soils under the nonplastic category did not exhibit as significant of a drop in resilient modulus with increasing moisture content except for soil S_2. More importantly, three (S_5, S_7, and S_8) of the four soil types exhibited increasing trends in M_R values with increasing deviator stress magnitude. Even for soil S_2, this trend was observed for the wet conditions. This clearly shows that in most cases, the soils with nonplastic fines exhibited stiffening response as the deviator stress magnitude was increased. This is demonstrated by positive values for the k_3 parameter for all these cases except for the case when soil S_2 was tested under dry conditions.

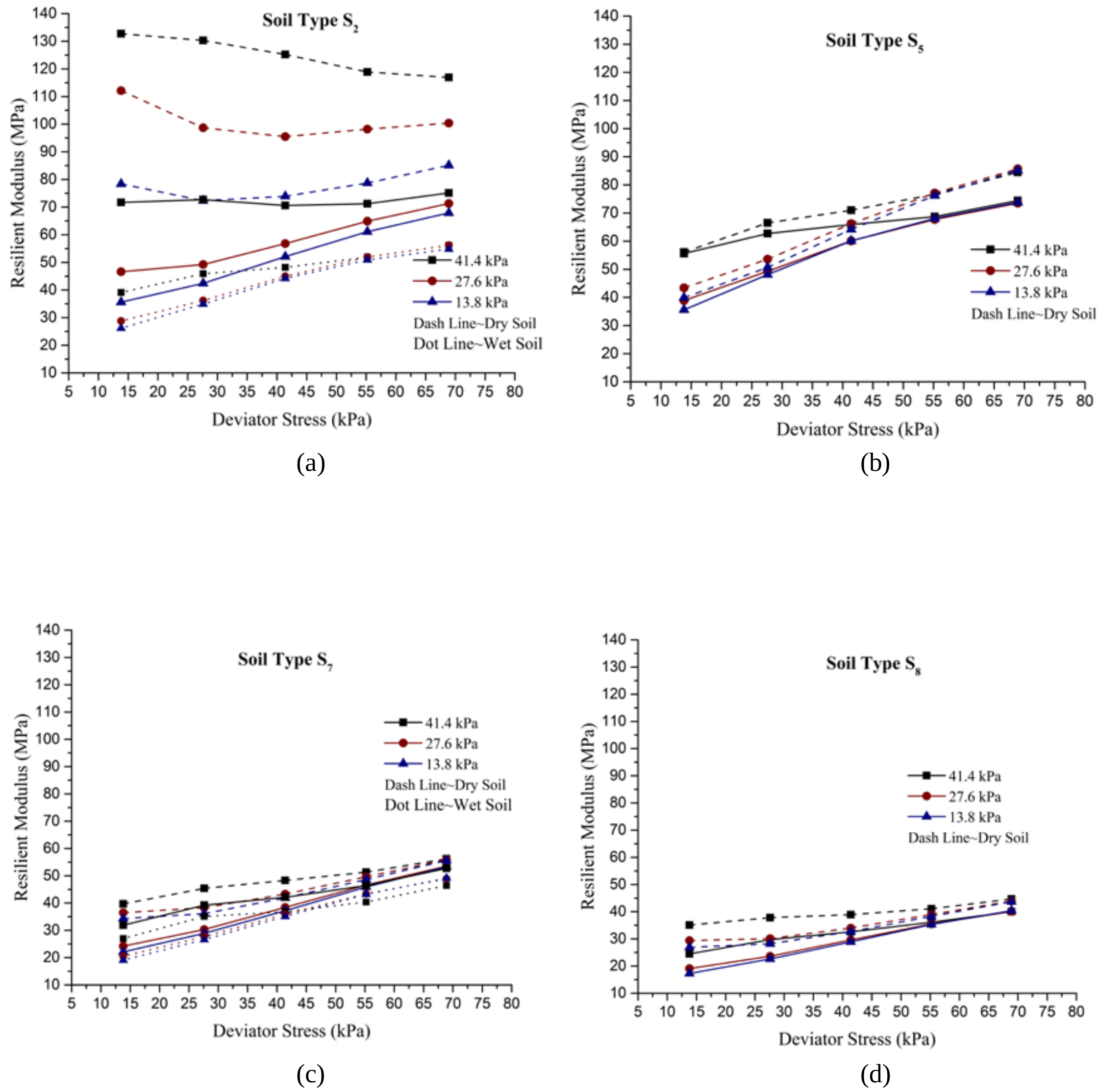


Figure 2-3 Variation of M_R with different moisture content for non-plastic soils

From the above results, it can clearly be seen that although moisture content adversely affects the resilient modulus values for all soil types, that effect is much more pronounced in case of soils with plastic fines.

Predicting Soil Resilient Modulus from Index Properties

As already mentioned, one of the objectives of this research effort was to evaluate whether or not resilient modulus values for subgrade soils could be predicted with reasonable accuracy from easy-to-determine index and mechanical properties. This would help state and local highway agencies acquire relevant subgrade soil input parameters for mechanistic-empirical pavement design without having to run cumbersome and resource-intensive repeated load triaxial tests. The current research study utilized soil gradation information, index properties, compaction characteristics, as well as CBR and UCS values to try and develop correlation equations to predict the resilient modulus value. However, note that resilient modulus is a stress-dependent property and cannot be specified using a constant value for a particular material. Nevertheless, the current version of the AASHTOWare® Pavement ME Design software requires a single value of resilient modulus as input. To help the reader overcome this dilemma, some discussion on how to arrive at a single resilient modulus value for a soil is warranted. This discussion is presented in the following subsection.

Specifying One Resilient Modulus Value for an Unbound Material

Resilient modulus is a stress-dependent parameter and can take up different values depending on the stress state that a particular unbound material (soil or aggregate) is subjected to. A material that is very well-confined, will usually be stiffer compared to a material that is not well-confined. This is particularly relevant in a pavement structure because the confinement levels as well as vehicle-imposed stress levels change spatially within an unbound layer. An element closer to the pavement surface is subjected to much higher stress levels (from vehicular loading) compared to an element further down in the pavement system. This is also true within a particular layer. The vehicle-imposed stresses are much higher closer to the point of loading and dissipate gradually as we move away vertically or laterally. Considering that resilient modulus is highly influenced by the stress conditions, it is expected that the resilient modulus of an unbound layer will exhibit significant spatial variation.

Traditional pavement analysis approaches use the linear elastic theory where each layer, including unbound soil and aggregate layers, is represented by a single modulus value. Although this can be thought of as an over-simplification of actual material behavior, considering spatially variable modulus values within a layer requires the use of computationally expensive approaches such as the finite element method. Although several advanced pavement analysis codes such as ILLI-PAVE (Thompson & Elliott, 1985) and GT-PAVE (Tutumluer, 1995) have been developed with the capability to consider stress-dependent (and hence spatially variable) modulus values for soil and aggregate layers, using such analysis approaches during ME design would render the design and analysis process excessively time consuming. Therefore, the ME analysis and performance prediction approach implemented in AASHTOWare® Pavement ME Design makes use of JULEA, a layered-elastic analysis program developed by Uzan (1976). This in turn requires the user to specify a single modulus value to be used during the design process. Note that this departure from actual soil and aggregate behavior has been deemed to be acceptable because the stress-induced variations in soil and aggregate modulus is not likely to affect the pavement response and performance as significantly as other factors such as variations in HMA layer thickness/modulus and/or variations in climatic conditions.

In an effort to propose a single modulus value that can be used to represent the entire soil/aggregate layer, NCHRP project 1-28 (Barksdale et al. 1997) proposed the concept of ‘Summary Resilient Modulus’. This approach recommended using the resilient modulus value obtained from laboratory testing at 14 kPa confining pressure and 41 kPa cyclic stress as the ‘Summary Resilient Modulus (SRM)’ for fine-grained subgrade soils. This stress state is assumed to be representative of the typical stress field experienced by a subgrade element in a pavement section under vehicular loading. Therefore, the current study used the concept of summary resilient modulus to define one single modulus value for each of the soils tested. Note that the stress states used for defining the SRM for fine-grained soils correspond to stress sequence number 13 during AASHTO T 307 testing.

Correlation Development to Predict Summary Resilient Modulus Values

Once the decision was made to use the SRM as the representative modulus value for each tested soil, the first task involved analyzing the trends in SRM values under different test conditions. A Box-and-Whisker plot was developed for all the SRM values corresponding to different moisture conditions (see Figure 2-4). Note that the horizontal line within the shaded boxes in represents the median value for that particular distribution. The boundaries of the box correspond to the 25th and 75th percentiles, respectively. The straight line beyond the box extend to show the scatter of data for that particular moisture condition.

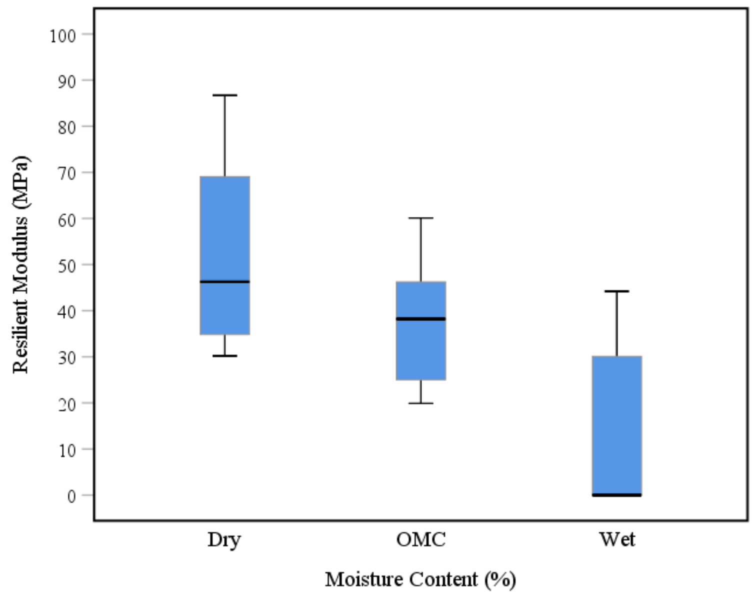


Figure 2-4 Distribution of summary modulus data tested at each moisture content

As seen from the plot, the median SRM value for specimens tested under dry conditions was the highest, whereas the median value corresponding to wet conditions was lowest. Note that the median value for the wet conditions lies at ZERO because several of the specimens (more than 50%) failed during resilient modulus testing, and their SRM value was taken to be zero for this analysis. It is important to note that although the SRM value reduces as the moisture condition is increased from dry condition to OMC, the most significant drop in SRM occurs when the moisture content is increased beyond OMC to wet conditions. This wide variation in SRM value with changing moisture content clearly implies that any equation developed to predict the SRM value must take the moisture content of the specimen into consideration. Extensive statistical analysis of the test data was carried out to explore whether it was possible to develop generalized correlation equations to predict the SRM values at different moisture contents. However, it was observed that no such generalized equation resulted in statistically significant correlations.

Therefore, the discussion in this chapter focuses on the development of correlation equations to predict the SRM when the moisture content equals OMC.

Multiple regression analysis was carried out using the R statistical package (Team, 2013) to develop correlations between SRM and different soil index and mechanical properties. The model equations for predicting SRM are shown in Equation (2) and Equation (3) where independent variables are percent finer than #200 sieve (P#200), compaction moisture content (W), unconfined compressive strength (q_u), and percentage clay present (% clay).

$$M_R = 69.41 - 0.51 \times P\#200 - 1.44 \times W \quad \textbf{(2)}$$

$$M_R = 74.28 - 0.18 \times q_u - 3.99 \times \%Clay \quad \textbf{(3)}$$

Close inspection of the two equations presented above leads to several interesting observatios. For example, the first equation (correlating Mr with ‘P200’ and ‘W’) has negative coefficients associated with the two independent variables. This means, the SRM value decreases as both P200 and W values are increased. This is consistent with commonly observed soil behavior, as both P200 and W have been shown to have negative correlations with resilient modulus. The second equation, on the other hand, proposes relationships that do not agree with engineering observation. For example, both the independent variables (q_u and %clay) in the second equation are associated with negative coefficients. Although reduction in SRM with increasing %clay seems logical, the negative coefficient for q_u has no physical significance. There is no scientific reason for SRM to have a negative correlation with q_u. Therefore, it is quite clear that the above-mentioned equation is merely an outcome of an error minimization process, and does not have any significant mechanistic standing. Therefore, the implementation of such an equation into practice cannot be justified.

Besides attempting to develop correlations equations for predicting the SRM values, the current study also tried to develop correlation equations to predict the MEPDG model (also known as the universal model) parameters (k_1, k_2, and k_3) using the index and other mechanical properties. If the model parameters can be predicted with reasonable accuracy, this would help in calculation of representative resilient modulus values if typical stress states experienced by a soil element in a pavement system can be estimated. The equations for predicting MEPDG model parameters are shown in Equation (4), Equation (5) and Equation (6) below.

$$K_1 = -93.1 - 15.5 \times P\#200 + 73.98 \times W \quad \textbf{(4)}$$

$$K_2 = -0.47 - 0.01 \times P\#200 + 0.10 \times W \quad \textbf{(5)}$$

$$K_3 = 8.95 + 0.10 \times P\#200 - 0.93 \times W \quad \textbf{(6)}$$

Statistical parameters indicating the quality of these regression equations were obtained from R, and have been listed in in **Error! Reference source not found.**. The parameters that were considered to measure the quality of the fit were: residual standard error (RSE), F statistics, model significance in term of P-value and precision of developed model in term of Adjusted R-square.

Table 2-7 Multiple Regression Analysis Output for the Five Models

Models	RSE	DF	F-value	P-value	Adj. R-squared
Equation (2)	3.356	5	59.04	0.00033	0.94
Equation (3)	5.855	5	17.71	0.0054	0.83
Equation (4)	96.86	5	16.48	0.0063	0.82
Equation (5)	0.1464	5	9.62	0.0193	0.71
Equation (6)	1.339	5	9.26	0.0208	0.70

Once again, it is important to closely inspect the equations resulting from regression analysis to assess whether or not they carry any physical meaning. Taking a look at the first equation (to predict k_1), it can be seen that the intercept has a negative value, and the coefficients associated with 'P#200' and 'W' are negative and positive, respectively. This is questionable at several levels. Firstly, based on this equation, if a soil has zero percent of particles passing the #200 sieve (finer than 0.075 mm), and is compacted under completely dry conditions (moisture content close to zero), then the k_1 value will be negative. As the k_1 parameter is supposed to correspond to the Young's modulus of the material, a negative k_1 parameter would mean a negative Young's modulus, which has no meaning whatsoever. Additionally, the equation shows that k_1 value would increase with increasing moisture content; this is also erroneous as increasing the moisture content (particularly on the wet side of OMC) leads to significant weakening of soils. Similar limitations can be easily identified for the equations to predict k_2 and k_3. Therefore, it can be clearly seen that the equations to predict the MEPDG model parameters, though lead to reasonably good statistical significance, have no physical meaning associated with them.

Therefore, using these equations to predict the M_R for soils will lead to erroneous pavement designs.

Effect of Soil Resilient Modulus on Predicted Pavement Performance Using AASHTOWare® Pavement ME Design

The ME pavement design procedure presents the user with the option to perform the analysis at three different levels based on the amount of information available. Level 1 corresponds to the case when all design inputs have been established experimentally. Level 3 on the other hand corresponds to the case when very little information is available regarding the design input data, and nation-wide assumed/default values are used during the design process. Level 2 presents an intermediate scenario as far as the availability of design data is concerned. As far as resilient modulus value of soil is concerned, level 1 corresponds to the case where the user has data from laboratory resilient modulus tests conducted for the particular subgrade soil type under consideration (currently, AASHTOWare® Pavement ME Design does not have the ability to incorporate level 1 inputs for unbound materials). Level 3 on the other extreme corresponds the case where no laboratory test data is available for the soil, and the resilient modulus needs to be assumed based on generic information such as the USCS or AASHTO soil classification. Level 2 lies between these two extremes and represents a case where although M_R test results are not available, data from certain other tests such as CBR, UCS, R-value, etc. are available, and can be used to predict the resilient modulus values through commonly accepted correlation equations.

The previous section clearly proved that no correlation of statistical significance and following commonly observed material behavior trends could be developed to predict soil resilient modulus from index and other commonly measured mechanical properties. The next task involved evaluating the effect of soil resilient modulus on pavement performance prediction using Pavement ME Design. The objective was to compare predicted pavement performance when laboratory-measured soil properties are used as design inputs in place of commonly made assumptions. If the difference in predicted pavement performance is found to be significant, then state highway agencies would be recommended to somehow compile a database with actual resilient modulus test data for typical soil types rather than using previously established correlation equations.

This task was completed by randomly selecting a state highway in Idaho for case study. Traffic and other relevant inputs required by AASHTOWare® Pavement ME Design (version 2.3.1) were obtained from the User's Guide developed for ITD by Mallela et al. (2014). A conventional flexible pavement section was designed comprising HMA, base, subbase, and subgrade layers. Two different designs were performed using two differen types of subgrade inputs. The first design assumed no first-hand information about the subgrade properties and used the default values recommended by Mallela et al. (2014). The second design used the test data from the current laboratory testing effort. Results from the two cases have been discussed in the following sections. Note that during the discussions, the first design has been referred to as "Recommended Subgrade" indicating that this design alternative uses subgrade properties as recommended by the User's guide developed for ITD by Mallela et al. (2014); the second design has been termed as the "Tested Subgrade". The traffic data was obtained from ITD's Weigh-in-Motion (WIM) records. Note that material properties for the HMA, Base, and subbase layers, as well as traffic related inputs were kept unchanged between the two cases; the only difference was in the subgrade properties.

At first, suitable layer thicknesses were established by using all inputs as recommended by the User's Guide (Mallela et al. 2014). While converging on the layer thickness values, it was ensured that the resulting pavement section did not fail in any category during its design life (for the specified design reliability level). Minimum required layer thicknesses obtained from this approach have been marked in Figure 2-5.

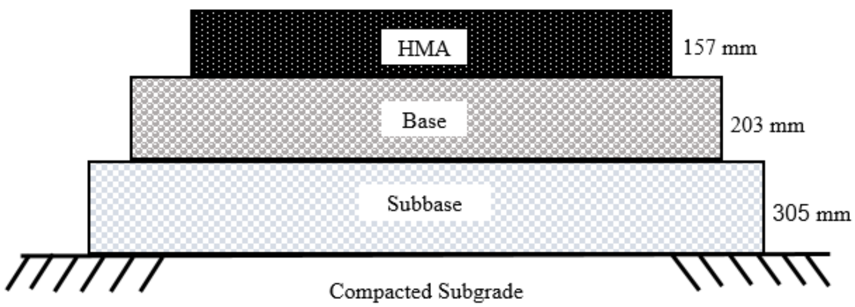


Figure 2-5 Minimum required layer thicknesses while using all ITD-recommended values as material inputs.

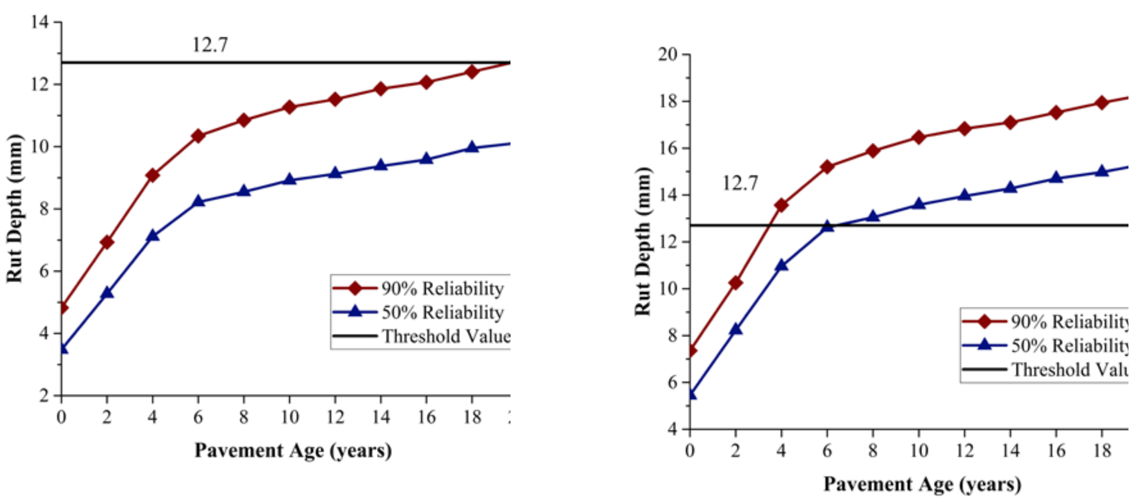
Once the minimum required layer thicknesses were established, the analysis and performance prediction process was repeated by replacing the ITD-recommended material property values for the subgrade with those obtained from the laboratory testing carried out in this study. Note that all other aspect of the pavement structure (layer thicknesses as well as loading conditions, etc.) were unchanged from the first case. Performance predictions for the two design cases have been compared in Table 2-8.

Table 2-8 Comparison of Pavement Performances Obtained from Pavement ME.

	Target values at 90% reliability	Recommended Subgrade		Tested Subgrade	
		Predicted	% Reliability	Predicted	% Reliability
Terminal IRI (mm/km)	2762	1850	99.99	2020	99.89
Total Permanent Deformation (mm)	12.7	12.7	90.41	18.5	12.68
AC bottom up fatigue cracking (% lane area)	15	1.45	100	1.45	100
AC thermal cracking (m/km)	284	51.2	100	51.2	100
AC top-down fatigue cracking (m/km)	947	57.2	100	55.5	100
AC-permanent deformation (mm)	12.7	2.54	100	2.54	100

As seen from the above table, several of the performance criteria such as: Terminal IRI, AC bottom up and top down fatigue cracking, AC thermal cracking and AC permanent deformation were within target reliability levels for pavement structures built using both ITD-recommended as well as laboratory-established subgrade soil properties. However, it can be seen that the total predicted permanent deformation value changed significantly (from 12.7 mm to 18.5 mm) when the assumed soil properties were replaced with those established through laboratory testing. The limiting criterion for total permanent deformation was met in the first design case, whereas it failed for the second design case. Note that the Guide-recommended subgrade soil modulus values was 103 MPa, whereas a value of 40 MPa was used as the Summary Resilient Modulus (SRM) from the laboratory testing results. It is quite evident that using a lower modulus value for the subgrade soil resulted in a significantly higher predicted value for the total permanent deformation.

Figure 2-6 shows the accumulation in total permanent deformation with pavement age for the two design cases. The pavement structure using ITD-recommended values as inputs (design case 1) did not exceed the limiting value (12.7 mm) for total permanent deformation accumulation during the pavement's lifetime. On the other hand, the pavement structure with laboratory-established soil input properties (design case 2) exceeded the limiting value for total permanent deformation accumulation within three years of construction (for 90% design reliability). Even for the case with 50% design reliability, the limiting value for total permanent deformation accumulation is exceeded within six years of construction.



(a) (b)

Figure 2-6 Predicted rutting for (a) recommended subgrade (b) tested subgrade

Effect of Soil Resilient Modulus on Critical Pavement Response Parameters

The pavement performance prediction approach inherent to M-E design relies on complex transfer functions that use relevant pavement responses, such as stresses, strains, deflections, etc., along with several other variables to predict pavement performance. Note that the nature of these transfer functions is often not evident to the user. Therefore, this study directly compared critical pavement response parameters for the different pavement sections to discuss how the performances are likely to be affected. Critical pavement response parameters for different pavement sections can be directly compared to categorically isolate the effects of certain factors of interest on pavement behavior. The critical pavement response parameters considered were: (1) Horizontal strain at bottom of HMA layer (HStrn-HMA); (2) deviatoric stress on top of the subgrade (DStrss-Sub); and (3) vertical strain on top of the subgrade (VStrn-Sub). Simple damage models such as the ones proposed by the Asphalt Institute would help translate the critical pavement response parameters into performance (in terms of number cycles to failure).

Note that AASHTOWare® Pavement ME Design is based on JULEA (Jacob Uzan Layered Elastic Analysis) as the background analysis engine, and is therefore based on the multi-layer linear elastic theory. In its current version, the software does not account for stress-dependent unbound material behavior (Ceylan & Gopalakrishnan 2011).

To quantify how resilient modulus value affects critical pavement response parameter calculations, the current study used the commonly used layered elastic analysis program: KENLAYER (Huang, 1993). The pavement section shown in Figure 2-5 was used for this analysis, and the input parameters were kept similar to those used during the Pavement ME Design runs. The analysis case that used ITD-recommended subgrade soil properties has been referred to here as the 'ITD recommended pavement section'. The alternate analysis case used laboratory-generated subgrade soil properties and is referred to as the 'Tested subgrade' case. A standard wheel load with 552 kPa contact pressure and 150 mm radius was applied from top of the HMA layer. The critical pavement response parameter values were extracted at different points within the pavement structure and have been plotted in Figure 2-7.

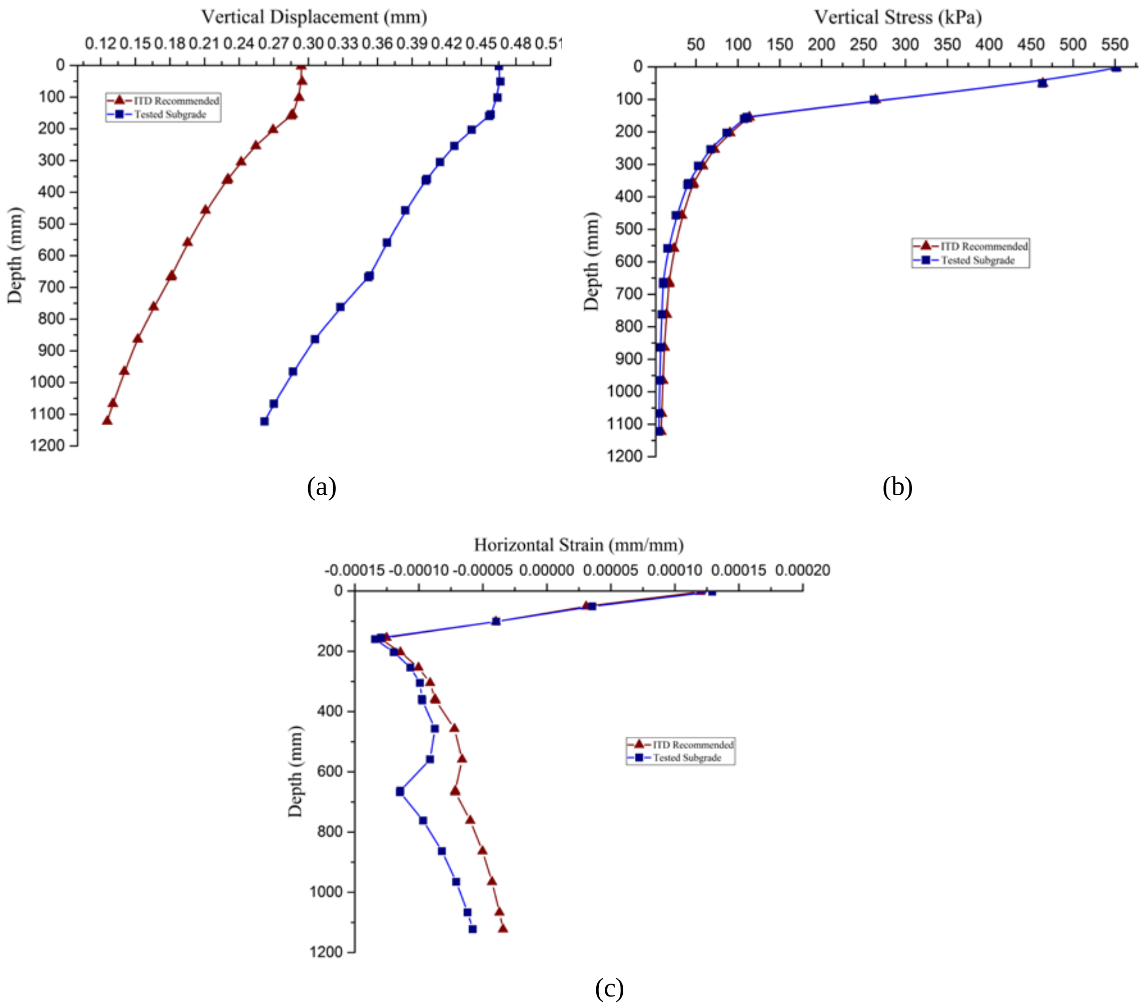


Figure 2-7 Variation in predicted pavement performances; a) vertical displacement b) vertical stress and c) horizontal strain with depth starting from top of pavement.

Remember that the ITD-recommended subgrade case was assigned a higher resilient modulus values compared to the tested case. Accordingly, all layers in ITD-recommended pavement structure had lower vertical deformation magnitudes compared to the tested case. The vertical stress magnitudes were found to be relatively similar for the two cases. The horizontal strain at the bottom of the asphalt layer was found to be quite similar for the two cases. Note that Figure 2-8c shows tensile strain in the unbound layers, which is not possible in the real world. This is nothing but an outcome of layered-elastic analysis approach, which treats every layer as a continuum with equal modulus in compression as well as tension. From the above figure, it can clearly be seen that changing the subgrade modulus is likely to primarily affect the rut accumulation in the pavement (a function of the vertical strain/displacement. Other distresses such as fatigue cracking are not likely to be affected significantly by changes in subgrade modulus.

Use of Simple Damage Models to Compare Pavement Life

Once the critical pavement response parameters were calculated for the two alternate designs, the next step involved using simple damage models proposed by the asphalt institute to compare the number of cycles to failure for these pavement sections under fatigue and rutting criteria. The horizontal tensile strain at the bottom of HMA layer was used in fatigue cracking model shown in Equation (7) to predict number of load repetitions. Similarly, vertical compressive strain at the top of subgrade were used in Equation (8) to predict number of load repetitions. Both of the equations were proposed by Asphalt Institute for damages analysis.

$$N_f = 0.0796(\in_t)^{-3.291} \left|E^*\right|^{-0.854} \tag{7}$$

$$N_d = 1.365 \times 10^{-9} (\in_c)^{-4.477} \tag{8}$$

Table 2-9 lists the critical pavement response parameters and the corresponding load repetitions to failure calculated using the damage models for rutting and fatigue cracking.

Table 2-9 Sensitivity of Subgrade Resilient Modulus in Damage Predictions

Subgrade Types	Subgrade soil Modulus MPa	Tensile strain $\in_t$	Vertical strain $\in_c$	Load repetitions N_f	Load repetitions N_d
ITD Recommended	103	-1.25×10^{-4}	1.757×10^{-4}	5.35×10^{6}	88.56×10^{6}
Tested Subgrade	40	-1.295×10^{-4}	2.794×10^{-4}	4.765×10^{6}	11.10×10^{6}

Results from the rutting model calculations show that ITD-assumed subgrade modulus allows 8 times greater number of load repetitions compared to the tested subgrade case. Similarly, calculations from the fatigue cracking model estimated 12.3% higher number of load repetitions before failure for the pavement with assumed soil modulus properties. These observations clearly highlight the importance of laboratory testing to establish resilient modulus properties of subgrade soils rather than using nationally assumed standard values.

Effect of Moisture-Induced Subgrade Resilient Modulus Variation on Pavement Performance Prediction

As already mentioned, the subgrade soils in the current study were tested under three different moisture conditions (90% of OMC, OMC, and 110% of OMC) to evaluate their moisture sensitivity. The next task in the pavement analysis and performance prediction using AASHTOWare® Pavement ME Design involved determining minimum required pavement layer thicknesses to ensure adequate pavement performance when the subgrade moisture content varied between these extremes. These variations in subgrade moisture can be due to seasonal effects, or due to inadequate drainage conditions that cause unexpected ingress of water into the pavement structure. As is common practice, the User's Guide developed for ITD by Mallela et al. (2014) recommends using subgrade properties under OMC conditions. However, it is quite possible for the subgrade moisture condition to deviate significantly from OMC conditions, which would lead to different required pavement layer thicknesses. The following paragraphs present results from this analysis.

Subgrade soil at OMC conditions was treated as the control case in this analysis. Two other design cases, one with the subgrade moisture content set to 90% of OMC, and the other at 110% of OMC were considered, and the minimum required pavement layer thicknesses were compared. Note that properties for the HMA, base, and subbase layers, as well as the traffic inputs were kept unchanged for all three cases. Required pavement layer thicknesses for the three different subgrade moisture conditions have been listed in Table 2-10.

Table 2-10 Required Layer Thickness of Pavement Built in Tested Subgrade

Layer type	Layer Thickness (mm)		
	Control	Dry	Wet
HMA	190	173	216
Base	254	203	254
Subbase	305	305	305

From the above table it can clearly be seen that when the subgrade moisture condition was set to "wet", the HMA layer thickness needed to be increased (from 190 mm to 216 mm) to satisfy the preset performance criteria. On the other hand, when the subgrade moisture condition was set to "dry" reductions in HMA layer (from 190 mm to 173) as well as base layer (from 254 mm to 203 mm) thicknesses could be accommodated while still meeting all performance criteria. This clearly highlights the importance of preventing moisture ingress into the subgrade layer if adequate pavement performance is to be maintained.

The next step in this analysis involved studying the case where the pavement section has been designed assuming OMC conditions for the subgrade, but a significant fluctuation in the moisture content occurs during the design life of the pavement. To study this scenario, all layer thicknesses were kept constant at the values for the control case (subgrade moisture at OMC); only the subgrade modulus value was altered to simulate moisture content variations. Figure 2-8 shows the variation of IRI (Figure 2-7a) and Rut Depth (Figure 2-7b) with age for a flexible pavement structure built on subgrade soils compacted at three different moisture conditions.

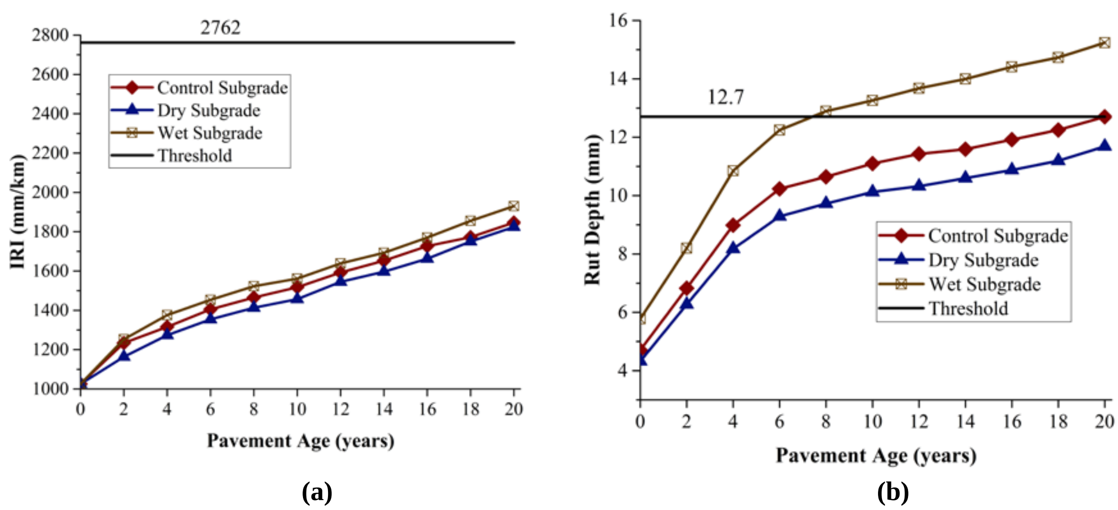


Figure 2-8 Effect of subgrade moisture on a) IRI and b) rut depth of flexible pavement

Higher moisture contents result in lower modulus values which ultimately lead to higher IRI values. Among the three moisture conditions being analyzed, the pavement section with dry subgrade corresponded to the lowest predicted IRI value. Although the IRI values predicted for pavement sections with OMC and wet subgrade conditions were higher than that for the one with dry subgrade, it should be noted that all three pavement sections passed the IRI performance criteria. As seen from Figure 2-7b, the pavement structure built over wet subgrade exceeded the pre-established threshold value for rutting within seven years after construction; on the other hand, the pavement section with dry subgrade did not reach the threshold life till the end of its design life. Excess moisture in the subgrade soil makes it weaker which results in higher permanent deformation accumulations under traffic loading. From the analysis results it was clear that rut depth predictions changed significantly with changing subgrade moisture conditions, whereas predictions for several other performance indicators such as bottom up cracking and thermal cracking remained unchanged. Note that the effect of moisture content on IRI, although noticeable, was not as significant as that for rut accumulation.

Effect of Soil Moisture on Critical Pavement Response Parameters – Results from Finite Element Analysis

The above section discussed the effects of changing soil moisture conditions on pavement performance prediction using AASHTOWare® Pavement ME Design. However, as already mentioned, rather than comparing the predicted performances, sometimes a better understanding of pavement behavior can be obtained by comparing the critical pavement response parameters. This was accomplished in the current study by analyzing the pavement sections using ILLI-PAVE, a finite element-based pavement analysis program that is capable of considering stress-dependent unbound material behavior. Developed at the University of Illinois by Thompson and Elliott (1985), ILLI-PAVE considers stress hardening behavior of aggregates and stress softening behavior of fine-grained soils. The ILLI-PAVE analyses carried out in this study would help determine whether ignoring the stress-dependent behavior of unbound materials is likely to affect the performance predictions significantly or not.

ILLI-PAVE 2005 (version 5.0) finite element tool uses deviator stress dependent bilinear approximation for subgrade soils, and and the K-θ model for unbound aggregates to account for stress-dependent material behavior (Tutumluer 1995). The laboratory-generated resilient modulus test results were fitted with adequate models, and the model parameters were used as inputs for ILLI-PAVE analysis. Three different sets of resilient modulus modulus parameters were considered for the subgrade soil corresponding to the three different moisture conditions. The pavement section with subgrade corresponding to OMC conditions was treatd as the 'control section'. Values for the critical pavement response parameters were extracted from the ILLI-PAVE output, and have been listed in Table 2-11.

Table 2-11 Predicted Pavement Responses Found from ILLIPAVE Stress Dependency Tool

Critical Responses	Control Section		Dry Subgrade		Wet subgrade	
	Values	Depth mm	Values	Depth mm	Values	Depth mm
HStrn-HMA	137.6×10^{-6}	216	131.9×10^{-6}	216	151.6×10^{-6}	216
DStrss-Sub (kPa)	13.1	483	13.8	483	9.6	483
VStrn-Sub	341.5×10^{-6}	483	259.2×10^{-6}	483	551.9×10^{-6}	483

*HStrn-HMA: Horizontal Strain at the bottom of the asphalt layer

**DStrss-Sub: Deviator Stress at the top of the subgrade

***VStrn-Sub: Vertical Strain on top of the subgrade

As seen from the table, the subgrade soil under dry conditions leads to lower tensile strain values at the bottom of HMA layer and lower vertical strain on top of soil surface. It can clearly be seen that the change in critical pavement response parameter magnitudes is not as drastic when the moisture conditions are changed from OMC (control) to dry conditions compared to the case when moisture content changes from OMC to wet condition. The wet subgrade condition results in the highest (elastic) vertical strain on top of the subgrade, which in turn can be linked to the highest rut susceptibility.

Summary and Conclusions

The manuscript focused on laboratory characterization of typical subgrade soil types encountered in the state of Idaho to facilitate state-wide implementation of M-E pavement design practices. Eight different soil materials were collected from different parts of Idaho and were tested in the laboratory. Preliminary laboratory tests such as grain size distribution, moisture density relationship, Atterberg's limit testing, and CBR were first carried out before the performance of resilient modulus and UCS tests at three different moisture contents. Extensive statistical analysis was performed to try and establish correlation equations that could predict the resilient modulus value of a particular soil from other easy-to-measure index and mechanical properties. Later this investigation was extended to evaluate the effects of changes in soil moisture as well as assumed default resilient modulus properties in predicted pavement performance.

The following conclusions are drawn from this study:

- Moisture content variation has a significant effect on the resilient modulus and shear strength of plastic soils; increasing moisture content results in rapid reduction in modulus and strength properties. The effects of these variations on nonplastic soils was not as clearly evident.

- Most of the soil materials tested exhibited highest strength and modulus properties when tested on the dry side of OMC. Several soils failed to withstand the stress levels applied during resilient modulus testing, highlighting the importance of minimizing moisture content fluctuations in the subgrade during the service life of a pavement.
- Correlation equations developed to predict summary resilient modulus or MEPDG model parameter values did not have any physical meaning. Although some of the equations gave reasonably good coefficient of determination (R^2) values, inconsistencies with commonly observed engineering trends means these equations should not be implemented into practice.
- Total permanent deformation (rutting) was the controlling pavement performance criterion from pavement analysis and performance prediction using AASHTOWare® Pavement ME Design. Laboratory-established resilient modulus values were consistently lower than the default values used by ITD; this led to significant over-prediction in pavement service life when subgrade soil properties were assumed instead of being established through laboratory testing.
- Linear elastic analysis of typical pavement sections showed that changing the subgrade resilient modulus primarily affected the vertical (elastic) deformation of the pavement surface, and can be directly linked to higher rut accumulation in the subgrade. On the other hand, changes in subgrade resilient modulus did not affect the horizontal strain at the bottom of the asphalt layer, therefore, not impacting the fatigue service life of the pavement significantly.

- Findings from finite element analysis also gave evidence in favor of the previous statement that vertical strain at the top of subgrade changed drastically when moisture level was changed from OMC to wet of OMC conditions.

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CHAPTER THREE: MANUSCRIPT TWO – MOISTURE SENSITIVITY OF TYPICAL IDAHO SUBGRADE SOILS AND THEIR IMPLICATION ON M-E PAVEMENT DESIGN

Abstract

Subgrade plays an important role in the pavement structure, and mostly contains fine-grained soils. The moisture sensitivity of subgrade soil has drawn the attention of several researchers, and has led them to study soil behavior at different moisture contents. This research study focused on two different soil types collected form two different regions in the state of Idaho. Special care was taken while selecting the soils to ensure that the soils were from geographic regions with significantly different annual precipitation trends. California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS) tests carried out on the two soils indicated very high degree of moisture sensitivity, highlighted the importance of preventing excessive moisture ingress into the subgrade. The recently developed Mechanistic Empirical Pavement Design method considers resilient response of soils to predict pavement performance under traffic and environmental loading; this necessitates determination of resilient modulus of the soils. Both the soils experienced significant drop in resilient modulus with increasing moisture content, with one of the soils accumulating more than 5% strain during conditioning cycles. Among different pavement performance indicators, the M-E design method predicts subgrade rutting using resilient modulus and several other index properties. Resilient modulus properties for unbound materials are usually determined in the laboratory using standard test protocols such as

AASHTO T 307; these test protocols only subject the specimen to limited number of repeated load cycles. This is not representative of the loading scenario in actual pavement sections, were the subgrade may be subjected to millions of load cycles. To accurately characterize the permanent deformation potential of soils, a new test protocol was developed in this study. Stress states to be applied during the permanent deformation testing were first determined through analysis of typical pavement sections under loading. Repeated load triaxial tests were performed on the soil specimens by applying the pre-determined stress levels. Each specimen was tested by applying 10,000 load cycles. Plastic strain potential of both soils increased significantly when the moisture content was increased beyond the Optimum Moisture Content (OMC). Soil specimens prepared on the dry side of OMC, on the other hand, performed significantly better with no permanent strain accumulation beyond the initial few cycles. Significant change in soil behavior under loading highlighted the importance of considering different moisture contents and taking special measures to prevent excessive ingress of moisture into the subgrade during pavement design and construction, respectively.

Key Words: Subgrade, California Bearing Ratio, Unconfined Compressive Strength, Mechanistic Empirical Pavement Design, Resilient Modulus, Permanent Deformation, Moisture Sensitivity

Introduction

Subgrade is considered to be the foundation for a pavement structure, and requires adequate characterization for the overlying pavement structure to perform satisfactorily under vehicular and environmental loading. Generally, pavement structures are built on the existing soil at a site unless special circumstances require its replacement with borrow material. As subgrade soils are generally fine grained, they are often susceptible to moisture related failure. Standard construction specifications require the subgrade soil to be compacted to target density levels (as a percentage of the Maximum Dry Density of MDD) at or near the Optimum Moisture Content (OMC). The OMC and MDD values are established through laboratory testing where compaction curves are developed using the standard or modified compactive efforts. Therefore, most laboratory tests aimed at establishing strength and modulus properties of subgrade soils use the OMC as the standard moisture content for specimen preparation. However, establishing all relevant soil properties at OMC may not present the full picture regarding soil behavior under varying moisture contents.

The recently developed Mechanistic Empirical (M-E) Pavement Design approach implemented through AASHTOWare® Pavement ME Design requires subgrade resilient modulus as the primary input to define its behavior under loading. As the resilient modulus value is input during M-E pavement design as a single parameter, the test is often carried out in the laboratory under OMC conditions. The Enhanced-Integrated Climatic Model (EICM) inherent to Pavement ME Design then modifies the specified subgrade resilient modulus based on changes in soil moisture and temperature. However, this variation incorporated through the EICM may not adequately capture the moisture sensitivity exhibited by subgrade soils. Resilient modulus has been shown by researchers in the past to exhibit high level of moisture sensitivity (George, 2004). Nazzal et al. (2008) observed 50% to & 70% decrease in resilient modulus with increasing moisture content after studying four groups of soils at different moisture contents. Subgrade soils of Manitoba, Canada were tested at wet side of optimum moisture content and it was found that specimens often accumulated greater than 5% plastic strain during resilient modulus testing (Soliman and Shalaby 2010). These previous studies indicated the significance of considering moisture variation during repeated load testing of subgrade soils.

Besides being used to for resilient modulus determination, repeated load triaxial testing is also used to characterize the permanent deformation properties of unbound materials. Note that the rutting potential of unbound materials can be evaluated through this test; rutting is considered to be one of the controlling pavement performance indicators during M-E design. Note that this is more significant for flexible pavement structures compared to rigid pavements because of the difference in which loads are dissipated in the two pavement types. Excessive subgrade rutting can significantly reduce the service life of a pavement (Monismith et al. 1972; Barksdale 1972). Majidzadeh et al. (1978) reported that weak subgrade soils were generally susceptible to rutting, and can contribute more than 40% towards total pavement rutting. The M-E pavement design approach currently uses soil resilient modulus and index properties to make predictions regarding the permanent deformation of soils; this may not represent the best approach because resilient modulus is an indicator of the “elastic” behavior of a soil, whereas permanent deformation accumulation is a “plastic” phenomenon. Puppala et al (1996) found that fine grained soils such as silty clay and silty sand showed good resilient modulus, but experienced significant rutting under traffic loading during their service lives. It is therefore necessary to characterize both resilient as well as plastic deformation properties of subgrades to facilitate the design and construction of well-performing pavement structures. Adequate information concerning the permanent deformation behavior of soils can help in the determination of suitable pavement layer thicknesses to minimize excessive rut accumulation under loading (Puppala et al, 1999).

A research study was recently undertaken in collaboration with the Idaho Transportation Department (ITD) to evaluate the permanent deformation characteristics of some of the common soil types used in pavement applications across the state of Idaho. One of the primary goals was to evaluate the moisture sensitivity of these soils, and quantify how the resilient modulus and permanent deformation properties change with changes in moisture content. These investigations would help transportation agencies to make important decisions for implementing M-E pavement design practices, and would encourage them to develop state-specific guidelines and practices to restrict rut accumulation in the subgrade within allowable limits. Tasks undertaken under the scope of this research effort to accomplish the overall objective have been listed:

1) Identify regions in the state of Idaho that experience significantly different precipitation amounts throughout the year, and collect soils from these two regions;
2) Conduct laboratory tests to characterize the resilient modulus, shear strength, and permanent deformation properties, and study the effect of moisture content variation on these properties;

3) Develop a new testing protocol to characterize the permanent deformation behavior of fine-grained soils. Use this new test protocol to study the effect of moisture content variation on the susceptibility of fine-grained soils to rutting.

Material Selection

As already mentioned, the primary objective during the material selection phase was to identify two locations in the state of Idaho that receive significantly different annual precipitations, and collect soil samples to study how the differences in soil moisture content may affect pavement performance. A precipitation map for Idaho was prepared to aid identification of the candidate locations for soil sampling. Precipitation data was collected from Idaho State Climate Service (ISCS) and then the annual average precipitations were plotted on a map (see Figure 3-1) using the ArcGIS software.

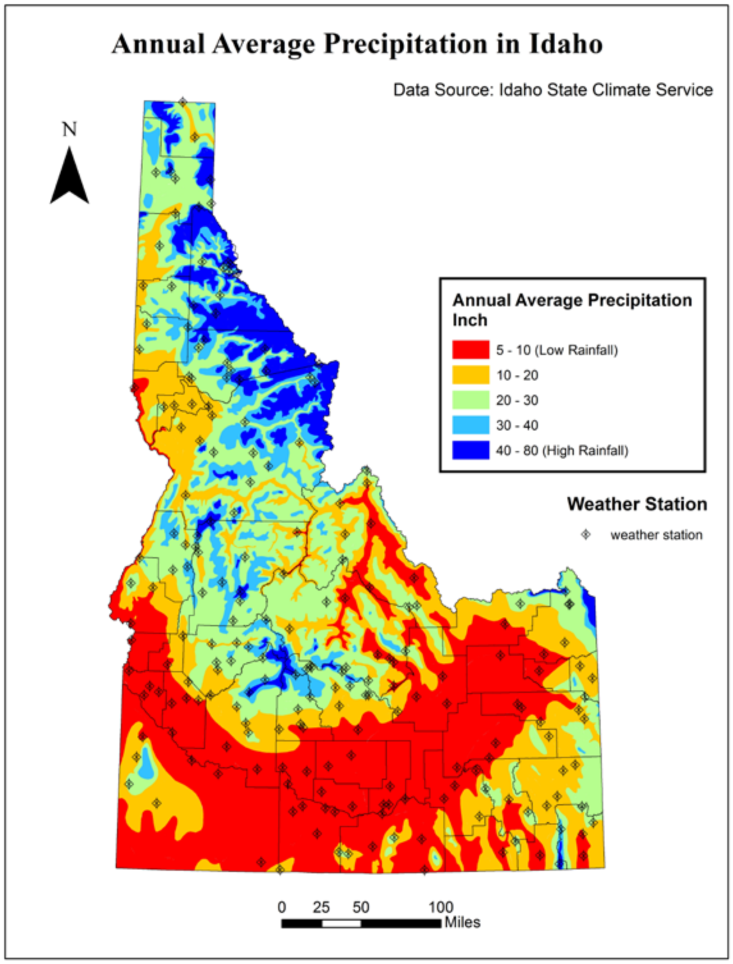


Figure 3-1 Annual average precipitation map for Idaho

The state of Idaho has a widely varying geography and weather where some regions are very dry, and some regions experience significantly higher amounts of rainfall. As seen from Figure 3-1, the northern part of the state receive significantly higher precipitation compared to the southern parts. One soil (termed S_1) from the northern part of Idaho was selected for this study to represent 'wet zone' conditions. Another soil (termed S_2) was collected from the south-west region of the state, and represented the 'dry zone' conditions.

Preliminary Soil Characterization and Shear Strength Measurements

<u>Grain Size Distribution</u>

The grain size distribution for the two soils was established using the AASHTO T 88 test method, and have been plotted in Figure 3-2. The amount of sand, silt and clay particles of these soils have been listed in Table 3-1. Atterberg's limit tests were carried out per AASHTO T 90 standard; both soils exhibited low plasticity index (PI), and could be classified as 'A-4' according to the AASHTO classification system. The equivalent USCS classifications were: CL for S_1 and CL-ML for S_2.

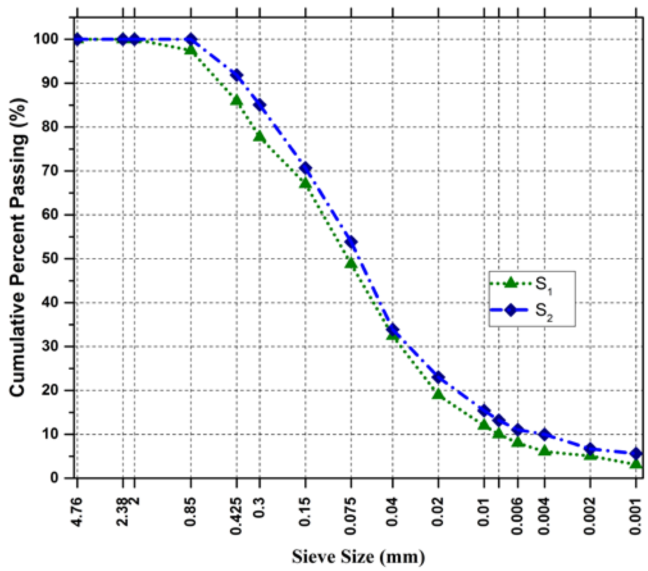


Figure 3-2 Grain size distribution of selected two soil types

Table 3-1 Soil Index Properties and Classification

Material	Liquid Limit LL	Plasticity Index PI	Sand	Silt	Clay	Soil Classification	
	%	%	%	%	%	USCS	AASHTO
S_1	30	3	51.2	43.7	5.1	CL	A-4
S_2	26	5	46.1	47.2	6.7	CL-ML	A-4

Moisture Density Relationship

The optimum moisture content (OMC) and maximum dry density (MDD) of the two soils were established in the laboratory following the AASHTO T 99 test method. This information was subsequently used in this study to prepare specimens for resilient modulus, shear strength, and permanent deformation testing. Figure 3-3 shows the moisture-density plots for the two soil types. As expected, the dry density increased with the increasing moisture content until OMC, and decreased beyond that. To study the effect of moisture content variation on soil behavior, specimens were also prepared in this study at 90% OMC (dry) and 110% OMC (wet) conditions. Table 3-2 lists the moisture contents for the two soils corresponding to the dry, OMC, and wet conditions.

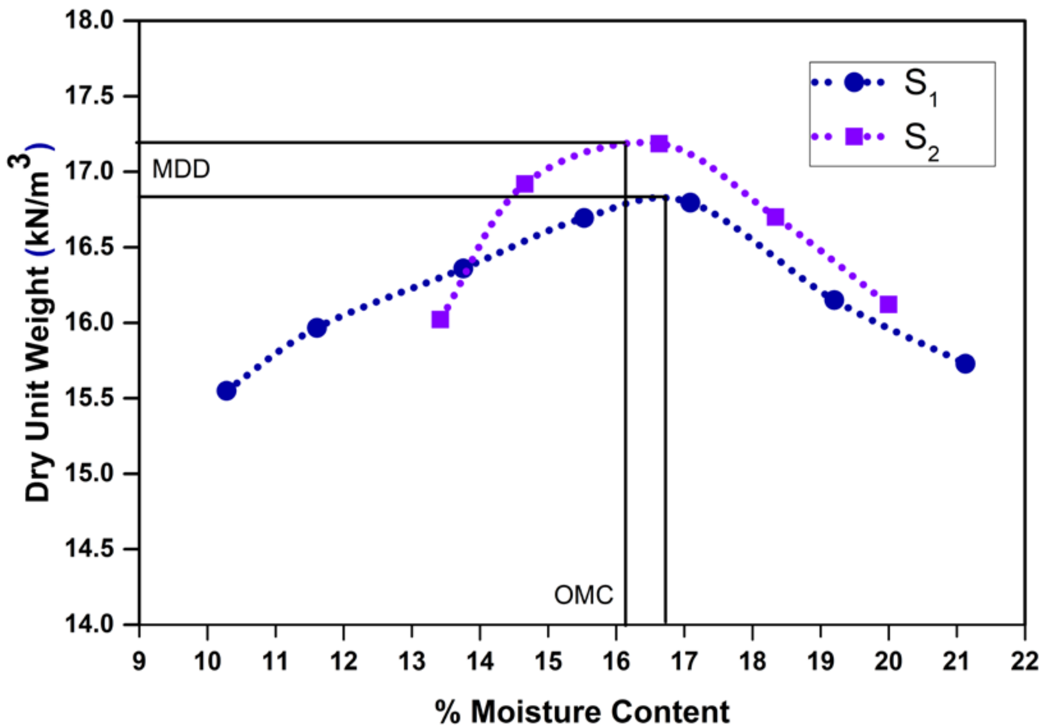


Figure 3-3 Compaction moisture-density characteristics for the two soils established following the AASHTO T 99 procedure

Table 3-2 Moisture Contents Corresponding to Dry, OMC, and Wet Conditions for the Two Soil Types

Soil ID	Compaction Moisture Content		
	Dry	OMC	Wet
S_1	15.1	16.8	18.5
S_2	17.8	16.2	14.6

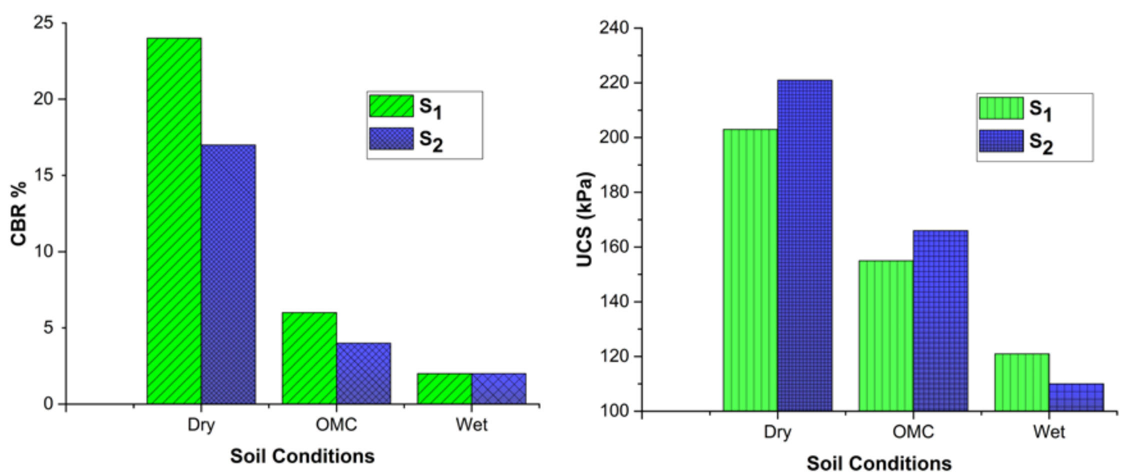
Strength Properties of Soils for Typical Pavement Design

California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS) were used in the current study to determine establish shear strength properties for the two soil types. Although the CBR is an index property, it has been used extensively by researchers and practitioners as an indicator of unbound material shear strength. CBR is also used as an indirect method to predict the resilient modulus of soils (Patel et al 2010). Several different correlations equations have been used by state and local highway agencies to predict resilient modulus from CBR and UCS value (Jackson 2015); the resilient modulus values thus predicted, are used as inputs during M-E pavement design instead of having to perform complex and cumbersome repeated load triaxial testing. CBR values for the two soil materials were determined in the current study under unsoaked conditions, but otherwise following the AASHTO T193 method. Each soil material was tested at three different moisture conditions to quantify the moisture sensitivity; CBR values for the two soils under the three different moisture conditions have been listed in Table 3-3. The same data has been plotted in the form of a column plot in Figure 3-4 (a). As seen from Table 3-3 as well as Figure 3-4 (a), both soil types showed significant degree moisture sensitivity. Under dry conditions, both soils exhibited CBR values that were at least four times higher than the corresponding values under OMC conditions. On the other hand, when the moisture content was increased to 110% OMC, the CBR value underwent a rapid reduction. Soil S_1 exhibited a greater level of moisture sensitivity compared to S_2 during CBR testing.

Table 3-3 California Bearing Ratio and Unconfined Compressive Strength of Soils

Soil Type	Conditions	CBR (%)	UCS (kPa)
S_1	Dry	24	203
	OMC	6	155
	Wet	2	121
S_2	Dry	17	221
	OMC	4	166
	Wet	2	110

Unconfined compressive strength values for the soils were established following guidelines provided in AASHTO T208. After completion of the resilient modulus tests (discussed in the next section), the same specimen was used to test for UCS under controlled strain conditions at a rate of 1 mm/min. Similar to the CBR testing, specimens were prepared at three different moisture conditions and tested to quantify their moisture sensitivity. UCS values for the two soils under three different moisture conditions were also listed in Table 3-3, and the same data has been plotted in the form of column plot in Figure 3-4 (a). As seen from the table and the plot, both soil types exhibited highest UCS values when tested under dry conditions. UCS values decreased significantly as the moisture contents were increased to OMC and subsequently to wet of OMC conditions.

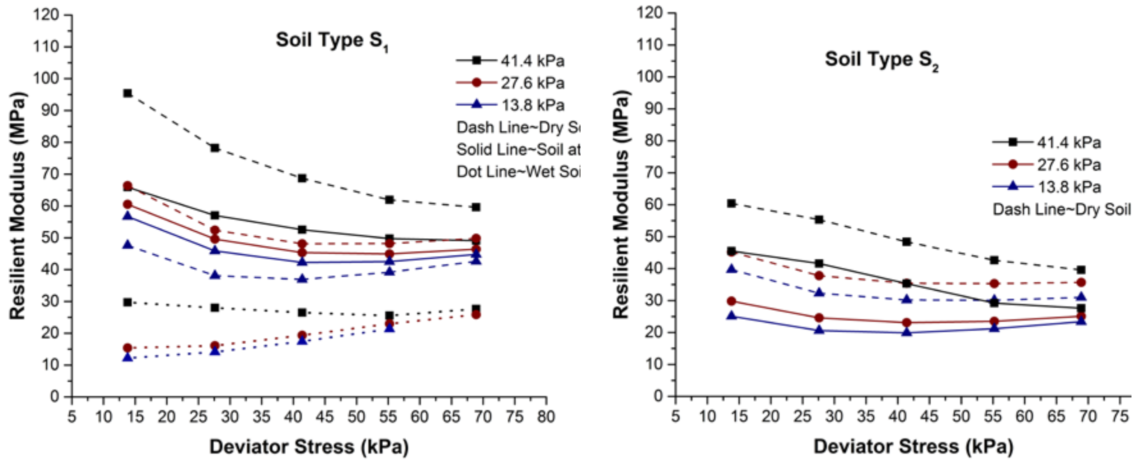


(a) **(b)**

Figure 3-4 (a) California Bearing Ratio and (b) Unconfined Compressive Strength of soils at different conditions

Resilient Response for Pavement ME Design

Resilient modulus properties of the two soils were established in the laboratory using the AASHTO T 307 test protocol. Besides providing desired information for M-E pavement design, the resilient modulus data was also used during pavement analyses to calculate the stress states on top of the subgrade, which would then be used to develop the new permanent deformation test protocol. Resilient modulus characterization is representative of pavement layer response under stress levels that are significantly lower than the corresponding shear strength values which is necessary to model the performance of pavement layers (Taylor 2008). Resilient modulus testing was conducted at three different moisture contents to study the effects of varying moisture contents. Two replicate specimens were tested for each soil type at the three different moisture contents. Figure 3-5 shows the variation in resilient modulus of the two soils due to changing moisture content. For the soil S_1, the resilient modulus value decreases consistently with increasing moisture content; this effect is more prominent at high confining stress levels. At low confining stress levels, although the transition from 'dry' to OMC conditions does not appear to have a drastic effect, there is a sudden drop in M_R observed when the moisture changes from OMC to 'wet'. Note that the confining pressures felt by a subgrade soil element underneath a typical pavement structure is usually very low (of the order of 7-14 kPa). Accordingly, resilient modulus values for the soils at low confining pressure levels would be representative of field conditions.



(a) **(b)**

Figure 3-5 Resilient Modulus values for the two soils under different moisture conditions: (a) soil type S_1 and (b) soil type S_2

On the other hand, soil S_2 experienced greater than 5% permanent strain accumulation during the conditioning phase under wet conditions; this is taken to be an indicator of ‘specimen failure’ during M_R testing, therefore rendering this soil unsuitable for pavement applications under wet conditions. It should be noted that under no stress level, did the dry specimens of S_2 exhibit significantly stiffer behavior compared to the specimens tested under OMC conditions. There are several models available to predict the resilient modulus of soils based on different stress states. These model parameters are required during the pavement analysis to compute the stress-strain experienced at different points within the pavement structure under traffic loading. In this study, the MEPDG model and Uzan model were chosen and fitted to the laboratory test data; the model parameters established for the two soil types have been listed in Table 3-4. The two model forms have been given below.

$$M_R = k_1 \times p_a \times \left(\frac{\theta}{p_a} \right)^{k_2} \times \left(\frac{\tau_{oct}}{p_a} + 1 \right)^{k_3}$$ MEPDG Model

$$M_R = k \times (\theta)^n \times (\sigma_d)^m$$ Uzan Model

where

θ is known as bulk stress or the first stress invariant $(\theta = \sigma_1 + \sigma_2 + \sigma_3)$

σ_1 is the major principal stress

σ_2 is the intermediate principal stress

σ_3 is the minor principal stress

σ_d is the deviator stress

For a cylindrical triaxial test set-up

$\sigma_2 = \sigma_3$

$\sigma_1 = \sigma_3 + \sigma_d$

$\sigma_1 - \sigma_3 = \sigma_d$

p_a is atmospheric pressure, and equals 101.3 kPa

The p_a term is used for normalization in the equation

τ_{oct} represents the octahedral shear stress

$$\tau_{oct} = \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}$$

For a cylindrical triaxial test specimen,

$$\tau_{oct} = \frac{\sqrt{2}}{3}(\sigma_1 - \sigma_3) = \frac{\sqrt{2}}{3} \times \sigma_d$$

Table 3-4 MEPDG and Uzan Model Parameters of Soils at Different Moisture Conditions

Soil Type	Condition	MEPDG Model Parameters				Uzan Model			
		k_1	k_2	k_3	R^2	k	n	m	R^2
S_1	Dry	788.4	0.82	-3.11	0.88	1466.75	0.84	-0.44	0.95
	OMC	637.21	0.24	-1.73	0.75	5354.27	0.25	-0.25	0.89
	Wet	164.54	0.62	0.54	0.82	440.86	0.63	0.06	0.81
S_2	Dry	555.85	0.57	-2.66	0.86	1997.5	0.58	-0.36	0.91
	OMC	377.2	0.68	-2.68	0.76	1025.7	0.68	-0.37	0.81
	Wet	-	-	-	-	-	-	-	-

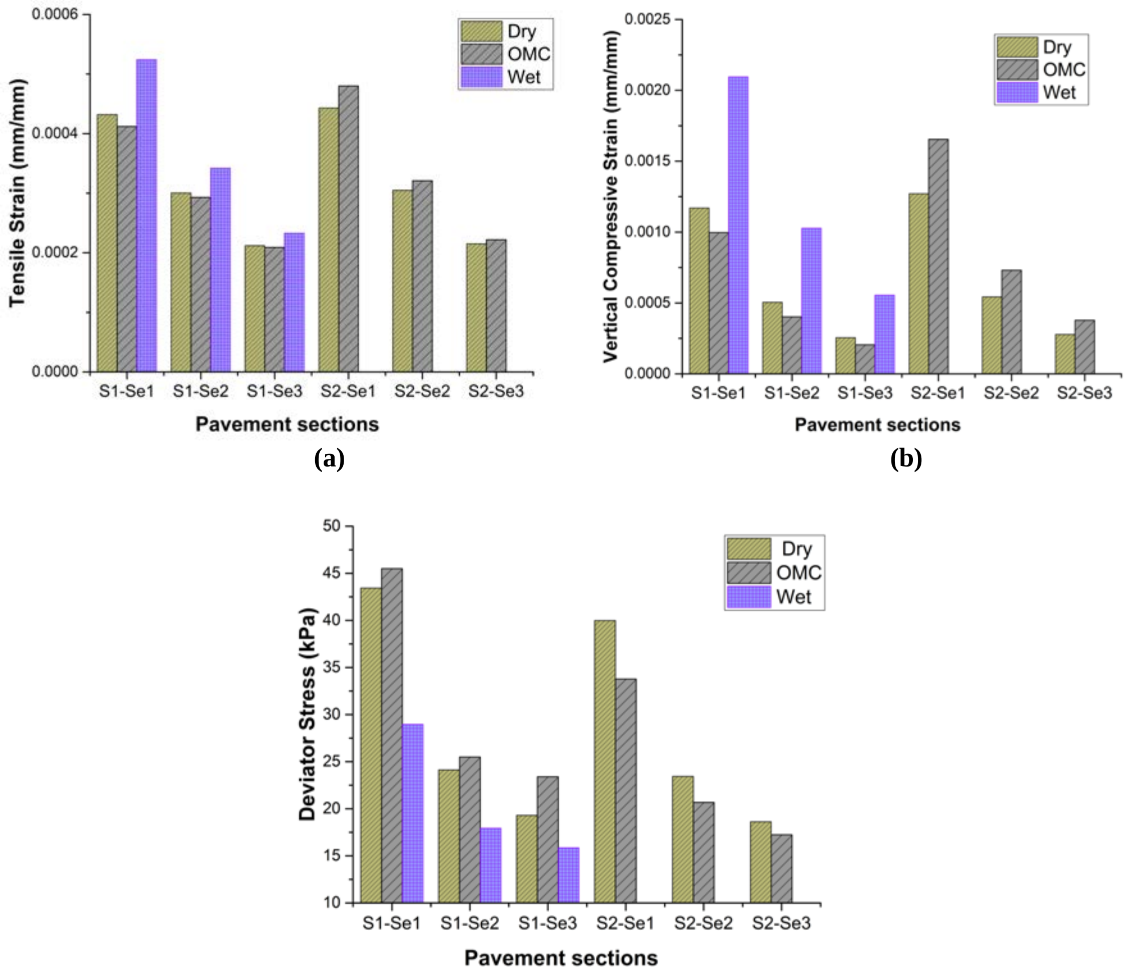
As seen from the above table, both k_3 and m parameter are negative for dry and OMC condition, but it becomes reverse at wet condition. This means that response of soils due to deviator stress depends on moisture condition. The Uzan model was used in this study to analyze the response of typical pavement structures under vehicular loading; this information was then used to develop the new permanent deformation test protocol as discussed in the next section.

Finite Element Analysis of Typical Pavement Sections

Tensile strain at bottom of HMA layer, as well as vertical compressive strain and deviator stress at the top of subgrade layer are considered as the critical pavement responses which are often used to predict the service life of pavements. In this section, the effect of subgrade moisture condition was evaluated on pavement response by monitoring changes in the critical pavement response parameters. It was decided to use ILLI-PAVE 2005 software to compute the critical pavement responses as ILLI-PAVE considers the stress dependent nature of unbound materials (Thompson & Elliott 1985). Table 3-5 lists the material properties that were used as inputs in ILLI-PAVE. The HMA and base layer properties were taken from the Idaho AASHTOWare® Pavement ME design user guide (Mallela et al, 2018). The subgrade properties were assigned based on values determined in the laboratory for S_1 and S_2. Soil properties for all three moisture conditions (dry, OMC, and wet) were used to analyze different pavement sections. Finally, three different pavement sections with different layer thickness were considered to cover the typical range of pavement structures constructed in the state of Idaho. Note that specimens of soil S_2 prepared under wet conditions were unable to sustain the stress levels applied during repeated load triaxial testing, and therefore, that particular case has not been included in this analysis. Figure 3- 6 shows the critical pavement response parameters obtained upon subjecting the pavement sections to a 40-kN wheel load at 80 psi tire pressure.

Table 3-5 Summary of Material Properties for ILLI-PAVE Analysis

Materials			Density (kg/m^3)	Modulus (MPa)	Poisson's Ratio	Layer Thickness (mm)		
						Sec-1	Sec-2	Sec-3
HMA			2323	3447	0.35	101.6	152.4	203.2
Base			2274	K-θ model	0.35	304.8	457.2	609.6
Subgrade Soils	S_1	Dry	1730	Uzan model	0.40	∞	∞	∞
		OMC	1730					
		Wet	1698					
	S_2	Dry	1762					
		OMC	1762					



(a) (b) (c)

Figure 3-6 Tensile strain at bottom of HMA layer (b) Compressive strain at top of subgrade (c) Deviator stress at top of subgrade

As seen from the figure, under OMC conditions, soil S_1 exhibits lowest tensile strain at the bottom of HMA layer but the same response parameters is lower for soil S_2 under dry conditions. Vertical Strain on top of subgrade becomes worse when moisture content changes from OMC to wet conditions for soil S_1; the value is much lower under OMC conditions. However, for soil S_2, the response under dry conditions is better compared to OMC. Deviator stress on top of the subgrade is governed by stiffness of the subgrade layer, which is significantly influenced by the moisture content. High amount of moisture makes a soil weaker, and that leads greater settlements (or strains) and lower stresses (as stress = stress x modulus). As shown from the plots, lower deviator stresses on top of the subgrade were obtained for soil S_1 under wet conditions. Also, this value was not found to change significantly for either soil when the moisture level was changed from dry to OMC.

Laboratory Tests to Characterize the Permanent Deformation Potential of Soils

<u>PD test program</u>

As already mentioned, no standard specification is available to characterize the permanent deformation (PD; or rutting) potential of unbound materials in the laboratory. Accordingly, this study attempted to develop a new test protocol that would consider the typical stress states experienced by soils in a pavement structure rather than using some pre-established stress levels. Several researchers have recently (Rahman et al. 2019)) used data from resilient modulus tests to evaluate the permanent deformation potential of unbound materials. However, it should be noted that the M_R test procedure applies only around 2500 cycles of loading to the specimen, which is not sufficient to study the PD characteristics of soils and aggregates. Moreover, the M_R test procedure (per AASHTO T307) applies 15 different stress states sequentially to the specimen. This creates stress-history effects on the specimen behavior, and therefore, can be misleading if used for PD characterization. Finally, some of the stress levels applied during the M_R testing are higher than those experienced by soil elements underneath typical pavement structures.

Considering the above-mentioned uncertainties, the current study selected 41.4 kPa and 55.2 kPa deviator stress levels based on results from ILLI-PAVE analysis (shown in Figure 3-6c). Both the soils were tested under two different deviator stress levels to characterize the permanent deformation response. Generally, confining pressure levels in subgrade are very low, and can often be lower than 34.5 kPa (Thompson and Robnett 1976). The current study chose to perform the PD tests at a confining pressure level of 20.7 kPa. The soil specimens were 100 mm in diameter, and 200-mm tall. A standard compaction drop hammer was used to compact the specimen in three layers targeting 100% maximum dry density. Moreover, adhesion between the layers was ensured by scarifying the surface of the compacted layer before placement of the next layer. Specimens were prepared under moisture contents corresponding to dry, OMC, and wet conditions. The deviator stress levels calculated from ILLI-PAVE analyses were applied to the specimen; haversine load pulses were applied with a 0.1s loading period followed by a 0.9s rest period. Each specimen was subjected to 10,000 cycles of loading; axial strain data was recorded during the testing using two externally mounted LVDTs on top of the triaxial cell. Two replicate specimens were tested at each soil-deviator stress-moisture condition combination to confirm repeatability of the test results.

Analysis of PD Test Results

After completion of the PD tests, axial permanent (plastic) strain data was plotted against the number of loading cycles (see Figure 3-7 for S_1 and Figure 3-8 for S_2). Accumulated plastic strain under repeated loading is representative of the susceptibility of soils to rutting under traffic loading. Under the first few load cycles, the permanent strain accumulation increases rapidly, with the rate of permanent strain accumulation reducing significantly as the specimen approaches “stable” conditions; ultimately, the specimen attains hysteretic stress-strain response under repeated loading.

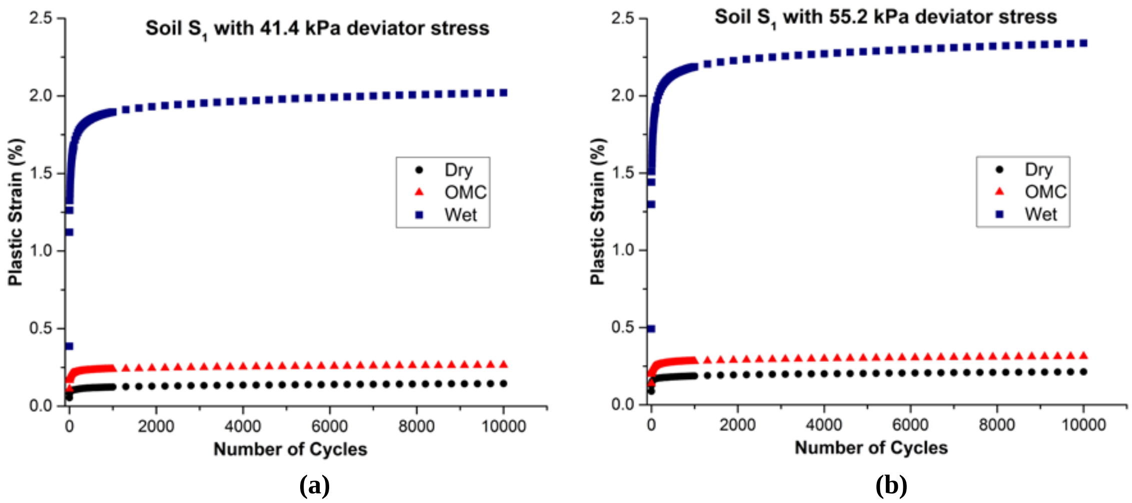
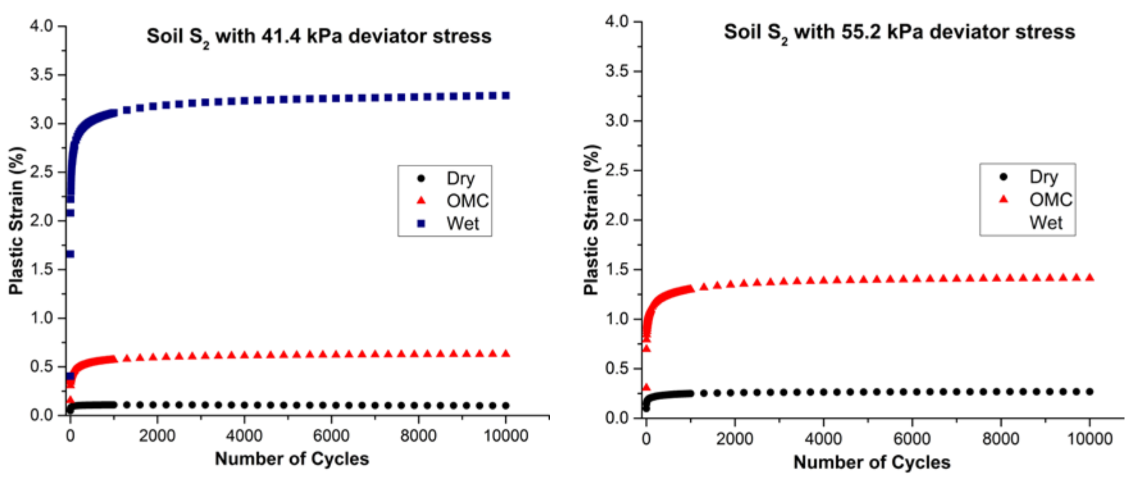


Figure 3-7 Accumulation of plastic strain in soil S_1 when tested at two different deviator stress values: (a) $\sigma_d = 41.4\ \text{kPa}$ **; and (b)** $\sigma_d = 55.2\ \text{kPa}$

As seen from the above figure, soil S_1 under dry and OMC conditions became stabilized after only 200 cycles of loading; the S_1 specimen under wet conditions took 1000 cycles to reach 'stable' behavior (characterized by no significant permanent strain accumulation under further loading). The figure also shows that plastic strain accumulation after 10,000 cycle under wet conditions is greater than that for dry and OMC conditions. Note that the S_1 specimen under wet conditions accumulated plastic strains that were 8 times greater than that under OMC conditions at end of testing. S_1 soil specimens at both dry and OMC conditions exhibited relatively low plastic strain accumulation upon 10,000 cycles of loading; this was consistent for both the deviator stress levels. Note that soil S_1 was collected from the northern part of Idaho form a region that experiences significantly high amounts of annual precipitation. This soil also exhibited very high susceptibility towards rutting under wet conditions. It is therefore recommended that during the design and construction of pavements in this region, extra care should be taken to prevent the ingress of water into the subgrade layer. Moreover, the design should be checked by using lower subgrade modulus values (to represent wet conditions) to ensure that the pavement layer thicknesses are adequate to protect the subgrade from excessive rutting.

Figure 3-8 shows similar plots for the S_2. Note that S_2 exhibited very high levels of moisture sensitivity, and the specimen prepared under wet conditions failed during PD testing when the deviator stress level was equal to 55.2 kPa. Note that when subjected to a deviator stress level of 41.4 kPa, the S_2 specimen at wet condition accumulated more than 3% plastic strain only after 500 load cycles. Unlike the S_1 specimen, specimens for S_2 exhibited high sensitivities even to changes in deviator stress magnitudes. Figure 3-8 shows that plastic strain accumulation rate under OMC conditions were significantly different from those under dry conditions. Under dry conditions, the specimens reached 'stable' behavior after only 100 cycles. As the S_2 material was collected from a region of Idaho with relatively low precipitation, designing for OMC conditions may be appropriate in this case.



(a) **(b)**

Figure 3-8 Accumulation of plastic strain in soil S_2 when tested at (a) 41.4 kPa deviator stress (b) 55.2 kPa deviator stress

Summary and Conclusions

This chapter presented results from a laboratory testing effort targeted at characterizing the permanent deformation potential for two different subgrade soil types under different moisture conditions. Laboratory tests performed on the soils included: (1) gradation; (2) Atterberg's limits; (3) moisture-density characteristics; (4) resilient modulus; (5) unconfined compressive strength; and (6) permanent deformation. The modulus, shear strength, and permanent deformation tests were carried out under three different moisture conditions. The resilient modulus test data were fitted with MEPDG, Uzan models to establish the model parameters, which were then used during finite element analysis of the pavement sections to calculate critical pavement response parameters under traffic loading. The analysis was performed using ILLI-PAVE as it can adequately capture the stress-dependent behavior of unbound materials. Stress states experienced by the subgrade layer under typical pavement configurations were calculated corresponding to the three different subgrade moisture conditions. These stress states were then used to develop a new permanent deformation test protocol, and the effects of moisture content and stress levels on permanent deformation behavior were studied. The following important conclusions can be drawn from this study:

(1) Both soils exhibited high levels of moisture sensitivity. CBR, UCS, as well as M_R values decreased significantly as the moisture content was increased. The change in these values was worse when the moisture content was increased from OMC to 110% OMC compared to the change from 90% OMC to OMC.

(2) Pavement response calculations under standard wheel loading indicated that vertical strain on top of the subgrade increased and the deviator stress decreased under wet conditions.

(3) Permanent deformation testing of the soils showed that both soils can undergo significant rutting under wet conditions. Both soils may reach 'stable behavior' under construction traffic if dry conditions are maintained by ensuring proper drainage. However, significant rutting may result if the moisture content increased during the service life of the pavement. New pavement design or rehabilitation projects in regions experiencing high amounts of precipitation should consider these factors and need to ensure proper drainage of pavement layer to protect the subgrade layer. Using subgrade modulus values at OMC conditions for all designs is not the best approach at locations where significant fluctuations in subgrade moisture content is likely.

Limitations of Current Study and Recommendations for Future Research

Although this research effort successfully accomplished many of its objectives, several limitations associated with the testing effort need to be considered. These limitations are listed below, and should be considered during future research efforts.

(1) The soil specimens in this study were tested for PD characteristics under the application of 10,000 load cycles, which is which is well below the number of traffic load repetitions experienced by pavement layers throughout their lifetime. A more extensive PD study needs to test the specimens up to 100,000 load applications.

(2) In this study, soils were only tested at two different deviator stress levels. This is not sufficient to develop an extensive rutting prediction model. The

AASHTOWare® Pavement ME Design software uses the MEPDG rutting model which predicts unbound material rutting based on resilient modulus and other soil index properties. Adequacy of the MEPDG model needs to be evaluated by comparing model-predicted rutting with actual lab tested rutting values for subgrade soils. However, such an effort will require the testing of several soil types and testing under several different stress states. As only two soils were tested in this effort, and due to time and material availability related constraints, the current study could not undertake the effort to calibrate the MEPDG model for Idaho conditions. This will be addressed in future research efforts.

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CHAPTER FOUR: SUMMARY AND CONCLUSIONS

Summary

The primary focus of this thesis was to characterize different subgrade soil types collected from across the state of Idaho through resilient modulus and permanent deformation testing along with other conventional, commonly used laboratory tests. Preliminary tests carried out in the laboratory included sieve analysis, Atterberg's limit testing and moisture-density characterization. Moreover, other commonly used strength and index tests such as Unconfined Compressive Strength (UCS) and California Bearing Ratio (CBR) were also carried out. Repeated load triaxial testing was carried out according to the AASHTO T307 protocol to measure resilient properties of the subgrade soils. Specimens were prepared and tested at three different moisture contents (dry of optimum, optimum, and wet of optimum) to quantify the effects of moisture variation on resilient modulus. The conventional stress-dependent resilient modulus models such as the ones proposed by Uzan, and the one used in the MEPDG (known as the universal model) were used to fit the laboratory data; the corresponding model parameters were thoroughly analyzed. The resilient modulus test results were analyzed along with the UCS and CBR test results to investigate the feasibility of predicting the resilient modulus values of subgrade soils without the need to run the complex repeated load triaxial tests. The effect of assumed resilient modulus and moisture variation in soils were investigated by computing pavement response and predicted pavement performances.

Stress levels on top of the subgrade under standard wheel loads were estimated through finite element analysis of typical pavement sections by considering stress-dependent unbound material behavior. This information was later used to develop a new permanent deformation (PD) testing protocol. PD tests were conducted on soil specimens prepared at different moisture contents to study moisture sensitivity of plastic strain accumulation in soils.

Conclusions

The following conclusions are drawn from this study:

- Fine grained soils showed high level of moisture sensitivity with little variation in moisture content; deviator stress levels had varied impact on M_R based on whether the soil comprised non-plastic or plastic fine.
- The reduction in soil modulus and strength properties when moisture content is increased from dry to optimum conditions is not as drastic as when the moisture content is increased beyond optimum condition.
- Although some correlation equations can be developed to predict the resilient modulus value of subgrade soils from easy-to-establish index and mechanical properties, most of these equations do not match commonly-observed material behavior trends. Therefore, generic equations to predict resilient modulus from other soil properties can lead to erroneous pavement designs.
- Subgrade soils exhibit very high levels of moisture sensitivity. Ultimate care needs to be taken during pavement construction to ensure the subgrade layer is compacted under dry conditions; adequate measures should be taken to prevent ingress of excessive moisture into the pavement structure during its service life.

- Even if some soils exhibit reasonable resilient modulus values when tested at OMC or wet of OMC conditions, they may accumulate excessive permanent deformation when subjected to very high number of load applications. Therefore, predicting rutting potential of soils using resilient modulus is not a recommended approach.
- Moisture sensitivity of soils should be considered during the design and construction of pavement sections in regions that experience very high amounts of precipitation. Using the soil properties determined at optimum moisture conditions for all design types is not recommended.