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RESEARCH REPORT

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RP 263

Unbound Materials Characterization for Pavement ME Implementation in Idaho

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16. Abstract

This research study focused on laboratory testing of typical subgrade soils and unbound aggregates used in pavement applications across the state of Idaho. A total of eighteen (18) aggregates and sixteen (16) subgrade soils were tested in the laboratory to support ITD's efforts towards statewide implementation of Mechanistic-Empirical (M-E) pavement design practices. Standard soil and aggregate tests were performed in the laboratory along with repeated load triaxial testing for resilient modulus characterization. The research team attempted to develop statistically significant correlations to predict soil and aggregate resilient modulus values from easy-to-establish index and/or mechanical properties; this would ITD avoid the cumbersome and resource-intensive resilient modulus test. Although aggregate base/subbase modulus values could be predicted from index properties with reasonable accuracy, no acceptable correlation could be developed for the subgrade soils. A database with all laboratory-established soil/aggregate properties was developed as a project deliverable; it can be used as a source for unbound material input properties required by AASHTOWare Pavement ME Design. Although it was not possible to develop unique correlation equations to predict soil and aggregate resilient modulus values from index properties, the developed database will significantly enhance the confidence level associated with unbound material input properties during pavement design in Idaho. When direct laboratory testing data is not available, ITD engineers are recommended to use this database to obtain unbound material input properties rather than relying on nation-wide or regional default values.

17. Key Words Unbound Materials, Mechanistic-Empirical Pavement Design, Resilient Modulus, MEPDG, Pavement ME Design	18. Distribution Statement Copies available online at http://itd.idaho.gov/alt-programs/?target=research-program

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METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS					APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	When You Know	Multiply By	To Find	Symbol	Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH					LENGTH		
in	inches	25.4	millimeters	mm	mm	millimeters	0.039	inches	in
ft	feet	0.3048	meters	m	m	meters	3.28	feet	ft
yd	yards	0.914	meters	m	m	meters	1.09	yards	yd
mi	miles (statute)	1.61	kilometers	km	km	kilometers	0.621	miles (statute)	mi
		AREA					AREA		
in^2	square inches	645.2	millimeters squared	cm^2	mm^2	millimeters squared	0.0016	square inches	in^2
ft^2	square feet	0.0929	meters squared	m^2	m^2	meters squared	10.764	square feet	ft^2
yd^2	square yards	0.836	meters squared	m^2	km^2	kilometers squared	0.39	square miles	mi^2
mi^2	square miles	2.59	kilometers squared	km^2	ha	hectares (10,000 m^2)	2.471	acres	ac
ac	acres	0.4046	hectares	ha					
		MASS (weight)					MASS (weight)		
oz	Ounces (avdp)	28.35	grams	g	g	grams	0.0353	Ounces (avdp)	oz
lb	Pounds (avdp)	0.454	kilograms	kg	kg	kilograms	2.205	Pounds (avdp)	lb
T	Short tons (2000 lb)	0.907	megagrams	mg	mg	megagrams (1000 kg)	1.103	short tons	T
		VOLUME					VOLUME		
fl oz	fluid ounces (US)	29.57	milliliters	mL	mL	milliliters	0.034	fluid ounces (US)	fl oz
gal	Gallons (liq)	3.785	liters	liters	liters	liters	0.264	Gallons (liq)	gal
ft^3	cubic feet	0.0283	meters cubed	m^3	m^3	meters cubed	35.315	cubic feet	ft^3
yd^3	cubic yards	0.765	meters cubed	m^3	m^3	meters cubed	1.308	cubic yards	yd^3
Note: Volumes greater than 1000 L shall be shown in m^3									
		TEMPERATURE (exact)					TEMPERATURE (exact)		
^oF	Fahrenheit temperature	5/9 (^{o}F-32)	Celsius temperature	^oC	^oC	Celsius temperature	9/5 ^{o}C+32	Fahrenheit temperature	^oF
		ILLUMINATION					ILLUMINATION		
fc	Foot-candles	10.76	lux	lx	lx	lux	0.0929	foot-candles	fc
fl	foot-lamberts	3.426	candela/m^2	cd/cm^2	cd/cm^2	candela/m^2	0.2919	foot-lamberts	fl
		FORCE and PRESSURE or STRESS					FORCE and PRESSURE or STRESS		
lbf	pound-force	4.45	newtons	N	N	newtons	0.225	pound-force	lbf
psi	pound-force per square inch	6.89	kilopascals	kPa	kPa	kilopascals	0.145	pound-force per square inch	psi

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Technical Advisory Committee

Each research project is overseen by a technical advisory committee (TAC), which is led by an ITD project sponsor and project manager. The Technical Advisory Committee (TAC) is responsible for monitoring project progress, reviewing deliverables, ensuring that study objective are met, and facilitating implementation of research recommendations, as appropriate. ITD's Research Program Manager appreciates the work of the following TAC members in guiding this research study.

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List of Acronyms

AASHTO	American Association of State Highway and Transportation Officials
Caltrans	California Department of Transportation
CBR	California Bearing Ratio
CDOT	Colorado Department of Transportation
GDOT	Georgia Department of Transportation
HMA	Hot Mix Asphalt
ITD	Idaho Transportation Department
LVDT	Linear Variable Differential Transformers
MDD	Maximum Dry Density
MDDOT	Maryland Department of Transportation
MEPDG	Mechanistic Empirical Pavement Design Guide
M_R	Resilient Modulus
MTS	Material Testing System
NCHRP	National Cooperative Highway Research Program
OMC	Optimum Moisture Content
PCC	Portland Cement Concrete
RLT	Repeated Load Triaxial
SMR	Summary Resilient Modulus
UCS	Unconfined Compressive Strength
USCS	Unified Soil Classification System
WSDOT	Washington State Department of Transportation

Executive Summary

The Idaho Transportation Department (ITD) has invested significant resources over the past decade to facilitate statewide implementation of Mechanistic-Empirical (M-E) pavement design practices. Several research studies have been initiated by ITD over the past few years to establish required input parameters for Hot-Mix Asphalt (HMA) and Portland Cement Concrete (PCC) materials. Along the same lines, this research study was undertaken in collaboration with Boise State University and the University of Idaho to characterize unbound materials (soils and aggregates) commonly used in Idaho for pavement applications. Current pavement design in Idaho relies on the R-value to quantify the stiffness of unbound aggregates and soils underneath asphalt layers. The R-value method is based on the original California Department of Transportation (Caltrans) method, and was later modified to accommodate Idaho conditions. Previous research showed that this method might not provide an optimal pavement thickness.

The current approach to determine the required input parameters include predicting the Resilient Modulus (M_R) values through a correlation established with the R-values. However, such correlations can often over predict or under-predict the actual M_R values, moreover it is not applicable across different non-traditional and recycled material types. This research study tested eighteen (18) different base/subbase materials and sixteen (16) different subgrade soils collected from different districts across Idaho to develop a database of relevant properties to be used as inputs during M-E pavement design. In addition, the study attempted to develop statistically significant correlation equations to predict the resilient modulus of soils and aggregates from other easy-to-establish index and strength properties. This would help ITD avoid the cumbersome and resource-intensive repeated load triaxial testing procedure required to measure resilient modulus properties of soils and aggregates in the laboratory.

Different tests performed on the soil and aggregate materials were: (1) particle size distribution; (2) Atterberg's limits; (3) moisture-density characterization; (4) California Bearing Ratio (CBR); (5) unconfined compressive strength for subgrade soils; (6) R-value tests on selected soils through the ITD central labs; and (7) repeated load triaxial testing to establish the resilient modulus properties. Additionally, effect of moisture variation on soil behavior was studied by testing all subgrade soils at three different moisture contents: 90% of Optimum Moisture Content (90% of OMC), at OMC, and at 110% of OMC. Resilient modulus testing was carried out in the laboratory per AASHTO T 307 test specifications. All aggregate materials were tested using cylindrical specimens of 150 diameter and 300 mm height, whereas the subgrade soils were tested using 100 mm diameter x 200 mm height specimens. The test results were fitted with commonly used resilient modulus model, and the corresponding model parameters were determined. The test results were thoroughly analyzed to evaluate the feasibility of predicting resilient modulus from other easy-to-measure index and strength properties.

Extensive statistical analysis of the test results indicated that although a reasonable correlation could be established to predict the resilient modulus value for aggregate base/subbase materials using index

properties, no such reliable correlation could be developed for the subgrade soils. ITD engineers are recommended to use typical values for subgrade soils listed in the database (accompanying deliverable with this report) to increase the reliability associated with these input properties.

Chapter 1
Introduction

Background

The foundation plays a vital role in building durable structures. Accurate material characterization is necessary for adequate design. Pavements are no exception to this requirement. Numerous studies have been conducted to improve the test methods used to characterize the behavior of pavement materials. A conventional flexible pavement consists of a prepared subgrade or foundation and layers of subbase, base and surface courses.(1) As pavements are layered structures, the lowermost layers of a flexible pavement structure are often layers of unbound materials (e.g., granular bases or compacted fill) above the existing soil material. A significant portion of the structural capacity of layered flexible pavement systems is offered by these unbound materials. These unbound materials are multiphase materials, consist of aggregate particles, air voids and water. Characterization of these materials should ideally be based on the behavior of the individual constituent elements and their interaction.

Almost every civil engineering agency works on a set of design procedures and regulations for both design and construction purposes. The need of these procedures or protocol are also apparent in issues related to pavement design. With the passage of time, several guidelines have been developed for the pavement design. Such as the American Association of State Transport and Highway Officials (AASHTO).(1-3) All these guidelines are empirical based, i.e. mostly they are based on the experiences rather than the mechanistic principle. But recently there has been a shift towards mechanistic based procedures from empirical procedures.

Research Need Statement

To achieve the goal of the trend, the National Cooperative Highway Research Program (NCHRP) has led to the development of a Mechanistic-Empirical Pavement Design Guide (MEPDG), documented in the final report of NCHRP project 1-37A. (4) After development of the MEPDG, most state transportation agencies have gradually initiated research efforts to facilitate its implementation into practice. The MEPDG procedure requires a large number of input variables for design, classified as traffic, climatic, structural and material inputs. Resilient modulus of the subgrade and Base/Subbase material or unbound materials, recommended by MEPDG, is one of them.

The test procedure for determining the resilient modulus for pavement materials is quite complex and requires sophisticated equipment. Also, the required test setup for resilient modulus testing is not generally available in most material testing laboratories due its high cost. The whole process of specimen preparation, loading, and measurement of sample deformation is very complex, time consuming, and requires trained and experienced operators.

Currently Idaho Transportation Department (ITD) is working towards full-scale implementation of the MEPDG, which is implemented in the software package known as AASHTOWare Pavement ME Design. Several research activities have been undertaken to locally calibrate the design guide to reflect design and construction practices in the State of Idaho. Laboratory characterization of the unbound materials (soils and aggregates) to establish input parameters to facilitate the implementation of the MEPDG is one of the basic steps.

Objective and Scope

The overall objective of this research study was to characterize representative, local material properties for unbound layers to facilitate comprehensive MEPDG implementation in the State of Idaho. Moreover, a database was developed with all test results to help ITD engineers quickly obtain input parameters for unbound materials during pavement design and performance prediction using Pavement ME Design. This database will significantly enhance the reliability associated with unbound material properties required as inputs during M-E pavement design. ITD engineers will no longer need to use Level 3 input properties; all properties will be at Level 2 or better, thereby increasing the overall reliability of the design.

The scope of this study mainly included establishing resilient modulus values of different types of unbound materials (soils and aggregates) typically used in the state of Idaho for pavement base/subbase and subgrade applications. Moreover, examining mathematical expressions, or models, for relating resilient modulus and stresses, and devising an easy means for ITD engineers to access the resilient modulus data and mathematical models are also included in the scope. A summary of the resilient modulus data generated in this study is contained in this report.

Report Layout

The contents reported in this report have been broken down into six chapters:

Chapter 2 presents a summary of findings from an extensive literature review that was carried out to identify practices adopted by different state highway agencies to establish resilient modulus properties of typical soils and aggregates used in pavement applications. Special attention was paid to models that have been developed for “easy prediction” of resilient modulus properties without the need to perform repeated load triaxial testing. Moreover, the research team also compiled regression equations that have been developed in the past by researchers to predict the resilient modulus values of soils and aggregates from easy-to-establish index properties. Note that a detailed literature review report was submitted to ITD as a separate deliverable. Therefore, Chapter 2 of this report only provides a summary of the information in a concise form.

Chapter 3 of the report describes the procedure adopted in the current study to collect representative soils and aggregate materials from different ITD districts for testing in the laboratory. This is followed by results from preliminary laboratory characterization efforts to establish the grain size distribution, moisture-density, and other index properties.

Chapter 4 describes results from repeated load triaxial testing efforts undertaken to establish resilient modulus properties for the soils and aggregates selected from different ITD districts. Inferences are drawn regarding different trends observed through the testing.

Chapter 5 presents details of efforts to establish correlations between resilient modulus and other "easy to establish" index properties of soils and aggregates. The ultimate objective of this effort was to assess whether or not state transportation agencies can forego the need to perform repeated load triaxial testing, and still predict resilient modulus values for soils and aggregates with reasonable accuracy. Chapter 5 also briefly discusses the database that was prepared as a deliverable from the current study compiling all laboratory test results for the aggregate and soil materials tested. The primary objective behind the development of this database was to provide a resource for ITD engineers to conveniently obtain unbound material properties as input parameters during mechanistic-empirical pavement design and performance prediction. Chapter 6 presents the summary and conclusions from this research study.

Chapter 2
Unbound Material Properties Affecting Pavement Performance

Typical Stress-Strain Response of Unbound Pavement Layers

In mechanistic-based pavement analysis and design methods, it is essential to understand the mechanical behavior of unbound layers underneath the surface layers. This is particularly relevant for flexible (or asphalt-surfaced) pavement systems, as a major portion of the structural capacity in these systems is contributed by the unbound layers as compared to rigid pavement systems. Due to the moving nature of traffic wheel loads, elements within a pavement system experience a stress pulse comprising vertical, horizontal and shear components. [5] The horizontal and vertical stress components are positive in unbound layers, whereas the shear stress turns to negative from positive as load passes over the element being considered. This reversal of the shear stresses causes a rotation of the principal stress axes, and therefore, leads to complex load-deformation behavior. [5] Figure 1 shows a schematic of the stress states experienced by a typical element within the pavement system due to movement of the wheel load. [5]

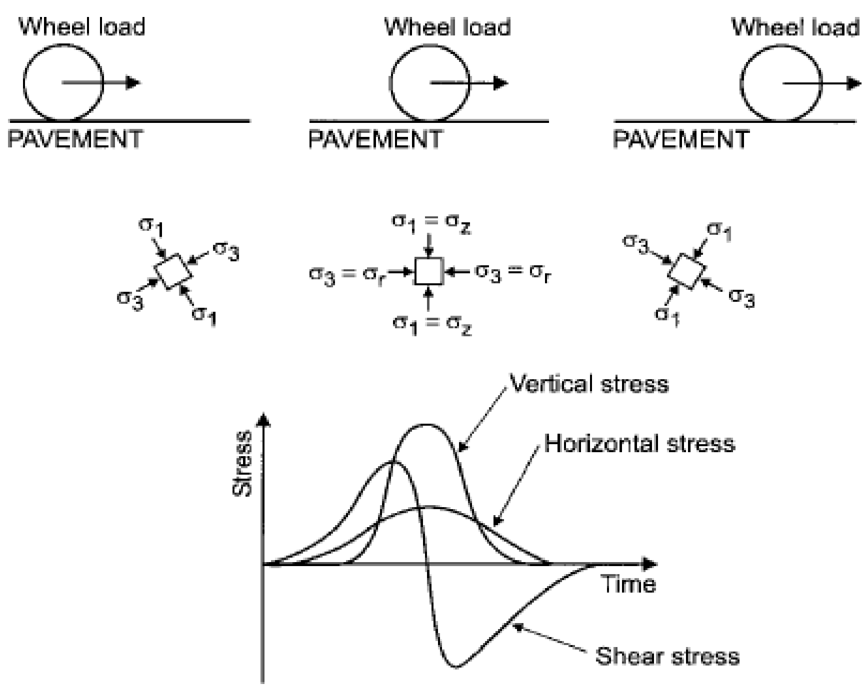


Figure Source: Lekarp et al. (2000)[5]

Figure 1. Schematic Representation of Stress States Experienced by a Typical Pavement Element Subjected to Moving Wheel Loads

Unbound layers undergo both elastic (commonly known as resilient for pavement applications) as well as plastic (permanent) deformations while subjected to traffic-imposed repeated stress pulses. Figure 2 presents a schematic of unbound material behavior under repeated loading with the help of a stress-strain diagram.[6] The relative magnitudes of elastic and plastic components of the total strain depend on several different factors, i.e. traffic load levels and speed of operation, thickness and quality of overlying pavement layers (if any), quality of materials used in construction of the unbound layer, and strength of the underlying layers, etc. In a typical unbound layer, the accumulation of permanent deformation for each load repetition gradually decreases with increased number of load applications. Once the layer has been well compacted to achieve a "stable" condition, all subsequent load applications should ideally result in deformations that are mostly elastic in nature, and thus well-constructed unbound layers should not accumulate any permanent deformation during the pavement's service life. Accordingly, mechanistic-based pavement design approaches have traditionally focused on the elastic or resilient response of unbound layers to predict the critical pavement responses under traffic loading. Resilient Modulus (M_R) is the most important property for incorporating repeated load behavior of unbound layers into pavement analysis. Defined as a secant modulus representing hysteretic stress-strain behavior of materials, the resilient modulus (M_R) is a critical material input property into (M-E) pavement design practices. The concept of resilient modulus was first introduced into pavement design in 1980. Figure 3 shows a schematic of typical hysteretic response exhibited by unbound materials under repeated loading.**[7]**

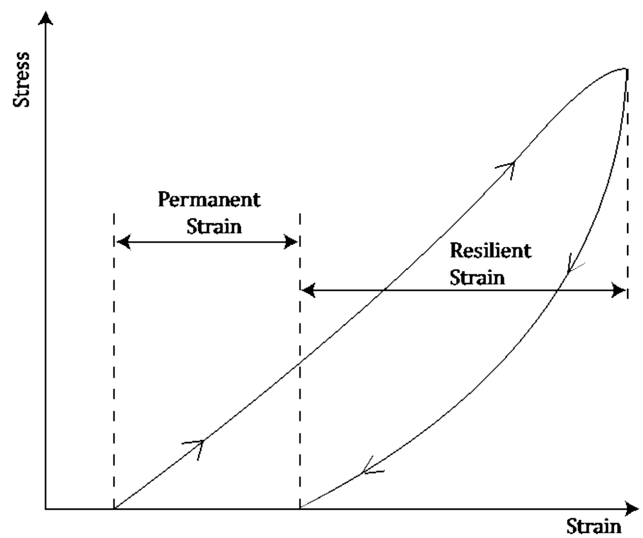


Figure Source: Mishra (2012)[6]

Figure 2. Strains in an Unbound Layer during One Cycle of Load Application

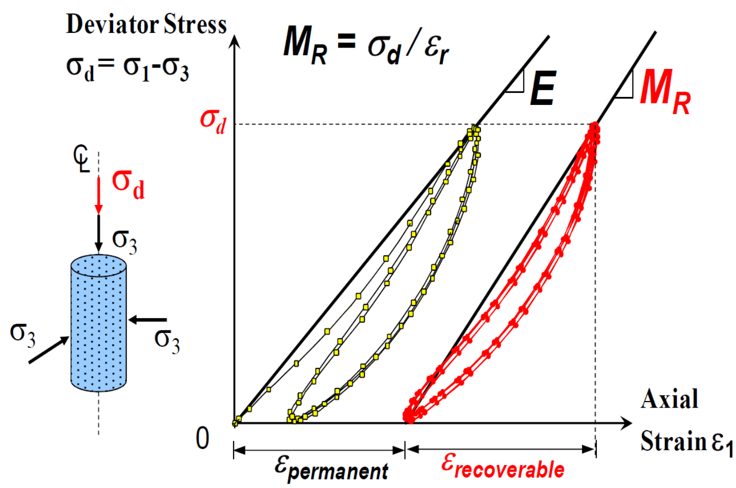


Figure Source: Tutumluer (2013) [7]

Figure 3. Resilient Modulus Defined as the Elastic Modulus of a Deformed Material

Resilient modulus is a fundamental material property used to characterize unbound pavement materials and is defined as the ratio of the peak axial repeated deviator stress to peak recoverable axial strain measured during a repeated load triaxial test. Figure 4 illustrates the definition of resilient modulus for unbound materials. It is a measure of material stiffness and provides a means to quantify material stiffness under different compaction conditions and applied stress levels. Accurate resilient modulus characterization is necessary to model the performance and life span of a given pavement structure **[8]** and is considered to be representative of pavement layer response under stress levels that are significantly lower than the corresponding shear strength values.

$$M_R = \sigma_d / \varepsilon_r$$

Figure 4. Equation to Define Resilient Modulus of Unbound Materials

where,

σ_d = Applied peak deviator stress

ε_r = Peak recoverable axial strain

M-E design procedures require numerous input variables for design; these variables are broadly classified as (1) traffic, (2) climatic, (3) structural, and (4) material inputs. The current M-E pavement design procedure (implemented into AASHTOWare Pavement ME Design) has three levels for design inputs**.** *Level 1* is the most accurate and uses site-specific data collected at or near the project site. *Level 2* requires the designer to input regional data that is representative of the local conditions. *Level 3* is the least accurate, utilizing national default values as inputs. Accordingly, M_R values for unbound materials can be input into Pavement ME Design in one of three ways.

- **Level 1:** Level 1 design efforts require the determination of M_R through laboratory or field testing of project- or site-specific materials. Common laboratory testing methods include, those specified by

NCHRP 1-28A [9] and AASHTO T307.[10] Note that the current version of AASHTOWare Pavement ME design does not allow the user to select Level 1 while providing unbound material resilient modulus inputs.

- **Level 2:** Level 2 involves estimation of the input parameter form correlation or regression equations. Accordingly, the M_R value for a particular unbound material is estimated from other easy-to-establish index properties such as the California Bearing Ratio (CBR), Resistance value (R- Value), etc. The fundamental approach behind a Level 2 design is that the input value is calculated from other site-specific data or parameters that are less expensive to measure.

- **Level 3:** In a Level 3 design, the input parameters are "best-estimated" based on global or regional default values. These default values usually represent median values for data groups comprising values with similar characteristics. Accordingly, in a Level 3 design, M_R values would be estimated from generic information such as soli classification.

The M-E Pavement Design Guide (MEPDG) strongly recommends Level 1 or Level 2 inputs for the resilient modulus of unbound layers. However, Level 1 testing may not be possible for many state and local transportation agencies due to complexities associated with repeated load triaxial testing. In such cases empirical relationships between M_R and different material properties are critical during the pavement design process. Many states, including Idaho, have vast amounts of test data on unbound materials collected throughout the years that may be utilized. The current research effort will test unbound materials commonly used in the state of Idaho for pavement applications to establish their resilient modulus properties and will also evaluate the suitability of different empirical correlations available to predict the M_R values from different soil and aggregate properties.

Determining Resilient Modulus Values for Unbound Pavement Layers

Methods to determine the resilient modulus of unbound pavement layers can be broadly classified into three categories: (1) Laboratory Test Methods, (2) Field Test Methods, and (3) Direct Correlations with Different Soil/Aggregate Properties.

Laboratory Test Methods

Typical examples of laboratory test methods include: (a) Repeated Load Triaxial Testing, (b) Resonant Column Test, (c) Hollow Cylinder Test, and (d) Simple Shear Test.

Field Methods

Field test methods that can be used to determine the in-place modulus values for unbound pavement layers include: (a) Falling Weight Deflectometer (FWD), (b) Light Weight Deflectometer (LWD), (c) Soil Stiffness Gauge or GeoGauge®, and (d) Seismic Pavement Analyzer. The in-place modulus values measured using these devices can often be used in place of the M_R value for pavement analysis and design purposes. Besides the above-mentioned methods for direct measurement of in-place modulus, results from the following field tests can be correlated with the layer resilient modulus values: (a) Dynamic Cone Penetrometer (DCP), (b) Cone Penetration Test (CPT), and (c) Plate Load Test.

Direct Correlations with Different Soil/Aggregate Properties

In this approach, the M_R value can be estimated using correlation equations with other soil index and strength properties. Some of the properties commonly used to predict M_R are: CBR, R-value, and AASHTO layer coefficient. Additionally, other indices such as PI and gradation as well as Dynamic Cone Penetrometer (DCP) Index have been used through indirect correlations to first predict CBR, which is then used to predict the M_R.[4]

Out of the above-listed methods, repeated load triaxial testing in the lab is most commonly used in the laboratory for M_R determination. Figure 5 shows a schematic representation of different stresses applied to a cylindrical specimen during repeated load triaxial testing.

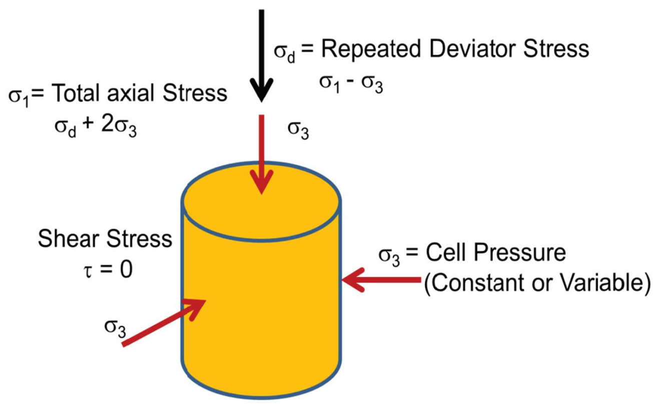


Figure Source: Tutumluer (2013)[7]

Figure 5. Stresses Applied to a Cylindrical Specimen during Repeated Load Triaxial Testing

Factors Affecting Resilient Modulus Properties of Unbound Materials

Particle size distribution or the gradation of the material has significant impact on the resilient modulus of the aggregates/soils. Coarse-grained particles exhibit more interlocking under the repeated loading, results higher resilient modulus. On the other hand, fine grained particles act as bonding or filler material to aggregate matrix, and minimize the movement of the coarse particles. [11-13]

Models to Predict Resilient Modulus of Unbound Materials

Several constitutive models have been developed over the years to predict the resilient modulus of unbound materials based on different stress state parameters.

Table 1 presents a list of different resilient modulus models developed over the years and reported in the literature. Among those listed in Table 1, the most-commonly used models are:

1. K-θ model (developed by Hicks and Monismith in 1971) [14]
2. Uzan Model (developed by Uzan in 1985) [15]

3. Modified Uzan Model (developed by Witczak and Uzan in 1988) [16]
4. MEPDG Model (developed by ARA inc. in 2004) [4]

Table 1. Chronological List of Different Resilient Modulus Models

SL #	Model Proposed By	Model Formulation	Notes
1	Biarez (1961) [17]	$E = K (\sigma_m)^n$	E = Secant modulus K, n are empirical constants
2	Dunlap (1963) [18]	$M_R = k_1 p_a \left(\frac{\sigma_3}{p_a}\right)^{k_2}$	
3	Seed et al. (1967) [19,20]	$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2}$	Primarily for granular soils
4	Hicks and Monismith (1971) [14]	$M_R = k_1 (\theta)^n$	$K - \theta$ Model
5	Shackel (1973) [21]	$M_R = k_1 \left\{\frac{(\tau_{oct})^{k_2}}{(\sigma_{oct})^{k_3}}\right\}$	
6	Boyce (1980) [22]	$K = \frac{k_i p^{(1-n)}}{1-\beta\left(q/p\right)^2}$ $G = G_i p^{(1\text{-}n)}$	Bulk Shear Model
7	Moossazadeh and Witczak (1981) [23]	$M_R = k_1 p_a \left(\frac{\sigma_d}{p_a}\right)^{k_2}$	Deviatoric Model
8	Uzan (1985) [15]	$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\sigma_d}{p_a}\right)^{k_3}$	Normalized Shackel (1973) Model
9	Lade and Nelson (1987) [24]	$E = M p_a \left[\left(\frac{I_1}{p_a}\right)^2 + R \frac{j_2}{p_a}\right]^{\lambda}$	Lade and Nelson Model
10	Witczak and Uzan [16] (1988); Modified Uzan Model	$M_R = k_1 p_a \left(\frac{\sigma_\theta}{p_a}\right)^{k_2} \left(\frac{t_{oct}}{p_a}\right)^{k_3}$	
11	Itani (1990) [25]	$M_R = k_1 p_a \left(\frac{\sigma_\theta}{p_a}\right)^{k_2} (\sigma_d)^{k_3} (\sigma_3)^{k_4}$	Itani Model
12	Crockford et al. (1990) [26]	$M_R = \beta_0 \left(\theta + 3\psi \frac{v_w}{v_t}\right)^{\beta_1} (\tau_{oct})^{\beta_2} \left(\frac{\gamma}{\gamma_w}\right)^{\beta_3}$	Crockford et al. Model
13	Pezo (1993) [27]	$M_R = k_1 p_a \left(\frac{\sigma_3}{p_a}\right)^{k_2} \left(\frac{\sigma_d}{p_a}\right)^{k_3}$	UT-Austin Model

Table 1 (continued). Chronological List of Different Resilient Modulus Models

SL #	Model Proposed By	Model Formulation	Notes
14	Lytton (1995) [(28)]	$M_R = k_1 p_a \left(\frac{I_1 - 3\theta f h_m}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a}\right)^{k_3}$	Lytton Model
15	Kolisoja (1997) [(29)]	$M_R = A\,(n_{max} - n) p_a \left(\frac{\theta}{p_a}\right)^{0.5}$	Effect of density included n = porosity of the aggregate
16	Ni et al. (2002) [(30)]	$M_R = k_1 p_a \left(1 + \frac{\sigma_3}{p_a}\right)^{k_2} \left(1 + \frac{\sigma_d}{p_a}\right)^{k_3}$	UKTC Model
17	Ooi et al. (2004) [(31)]	$M_R = k_1 p_a \left(1 + \frac{\theta}{p_a}\right)^{k_2} \left(1 + \frac{\sigma_d}{p_a}\right)^{k_3}$	
18	Ooi et al. (2004) [(31)]	$M_R = k_1 p_a \left(1 + \frac{\sigma_3}{p_a}\right)^{k_2} \left(1 + \frac{\tau_{oct}}{p_a}\right)^{k_3}$	
19	ARA, Inc. (2004) [(4)]	$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(1 + \frac{\tau_{oct}}{p_a}\right)^{k_3}$	Adopted in the MEPDG (ARA Inc., 2004).[(4)]
20	Gupta et al.(2007) [(32)]	$M_R = k_1 p_a \left(\frac{\sigma_b - 3 k_6}{p_a}\right)^{k_2} \left(k_7 + \frac{\tau_{oct}}{p_a}\right)^{k_3} + \alpha_1 (U_a - U_w)^{\beta_1}$	$(U_a - U_w)$ = Matric Suction Model

Correlations to Predict Resilient Modulus of Unbound Materials

Standard laboratory repeated-load triaxial tests used to determine the resilient modulus of unbound materials (NCHRP 1-28A and AASHTO T 307)[(9,10)] are complex and time consuming. Moreover, the equipment required to conduct this test is not readily available to all state agencies. During his survey of U.S. state and Canadian provincial transportation agencies, Tutumluer (2013) [(7)] observed that only fourteen (14) out of forty-six (46) agencies reported conducting resilient modulus testing on unbound materials. Accordingly, most other agencies relied on numerous correlation equations available in the literature to predict M_R values based on other easy-to-measure properties. Most of these correlations have been developed using regression analyses in which resilient modulus test results are correlated with results obtained from less expensive or more conventional test such as R-value, CBR, unconfined compression test, and index property tests. [(33)] However, it should be noted that because of large diversity in subgrade and weather condition across the U.S. and uncertainties associated with test results, correlation equations developed in a particular region is often not applicable to other regions with satisfactory reliability. For example, Montana Department of Transportation (MDT) selected over 30 such correlation equations and examined them using data from two MDOT soil survey reports.[(34)] Their evaluation indicated that there was little to no consistency between the equations for predicting M_R from soil index properties. These correlations were typically developed for specific groups of soil types or for soils obtained from specific geographic regions. There is no currently recognized unified general approach for using correlation equations, and the reliability of these equations are often uncertain. As most correlations are site or region specific, and most correlations do not account for important variations in soil type and consistency, different states revert to developing correlation equations for their regional soil and aggregate types to get the rational input value in ME design programs. The following sections discuss different correlations available in the literature and used by different agencies, to predict M_R values for soils and aggregates based on other material properties.

Correlation with Soil/Aggregate Gradation and Index Properties

The primary soil (both fine and coarse-grained) properties that influence the resilient modulus of pavement subgrade and base/subbase layers are: (1) moisture content, (2) compaction level (or density), (3) gradation, often represented as the percent passing #200 sieve or material finer than the 0.075 mm sieve, (4) liquid limit, and (5) plasticity index. In addition to this, compressive strength, degree of saturation, percent clay and percent silt also influence the M_R values of subgrade soils. The potential benefit of calculating M_R from soil properties is that the effect of seasonal variations and physical properties can be incorporated in the M_R prediction. [35] Some of the correlation equations reported in the literature have been given below (Table 2).

Table 2. Correlation of M_R with Material Index Properties

Soil Types	Correlation Equation	M_R Test Condition	Coefficient of Determination (R^2)	Studied by
A-7-6	M_R (ksi) = -0.1328xW + 0.0134xS + 2.319	Disturbed Samples	0.94	Jones and Witczak(1977)[36]
	M_R (ksi) = -0.111xW + 0.0217xS + 1.179	Undisturbed Samples	0.45	
A-4, A-6 and A-7	M_R (ksi) = 6.37 + 0.45(PI) – 0.0038x(%Silt) + 0.034x(%Clay) – 1.64x(%OC) – 0.244(GI)	$\sigma_3 = 0$ $\sigma_d = 6$ psi	0.64	Robnett and Thompson (1973)[37] & Thompson and Robnett (1976)[38]
A-4, A-6 and A-7	M_R (ksi) = 4.46 + 0.098x(%Clay) + 0.119 (PI)	Break Point Resilient Condition	0.4	Thompson and Robnett (1979)[39] and Thompson and LaGrow (1988)[40]
	M_R (ksi) = 0.86 + 0.307 x q_u		0.47	
	M_R (ksi) = 32.9 – 0.334 x S	95% Standard Proctor Dry Density	0.41	
	M_R (ksi) = 45.2 – 0.428 x S	100% Standard Proctor Dry Density	0.50	
Fine-grained soils (CH, MH, ML, and CL)	M_R (ksi) = 37.43 – 0.457xPI – 0.618xW – 0.142x P#200 + 0.179x σ_3 -0.325x σ_d +36.42 x CH + 17.10 MH Here, SM = 1, for SM soil, 0 otherwise MH = 1, for MH soil, 0 otherwise	-	0.76	Carmichael and Stuart (1985)[41]
Coarse-grained soils (GW, GP, GM, GC, SW, SP, SM, and SC)	M_R (ksi) = 0.523 – 0.0225xW + 0.544xlog σ_t + 0.173 x SM + 0.197xGR Here, SM = 1, for SM soil, 0 otherwise GR = 1, for GR soil, 0 otherwise	-	0.84	

Table 2 (continued): Correlation of M_R with Material Index Properties

Soil Types	Correlation Equation	M_R Test Condition	Coefficient of Determination (R^2)	Studied by
A-4, A-6, A-7	M_R (ksi) = 11.21 + 0.17 x %Clay + 0.20x PI - 0.73 x W_{opt}	$\sigma_d = 4\ psi$	0.80	Elliott et al. (1988) [42]
	M_R (ksi) = 9.81 +0.13x%Clay + 0.16xPI – 0.60 x W_{opt}	$\sigma_d = 8\ psi$	0.77	
A-2, A-4, and A-7	M_R (ksi) = 0.001x[45.8 + 0.00052x(1/a) + 0.188xq_u +0.45 x PI - 0.216xσ_d - 0.25xS - 0.15xP#200)	AASHTO T-274	0.83	Drumm et al. (1990) [43]
Specific Soil classification data are not available	M_R (ksi) = (a' + b'x σ_d) / σ_d a' = [318.2 +0.337xq_u + 0.73x%Clay + 2.26xPI – 0.915x σ - 2.26xS 0.304 x P#200)] b' = [2.10 +0.00039 x(1/a) + 0.104xq_u + 0.09xLL – 0.10xP#200]		0.80	
A-2-4, A-4, A-6, A-7-6, A-7-6	M_R (ksi) = 100[B_0 + B_1G +B_2G^2 +B_3G^3+B_4G^4 Here, G = Nebraska Group Index B = Regression Coefficient (function of moisture contained in the soil)		3<NGI < 28	Woolstrum (1990) [44]
A-6, A-7-6 and A-4	M_R (ksi) = [0.001xe^a] a = 7.16 + 0.0389 R_m - 0.049 σ_d + 0.0040 σ_3 + 1.01 X_c X_c = 1 for clayey soils, otherwise 0	AASHTO T-274	0.59	Farrar and Turner (1991) [45]
	M_R = 30.28 – 0.359 S – 0.0325 σ_d + 0.237 x σ_3 + 0.086 PI + 0.107 (p#200)	AASHTO T-274	0.66	
Fine	M_R (ksi) = 2.429 [(LL/W) x (γ_d / $\gamma_{d\text{-}s}$)$^{2.06}$ + (p#200/100)$^{-0.59}$	AASHTO TP-46	0.70	Rahim (2005) [35]
Coarse	M_R (ksi) = 44.58 [(γ_d/W)$^{0.86}$ x (p#200/log C_u)$^{-0.46}$	AASHTO TP-46	0.75	
Coarse	M_R (ksi) = (- 200 - 1.51 x P_{200} – 418 x D_{30} - 3.09 x OMC + 1.94 x MDD) x γ_{dr}	AASHTO T-307	0.968	Guthrie and Jackson (2015)[46]

Correlation with R value

The resistance value of a soil (remolded sample) is determined using the stabilometer device. Sandy gravel and crushed base course aggregates generally higher R-values, typically between 60 to 80; silt and clays on the other hand have relatively low R-values. [34] Though the R-value is not mechanistically related with M_R, still some of the states are using R-value in pavement design because of its familiarity and historical use in pavement design and the availability of R-Value data. Many states have developed correlation equations to predict M_R from R-value. [34] The common test methods for R-value are ASTM D 2844 and AASHTO T 190. It is important to note that the R-value used by ITD (determined using method IT-8) is not the same as the one established using AASHTO T 190. Therefore, special caution needs to be used while using correlation equations to establish M_R from R-value test results. Some of the correlations are tabulated below in Table 3.

Table 3. Correlation of M_R with R-value

Soil Types	Correlation Equation	Limitation	Coefficient of Determination (R^2)	Studied by
Fine-grained	M_R (ksi) = 1.455 + 0.057 x R	$46 \le R \le 68$	0.10	Buu (1980)[47] ITD
Coarse-grained	M_R (ksi) = 1.600 + 0.038 x R	$9 \le R \le 82$	0.80	
Sand, Sandy Loam, Silt-Clay Loam, Silt Silty-Clay, Heavy Clay	M_R (ksi) = 0.772 + 0.369 x R	$R < 60$	Limited number of tested samples	Asphalt Institute (1982)
Clay	M_R (ksi) = 1.859 + 0.219 x R	$5 < R < 40$	0.97	Yeh and Su (1989) [48]
A-1, A-4, A-6, A-7	M_R (ksi) = 3.5 + 0.125 x R	$R < 50$	0.50	
Fine Grained Soils	M_R (ksi) = 1.0 + 0.555 x R	$R < 20$	-	AASHTO 1993[1]
Coarse-Grained Soil (A-1) Clayey or Silty Soil(A-7)	M_R (ksi) = $[0.72x(e^{0.0521xR} - 1.0)]$	$25 < R < 75$	0.67	Muench et al. (2009)[49]; WSDOT
Lack of information about soil type	M_R (ksi)=0.001x10(S1+18.72)/6.24, where, S1 = [(R-5)/11.29] + 3 The above equation ultimately yields $M_R = 0.001x10^a$ Where, a = [{(R-5)/11.29} + 21.72]/6.25	$1 < S_1 < 10$ Where S_1 is the soil support value	Sufficient Information not available	(Yeh and Su, 1989)[48]; CDOT

Correlation with California Bearing Ratio (CBR)

The California Bearing Ratio (CBR) is a measure of a material resistance to penetration by a 3 in^2 piston at a rate of 0.05 inch per minute. Typical sample dimensions used for CBR testing are: 6 in diameter and 4.58 in. tall. These specimens are compacted at different moisture contents and are subsequently soaked for 96 hours. Details of the test procedure are discussed in ASTM D 1883 and AASHTO T 193. Although the CBR value is not intrinsically related to M_R, owing to the simplicity of the test procedure, many state agencies have developed correlation equations between CBR and M_R, tabulated in Table 4.

Table 4. Correlation between CBR and M_R

Correlation Equation	Limitation	Remarks	Studied by
M_R (ksi) = 1.42 x CBR	2 < CBR < 200	The original equation was developed to predict the dynamic modulus value. However, it has been widely referenced as correlation to predict M_R	Heukelom and Klomp (1962) (50)
M_R (ksi) = 5.409 x $CBR^{0.711}$	2 < CBR < 200	The Army Crpos of Engineers developed a correlation between dynamic modulus and insitu CBR values by comparing results obtained from different road test projects.	Green and Hall (1975)(51)
M_R (ksi) = 2.554 x $CBR^{0.64}$	2 < CBR < 12	No information available about soil types or theoretical/ empirical correlations used in this study	Powell et al. (1984) (52)
M_R (ksi) = 1.0016+0.043xCBR – 1.9557 x $(\frac{\log\sigma_d}{CBR})$ – 0.1705 $\log\sigma_d$	Kaolinite clay USCS classification = ML, LL=48.2%; PI = 41.9%,	R^2 = 0.93, Confining pressure 3 psi and 5% OMC	Lofti (1984) (53)
M_R (ksi) = 1.2 x CBR	AASHTO classes A-1, A-2,A-3,A-4,A-6,A-7-6	NP	Ohio DOT (2008) (54)
M_R (ksi) = 3.0 x $CBR^{0.65}$	NP	NP	Paterson and Maree (55); South African Council
M_R (ksi) = 3116 x $CBR^{0.4779707}$	Medium clay sand	± 1.5 % OMC	George (2004) (33), GDOT

Efforts to Determine Resilient Modulus Model Parameters

Besides the above-listed research studies, several studies have been carried out to establish typical values for resilient modulus model parameters (such as the k_1, k_2, and k_3 values in the MEPDG model and some other well-established models) for typical unbound materials. Once these model parameters have been established, the values can then be used as Level 1 input during M-E pavement design efforts without requiring repeated laboratory testing. Some efforts like these are tabulated below in Table 5.

Table 5. Resilient Modulus Model Parameters for Different Constitutive Models

Material	**K_1**	**K_2**	**K_3**	**R^2**	**Model Name**	**Studied by**
	Base Materials					
CRREL	803	0.931	-0.612	0.952	MEPDG Model	Wambura (2003)[56], MDDOT
	537	1.1	-0.0561	0.997		
	622	1.01	-0.585	0.954		
GTX	685	1.124	-0.664	0.986		
	866	1.034	-0.599	0.988		
	672	1.128	-0.716	0.985		
	741	1.091	-0.653	0.986		
MSU-1	1043	0.813	-0.476	0.858		
	871	1.008	-0.763	0.872		
	957	0.906	-0.641	0.851		
MSU-2	640	1.239	-0.651	0.976		
	727	0.974	-0.481	0.933		
	685	1.113	-0.581	0.971		
	Subgrade Materials					
CS	136	0.134	-3.033	0.973	MEPDG Model	Wambura (2003)[56], MDDOT
	145	0.255	-3.986	0.979		
	142	0.183	-3.138	0.96		
	139	0.187	-3.281	0.948		
SSS	301	0.928	-0.29	0.853		
	569	1.146	-2.88	0.97		
	477	0.966	-1.872	0.811		
	449	1.03	-1.856	0.792		
GTX	232	0.422	-19.818	0.935		
	178	0.426	-18.303	0.892		
	140	0.36	-13.993	0.906		
	181	0.408	-17.391	0.637		
CRREL	195	0.508	-19.416	0.901		
	204	0.467	-17.655	0.984		
	158	0.462	-18.048	0.976		
	170	0.45	-16.388	0.635		

Table 6 (continued). Resilient Modulus Model Parameters for Different Constitutive Models

Material	K_1	K_2	K_3	R^2	Model Name	Studied By
Limerock Base	22966.7	0.4773	-	0.5618	K-θ Model	Taylor and Timm (2009)[57], NCAT
Granite Base	10862.1	0.6267	-	0.886		
Seale Subgrade	6009.8	-0.1201	-	0.0288		
Type 5 Base	14049.7	0.671	-	0.8721		
Track Soil	26833.28	0.0447	-	0.0179		
Limerock Base	39001.4	0.2174	-	0.2204	Deviatory Model	
Granite Base	21350	0.3866	-	0.5765		
Seale Subgrade	4305.81	-0.5571	-	0.7834		
Type 5 Base	29487.2	0.3876	-	0.5334		
Track Soil	28878.92	-0.0572	-	0.0478		
Limerock Base	717.04	1.2338	-0.5645	0.8562	Uzan Model	
Granite Base	581.08	0.8529	-0.187	0.9172		
Seale Subgrade	225.09	0.3598	-0.7551	0.9786		
Type 5 Base	643.69	1.0318	-0.2833	0.9349		
Track Soil	1095.43	0.593	-0.4727	0.6642		
Limerock Base	1266.83	1.2081	-1.2332	0.9326	MEPDG Model	
Granite Base	716.28	0.8468	-0.4632	0.9253		
Seale Subgrade	817.63	0.3305	-3.3946	0.957		
Type 5 Base	883.54	1.005	-0.6575	0.9478		
Track Soil	1878.97	0.4067	-0.7897	0.4202		
Aggregate/	1,032.05	0.584	-0.028	0.997	Uzan Model	Ceylan and Kim (2009) [58]
Aggregate/	1,080.55	0.585	-0.103	0.997	MEPDG Model	

Table 7 (continued). Resilient Modulus Model Parameters for Different Constitutive Models

Material	K_1	K_2	K_3	R^2	Model Name	Studied By
Select/ OMC+4	134.309	0.337	-0.319	0.896	Uzan Model	
Select/OMC	464.692	0.301	-0.281	0.978		
Select/OMC-4	612.569	0.273	-0.201	0.97		
Class10/OMC+4	123.125	0.251	-0.359	0.957		
Class10/OMC	384.965	0.241	-0.192	0.924		
Class10/OMC-4	603.274	0.231	-0.175	0.953		
Unsuitable/ OMC+4	146.05	0.319	-0.369	0.901		
Unsuitable/ OMC	428.226	0.228	-0.182	0.937		
Unsuitable/ OMC-4	515.084	0.156	-0.173	0.904		
Select/OMC+4	284.582	0.322	-2.217	0.777	MEPDG Model	
Select/OMC	921.706	0.305	-2.105	0.983		
Select/OMC-4	1,002.83	0.277	-1.523	0.99		
Class10/ OMC+4	293.805	0.252	-2.658	0.94		
Class10/OMC	618.125	0.247	-1.476	0.969		
Class10/OMC-4	927.177	0.236	-1.335	0.993		
Unsuitable/ OMC+4	363.946	0.335	-2.855	0.968		
Unsuitable/ OMC	671.567	0.234	-1.401	0.983		
Unsuitable/OMC-4	792.418	0.164	-1.352	0.983		

Apart from the above listed works, other states like Wisconsin,[59] Kentucky [60] etc. have carried out similar kind of studies and developed the database for the implementation of MEPDG in their states. Hossain et al. (2013) [61] took one step ahead to predict these coefficients of MEPDG model with the help of inherent material properties and some index properties of the material. Accordingly, Hossain et al. (2013) [61] recommended the following three equations to predict the k_1, k_2, and k_3 values.

$k_1 = 259.44 \times P_{200} - 1.951 \times \mathrm{UCS}$ $\quad R^2 = 0.38$

$k_2 = 0.53 - 0.902 \times k_3$ $\quad R^2 = 0.78$

$k_3 = -0.044 \times \mathrm{OMC} + 0.087 \times \mathrm{PI}$ $\quad R^2 = 0.42$

Summary

This chapter provided a brief discussion on test methods commonly used to determine the resilient modulus of unbound materials and discussed factors that affect the resilient modulus values of soils and aggregates. In addition, this chapter also summarized the historical development of different correlation equations to predict the resilient modulus (M_R) values of unbound materials (soils and aggregates) based on different material and index properties. Several previous studies have attempted to predict the resilient modulus values of unbound materials using other 'east-to-establish' material and index properties such as gradation, moisture content, density, Atterberg limits, etc. Finally, this chapter also presented the correlations between resilient modulus with CBR and R-value that have been explored by researchers in the past.

Chapter 3
Material Collection and Preliminary Laboratory Characterization

Introduction

As already discussed, the primary objective of this research project was to develop a database with unbound material properties that can be used by ITD engineers during pavement design and performance prediction using Pavement ME Design. The first step towards fulfilling this objective involved identifying representative unbound materials (soils and aggregates) that are used in pavement subgrade and base/subbase applications in the state of Idaho. It is important to note that identification of typical aggregates used for base/subbase applications is relatively simple compared to typical soils encountered as subgrades. This is because ITD districts use certain state-approved quarries to obtain aggregates for use in base/subbase layers. However, the subgrade encountered during a project is entirely a function of the geographical location where the pavement is being constructed. Nevertheless, ITD district materials engineers were asked to identify typical soil types that were encountered in their respective districts. This information was then used to coordinate with district materials engineers to collect subgrade samples from projects where the construction/rehabilitation efforts led to exposure of the subgrade layer. Although it was not possible to obtain all different desired subgrade types, the research team ensured samples were obtained from as many locations as practically possible, so that the laboratory test results will be representative of typical pavement conditions in the state.

A Survey Questionnaire for Material Selection

A short questionnaire was developed and sent to all six ITD district materials engineers to identify representative soils and aggregates from each district for laboratory characterization. Table 8 lists the questions that were included in this questionnaire. Once the materials were selected, the research team coordinated with the district materials engineers to deliver all aggregate (base and subbase) materials to University of Idaho for laboratory testing, whereas all subgrade soils were tested at Boise State University. Typical aggregate types used for base/subbase applications selected through questionnaire are listed in Table 9, whereas Table 10 lists typical subgrade soil types collected for laboratory characterization under the scope of the current study.

Table 8. Questionnaire Sent to ITD Districts to Identify Representative Unbound Materials for Laboratory Characterization

Q.1	How many different aggregate sources (identified as material types; example: Vy-10) do contractors in your district commonly use for pavement base layer applications? Please note that this question is referring to ¾ inch minus material meeting ITD's base material specifications. Please list them in the decreasing order of frequency of usage (the most-used material source gets listed first)		
Material # 1			
Material # 2			
Material # 3			
Material # 4			
Material # 5			
Q.2	How many different aggregate sources do contractors in your district commonly use for pavement subbase layer applications? Please note that this question is referring to materials that may not meet ITD specifications for base layer usage, but can be used in pavement subbase layers. Please list them in the decreasing order of frequency of usage (the most-used material source gets listed first)		
Material # 1			
Material # 2			
Material # 3			
Material # 4			
Material # 5			
Q.3	How many different types of subgrade soil materials (e.g. CL, CH, ML, SM, etc.) do you encounter across your district during pavement construction? Please list them in the decreasing order of frequency of usage (the most common soil type gets listed first)		
Material # 1			
Material # 2			
Material # 3			
Material # 4			
Material # 5			
Q.4	Which ones of the above listed materials (listed in questions 1 through 3) do you think should be tested in the laboratory for representative characterization of unbound materials used in pavement applications across your district?		
Types	**Aggregate Base**	**Granular Subbase**	**Subgrade Soil**
Material # 01			
Material # 02			
Material # 03			

Table 9. Representative Unbound Aggregate Materials Selected for Laboratory Characterization

Districts	Used in Layer (Base / Subbase)	Material ID
District 1	Base	KT-215
	Base	BR-2
District 2	Base	NP - 82
	Base	WCW
District 3	Base	EL-132
	Base	VY-63
	Base	CN-140
	Subbase	CN-148
	Subbase	PY-720
District 4	Base	LN -80
	Base	CS-184
	Subbase	CS-184
District 5	Base	PW-84
	Base	BK-181
	Base	BK-100
District 6	Base	LE-160
	Base	A
	Base	B

Table 10. Representative Subgrade Soils Collected for Laboratory Characterization

Source	Material Type	Material Name
District 1	Subgrade	D1-ML
		D1-GM
District-2	Subgrade	17-9SL-0101
District-3	Subgrade	TP-8
		D3-SM
		D3-SC
District-4	Subgrade	Cs-184
		LN-80
District-5	Subgrade	Bk-180c
		Bg-112c
District-6	Subgrade	BN-59
		JF-83
		17-9SL-0054
		17-9SL-0055
		17-9SL-0057
		17-9SL-0058

As listed in Tables 9 and 10, a total of nineteen (19) aggregate types were acquired; however, 18 materials were tested in the laboratory. One aggregate sample was not stable during compaction, so it was discarded. The total number of subgrade soils collected was: sixteen (16). Also, one of the soil samples obtained from District-5 (Bg-112c) was extremely sandy in nature and could not be tested using conventional soil compaction and sample preparation approaches.

Particle Size Distribution, Atterberg's Limits, and Soil Classification

Prior to resilient modulus testing, the following preliminary/routine tests were conducted: (1) Particle size distribution, (2) Atterberg limit testing, and (3) Moisture-density testing to establish the compaction characteristics. Information from the gradation and Atterberg's limit tests were used to classify the soils, whereas, the compaction characteristics were subsequently used to prepare specimens for repeated load triaxial testing. Finally, the subgrade soils were also tested for other common properties such as (a) Unconfined Compressive Strength, and (b) Unsoaked California Bearing Ratio (CBR). All laboratory tests were performed following the relevant AASHTO standards. For example, the particle size distribution (gradation) tests were carried out per AASHTO T 27 or ASTM C 136. The particle size distribution of the aggregates and soils are listed in Table 11 and Table 12, respectively. The corresponding gradation plots have been presented in Figures 6 and 7, respectively.

Table 11. Particle Size Distribution for Aggregate Materials Tested in the Current Study

Materials	Sieve Size (mm)										
	25.00	19.00	12.50	9.50	4.75	2.36	1.18	0.60	0.30	0.15	0.075
	% Passing										
A	100	100	88	74	48	34	26	19	12	4	1.4
B	100	100	81	60	36	31	26	20	13	4	1.3
BK - 100	100	98	87	79	57	35	20	12	8	5	3.4
BK - 181	100	96	77	65	41	25	17	12	8	4	2.4
BR - 2	100	99	83	67	37	21	12	7	4	3	1.8
CS - 184	100	97	79	64	45	32	22	14	8	4	2.6
EL - 132	93	89	78	70	53	42	33	25	14	6	2.3
IMC - 140	100	97	76	65	51	43	35	19	8	4	1.7
KT - 215	100	97	87	78	54	32	21	14	9	6	3.4
LE - 160	100	96	76	62	41	29	22	16	7	3	2.0
LN - 80	100	98	87	77	53	38	29	19	8	3	1.2
NP - 82	100	100	95	82	51	29	18	13	9	6	4.2
PW - 84	100	97	87	81	65	48	32	19	11	6	4.3
VY - 63	100	96	63	45	24	12	6	3	2	2	1.2
WCW	100	100	93	81	57	36	23	16	12	9	4.6
CN - 148 SB	98	90	79	71	61	55	49	34	15	8	5.1
CS - 184 SB	100	93	86	78	58	42	31	21	12	6	2.1
PY - 720 SB	100	84	73	69	62	57	49	36	21	7	2.6

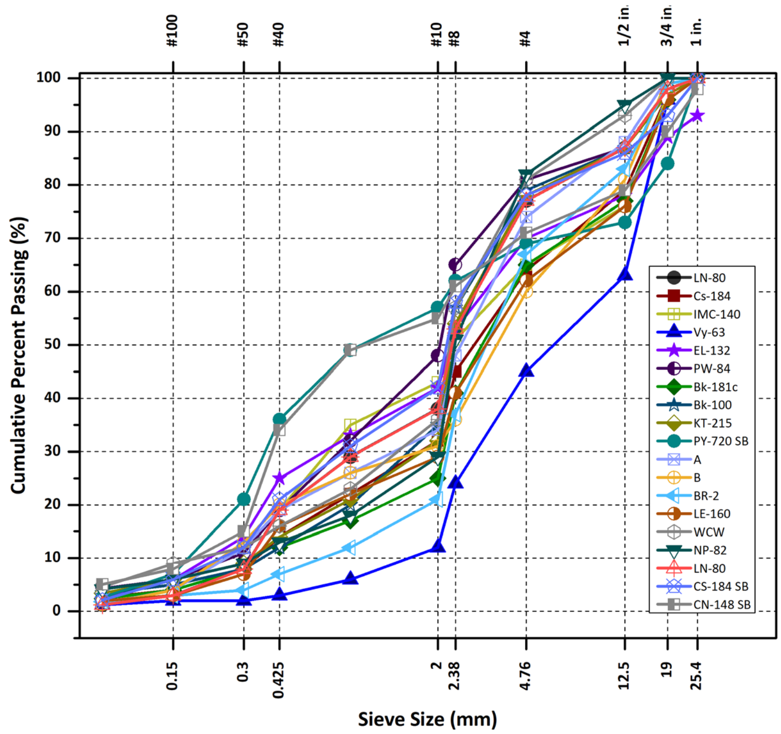


Figure 6. Gradation Curves for Aggregate (Base/Subbase) Materials Tested in the Current Study

Table 12. Particle Size Distribution for Subgrade Soils Tested in the Current Study

Materials	Sieve Size (mm)										
	25.0	19.0	12.5	4.75	2.38	2.0	0.85	0.425	0.3	0.15	0.075
	% Passing										
D1-ML	100	100	100	100	100	100	97	86	78	67	48.8
D1-GM	85	77	66	45	35	33	25	21.0	19.0	16	12.5
17-9SL-0101	81	77	70	52	48	45	40	36	34	28	22.2
TP-8	100	100	100	100	100	100	100	92	85	71	53.9
D3-SM	100	100	100	99	92	90	73	50	41	25	12.9
D3-SC	100	100	100	99	96	95	83	70	62	47	24.4
Cs-184	100	100	100	100	90	86	62	45	38	28	20.5
LN-80	92	89	82	63	53	51	41	28	19	9	3.4
Bk-180c	96	94	85	57	42	39	26	18	15	10	5.7
Bg-112c	86	79	59	35	32	31	29	21	16	6	2.2
BN-59	100	100	100	75	44	39	22	16	14	10	4.7
JF-83	100	100	96	55	39	35	27	23	21	11	4.7
17-9SL-0054	100	99	97	90	80	84	72	61	57	47	38.6
17-9SL-0055	100	100	97	88	82	79	64	53	48	38	31.5
17-9SL-0057	100	100	97	92	89	72	66	59	54	46	38.1
17-9SL-0058	100	100	100	99	97	96	79	67	63	56	49.9

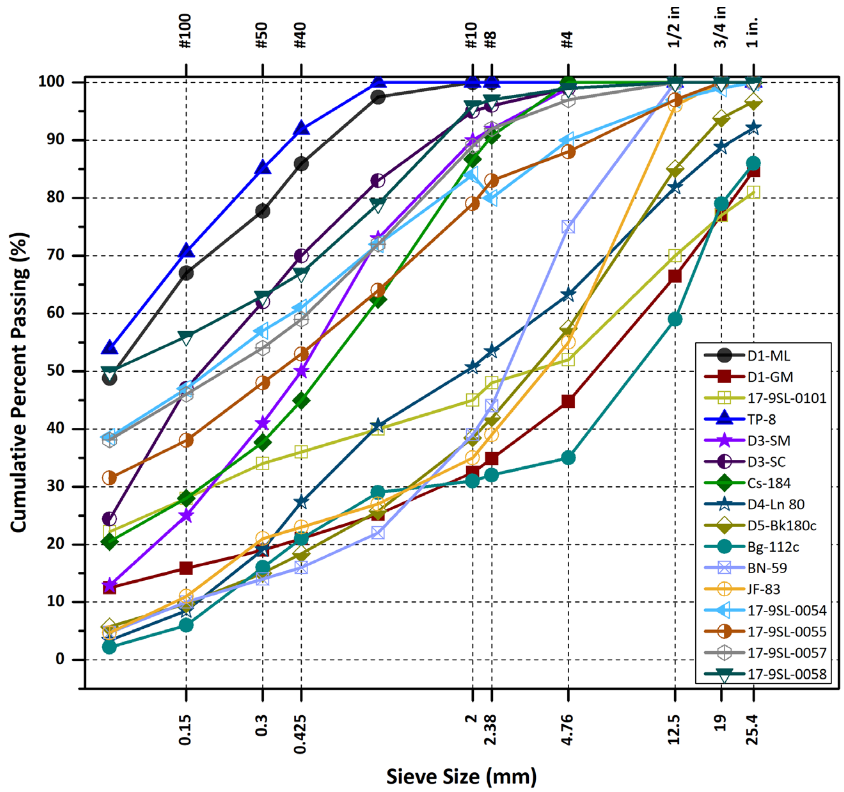


Figure 7. Gradation Curves for Subgrade Soils Tested in the Current Study

As seen from Figure 7, several of the subgrade materials delivered to Boise State laboratories comprised significant amounts of coarse particles. For example, the D1-GM material comprised only 45% particles by weight that passed the 4.75 mm sieve (boundary between coarse and fine-grained soils per USCS classification). Therefore, to homogenize all subgrade materials, and to ensure that the laboratory testing focused primarily on the fine fraction of the materials, all subgrade soils in the current study were tested on the material fraction passing through the 4.75 mm sieve. It should be noted that laboratory strength and modulus properties for these materials established by testing only the fine fraction might be lower than values that would have been established if the entire material gradation was tested. However, no clear differentiation between the subgrade and base/subbase materials would have been possible if the subgrade materials were tested by including all size fractions. Moreover, the strength and modulus values established by testing only the fine fraction would constitute more "conservative" inputs during pavement analysis and performance prediction using Pavement ME Design. *It should be noted that all test results for the subgrade soils presented in this report correspond to material fraction finer than 4.75 mm*.

Atterberg's limits represent the most commonly used index properties for unbound materials (soils and aggregates) and are often used to assess the suitability of a particular soil or aggregate type to be used in a pavement layer. As these index properties are relatively easy to establish in the laboratory, they are widely used in state transportation agency specifications developed to assess material suitability. The Plasticity Index (PI), which is the difference between the Liquid and Plastic Limits, is often used to differentiate between "good quality" and "poor quality" materials; soils and aggregates with high PI values are commonly accepted as "poor quality" materials. Table 13 and Table 14, respectively, list the Atterberg's limit values for the aggregates and soils tested in the current study. Cells designated as N/V (No value) correspond to instances where the particular index property for a material type could not be established.

Table 13. Atterberg's Limit Values for Aggregate Materials Tested in the Current Study

Material ID	Liquid Limit (LL, %)	Plastic Limit (PL, %)	Plasticity Index (PI, %)
A	N/V	N/V	N/P
B	N/V	N/V	N/P
BK – 100	18	14	4
BK – 181	19	14	5
BR – 2	21	15	6
CS – 184	N/V	N/V	N/P
EL – 132	N/V	N/V	N/P
IMC – 140	N/V	N/V	N/P
KT – 215	18	18	N/P
LE – 160	19	19	N/P
LN – 80	N/V	N/V	N/P
NP - 82	17	15	2
PW - 84	18	16	2
VY – 63	23	17	6
WCW	N/V	N/V	N/P
CN - 148 SB	N/V	N/V	N/P
CS - 184 SB	20	18	2
PY - 720 SB	N/V	N/V	N/P
*N/V: N/V **N/P: Nonplastic			

Table 14. Atterberg's Limit Values for Subgrade Soils Tested in the Current Study

Material ID	Liquid Limit (LL, %)	Plastic Limit (PL, %)	Plasticity Index (PI, %)
D1-ML	30	27	3
D1-GM	32	29	3
17-9SL-0101	19	N/V	N/P
TP-8	26	21	5
D3-SM	28	24	4
D3-SC	41	27	14
Cs-184	29	16	13
LN-80	N/V	N/V	N/P
Bk-180c	19	N/V	N/P
Bg-112c	N/V	N/V	N/P
BN-59	N/V	N/V	N/P
JF-83	N/V	N/V	N/P
17-9SL-0054	23	N/V	N/P
17-9SL-0055	21	N/V	N/P
17-9SL-0057	20	N/V	N/P
17-9SL-0058	30	17	13
* N/V: N/V **N/P: Nonplastic			

Results from the gradation and Atterberg's limit tests were used to classify the different aggregate and soil types. The final soil classifications (per AASHTO and USCS conventions) have been listed in Table 15 and Table 16.

Table 15. Classifications of the Aggregate (Base/Subbase) Materials Tested in the Current Study

Material ID	**Soil Classification**		**Material ID**	**Soil Classification**	
	Unified	**AASHTO**		**Unified**	**AASHTO**
A	GW	A-1-a	LE – 160	GW	A-1-a
B	GW	A-1-a	LN – 80	SP	A-1-a
BK – 100	SW	A-1-a	NP – 82	GW	A-1-a
BK – 181	GW	A-1-a	PW – 84	SW	A-1-a
BR – 2	GW	A-1-a	VY – 63	GW	A-1-a
CS – 184	GW	A-1-a	WCW	SP	A-1-a
EL – 132	SP	A-1-a	CN - 148 SB	SP	A-1-a
IMC – 140	SP	A-1-a	CS - 184 SB	SW	A-1-a
KT – 215	SW	A-1-a	PY - 720 SB	SP	A-1-a

Table 16. Classification of Subgrade Soils Tested in the Current Study

Material ID	Soil Classification		Material ID	Soil Classification	
	Unified	AASHTO		Unified	AASHTO
D1-ML	ML	A-4	Bk-180c	SW	A-1-a
D1-GM	GM	A-1-a	Bg-112c	GP	A-1-a
17-9SL-0101	GM	A-1-b	BN-59	SP	A-1-a
TP-8	CL-ML	A-4	JF-83	GP	A-1-a
D3-SM	SM	A-2-4	17-9SL-0054	SM	A-4
D3-SC	SC	A-2-6	17-9SL-0055	SM	A-2-4
Cs-184	SC	A-2-4	17-9SL-0057	SM	A-4
LN-80	SW	A-1-b	17-9SL-0058	SC	A-6

Moisture Density Relationships

Once the grain size distribution and Atterberg's limits for the materials were established, the next task involved establishing the moisture-density characteristics. This is particularly important because the optimum moisture content and maximum dry density values thus established, would subsequently be used during specimen compaction for repeated load triaxial testing. Depending on agency requirements, the compaction (or moisture-density) characteristics for unbound materials are established using Standard (ASTM D 698 or AASHTO T 99) or Modified (ASTM D 1557 or AASHTO T 180) compactive efforts. Following ITD engineers' recommendations, aggregate specimens in the current study were prepared using the modified compactive effort, whereas subgrade soil specimens were prepared using the standard compactive effort. The following sections present the compaction (moisture-density) characteristics of the soils and aggregates as established in the laboratory.

For the base/subbase materials the applicable compaction procedure was: AASHTO T 180 (Method D) or ASTM D 1557 (Method D). At least two replicates were prepared for the moisture density relationship test. Figure 8 presents an example curve showing the moisture-density relationship for one of the aggregate materials (LE - 160) tested in the current study. Table 17 lists the Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) values for all the aggregate materials tested.

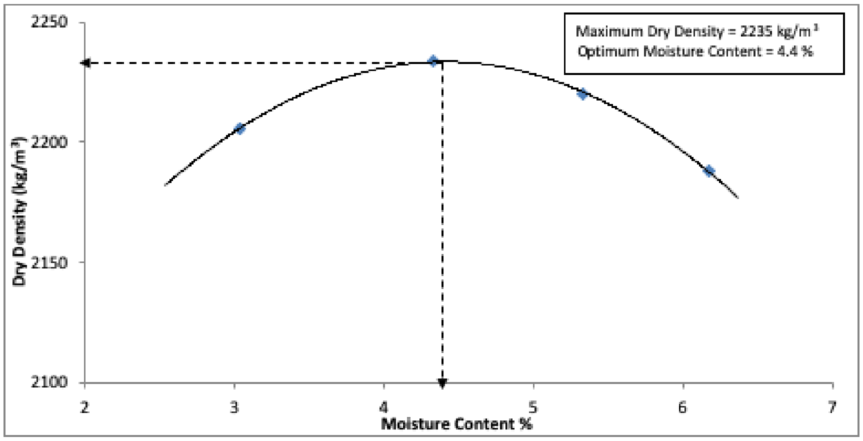


Figure 8. Example Moisture-Density Curve for One of the Aggregate Materials Tested

Table 17. Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) Values for the Aggregate Materials Tested in the Current Study

Material	OMC	MDD	
ID	%	lb./ft^3	Kg/m^3
A	4.4	140.2	2245
B	4.4	140.7	2253
BK – 100	4.7	142.9	2290
BK – 181	5.7	152.6	2445
BR – 2	5.6	142.3	2280
CS – 184	6.5	137.9	2210
EL – 132	6.0	142.1	2275
IMC – 140	6.2	142.9	2290
KT – 215	4.8	141.4	2265
LE – 160	4.4	139.5	2235
LN – 80	5.8	140.5	2250
NP – 82	3.5	141.1	2261
PW – 84	5.6	147.6	2365
VY – 63	4.3	141.4	2265
WCW	4.9	143.6	2300
CN - 148 SB	5.5	135.3	2168
CS - 184 SB	8.1	134.1	2148
PY - 720 SB	6.6	135.2	2165

Compaction characteristics for the selected subgrade soils were established in accordance with AASHTO T99 (method A). After air drying, materials were sieved thorough 4.75 mm sieve and compacted in three layers in a mold with a diameter of 101.6 mm (4 in.) and height of 116.43 mm (4.584 in.). Each layer was compacted using 25 blows. Figure 9 shows an example moisture-density curve for one of the materials

(CS-184). As expected, the dry density increases with compaction moisture content up to a maximum value, and then decreases as the compaction moisture contents is gradually increased. Table 18 lists the OMC and MDD values for all subgrade soils tested in the current study.

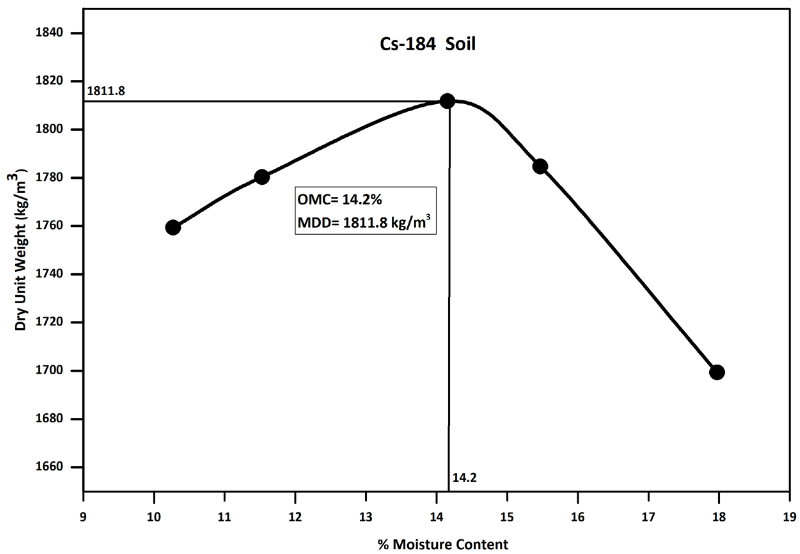


Figure 9. Graphical Representation of Moisture Density Test of Cs-184 Soil

Table 18. Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) Values for the Subgrade Soils Tested in the Current Study

Material ID	OMC	MDD	
	%	lb/ft^3	Kg/m^3
D1-ML	16.8	107.2	1717.2
D1-GM	15.0	109.8	1758.8
17-9SL-0101	16.0	108.0	1730.0
TP-8	16.4	109.5	1754.0
D3-SM	16.9	107.8	1726.8
D3-SC	22.8	92.7	1484.9
Cs-184	14.2	113.2	1811.8
LN-80	8.5	127.5	2042.4
Bk-180c	8.0	134.0	2146.5
BN-59	8.2	118.9	1904.6
JF-83	8.3	126.7	2029.5
17-9SL-0054	14.9	102.7	1645.1
17-9SL-0055	10.7	121.5	1946.2
17-9SL-0057	11.8	117.5	1882.2
17-9SL-0058	12.8	116.0	1858.1

California Bearing Ratio (CBR) Testing

As already mentioned, one of the objectives of this research study was to evaluate whether resilient modulus properties for unbound materials (soils and aggregates) could be predicated with reasonable accuracy using other 'easy to measure' index or mechanical properties. The California Bearing Ratio (CBR) is one such 'easy to measure' index property which has been correlated to the shear strength of soils and aggregates. Moreover, the CBR has often been correlated with the resilient modulus value of soils and aggregates. In fact, the mechanistic empirical pavement design guide (implemented through AASHTOWare Pavement ME Design) uses a correlation to calculate the resilient modulus value of soils and aggregates from measured CBR values (for design level 2). Therefore, the research team decided to conduct CBR tests on the soils and aggregates selected for testing in the current study and assess whether a 'reasonable' correlation between CBR and resilient modulus could be established. The research team is well aware of the fact that CBR testing is not part of the regular suite of tests conducted by ITD on soils and aggregates. However, if a reasonable correlation between CBR and resilient modulus can be found, including CBR testing into agency specifications would be significantly more convenient than requiring repeated load triaxial testing to determine resilient modulus values. California Bearing Ratio (CBR) testing in the current study was carried out with according to AASHTO T 193. Note that the CBR testing on the compacted specimens was performed under unsoaked conditions, and therefore no measure of aggregate/soil swelling under immersed conditions was carried out. Overall the CBR in unsoaked conditions provides higher values compared to soaked conditions. The following sections briefly discuss the CBR testing procedure as well as the test results.

Specimen Preparation for CBR Testing

Preparing Aggregate Specimens

For each specimen, the aggregate materials were compacted in a mold of 152.4 mm (6 in.) diameter and 177.8 mm (7 in.) height; a 61.4-mm (2.42-in.) thick spacer disk was placed inside the mold. Therefore, the compacted specimen had a diameter of 152.4 mm (6 in.), and was 116.3-mm (4.58-in.) tall. The aggregate specimens for CBR testing were prepared under OMC and MDD conditions established using the modified compactive effort. This was necessary because the CBR values at OMC and MDD conditions were later checked against resilient modulus values to determine whether consistent correlations existed between the two properties. Five lifts with fifty-six blows per layer were used to prepare the aggregate specimens for CBR testing.

Preparing Subgrade Specimens

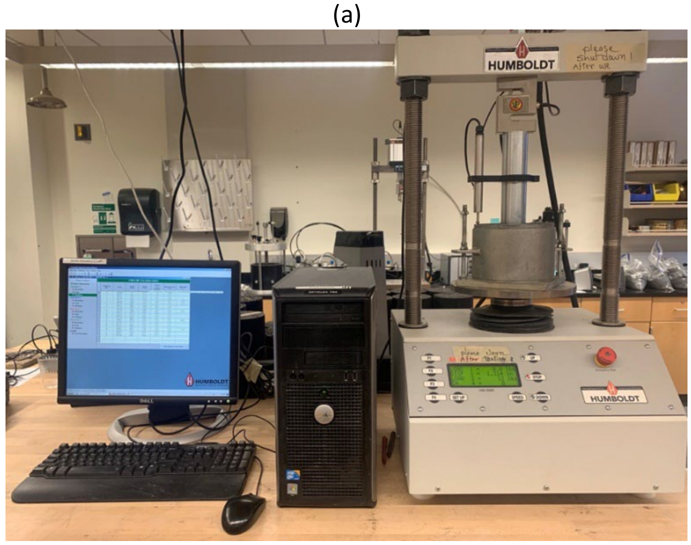
The collected soils were sieved thorough 4.75 mm sieve after drying overnight in the oven at 60^{o}C temperature. Soil specimens were compacted in 152.4 mm diameter and 177.8 mm height mold at three different moisture content to study the moisture sensitivity of materials. The rammer having weight of 2.5 kg were used to compact the materials at three different layers with 56 blows in each layer. The specimens were compacted at 90% of OMC, 100% of OMC, and 110% of OMC; the compactive energy was maintained constant at 100% of the standard compaction effort. After obtaining the load-deformation data, proper graphical correction was carried out to account for initial penetration of the piston into the specimen. Figure 10 shows a photograph of a subgrade soil specimen being prepared in the Boise State laboratory for CBR testing.

Figure 10. Subgrade Soil Specimen being Compacted for CBR Testing

Conducting the CBR Test

Once the specimens were compacted, a surcharge load of 2.27 kg (5 lb.) was placed on top of the compacted specimen. The test was conducted by a piston of 1935 mm^2 (3 in^2) cross-sectional area moving at a rate of 1.3 mm/min (0.05 in./min). Resistance felt by the penetrating piston was recorded using a load cell. Figure 11 shows photographs of the CBR test set-up at University of Idaho (Figures 11-a) and Boise State University (Figure 11-b).

(a)



(b)

Figure 11. CBR Test Set-ups at (a) University of Idaho; and (b) Boise State University

After obtaining the load-deformation data, proper graphical correction was performed in accordance with AASHTO T 193. Afterwards, the corrected stresses corresponding to penetration values of 2.54 mm (0.1 in.) and 5.08 mm (0.2 in.) were computed and divided by the standard stresses listed in AASHTO T 193 (6.9 MPa for 2.54 mm penetration, and 10.3 MPa for 5.08 mm penetration), to obtain the ratio of the measured stress to the standard stress at the same deflection. The greater of the two values was subsequently multiplied by 100 to establish the CBR value for the particular material type. To achieve the consistency and reliability, two replicates were prepared and tested for CBR values per material, and the average values were reported.

Table 19 lists CBR values established for the aggregate materials, whereas Table 20 lists the same for the subgrade materials. Note that CBR testing of aggregates was performed at OMC and MDD conditions only, whereas CBR tests on the subgrade materials were conducted at three different moisture contents (90% OMC, OMC, and 110% OMC) to also study the effect of moisture content. As seen from Table 19, the CBR values for most of the aggregate materials was greater than 100%, which is expected for crushed aggregates. The subgrade soil materials on the other hand, exhibited significantly lower CBR values, with most materials showing significant moisture susceptibility (see Table 20).

Table 19. Summary of the CBR Test Results for the Base/Subbase Materials

Material ID	California Bearing Ratio (%)	Material ID	California Bearing Ratio (%)
A	204.4	LN - 80	216.5
B	242.7	NP - 82	139.3
BK - 100	211.0	PW - 84	133.1
BK - 181	142.7	VY - 63	169.9
EL - 132	129.9	WCW	194.2
IMC - 140	239.8	CN – 148 SB	168.7
KT - 215	160.0	CS – 184 SB	78.7
LE - 160	168.0		

Table 20. Summary of the CBR Test Results for Soils

Materials Name	California Bearing Ratio (%)		
	0.9*OMC	OMC	1.10*OMC
D1-ML	23.7	5.2	1.6
D1-GM	36.1	12	2.0
TP-8	16.9	3.4	2.1
D3-SM	38.6	22.3	2.2
D3-SC	25.2	12.6	5.2
Cs-184	27.5	6.7	3.5
LN-80	61.4	55.8	10.5
Bk-180c	18.1	12.3	3.3
BN-59	8.2	4.5	1.3
JF-83	23.6	18.3	4.3
17-9SL-0055	60.4	38.1	9.2
17-9SL-0057	36.8	14.3	3.2

Summary

This chapter provided information about the materials (soils and aggregates) tested in this study. Representative aggregate and subgrade soil materials typically used in pavement applications across the state of Idaho were first selected through a survey of ITD’s district materials engineers. These materials were then transported to the Boise State and University of Idaho facilities for laboratory characterization. A total 18 aggregate (base/subbase) and 16 soil (subgrade) materials tested in the laboratory. This chapter also presented results from different preliminary tests, such as particle size distribution, Atterberg's limits, moisture-density relationship, and soil classification, performed on the collected aggregate and soil materials. Finally, results from CBR testing of the aggregates and soils were also included in this chapter. The next chapter will discuss details on repeated load triaxial testing of the aggregates and soils to establish resilient modulus properties.

Chapter 4
Repeated Load Triaxial Testing and Model Fitting

Introduction

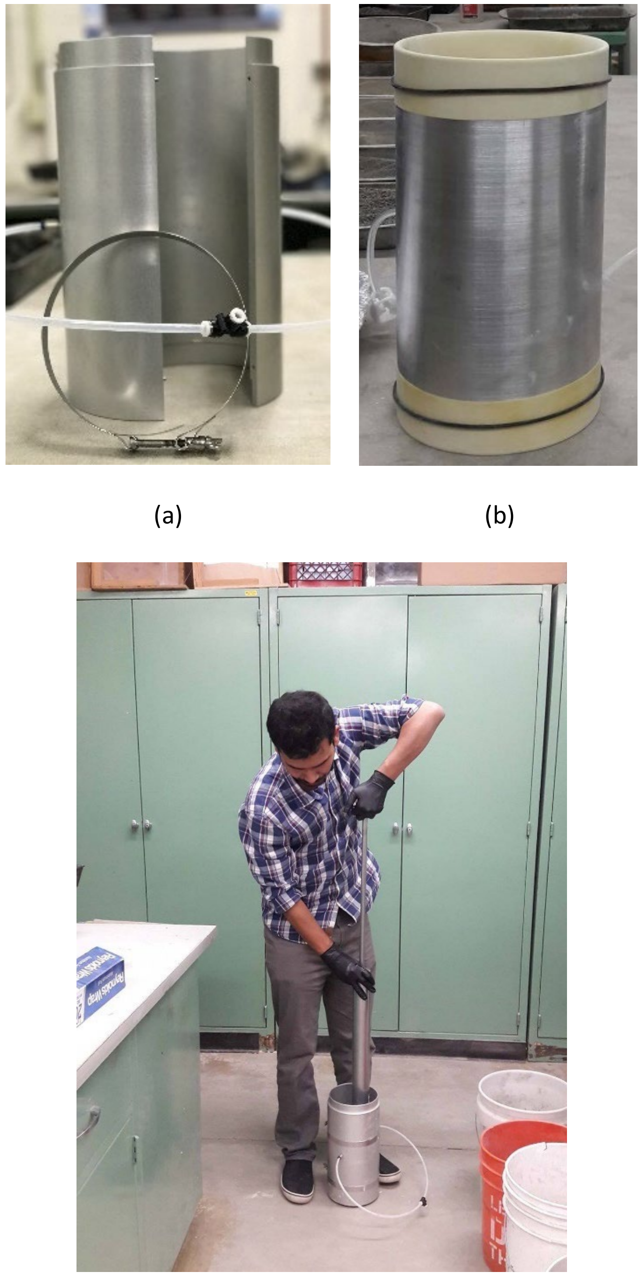
As already mentioned, the primary objective of this research study was to develop a database with resilient modulus properties of soils and aggregates commonly used in pavement applications in the state of Idaho to facilitate state-wide implementation of the mechanistic-empirical pavement design procedure. Laboratory testing to determine resilient modulus properties of the unbound materials was therefore the most important task in this study. This chapter presents details on the laboratory testing procedure followed by analysis of the test results.

Resilient Modulus Testing of Aggregates and Soils

Repeated load triaxial (RLT) testing was conducted in accordance to specifications in AASHTO T 307 to establish resilient modulus properties of the tested materials. To eliminate size effects from triaxial test results, it is recommended that the specimen diameter should be at least 5-6 times the top size of the material being tested. Considering the significantly different particle size distributions for aggregates and soils, the different specimen sizes were selected during the resilient modulus testing. Based on the gradation data presented in Chapter 3 of this report, it was decided that the aggregate materials will be tested by preparing 152 mm (6 in.) diameter by 305 mm (12 in.) cylindrical specimens. The subgrade soils, on the other hand, were tested by preparing cylindrical specimens that were 100 mm (~4 in.) in diameter and 200 mm (~8 in.) in height. The following paragraphs outline the detailed procedure adopted to prepare the specimens for each material type.

Aggregate Specimen Preparation

The aggregate materials were first oven dried at 60°C (140°F) for at least 24 hours, and subsequently cooled to room temperature. Pre-calculated amounts of moisture was then added to get the material to optimum moisture content. Cylindrical specimens for resilient modulus testing were compacted by placing the material into a split mold in six lifts, and compacting each layer using a drop hammer. Note that the target density for each specimen was 95% of MDD established using modified compactive effort. 95% of MDD was selected for specimen preparation instead of 100% MDD because field Quality Control / Quality Assurance (QC/QA) specifications often require contractors to achieve minimum compaction levels of 95% with respect to laboratory-established MDD values. Preparing several trial specimens, it was observed that 60 blows (of the modified Proctor hammer) per layer were usually sufficient to achieve 95% MDD. Therefore, all aggregate specimens in this study were prepared by applying 60 blows per layer; each specimen was compacted in six layers. Compacting in six layers ensured uniform distribution of density throughout the specimen height. Figure 12 shows a photographs of the split mold (Figure 12-a), the membrane expander (Figure 12-b), and specimen compaction inside the split mold (Figure 12-c).



(a) (b)

(c)

Figure 12. Photograph Showing Aggregate Specimen being Compacted inside a Split Mold for Resilient Modulus Testing

A porous stone was placed on either end (top and bottom) of the specimen to ensure dissipation of any excess pore water pressure. A metallic platen was placed on top, and was secured to with O-rings to the membrane.

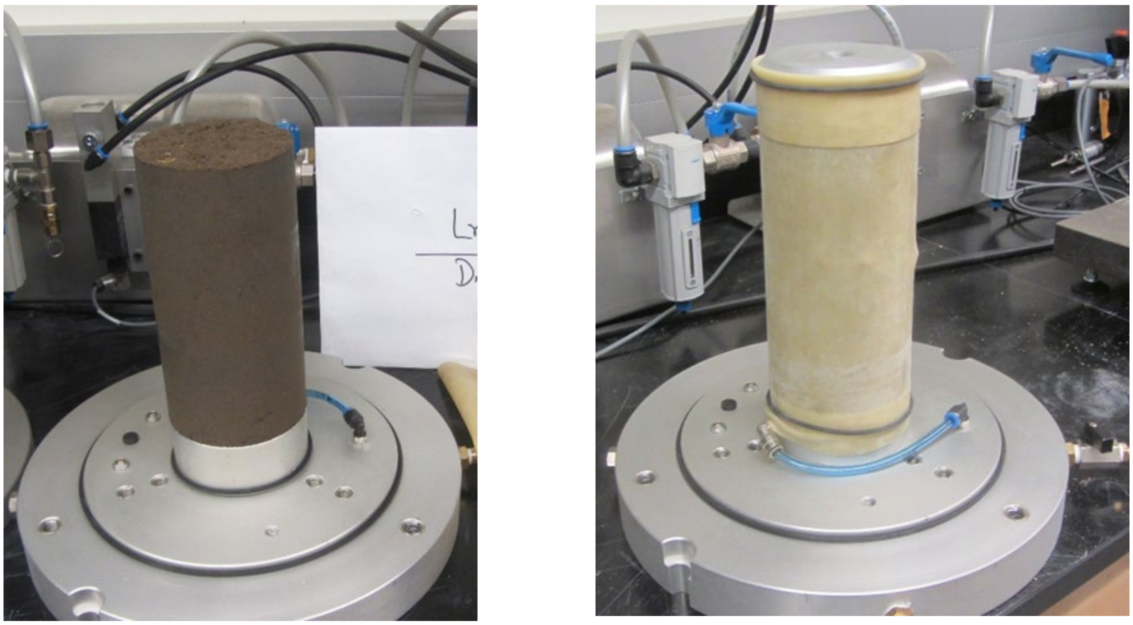
Subgrade Soil Specimen Preparation

Before testing the soil materials for resilient modulus properties, all materials were first sieved through the 4.75 mm sieve (US sieve size No. 4). This was necessary because the focus was to test the “fines” fraction of the materials only, without including any “coarse” fractions. The subgrade soil materials were tested for resilient modulus properties using specimens that were 100 mm in diameter, and 200 mm in height. Another significant difference between the aggregate and soil specimen preparation approaches was that the soil specimens were compacted at 100% of MDD established through the standard compactive effort. Note that it is common for subgrade layers to be compacted targeting MDD values established through the standard compactive effort. As the MDD values were relatively low to start with, the research team decided to compact the specimens at 100% MDD (standard compactive effort) instead of targeting 95% MDD. This would also ensure stability of the specimens during resilient modulus testing.

First, the amount of material required to produce a specimen of the mentioned dimensions at the pre-determined density levels were weighed and placed overnight in an oven at 60^{o}C. After removing from the oven, the material was air-cooled to attain room temperature. A pre-determined amount of water was then added to the dry soil to bring the mix up to the target moisture content. After thorough mixing, the material was compacted in three lifts to achieve 100% MDD. The surface between of each lift was scarified to ensure proper adhesion between lifts. Figure 13 shows a photograph of a soil specimen being compacted inside a split mold. As already mentioned, the subgrade materials selected in this study were also tested for moisture sensitivity. Therefore, resilient modulus specimens were prepared at OMC, 90% OMC, and 110% OMC to quantify the variation in material response under repeated loading with varying moisture contents.

Figure 13. Photograph Showing Subgrade Specimen Preparation inside the Split Mold

After compaction, the specimen was carefully demolded, and placed on top of the bottom platen mounted to the lower pedestal of the triaxial chamber. A membrane was mounted on top of the specimen using a membrane expander, and an O-ring was mounted to ensure adequate sealing between the membrane and the bottom platen. A similar procedure was adopted after placing a 100-mm diameter top platen on top of the specimen. Figure 14-a shows a photograph of a compacted specimen placed on top of the bottom pedestal, whereas Figure 14-b shows a photograph of the same specimen after the rubber membrane has been mounted, and the O-rings placed around the top and bottom platens.



(a) (b)

Figure 14. Photographs Showing: (a) Compacted Specimen Placed on Top of the Bottom Platen; and (b) Specimen Mounted with Membrane and O-rings

Aggregate Testing

The resilient modulus test is a stress-controlled test. AASHTO T 307 specifies the stress states to be applied to the specimen for resilient modulus testing. Note that different stress states are specified for unbound aggregate (base/subbase) and subgrade materials. A total of 15 stress states are applied to the specimen in addition to the pre-conditioning stress state (to simulate compaction and rearrangement of the material during construction and initial loading). Each load pulse comprises a 100 ms loading period followed by a 900 ms rest period. Table 21 lists the stress states applied during resilient modulus testing of unbound granular (base/subbase) materials per AASHTO T 307. Note that these materials are identified as Type I materials in AASHTO T 307.

Resilient modulus testing on the aggregate specimens in this study was carried out using a Material Testing System (MTS) loading frame (Model 810). Figure 15 shows a photograph of the repeated load triaxial testing set-up at the University of Idaho. Each aggregate specimen was pre-conditioned through the application of 750 load cycles. The confining pressure was checked at the start of each sequence and maintained constant for the whole time of each sequence. Specimen deflection during testing was measured using two external linear variable differential transformers (LVDTs) mounted to the loading rod.

Figure 15. Repeated Load Triaxial Test Setup at the University of Idaho Lab

Table 21. Stress States Applied during Resilient Modulus Testing of Base/Subbase Materials

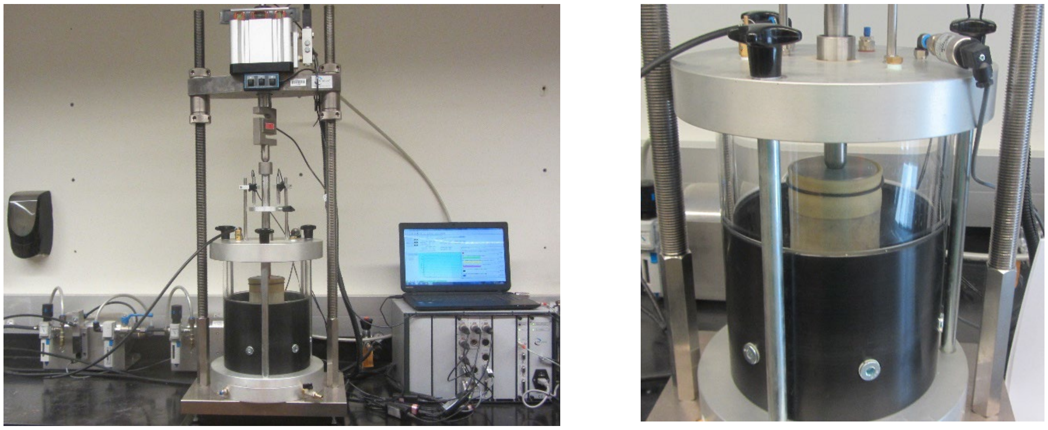
Sequence No.	Confining Pressure, σ_3		Maximum Axial Stress, σ_d		Cyclic Stress σ_{cyclic}		Constant Stress, $0.1\sigma_d$		No. of Load Applications
	kPa	psi	kPa	psi	kPa	psi	kPa	psi	
0	103.4	15	103.4	15	93.1	13.5	10.3	1.5	500-1000
1	20.7	3	20.7	3	18.6	2.7	2.1	0.3	100
2	20.7	3	41.4	6	37.3	5.4	4.1	0.6	100
3	20.7	3	62.1	9	55.9	8.1	6.2	0.9	100
4	34.5	5	34.5	5	93.1	13.5	3.5	0.5	100
5	34.5	5	68.9	10	62.0	9	6.9	1	100
6	34.5	5	103.4	15	93.1	13.5	10.3	1.5	100
7	68.9	10	68.9	10	62.0	9	6.9	1	100
8	68.9	10	137.9	20	93.1	13.5	13.8	2	100
9	68.9	10	206.8	30	186.1	27	20.7	3	100
10	103.4	15	68.9	10	62.0	9	6.9	1	100
11	103.4	15	103.4	15	93.1	13.5	10.3	1.5	100
12	103.4	15	206.8	30	93.1	13.5	20.7	3	100
13	137.9	20	103.4	15	93.1	13.5	10.3	1.5	100
14	137.9	20	137.9	20	124.1	18	13.8	2	100
15	137.9	20	275.8	40	248.2	36	27.6	4	100

Soil Testing

Testing of the soil specimens was carried out following a procedure quite similar to that for the aggregate specimens. The primary difference between the two test systems was that the confining pressure in the Boise State setup was maintained using an automated feedback-based system. Therefore, manual verification of the confining pressure at each stress sequence was not required. Moreover, the stress states applied during resilient modulus testing of subgrade soils (Type II materials per AASHTO T 307) were different from those for the base/subbase (Type I materials per AASHTO T 307) and have been listed in Table 22. All subgrade soil specimens tested in the current study qualified as Type II materials per AASHTO T 307 because all materials were first sieved through the 4.75 mm sieve. Prior to beginning of the actual test, each specimen was preconditioned by subjecting it to 1000 cycles at the stress states corresponding to Sequence 0 of AASHTO T 307 (see Table 22). Figure 16 shows two photographs of the resilient modulus testing setup at Boise State University.

Table 22. Stress State Sequence Used for Resilient Modulus Testing of Subgrade Soils

Seq. No.	Confining Pressure, σ_3		Maximum Axial Stress, σ_{max}		Cyclic Stress, σ_{cyclic}		Constant Stress, 0.1 σ_{max}		No. of Load Applications
	kPa	psi	kPa	psi	kPa	psi	kPa	psi	
0	41.4	6	27.6	4	24.8	3.6	2.8	0.4	1000
1	41.4	6	13.8	2	12.4	1.8	1.4	0.2	100
2	41.4	6	27.6	4	24.8	3.6	2.8	0.4	100
3	41.4	6	41.4	6	37.3	5.4	4.1	0.6	100
4	41.4	6	55.2	8	49.7	7.2	5.5	0.8	100
5	41.4	6	68.9	10	62.0	9.0	6.9	1.0	100
6	27.6	4	13.8	2	12.4	1.8	1.4	0.2	100
7	27.6	4	27.6	4	24.8	3.6	2.8	0.4	100
8	27.6	4	41.4	6	37.3	5.4	4.1	0.6	100
9	27.6	4	55.2	8	49.7	7.2	5.5	0.8	100
10	27.6	4	68.9	10	62.0	9.0	6.9	1.0	100
11	13.8	2	13.8	2	12.4	1.8	1.4	0.2	100
12	13.8	2	27.6	4	24.8	3.6	2.8	0.4	100
13	13.8	2	41.4	6	37.3	5.4	4.1	0.6	100
14	13.8	2	55.2	8	49.7	7.2	5.5	0.8	100
15	13.8	2	68.9	10	62.0	9.0	6.9	1.0	100



(a) (b)

Figure 16. Photographs Showing: (a) Repeated Load Triaxial Test Setup; and (b) Mounted Specimen inside the Confining Cell in the Boise State Laboratory

Analysis of Resilient Modulus Test Results

Per AASHTO T 307 specifications, data corresponding to the last five cycles of each stress sequence were used to calculate the resilient modulus value for that particular stress state. Therefore, for each material tested, fifteen (15) different resilient modulus values are obtained after completion of the test. A limiting value for maximum allowable permanent strain during testing was set at 5%. In other words, if the permanent strain accumulation in the specimen exceeded 5% under any of the stress states, the test was stopped automatically. This is particularly important because the resilient modulus test is a non-destructive test, and the specimen should not undergo significant deformation during the resilient modulus testing.

Table 23a and 23b list resilient modulus values for the eighteen different aggregate (base/subbase) materials tested under the scope of this study. As seen from the table, all aggregate materials exhibited stress hardening behavior, which is reflected through higher resilient modulus values at higher bulk stress levels. Note that all values listed in Table 23a and b correspond to OMC and 95% MDD conditions as the aggregate materials were tested under one moisture condition only.

The subgrade materials, on the other hand, were tested under three different moisture conditions (90% OMC, OMC, and 110% OMC). This was particularly important because subgrade soils are more likely to exhibit moisture sensitivity during the performance period of pavements. Figures 17 through 19 present resilient modulus test results (at different moisture contents) for all subgrade materials tested in this study. Note that soils corresponding to material IDs 17-9SL-0101, 17-9SL-0054 and 17-9SL-0058 were tested only at optimum moisture content as not sufficient material were available to carry out testing at multiple moisture contents. Specimens corresponding to material IDs D1-ML, D3-SM, TP-8, Cs-184, Bk-180c, JF-83 and 17-9SL-0057 prepared at 110% OMC failed during resilient modulus testing. At each moisture content, at least two replicate specimens were prepared to ensure repeatability of the test results. For instances where the results for the two specimens were significantly different, a third specimen was tested, and average data for the two specimens that showed the closest "match" was reported.

Table 23a. Resilient Modulus Test Results for all Base/Subbase Materials Tested in the Current Study (SI Units)

Material ID	Sequence Number (Resilient Modulus in MPa)														
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
A	79.9	95.3	107.2	118	137.5	151.9	200.7	227.1	237.3	229.5	248.3	289.4	289.2	314.1	350.7
B	84.1	98.6	113.1	123.9	146.4	157.5	210.7	238.8	245.4	236.9	256	294.3	293.9	318.8	355.4
BK - 100	60.5	87.2	100.9	103.6	128.5	141	183.1	207.8	216.5	213.2	227	263.7	263.9	281.6	319.9
BK - 181	42.0	58.7	80.2	68.1	97	110.1	127.9	159.7	182.1	144.5	159	211.1	187.5	204.9	277.8
BR - 2	119.5	135.2	151.8	168.5	190.5	207.6	269.2	300.3	311	316.9	329.6	371.1	382.6	401.3	441
CS - 184	61.8	77.7	89.5	93.6	109.4	121.8	149.2	172.5	184.2	174.3	184.7	216.6	217.1	230.1	264.7
EL - 132	57.6	82.7	95.7	102.1	128.6	138.9	184.8	209.3	218.2	212.6	228.5	265.5	271.3	288.5	328.4
IMC - 140	62.8	80.0	95.6	105.2	126.3	136.7	176.8	197.4	213.4	197.3	216	253	251.9	269.4	307.1
KT - 215	43.6	74.7	95.3	76.4	122.3	139.5	152.4	200.9	214	159.8	206.7	253.6	233.7	268.2	309.9
LE - 160	82.4	101.2	114.4	123.5	146.5	159.3	196.2	230	246.8	227.2	248.5	293.4	287.1	311	357.1
LN - 80	58.6	93.1	109.2	108.8	140.9	154.7	190	222.4	225.5	214.5	232.5	275.7	272.4	291.6	338.2
NP - 82	86.1	96.6	110.0	117.5	137.1	150.9	197.3	222.2	237.4	231.9	244.1	284.9	291.3	310.3	348.7
PW - 84	32.9	53.7	71.4	50.5	82.8	103.9	97.1	137.8	161.5	96.9	122.7	179.8	135	165.1	227.4
VY - 63	63.4	84.0	106.7	94.2	134.3	155.1	181.4	228.1	245.7	197.7	236.4	290.2	268.6	299.3	350.2
WCW	76.6	89.8	101.5	107.1	124.9	140.7	174.2	201	219.7	209.4	217.2	256.1	258.3	271.6	313.9
CN - 148 SB	88.1	105.5	120.7	137.9	152	161.4	212.3	234.8	243.4	249.4	255.1	286	303.3	316.4	352.8
CS - 184 SB	33.1	48.9	61.5	43.4	63.7	79.6	63.4	92.3	115.3	69.3	77.6	119.1	82	96.9	144.7
PY - 720 SB	84.5	100.5	114.3	123.4	144.3	153.4	197	217.7	227.4	226.7	243.6	273.4	280.1	295	321.9

Table 23b. Resilient Modulus Test Results for all Base/Subbase Materials Tested in the Current Study (English Units)

Material ID	Sequence Number (Resilient Modulus in ksi)														
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
A	11.6	13.8	15.5	17.1	19.9	22.0	29.1	32.9	34.4	33.3	36.0	42.0	41.9	45.6	50.9
B	12.2	14.3	16.4	18.0	21.2	22.8	30.6	34.6	35.6	34.4	37.1	42.7	42.6	46.2	51.5
BK - 100	8.8	12.6	14.6	15.0	18.6	20.5	26.6	30.1	31.4	30.9	32.9	38.2	38.3	40.8	46.4
BK - 181	6.1	8.5	11.6	9.9	14.1	16.0	18.6	23.2	26.4	21.0	23.1	30.6	27.2	29.7	40.3
BR - 2	17.3	19.6	22.0	24.4	27.6	30.1	39.0	43.6	45.1	46.0	47.8	53.8	55.5	58.2	64.0
CS - 184	9.0	11.3	13.0	13.6	15.9	17.7	21.6	25.0	26.7	25.3	26.8	31.4	31.5	33.4	38.4
EL - 132	8.4	12.0	13.9	14.8	18.7	20.1	26.8	30.4	31.6	30.8	33.1	38.5	39.3	41.8	47.6
IMC - 140	9.1	11.6	13.9	15.3	18.3	19.8	25.6	28.6	31.0	28.6	31.3	36.7	36.5	39.1	44.5
KT - 215	6.3	10.8	13.8	11.1	17.7	20.2	22.1	29.1	31.0	23.2	30.0	36.8	33.9	38.9	44.9
LE - 160	12.0	14.7	16.6	17.9	21.2	23.1	28.5	33.4	35.8	33.0	36.0	42.6	41.6	45.1	51.8
LN - 80	8.5	13.5	15.8	15.8	20.4	22.4	27.6	32.3	32.7	31.1	33.7	40.0	39.5	42.3	49.1
NP - 82	12.5	14.0	16.0	17.0	19.9	21.9	28.6	32.2	34.4	33.6	35.4	41.3	42.2	45.0	50.6
PW - 84	4.8	7.8	10.4	7.3	12.0	15.1	14.1	20.0	23.4	14.1	17.8	26.1	19.6	23.9	33.0
VY - 63	9.2	12.2	15.5	13.7	19.5	22.5	26.3	33.1	35.6	28.7	34.3	42.1	39.0	43.4	50.8
WCW	11.1	13.0	14.7	15.5	18.1	20.4	25.3	29.2	31.9	30.4	31.5	37.1	37.5	39.4	45.5
CN - 148 SB	12.8	15.3	17.5	20.0	22.0	23.4	30.8	34.1	35.3	36.2	37.0	41.5	44.0	45.9	51.2
CS - 184 SB	4.8	7.1	8.9	6.3	9.2	11.5	9.2	13.4	16.7	10.1	11.3	17.3	11.9	14.1	21.0
PY - 720 SB	12.3	14.6	16.6	17.9	20.9	22.2	28.6	31.6	33.0	32.9	35.3	39.7	40.6	42.8	46.7

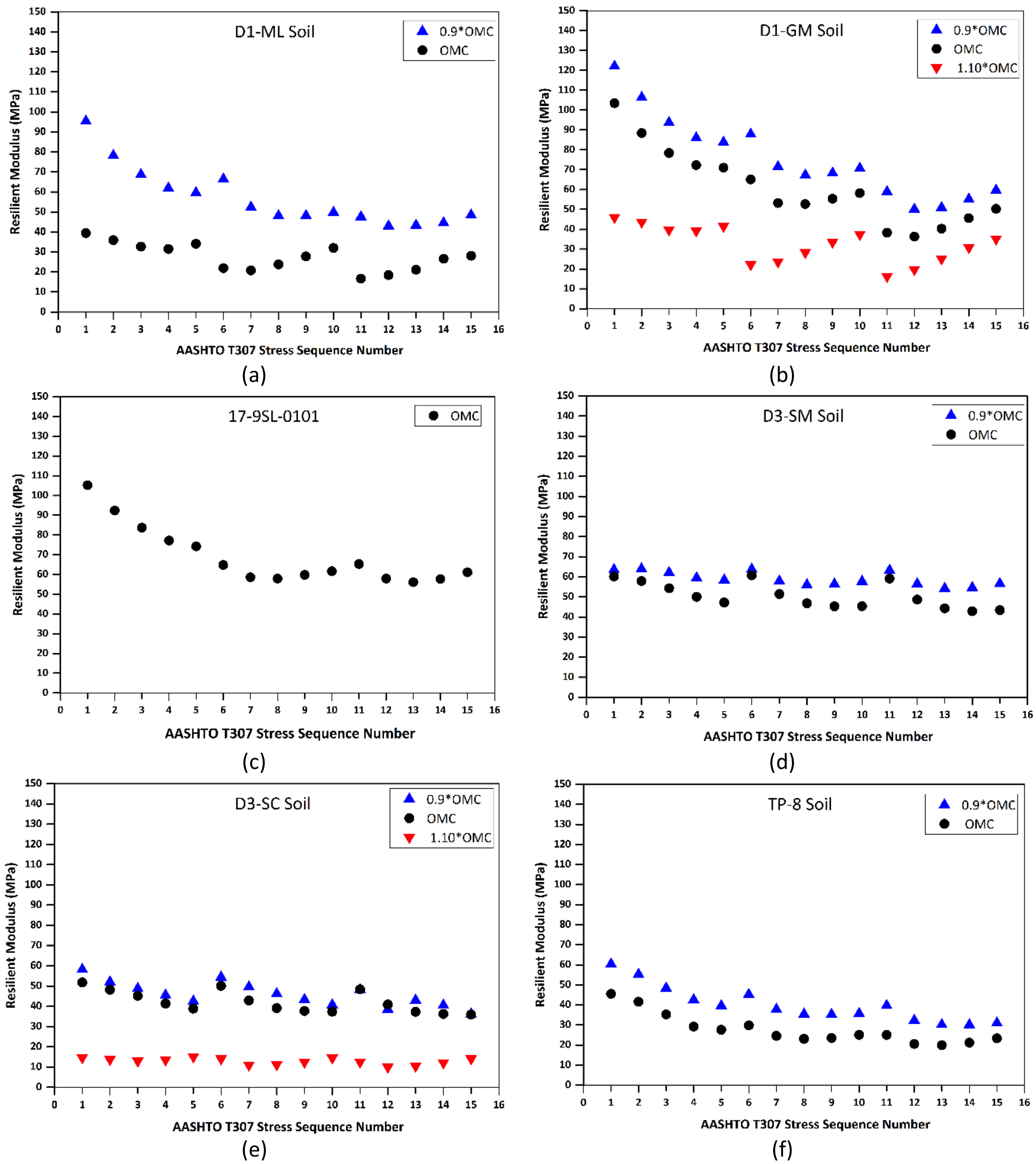


Figure 17. Resilient Modulus Values Corresponding to Different AASHTO T 307 Stress Sequences for the Subgrade Materials Tested in the Current Study: (a) D1-ML; (b) D1-GM; (c) 17-9SL-0101; (d) D3-SM; (e) D3-SC; and (f) TP-8

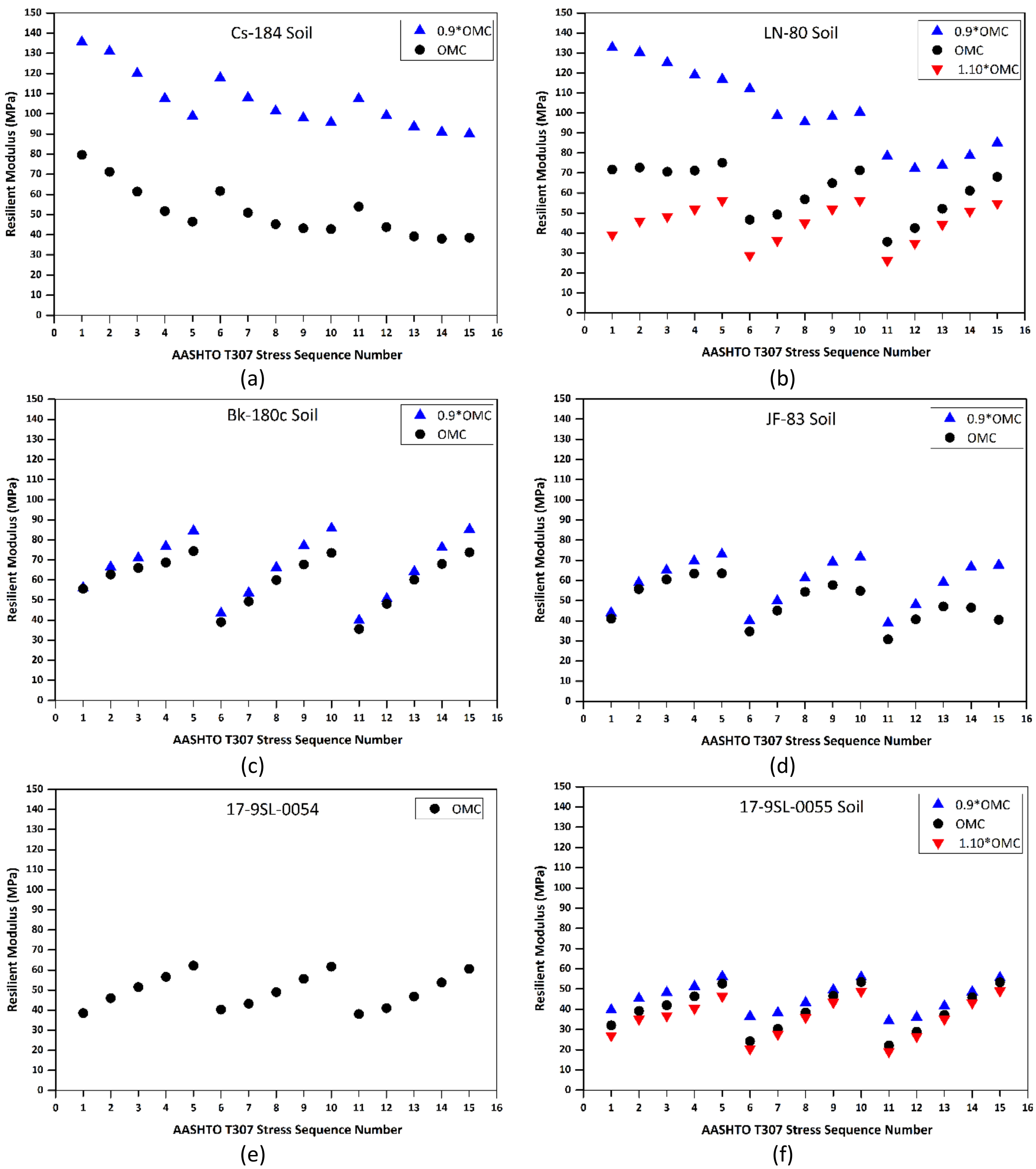


Figure 18. Resilient Modulus Values Corresponding to Different AASHTO T 307 Stress Sequences for the Subgrade Materials Tested in the Current Study: (a) Cs-184; (b) LN-80; (c) Bk-180c; (d) JF-83; (e) 17-9SL-0054; and (f) 17-9SL-0055

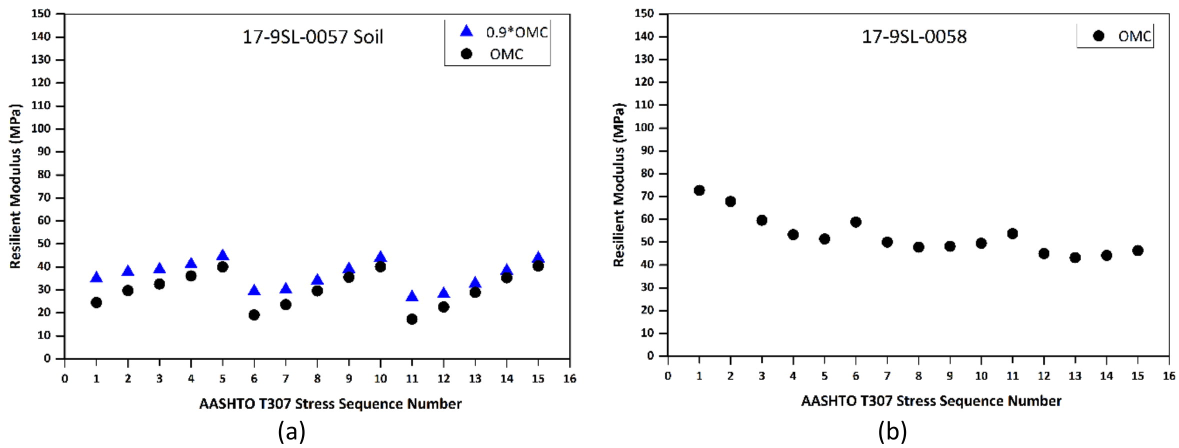


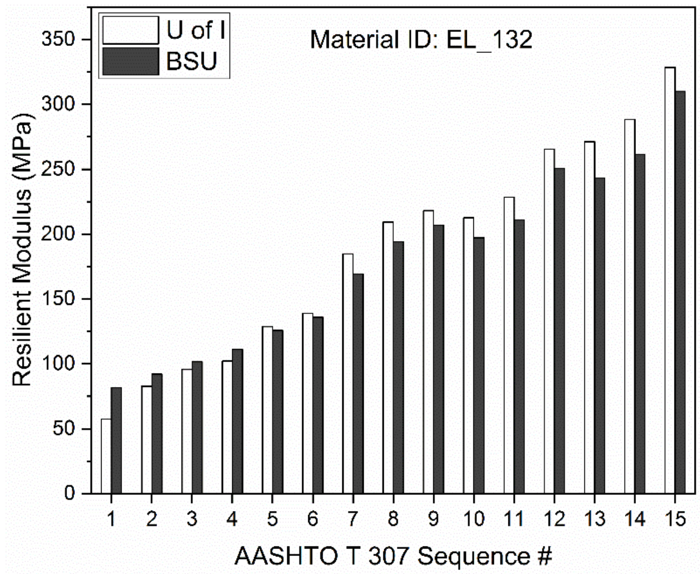
Figure 19. Resilient Modulus Values Corresponding to Different AASHTO T 307 Stress Sequences for the Subgrade Materials Tested in the Current Study: (a) 17-9SL-0057; and (b) 17-9SL-0058

From the above figures it can clearly be seen that for materials that could be tested at multiple moisture contents, the modulus values at 90% OMC were higher than those for OMC or for 110% of OMC. Although several of the materials could not be tested at 110% OMC, those that could be tested, demonstrated clear reduction in resilient modulus with increasing moisture content. This reinforces the concept that the pavement support conditions can deteriorate significantly with seasonal fluctuations in moisture content. It is therefore critical to ensure adequate drainage of moisture from the pavement structure. In cases where not all moisture can be adequately removed through installation of drainage features, deterioration in the support condition should be considered during design and construction of the pavement.

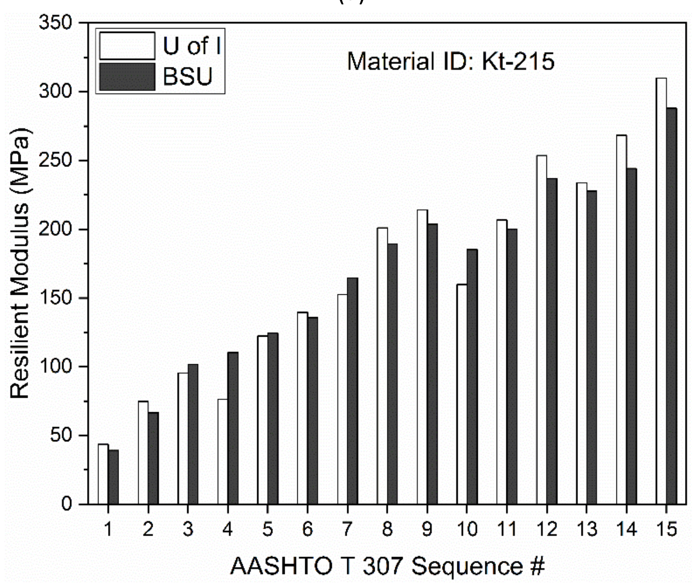
Checking for Inter-Laboratory Variations in Laboratory Test Results

Considering that laboratory testing in the current study were carried out at two different laboratories (Boise State and University of Idaho), it was important to ensure no significant inter-laboratory variance was introduced into the test data. This is particularly critical for the resilient modulus test results as the test procedure involves several complex steps, and slight variations in each step can potentially lead to significantly different test data. The research team randomly selected two different aggregate materials (EL-132 and Kt-215) to carry out inter-laboratory verification tests. These two materials were tested at both the University of Idaho as well as Boise State laboratories. Specimens for these two aggregate materials (two replicates each) were compacted at Optimum Moisture Content (OMC), and tested using the University of Idaho and Boise State test set-ups. Figure 20 compares the resilient modulus values for these two materials as determined using the Boise State and University of Idaho test set-ups. The values have been compared for the 15 different AASHTO T 307 stress sequences.

From the figures it is clear that the modulus values established using the two test setups were within 10% of each other for most of the stress states (except for a 1-2 stress states). Considering the uncertainties associated with material sampling, sample compaction, test set-up and equipment compliance, a 10% difference in the modulus values indicates excellent consistency among the two laboratories. This verification effort established that data generated from the two laboratories can be safely merged into one database without concerns related to data inconsistencies.



(a)



(b)

Figure 20. Inter-Laboratory Variation in Resilient Modulus for EL-132 Aggregate Material

Establishing Resilient Modulus Model Parameters

Once all testing for resilient modulus properties was complete, the next step involved fitting the data with commonly used constitutive models that can accurately capture stress-dependent behavior of the materials tested. A list of resilient modulus models for aggregates and soils was presented in Chapter 2 of this report. The models listed in Figure 21 were used to fit the laboratory test data this study.

Resilient Modulus Models used for unbound aggregates:

$K-\theta$ Model: $M_R = k \times \theta^n$

Uzan Model: $M_R = k_1 \times p_a \times \left(\frac{\theta}{p_a}\right)^{k_2} \times \left(\frac{\sigma_d}{p_a}\right)^{k_3}$

Modified Uzan Model: $M_R = k_1 \times p_a \times \left(\frac{\theta}{p_a}\right)^{k_2} \times \left(\frac{\tau_{oct}}{p_a}\right)^{k_3}$

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

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,

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

Figure 21. Resilient Modulus Models used for Unbound Aggregates

The model fitting can be easily carried out using a spreadsheet package such as Microsoft excel (the "SOLVER" tool in Excel can be particularly useful). Instead of using SOLVER, the model fitting can also be easily carried out by linearizing the equations and performing simple linear regression analyses. Figure 22 illustrates how resilient modulus test data can be easily fit with the K-θ model.

The $K-\theta$ model to predict the resilient modulus values for unbound aggregates can be written as:

$$M_R = K \times \theta^n$$

Taking logarithm of both sides of the above equation will linearize it.

$$\Rightarrow \log_{10}\left(M_R\right) = \log_{10}\left(K \times \theta^n\right)$$

$$\Rightarrow \log_{10}\left(M_R\right) = \log_{10}\left(K\right) + n\log_{10}\left(\theta\right)$$

Figure 22. Resilient Modulus Models used for Unbound Aggregates

The final equation in Figure 22 is of the form: $Y = mx + b$, which is the standard equation for a straight line. M_R values are available from laboratory testing and are the predicted variable in this equation. The θ value can be calculated from the measured stress states during resilient modulus testing. Once a straight line has been fitted between log (M_R) and log(θ), the Y intercept of the straight line represents log(K), and the slope of the line represents 'n'. It is important to note that similar linearization approaches can be followed for the other models listed above. However, multiple linear regression (and not a simple straight-line fitting) needs to be carried out to establish the regression parameters. This is important because the models have more than two parameters. Nevertheless, these tasks can be easily accomplished by spreadsheet applications such as Microsoft Excel.

The adequacy of a model to predict the resilient modulus value for a material at a given stress state can be assessed by looking at the coefficient of determination (R^2) value established during the regression analysis. Table 24 lists the model parameter values established for the unbound aggregate (base/subbase) materials tested in the current study. Parameters corresponding to all four models (K-θ, Uzan, Modified Uzan, and MEPDG) have been listed in Table 24. As seen from the table, for most of the cases, fitting these models to the test data resulted in high R^2 (coefficient of determination) values.

Table 24. Model Parameters for Four Constitutive Models used for Base/Subbase Material

K-theta Model						Uzan Model			
Material ID	**K (MPa)**	**n**	**R^2**			**K_1**	**K_2**	**K_3**	**R^2**
A	3.771	0.695	0.997			904.522	0.713	-0.027	0.996
B	4.308	0.678	0.995			968.483	0.684	-0.023	0.994
BK - 100	2.879	0.725	0.987			827.156	0.710	-0.020	0.994
BK - 181	1.567	0.780	0.972			706.275	0.586	0.195	0.995
BR - 2	7.419	0.629	0.996			1269.063	0.675	-0.059	0.998
CS - 184	3.712	0.652	0.996			762.195	0.633	0.011	0.997
EL - 132	2.319	0.763	0.986			790.765	0.750	-0.028	0.993
IMC - 140	3.026	0.710	0.991			823.871	0.679	0.003	0.995
KT - 215	1.630	0.806	0.946			841.240	0.606	0.156	0.981
LE - 160	4.656	0.662	0.998			999.389	0.642	0.021	0.997
LN - 80	3.034	0.724	0.971			922.999	0.655	0.027	0.988
NP - 82	4.282	0.673	0.997			902.220	0.714	-0.038	0.997
PW -84	1.551	0.745	0.898			748.558	0.349	0.407	0.993
Vy - 63	2.440	0.761	0.980			961.594	0.619	0.117	0.992
WCW	4.158	0.660	0.999			858.071	0.667	-0.006	0.999
CN - 148 SB	5.580	0.636	0.992			984.968	0.685	-0.065	0.995
CS - 184 SB	3.942	0.523	0.793			726.966	0.065	0.513	0.993
PY -720 SB	5.544	0.626	0.995			945.874	0.661	-0.051	0.996
Modified Uzan Model						**MEPDG Model**			
Material ID	**K_1**	**K_2**	**K_3**	**R^2**		**K_1**	**K_2**	**K_3**	**R^2**
A	886.345	0.713	-0.027	0.996		935.207	0.711	-0.075	0.996
B	951.572	0.684	-0.023	0.994		997.734	0.685	-0.073	0.994
BK - 100	814.795	0.710	-0.020	0.994		849.221	0.717	-0.081	0.994
BK - 181	817.793	0.586	0.195	0.995		561.249	0.621	0.453	0.993
BR - 2	1213.654	0.675	-0.059	0.998		1367.016	0.666	-0.151	0.998
CS - 184	757.615	0.639	0.007	0.997		748.290	0.643	0.006	0.997
EL - 132	774.373	0.750	-0.028	0.993		819.292	0.755	-0.098	0.994
IMC - 140	826.002	0.679	0.003	0.995		821.516	0.687	-0.016	0.995
KT - 215	945.925	0.606	0.156	0.981		699.299	0.664	0.282	0.974
LE - 160	1015.425	0.642	0.021	0.997		974.164	0.648	0.046	0.997
LN - 80	941.857	0.655	0.027	0.988		895.024	0.675	0.019	0.988
NP - 82	876.966	0.714	-0.038	0.997		944.597	0.706	-0.087	0.997
PW -84	1016.859	0.349	0.407	0.993		463.894	0.432	0.916	0.980
Vy - 63	1036.089	0.623	0.113	0.992		832.396	0.664	0.208	0.988
WCW	854.179	0.667	-0.006	0.999		864.330	0.666	-0.012	0.999
CN - 148 SB	938.181	0.685	-0.065	0.995		1067.651	0.673	-0.157	0.995
CS - 184 SB	1069.443	0.065	0.513	0.993		388.543	0.152	1.244	0.981
PY -720 SB	910.497	0.661	-0.051	0.996		1008.901	0.657	-0.142	0.997

Resilient Modulus Models used for Subgrade Materials

Unlike the unbound aggregate materials, the subgrade materials were fitted with the MEPDG model only. Aggregates have been known to exhibit stress-hardening behavior, whereas fine-grained soils commonly exhibit stress-softening behavior. The K-θ model was originally developed for unbound aggregates, and therefore, fail to adequately capture the stress-softening behavior of fine-grained soils. The MEPDG model on the other hand captures both the stress-hardening behavior of aggregates (through the bulk stress or θ term) and the stress-softening behavior of fine-grained soils (through the octahedral shear stress or τ_{oct} term).

As already mentioned, ultimate objective of this research study was to establish a database of resilient modulus model parameters that ITD engineers can use during pavement analysis and performance prediction using AASHTOWare Pavement ME Design. MEPDG model parameters for each of the subgrade soil materials tested at different moisture contents have been presented in Table 25. Note that for the values listed in the table, the units for stress and resilient modulus are in kPa; the term P_a represents atmospheric pressure, and has a value of 101.325 kPa.

Table 25. Summary of MEPDG Model Parameters Established for the Subgrade Soil Materials

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
D1-ML	768.4	0.67	-2.61	0.88	260.6	0.76	-0.72	0.77	-	-	-	-
D1-GM	1001.4	0.84	-2.72	0.91	744.8	0.99	-2.47	0.89	263.5	0.81	0.07	0.78
17-9SL-0101	-	-	-	-	888.4	0.58	-2.25	0.70	-	-	-	-
TP-8	555.9	0.57	-2.66	0.86	377.2	0.68	-2.68	0.76	-	-	-	-
D3-SM	608.9	0.10	-0.57	0.59	563.6	0.21	-1.60	0.79	-	-	-	-
D3-SC	587.8	0.26	-1.82	0.90	541.9	0.18	-1.75	0.89	124.4	0.25	-0.2	0.30
Cs-184	1346.5	0.34	-1.93	0.95	767.2	0.52	-3.1	0.90	-	-	-	-
LN-80	1182.4	0.69	-1.74	0.95	486.9	0.55	0.53	0.84	283.6	0.26	2.17	0.89
Bk-180c	410.2	0.23	2.35	0.92	397.6	0.26	1.94	0.85	-	-	-	-
JF-83	371.14	0.22	2.03	0.87	383.1	0.45	0.79	0.74	-	-	-	-
17-9SL-0054	-	-	-	-	340.2	0.09	1.95	0.98	-	-	-	-
17-9SL-0055	327.4	0.17	1.58	0.94	223.6	0.25	2.78	0.94	190.1	0.19	3.17	0.91
17-9SL-0057	273.9	0.25	1.25	0.89	175.2	0.24	2.7	0.94	-	-	-	-
17-9SL-0058	-	-	-	-	667.1	0.38	-1.88	0.78	-	-	-	-
Note: cells with no data indicate the specimen could not be tested for resilient modulus properties												

Analyzing the Model Parameter Values

Once the resilient modulus model parameters for all the aggregate and soil materials were established, the next task involved assessing whether the model parameter values established for the tested materials meet commonly observed patterns or not. Per the Mechanistic-Empirical Pavement Design Guide (MEPDG), *the coefficient k_1 is proportional to Young's modulus, and thus the values for k_1 should be positive*. k_1 values established for all soils and aggregates tested in the current study have been plotted in Figure 23. Note that the x axis of Figure 23 only refers to material numbers, (as different material types were tested) as aggregate and soils. In other words, base/subbase material number 1 and subgrade material 1 do not correspond to the same material, but the number 1 is just used as a material ID. As seen from Figure 23, the k_1 values for the base/subbase materials are consistently higher than that for the subgrade soils. This clearly agrees with the common observation that base/subbase layers are stiffer than subgrade layers and contribute significantly towards the load carrying capacity in flexible pavements. Note that the k_1 parameters in Figure 23 were determined at OMC conditions as the aggregate materials in the current study were tested at OMC conditions only.

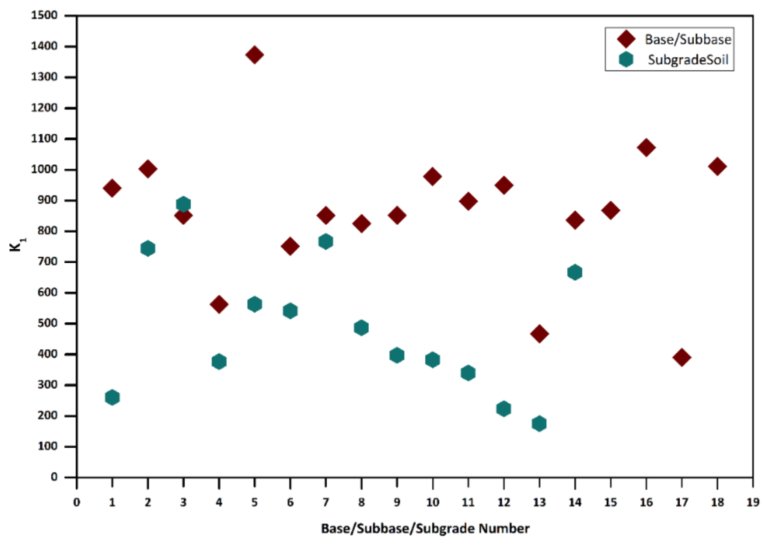


Figure 23. Comparing k_1 Parameter Values for the Base/Subbase and Subgrade Materials Tested in this Study at Optimum Moisture Content

As mentioned in the MEPDG, *increasing the bulk stress, θ, should produce a stiffening or hardening of the material, which results in a higher M_r. Therefore, the exponent k_2*, of the bulk stress term θ should also be positive. Figure 24 shows the k_2 parameters for all the soils and aggregates tested in the current study. As expected, the k_2 parameter is positive for all the materials. Moreover, it should be noted that the k_2 parameter values for the base/subbase materials are consistently higher than those for the subgrade soils. This clearly establishes that the stress hardening component in aggregates is much more significant compared to subgrade soils.

Finally, Figure 25 shows the k_3 parameter values for all the soils and aggregates tested in the current study. Per MEPDG, *the values for k_3 should be negative since increasing the shear stress will produce a softening of the material (i.e. a lower M_r).* From Figure 25, it can clearly be seen that the k_3 parameter values for most of the base/subbase materials are close to zero. This indicates that the stress-softening component in unbound aggregates is quite insignificant. k_3 values for the subgrade soils, on the other hand, were non-zero in value. Note that Yau and von Quintus (2002) [62] reported similar trends upon extensive analysis of resilient modulus test results for a total of 2014 resilient modulus tests for unbound aggregates and soils: "*coefficient k_3 was found to be zero for nearly 25 percent of the M_R tests* *performed on the unbound aggregate base/subbase materials and about 10 percent of the tests performed on the coarse-grained subgrade soils."*

Although one would expect all k_3 parameter values to be negative, positive values were observed for some of the materials tested. This is an outcome of the error minimization technique in multiple linear regression, as the coefficient values are adjusted until the best match between observed and predicted values are obtained. Note that from their analyses, Yau and von Quintus (2002) [62] did not report positive values for k_3, which was observed in the current study several times.

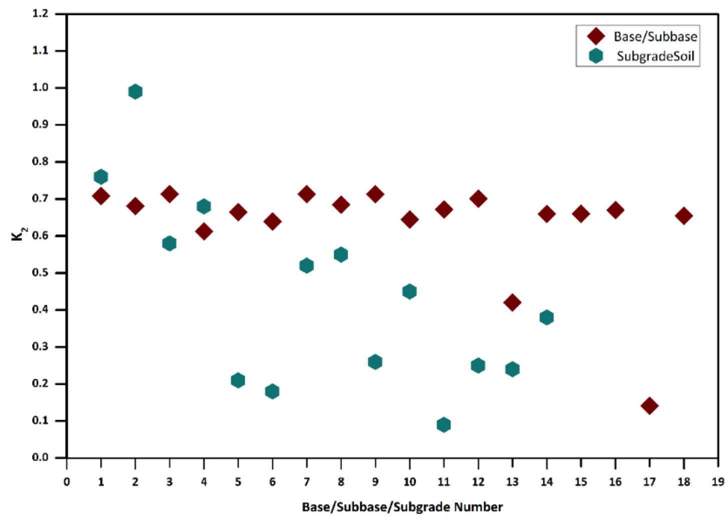


Figure 24. Comparing k_2 Parameter Values for the Base/Subbase and Subgrade Materials Tested in this Study at Optimum Moisture Content

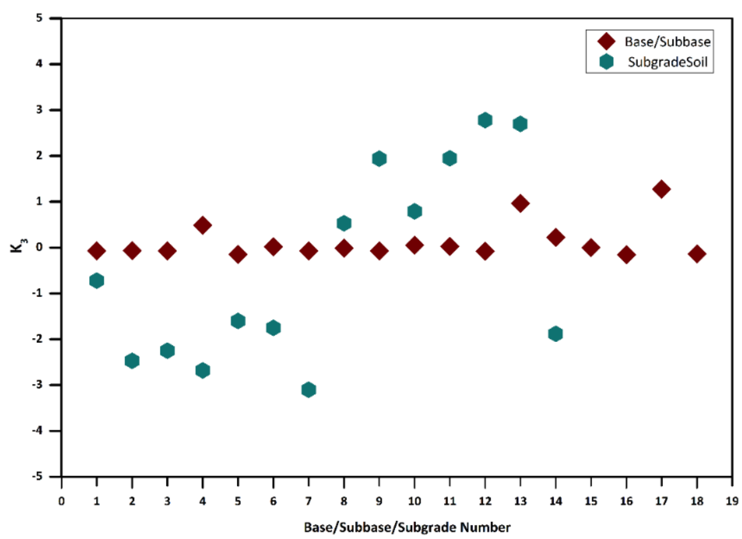


Figure 25. Comparing k_3 Parameter Values for the Base/Subbase and Subgrade Materials Tested in this Study at Optimum Moisture Content

Once comparison between the k_1, k_2, and k_3 parameter values for the aggregate and soil materials was complete, the next task involved studying the effect of moisture content on the k_1, k_2, and k_3 parameters for the subgrade soils tested in the current study. Figures 26 through 28 show the variation in k_1, k_2, and k_3 parameters for the subgrade soils with moisture content. As a reminder, each subgrade soil type was tested for resilient modulus properties at three different moisture contents (90% OMC, OMC, and 110% OMC). Therefore, regression model parameters were established for the subgrade materials at each of the moisture conditions. It is important to note that several of the materials could not be tested at 110% OMC as the specimens were too weak to sustain the repeated loading during resilient modulus testing. Nevertheless, from the data presented in Figure 26 through 28, it can be seen that the specimens tested at 90% OMC conditions often exhibited higher k_1 parameters compared to specimens tested at OMC or 110% OMC. This indicates, the specimens tested at 90% OMC were in general “stiffer” than those tested at the other two moisture conditions were. No particular trend could be found between k_2 and k_3 parameters for specimens tested at different moisture contents.

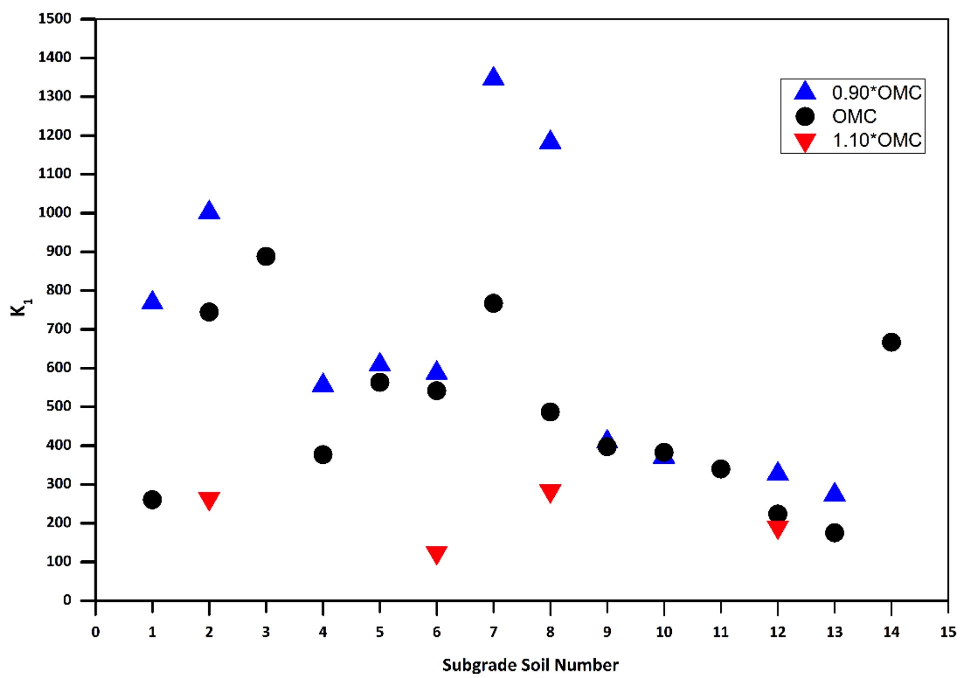


Figure 26. Effect of Compaction Moisture Content on k_1 Parameter of the Subgrade Soils Tested in the Current Study

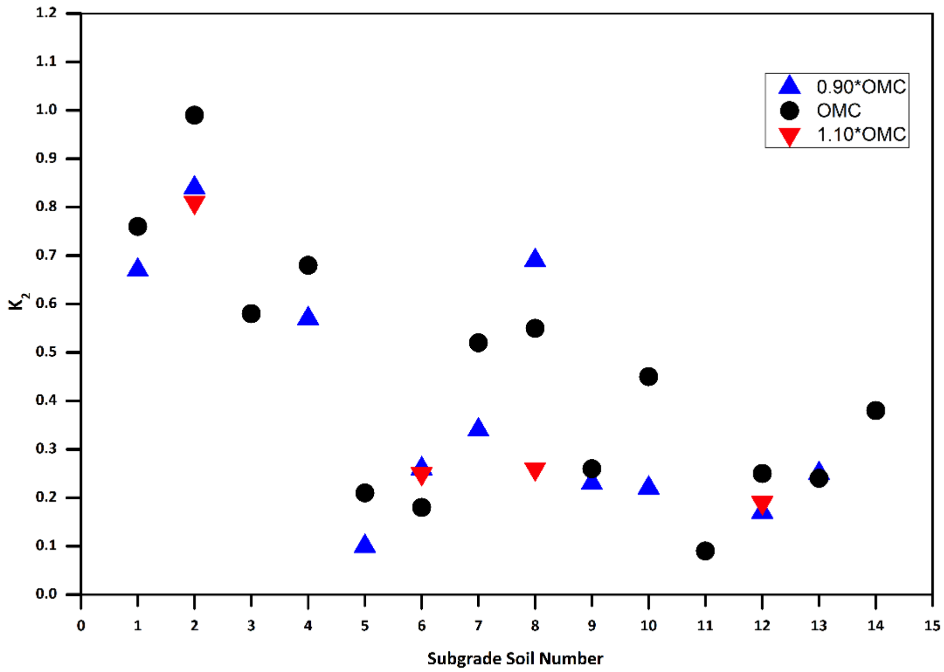


Figure 27. Effect of Compaction Moisture Content on k_2 Parameter of the Subgrade Soils Tested in the Current Study

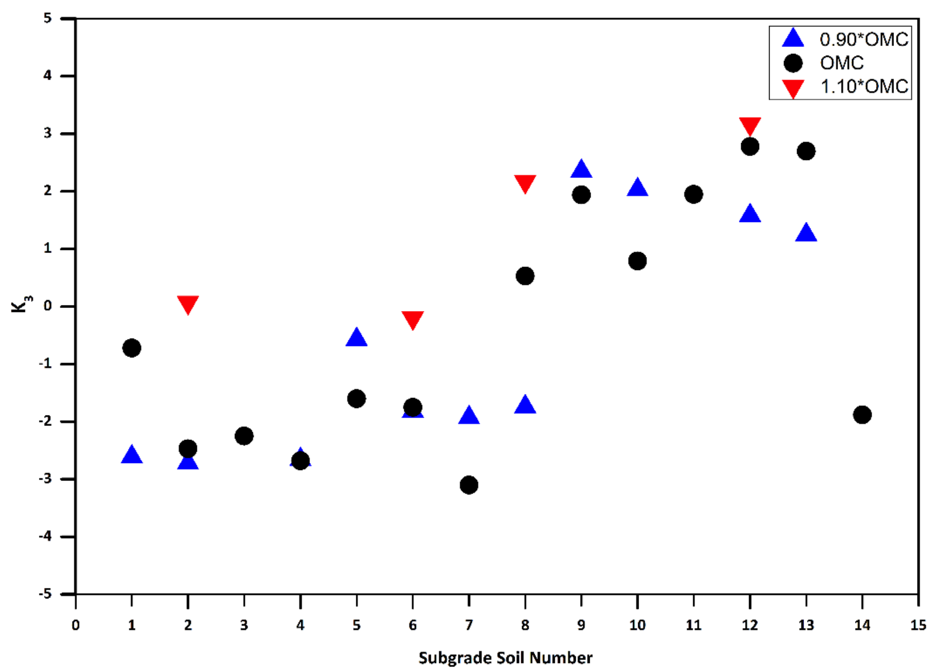


Figure 28. Effect of Compaction Moisture Content on k_3 Parameter of the Subgrade Soils Tested in the Current Study

Unconfined Compression Testing of Subgrade Soils

As already mentioned, one of the objectives of this research study was to explore the possibility of correlations between resilient modulus properties of soils and aggregates and other 'easy to measure' index as well as engineering properties. The Unconfined Compressive Strength (UCS) represents one of the most commonly used engineering properties for cohesive soils. Researchers in the past have attempted to establish relationships between the unconfined compressive strength and resilient modulus for cohesive soils.[43,61,63] Accordingly, the subgrade soils collected in this study were also tested for UCS properties to assess whether a statistically significant correlation could be established between UCS and M_R. The UCS tests were performed in accordance with AASHTO T 208 specifications. Note that to optimize the use of available materials, the same specimen that was tested for resilient modulus properties was subsequently tested for UCS. This is a common practice in soil testing as the resilient modulus test is non-destructive in nature, and it is assumed that subjecting a soil specimen to repeated loading during resilient modulus testing does not affect its engineering properties. The soil specimens were tested for UCS under controlled strain conditions at a rate of 1 mm/minute as required by AASHTO T 208. Figure 29 shows a photograph of a soil specimen being tested for UCS. The average maximum stress obtained from two replicate samples for each material type were calculated and have been listed in Table 26. As seen from the data, all subgrade soils exhibited moisture-sensitive behavior with significantly higher UCS values observed for the specimens compacted at 90% of OMC. Because of limited material availability, UCS tests on the following subgrade soils were performed under OMC conditions only: 17-9SL-0101, 17-9SL-0054 and 17-9SL-0058 soils.

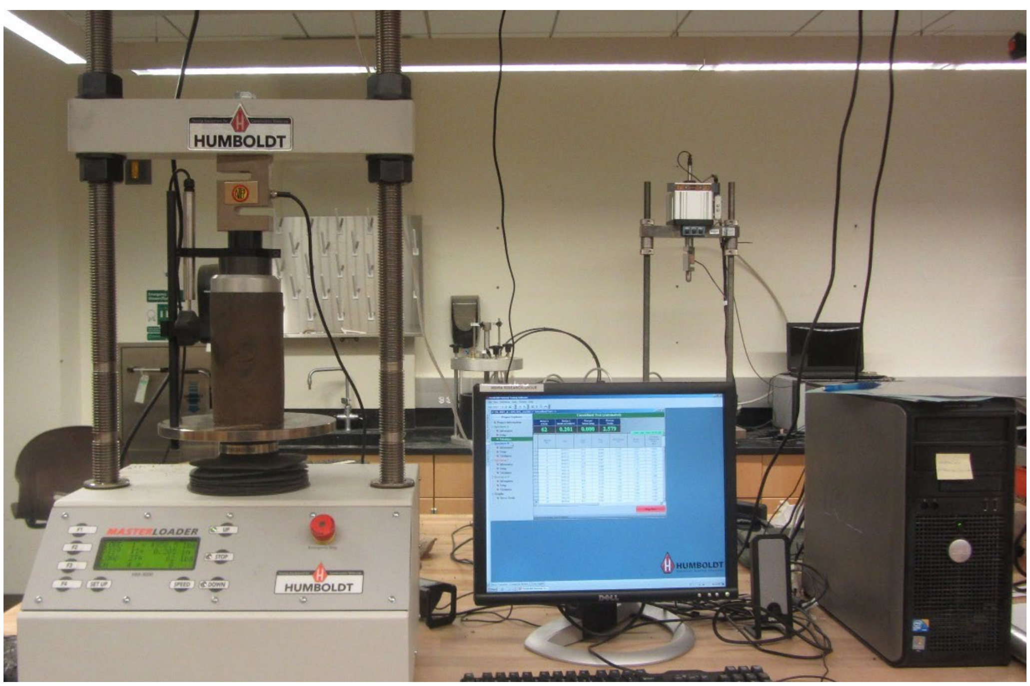


Figure 29. Photograph Showing Soil Specimen being Tested for Unconfined Compressive Strength (UCS)

Table 26. Unconfined Compressive Strength Values of Soils Tested at Different Moisture Condition

Materials	Unconfined Compressive Strength (UCS)					
	0.9*OMC		OMC		1.10*OMC	
	psi	kPa	psi	kPa	psi	kPa
D1-ML	29.4	202.7	22.5	155.1	17.5	120.7
D1-GM	25.9	178.6	16.6	114.5	16.5	113.8
17-9SL-0101	-	-	22.3	153.8	-	-
TP-8	32.0	220.6	24.1	166.2	15.95	110.3
D3-SM	38.3	264.1	34.1	235.1	19.3	133.1
D3-SC	39.1	269.6	34.5	237.9	21.5	148.2
Cs-184	36.1	248.9	23.5	162.7	10.99	75.8
LN-80	16.5	113.8	15.2	104.8	12.0	82.7
Bk-180c	19.5	134.4	14.0	96.5	7.0	48.3
JF-83	7.9	54.5	1.5	10.3	0.18	1.24
17-9SL-0054	-	-	18.0	124.1	-	-
17-9SL-0055	25.0	172.4	17.0	117.2	14.9	103.4
17-9SL-0057	24.0	165.5	21.0	144.8	7.5	51.7
17-9SL-0058	-	-	25.6	176.5	-	-

Summary

This chapter presented findings from repeated load triaxial testing of soils and aggregates to establish the resilient modulus properties. Details regarding specimen preparation and the test procedure were presented followed by detailed analyses of the test results. The test results were then fitted against commonly used constitutive equations that can be used to predict the resilient modulus values for soils and aggregates. Regression model parameters thus established were tabulated, and the values were analyzed to ensure the observed trends were consistent with those reported by other researchers in the past. The next chapter will detail the research team's efforts to assess if the resilient modulus properties for aggregates and soils in the state of Idaho can be predicted with reasonable accuracy from other "easy to measure" mechanical and index properties.

Chapter 5
Predicting Resilient Modulus Values from Other Index or Mechanical Properties

One of the primary obstacles in the path of agency-wide implementation of resilient modulus testing is that the test is time consuming and requires expensive equipment as well as well-trained laboratory personnel. Nevertheless, the resilient modulus is the most important input parameter for unbound materials (soils and aggregates) during M-E pavement design. Several states have therefore undertaken efforts to establish correlations between the resilient modulus of soils and aggregates and other ‘easy-to-measure’ index and engineering properties. Along the same lines, the current research study attempted to establish correlation equations between resilient modulus and other properties. Availability of reliable and statistically significant correlations will help ITD engineers predict resilient modulus values for soils and aggregates as inputs for Pavement ME Design. These correlations could be used as Level 2 inputs in Pavement ME Design. Although Level 2 inputs are not as accurate as Level 1 inputs, equations developed for local materials (for example in a given state) would ultimately lead to better representation of material behavior. This prediction equation can be developed with the help of multiple regression analysis, where the independent variables could be commonly tested soil/aggregate parameters such MDD, OMC, particle size distribution, etc. and/or index properties like CBR, R-value etc. The predicted value of the M_R of a certain material could be used as input during M-E pavement design.

Defining a Single Resilient Modulus Value for Unbound Materials

As discussed in Chapter 4 of this report, the resilient modulus for a particular unbound material (aggregate or soil) is commonly established through repeated load triaxial testing. The most common test specification used for this purpose is AASHTO T 307. While testing a particular aggregate or soil for resilient modulus per the AASHTO T 307 procedure, a cylindrical specimen is subjected to repeated loading under fifteen different stress sequences (after a conditioning sequence). During each of the 15 stress sequences, the specimen is subjected to 100 load pulses, and data from the last five cycles under each sequence is used to establish the resilient modulus value for that particular stress state. Therefore, at the end of the resilient modulus test procedure, fifteen different resilient modulus values (one corresponding to each stress sequence) are obtained for the material being tested. Considering the stress-dependent behavior for unbound aggregates (stress-hardening) and fine-grained soils (stress-softening), the resilient modulus values obtained corresponding to the 15 stress sequences can vary significantly from one another.

Starting from the 1986 AASHTO Pavement Design Method, Resilient Modulus is an important input parameter to characterize unbound materials in a pavement structure. However, the design methodology uses only one resilient modulus value to represent the “stiffness” of the unbound layers. The same is true for the 1993 AASHTO design guide as well as current implementation of the MEPDG through the AASHTOWare Pavement ME Design. It should be noted that the current version of

AASHTOWare Pavement ME Design (version 2.5.3) does not incorporate Level 1 design capabilities for unbound materials (aggregates and soils). In Level 2, a single value of resilient modulus for the entire layer is calculated from other index/mechanical properties using pre-established correlations. Similarly, in Level 3, a single value of resilient modulus for the entire layer is estimated from general information such as soil classification. *It is therefore critical to decide what single value for resilient modulus can be used (out of the 15 different values established from AASHTO T 307 testing) to represent the modulus of the entire aggregate/soil layer.* NCHRP 1-28 A [9] recommends reporting a "summary resilient modulus" for each material tested. For aggregate base/subbase materials, this corresponds to the stress state where the confining pressure equals 35 kPa or 5 psi (σ_3 = 35 kPa or 5 psi), and the deviator stress equals 103 kPa or 15 psi (σ_d = 103 kPa or 15 psi). Looking at the 15 different stress sequences specified by AASHTO T 307, this corresponds to sequence # 6. Therefore, the resilient modulus value calculated for sequence # 6 can be used as a "summary resilient modulus" to represent the "stiffness characteristics" of base/subbase materials. Similarly, NCHRP 1-28A recommends using σ_3 = 14 kPa (2 psi) and σ_d = 41 kPa (6 psi) to calculate the summary resilient modulus values for subgrade soils. This corresponds to stress sequence # 13 per AASHTO T 307.

The current study utilized the concept of summary resilient modulus when it was necessary to pick just one modulus values for a given material. Accordingly, any correlations developed to estimate the resilient modulus properties from other mechanical or index properties treated the summary resilient modulus as the primary "predicted" variable.

Developing Correlation Equations to Predict Resilient Modulus

Since the resilient modulus test is time consuming, and requires expensive equipment as well as well-trained personnel, having alternative approaches to estimate the resilient modulus values for base/subbase materials (eliminating the need for repeated load triaxial testing) is a desirable scenario for highway agencies. Accordingly, the current research study explored whether or not statistically significant correlations could be developed to predict soil/aggregate resilient modulus values from easy-to-measure mechanical/index properties.

Correlation Development for Base/Subbase Materials

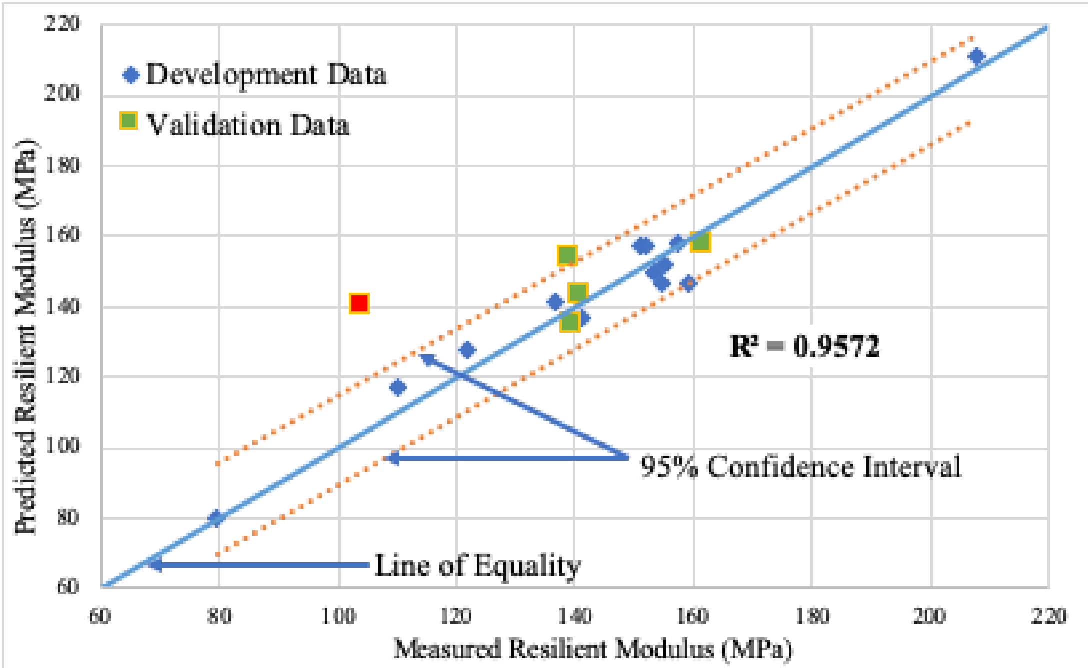
For the base/subbase materials, the correlation development effort involved use of the "Minitab" statistical software package. The Summary Resilient Modulus (SRM) values for fourteen (14) randomly selected base/subbase materials were used as "training data" or "development data", whereas the SRM values for the remaining four (4) materials were used as "test data" or "validation data". In Minitab software, the "stepwise" regression analysis technique was used to develop the prediction model. The average SRM of each material was considered as the response variable where the material properties like MDD, OMC, percent passing of #4 sieve, particle size of 10% (D_{10}) and 30% (D_{30}) finer were considered as predictor variables. Figure 30 presents the relationship between resilient modulus and various material parameters as established through this statistical analysis effort.

$$M_R = 198.2 + 1.405\,\frac{\mathrm{MDD}^{0.73}}{\mathrm{OMC}^{1.3863}} - 16.82\,(D_{10} + D_{30})^{1.18} - 1.081\,P_4 + 123\,L - 53.24\,H$$

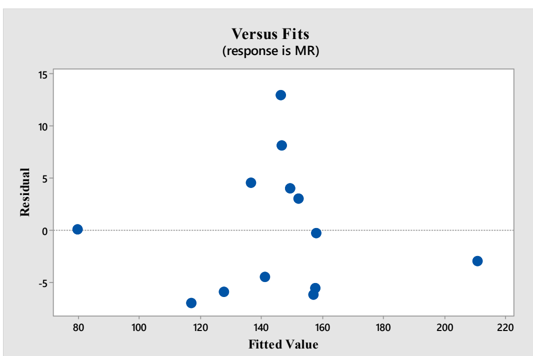
where,
M_R = summary resilient modulus (MPa)
MDD = maximum dry density (kg/m^3),
OMC = optimum moisture content (%),
D_{10} = particle diameter corresponding to 10 percent finer (mm),
D_{30} = particle diameter corresponding to 30 percent finer (mm),
P_4 = percent passing of #4 sieve,
L and H are categorical variables:
L = 1, if $D_{10} \geq 1.00$ mm; otherwise L = 0
H = 1, if OMC > 8.0%, otherwise H = 0.

Figure 30. Correlation between Resilient Modulus and Different Material Properties for the Base/Subbase Materials Tested in the Current Study

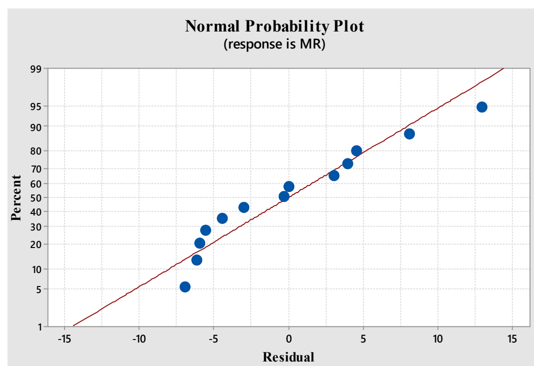
The measured and predicted resilient modulus values of test materials are depicted in Figure 31*a*. The validation data points are well covered by the 95% confidence interval envelop except for one material. The coefficient of determination (R^2) of the proposed model is high (0.9572) with adjusted R^2 of 0.9266. The basic assumptions of the regression analysis were met by the equation in Figure 30. The plot of the residuals against fitted value shows no significant pattern (see Figure 31-b). The normality Q-Q plot also satisfies the requirement, e.g. most of the data points are on the line or close to the line. (see Figure 31c).



(a)



(b)



(c)

Figure 31. Statistical Analysis of the Prediction Model (a) Measures vs Predicted value of Resilient Modulus, (b) Residual vs Fitted plot, (c) Q-Q plot

The p-value of each independent variables was less than 0.05 which means all the estimates for independent variables are significant. The following observations can be made from the equation presented in Figure 30.

- The resilient modulus increases with the increase of maximum dry density and decreases with the increase in optimum moisture content.
- The resilient modulus decreases with effective grain size (D_{10} and D_{30}) up to certain limit, and with percent passing of sieve No. 4 but at lower rate compared to effective grain size (D_{10} and D_{30}).

The analysis of variance of the model is provided in Table 27. Another aspect of this kind of multiple regression analysis is multicollinearity. Multicollinearity defines how the independent variables are correlated to each other. This can result in incorrect estimates of the coefficients and their standard errors. Variance Inflation Factor (VIF) is the measure of the multicollinearity. The lower the VIF value the less the independent variables are correlated each other. Several researchers used different cutoff values for the VIF. For this study the cutoff for the VIF is considered as 10.0 and all the VIFs for this model are found less than 10.0 (i.e., the independent variables are not correlated or highly correlated to each other). As expected, the highest VIF value was obtained for the (D_{10}+D_{30}) term as these two gradation parameters are correlated to each other; nevertheless, the VIF value for this term was less than 10.0.

Table 27. Analysis of Variance Table of the Developed Model

Coefficients					
Term	**Coef.**	**SE Coef.**	**t-Value**	**p-Value**	**VIF**
Constant	198.2	27.3	7.26	0.000	
MDD^0.73/OMC^1.3863	1.405	0.319	4.4	0.003	1.73
(D_{10} + D_{30}) ^ 1.18	-16.82	2.58	-6.51	0.000	9.96
P_4	-1.081	0.429	-2.52	0.040	4.25
Large					
1	123	13.8	8.94	0.000	4.88
High					
1	-53.24	9.65	-5.52	0.001	1.31
Analysis of Variance					
Source	**DF**	**Adj SS**	**Adj MS**	**F-Value**	**P-Value**
Regression	5	10278.6	2055.73	31.29	0.000
MDD^0.73/OMC^1.3863	1	1271.3	1271.32	19.35	0.003
(D_{10} + D_{30}) ^ 1.18	1	2783.7	2783.69	42.36	0.000
P_4	1	418.4	418.37	6.37	0.040
Large	1	5253.1	5253.09	79.95	0.000
High	1	2000	2000.02	30.44	0.001
Error	7	694.3	99.18		
Total	12				

Another attempt was made to correlate the summary resilient modulus with the CBR. The CBR values summarized in Table 19 were correlated with the corresponding summary resilient modulus values. Figure 32 shows the correlation between CBR and SRM at 5.08 mm and 2.54 mm penetration, respectively. The relationship between the two measures is nonlinear. For CBR at 5.08 mm (0.2 in) penetration, the coefficient of determination (R^2) is 0.5626, while the correlation yielded R^2 of 0.5668 at 2.54 mm (0.1 in) penetration. This correlation is considered fair given the fact that the SRM is calculated from repeated load testing, whereas, the CBR is calculated from a monotonic test where a loading piston penetrates into the specimen. Even though there is difference between the mechanics of resilient modulus and CBR tests, the parameter estimates were found statistically significant (i.e., *p-value* = 0.001 < 0.05), means correlation of summary resilient modulus and CBR is statistically significant.

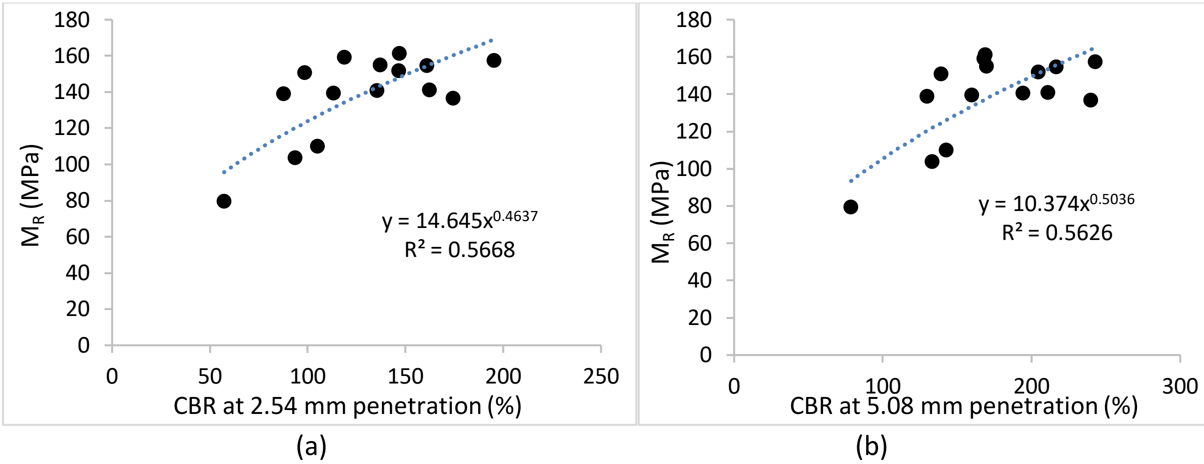


Figure 32. Correlation between Resilient Modulus and CBR Measured at (a) 2.54 mm (0.1 in) Penetration, (b) 5.08 mm (0.2 in) Penetration

From the above-reported results, it can be seen that resilient modulus values for unbound aggregate (base/subbase) materials tested in the current study *could be predicted with reasonable accuracy from other 'easy-to-measure' mechanical and index properties*. The equation presented in Figure 30 could be used with reasonable accuracy to estimate the resilient modulus values for the aggregate materials tested in this study. However, whether the correlation will hold for materials other than those tested in the current study, needs to be investigated.

Correlation Development for Subgrade Soils

A similar approach was used to assess whether the resilient modulus values for the subgrade soil materials could be predicted with reasonable accuracy using other mechanical and index soil properties. The summary modulus was used as the predicted variable, and properties such as Moisture Content, Dry Density, Plasticity Index, Liquid Limit, Percent Fines (fraction finer than then 0.075 mm sieve or passing sieve number 200), unconfined compressive strength (qu), and CBR were used as the predictor variables. The statistical analysis was carried out using the SAS software package. Note that after initial analysis, it was observed that only percent fines (fraction finer than # 200 sieve) had a statistically significant correlation (with a p-value of 0.05) with the SRM value. R-value test results were collected from ITD central labs for some of the soils, and those results were used to assess if SRM values could be predicted with reasonably accuracy. However, no significant statistical correlation was found. Figure 33 shows graphs of SRM values (only at the OMC) for the subgrade materials plotted against other easy-to-measure soil properties. As seen from the figure, only percent fines (Figure 33-c) shows a clear relationship with summary resilient modulus, with the SRM value clearly decreasing with increasing fines contents. Although the moisture content plot (Figure 33-a) does show a clear trend at the beginning (SRM decreases with increase in moisture content), the trend disappears for higher moisture contents. This is primarily because different subgrade soil materials tested exhibit different moisture sensitivities, and some of the soils were still "stable" at relatively high moisture contents. Similarly, a roughly increasing trend is observed for SRM plotted vs. dry density (Figure 33-b). However, when all data is taken together, even that trend tends to disappear. No clear trend between CBR and SRM were found to satisfactorily cover all soil types either (see Figure 33-e).

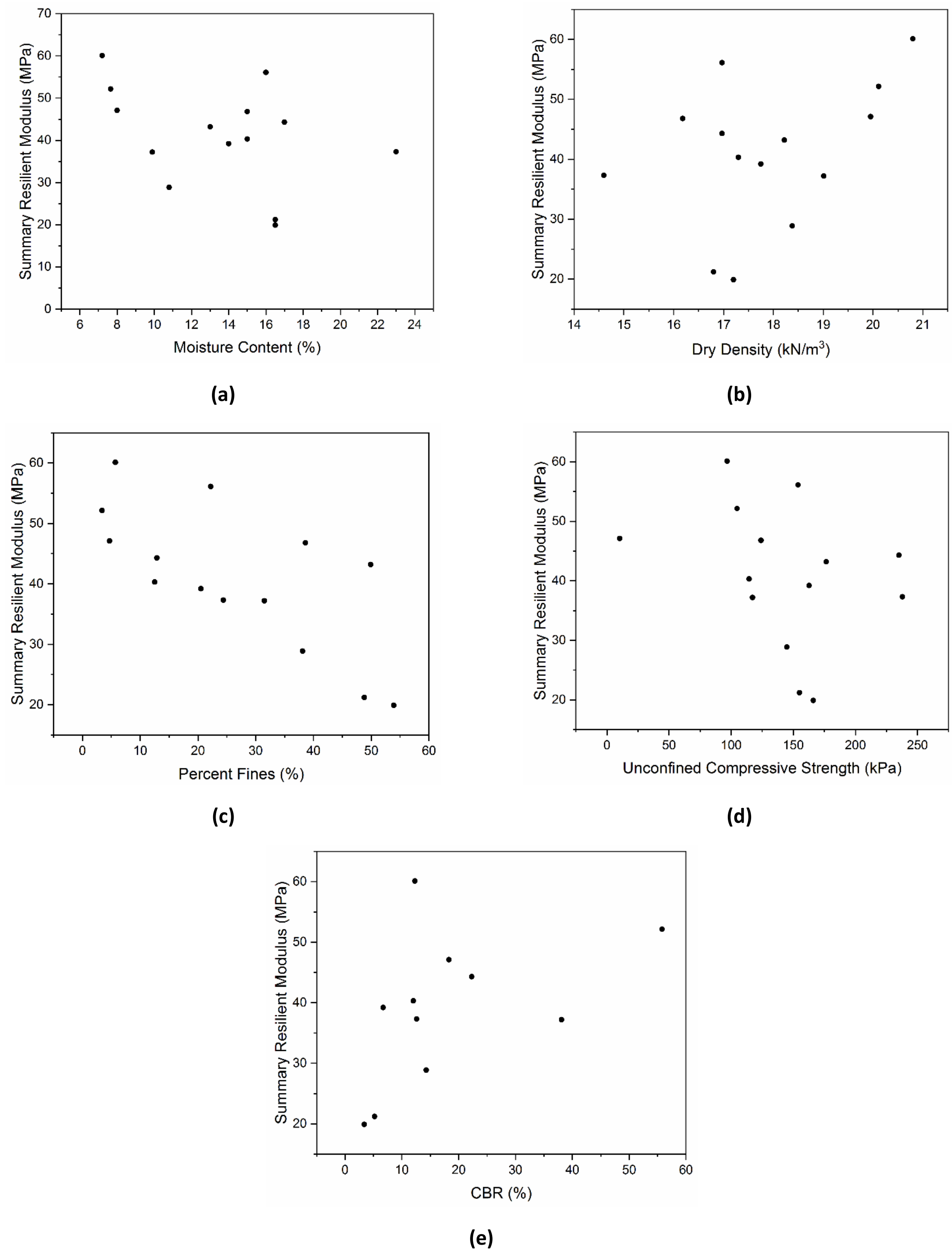


Figure 33. Scatterplots Showing Variation of Summary Resilient Modulus for the Subgrade Soils with Other Soil Properties: (a) Moisture Content (b) Maximum Dry Density (c) Percentage Passing #200 Sieve (d) Unconfined Compressive Strength and (c) California Bearing Ratio

The next step in the statistical analysis process involved carrying out a stepwise linear regression (using SAS) to check which predictor terms can be used to predict the SRM values with reasonable accuracy. Results from this analysis are presented in Figures 34 and 35. As seen from these figures, only the percent fines term could be used to predict the SRM value with reasonable accuracy (R^2 = 0.5124). No other terms were statistically significant to be included in the stepwise linear regression model.

Stepwise Selection: Step 1

Variable Fines Entered: R-Square = 0.5124 and C(p) = -2.1316

Analysis of Variance					
Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	1	932.63399	932.63399	12.61	0.0040
Error	12	887.32981	73.94415		
Corrected Total	13	1819.96380			

Variable	Parameter Estimate	Standard Error	Type II SS	F Value	Pr > F
Intercept	53.67024	4.24853	11800	159.58	<.0001
Fines	-0.48396	0.13627	932.63399	12.61	0.0040

Bounds on condition number: 1, 1

All variables left in the model are significant at the 0.1000 level.

No other variable met the 0.0500 significance level for entry into the model.

Summary of Stepwise Selection								
Step	Variable Entered	Variable Removed	Number Vars In	Partial R-Square	Model R-Square	C(p)	F Value	Pr > F
1	Fines		1	0.5124	0.5124	-2.1316	12.61	0.0040

Figure 34. Results from Stepwise Linear Regression of Resilient Modulus Test Results for Subgrade Soils Showing Percent Fines as the only Statistically Significant Predictor for Summary Resilient Modulus (SRM)

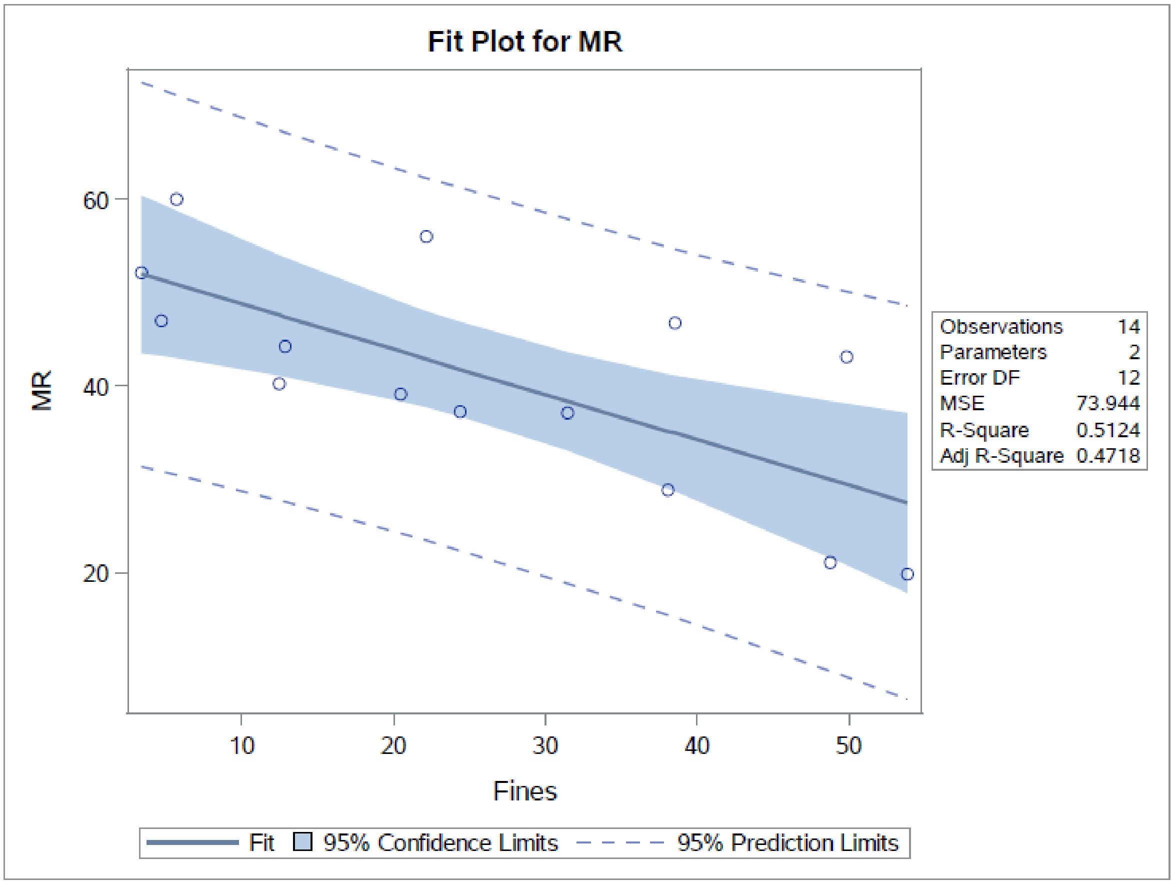


Figure 35. Fit Plot for Summary Resilient Modulus Prediction Using Percent Fines as the Predictor Variable

Summary

This chapter presented results from statistical analysis of the resilient modulus test data to assess whether or not the resilient modulus values for unbound aggregate (base/subbase) and soils typically used in pavement applications in the state of Idaho can be predicted with reasonable accuracy from other "easy-to-measure" mechanical and index properties. Extensive statistical analysis of the data indicated that <u>*although it was possible to predict the resilient modulus values for the aggregates using other properties, the same results could not be accomplished for the subgrade soils*</u>. The next chapter will summarize findings from the current study, and will present recommendations on how the findings from the current study can be used by ITD engineers during pavement analysis and performance prediction using AASHTOWare Pavement ME Design.

Chapter 6
Summary, Conclusions, and Recommendations

The Idaho Transportation Department (ITD) has traditionally relied on the R-value and gravel equivalency factors to determine the required thicknesses for pavement layers. This approach has been known to be overly conservative, and does not provide optimal pavement thicknesses. ITD has recently invested considerable amount of resources to transition into the Mechanistic-Empirical pavement design approach (implemented through the AASHTOWare Pavement ME Design). As a part of this effort, the current research study was initiated to characterize unbound materials commonly used in the state of Idaho for pavement applications.

The primary objective of this research study was to characterize representative, local material properties for unbound layers to facilitate comprehensive MEPDG implementation in the State of Idaho. The research team worked with engineers from all six ITD districts to collect representative base/subbase and subgrade materials for laboratory testing. A total of 18 base/subbase and 16 subgrade materials were tested in the laboratory to establish the resilient modulus properties along with other commonly used index and mechanical properties. Tests conducted in the laboratory included: particle size distribution, Atterberg limits, soil classification, moisture density relationships, and California Bearing Ratio (CBR) etc. Moreover, the subgrade soil materials were also tested for unconfined compressive strength (q_u) properties. Note that all tests on the base/subbase materials were carried out at the Optimum Moisture Content (OMC), whereas the subgrade soils were tested at three different moisture contents (90% OMC, OMC, and 110% OMC) to also study the effect of moisture content on material behavior. Resilient Modulus testing of the materials was carried out using the AASHTO T 307 protocol.

Extensive statistical analysis of the test results was carried out to assess whether or not the resilient modulus properties can be predicted with reasonable accuracy from other commonly used index/mechanical properties, thus eliminating the need for cumbersome repeated load triaxial testing. Finally, a database was developed with all test results to help ITD engineers quickly obtain input parameters for unbound materials during pavement analysis and performance prediction using Pavement ME Design.

Summary of Significant Findings

The following are the most significant findings from this research study.

1. After completion of all laboratory tests, different correlations developed in the past by researchers to predict the resilient modulus values of soils and aggregates (summarized in Chapter 2 of this report) were first tried with the test data generated in the current study. It was found that none of the existing models could work for materials available in Idaho.

2. An effort was made to develop correlation equation for predicting resilient modulus from less expensive laboratory test results such as index properties, CBR, R-value and unconfined compressive strength. The particle size and parameters from moisture density tests were used as independent variables successfully to predict the resilient modulus of aggregates with R^2 value of near 0.90. However because of wide variety of soils available in Idaho, none of listed properties showed any significant as an individual variable except percentage finer than 200 sieve.
3. A summary resilient modulus prediction model was developed for the base/subbase materials as a function of material properties including maximum dry density, optimum moisture content, particle diameter corresponding to 10% and 30% finer and percent passing through 4.75 mm (#4) sieve. The model provided good correlation with the laboratory-measured resilient modulus for typical unbound materials used in road construction in Idaho.
4. Direct measurement of resilient properties of aggregates is quite expensive, time-consuming and very complex to conduct. The developed model shall assist ITD to accurately estimate the resilient modulus values for coarse unbound materials from basic parameters that are typically determined from compaction and gradation tests.
5. The stress dependency of resilient modulus of the unbound aggregates and soils was examined and evaluated using four constitutive models (i.e., K-θ model, Uzan model, Modified Uzan model, MEPDG model) for the aggregates, and the MEPDG model for the subgrade soils.
6. A database was developed with all test results. The materials in this database have been divided by ITD district. That way, engineers from different ITD districts can go to the respective tabs in the database to obtain typical values for aggregates and soils used in pavement application in their respective districts.

Recommendations for Implementation of Project Findings

Based on the findings of this project, the following recommendation are reached:

1. The database developed in this study can be used by ITD Engineers during pavement M-E design instead of using the assumed data from nearby states.
2. If it isn't possible to perform the repeated load triaxial test because of time constrain or availability of equipment, *<u>it is suggested to use developed model for predicting resilient modulus for base and subbase aggregates</u>*. This predicted M_R value can be taken as Level-2 input in Pavement ME design.
3. Subgrade soils in Idaho have wide variability including silt, clayey sand, poorly graded sand and poorly to well-graded gravel. If possible, repeated load triaxial tests should be performed for the subgrade soils. *<u>However, in cases where performing such tests are not possible, the ITD engineers can obtain from the database, typical values for the soils tested from their respective district, and use as input into Pavement ME Design</u>*. Although no statistically significant correlation equation could be developed for the subgrade soils, using typical values for respective districts from the database will significantly enhance the reliability levels for these input properties.

4. Pavement M-E requires a single resilient modulus value while the AASHTO T307 test protocol gives 15 different modulus values corresponding to 15 different stress states. In such cases, the Summary Resilient Modulus (SRM) value should be used during pavement design. Per NCHRP 1-28A, the SRM value corresponds to the sixth AASHTO T 307 stress sequence for aggregates (Type I materials; σ_3 = 5 psi; σ_d = 15 psi) and the thirteenth AASHTO T 307 stress state for subgrade soils (Type II materials; σ_3 = 2 psi; σ_d = 6 psi).
5. Once pavement ME design has been updated to include Level 1 for unbound materials, the direct input of resilient modulus will be possible using the fitted model parameters. The model parameters found after regression analysis using tested data are also included in the database. Note that the model parameters for aggregate materials have been presented only for OMC conditions, whereas those for subgrade soils have been presented for three different moisture conditions (90% OMC, OMC, and 110% OMC). Accordingly, depending on the drainage conditions at the project location, the engineer can chose to use model parameters corresponding to relatively dry or wet conditions.
6. Moisture sensitivity study indicated that some of the soils are too weak either failed during testing or showed significantly decreased resilient modulus values. It is recommended to be more conservatives regarding drainage provisions to ensure that subgrade are not getting wet in that area having sensitive soils.

References

1. **American Association of State Highway and Transportation Officials (AASHTO)**. (1993). AASHTO guide for design of pavement structure, Washington, D.C

2. **AASHTO**. Interim Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials. 1972.

3. **AASHTO** Guide for Design of Pavement Structures. American Association of State Highway and Transportation Officials, Wnshington, D.C., 1986.

4. **ARA, Inc.**, ERES Consultants Division. Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures. Final Report, NCHRP Project 1-37A. Transportation Research Board, National Research Council, Washington, D.C., 2004. http://www.trb.org/mepdg/guide.htm.

5. **Lekarp, F., U. Isacsson, and A. Dawson.** State of the art. I: Resilient response of unbound aggregates. Journal of Transportation Engineering, 126(1), 2000, 66-75 http://ascelibrary.org/doi/10.1061/%28ASCE%290733-947X%282000%29126%3A1%2866%29.

6. **Mishra, D.** Aggregate Characteristics Affecting Response and Performance of Unsurfaced Pavements on Weak Subgrades. PhD dissertation, University of Illinois at Urbana-Champaign, 2012.

7. **Tutumluer, E.** Practices for Unbound Aggregate Pavement Layers. NCHRP Synthesis No. 445, NCHRP Project 20-05, Topic 43-03, 2013. https://trid.trb.org/view.aspx?id=1278501.

8. **Taylor, A.** Mechanistic characterization of resilient moduli for unbound pavement layer materials. PhD dissertation, 2008. https://etd.auburn.edu/handle/10415/1159

9. **National Cooperative Highway Research Program Project 1-28A.** Harmonized Test Methods for Laboratory Determination of Resilient Modulus for Flexible Pavement Design. 2004.

10. **AASHTO T307**. Standard Method of Test for Determining the Resilient Modulus of Soils and Aggregate Materials. AASHTO'S Standard Specification for Transportation Material and Methods of Sample and Testing, 1999.

11. **Knight, J. A.** Gradation of Aggregate as Applied to Stabilization of Gravel Roads. Canadian Engineer, 13(23), 1935, pp. 9-10.

12. **Radjai, F., D. E. Wolf, M. Jean, and J. J.** Moreau. Bimodal Character of Stress Transmission in Granular Packings. Physical Review Letters, 80(1), 1998, pp. 61-64.

13. **Voivret, C., F. Radjai, J. Y. Delenne, and M. S. El Youssoufi.** Multiscale Force Networks in Highly Polydisperse Granular Media. Physical Review Letters, 102(17), Article 178001, 1-4, 2009.

14. **Hicks, R.G., and C.L. Monismith.** Factors Influencing the Resilient Properties of Granular Materials. In Transportation Research Record 345, Transportation Research Board, National Research Council, Washington D.C., 1971, pp. 15-31.

15. **Uzan, J.** Characterization of Granular Material. In Transportation Research Record: Journal of the Transportation Research Board, No. 1022, Transportation Research Board of the National Academies, National Research Council, 1985 pp. 52-59.

16. **Witczak, M.W., and J. Uzan.** The Universal Airport Pavement Design System, Report I of V: Granular Material Characterization. University of Maryland, Department of Civil Engineering, MD, 1988.

17. **Biarez, J.** Contribution à l'étude des propriétés mécaniques des sols et desmatériaux pulvérulents. D.Sc, Grenoble, 1961.

18. **Dunlap, W.S.** A Report on a Mathematical Model Describing the Deformation Characteristics of Granular Materials. Technical Report 1, Project 2-8-62-27, Texas Transportation Institute, Texas A&M University, College Station, 1963. http://d2dtl5nnlpfr0r.cloudfront.net/tti.tamu.edu/documents/TTI-1963-ID1100.pdf

19. **Seed, H.B., F.G. Mitry, C. L. Monismith, and C.K. Chan.** Factors Influencing the Resilient Deformations of Untreated Aggregate Base in Two Layer Pavements Subjected to Repeated Loading. Highway Research Record, Highway Research Board, No.190, 1967, pp. 19-55.

20. **Seed, H.B., Mitry, F.G., C.L. Monismith, and C.K. Chan.** Prediction of Flexible Pavement Deflections from Laboratory Repeated Load Tests. NCHRP Rep. No. 35, National Cooperative Highway Research Program, 1967, pp. 117

21. **Shackel, B.** Repeated Loading of Soils–A Review. Australian Road Research, Vol. 5, No. 3, 1973, pp. 22-49.

22. **Boyce, J.R.** A Nonlinear Model for the Elastic Behavior of Granular Materials Under Repeated Loading. International Symposium on Soils Under Cyclic and Transient Loading, Swansea, 1980.

23. **Moossazadeh, J., and M. W. Witczak.** Prediction of subgrade moduli for soil that exhibits nonlinear behavior. In Transportation Research Record 810, TRB, National. , Transportation Research Board, National Research Council, Washington, D.C., 1981, pp 9-17. https://trid.trb.org/view.aspx?id=173842.

24. **Lade, P.V., and R.B. Nelson.** Modeling the Elastic Behavior of Granular Materials. International Journal for Numerical and Analytical Methods in Geomechanics, Vol. 11, 1987, pp. 521-542.

25. **Itani, S.Y.** Behavior of Base Materials Containing Large-Sized Particles. PhD dissertation, Georgia Institute of Technology, School of Civil and Environmental Engineering, September, 1990.

26. **Crockford, W.W., L. J. Bendana, W.S. Yang, S.K. Rhee, and S.P. Senadheera.** Modeling Stress and Strain States in Pavement Structures Incorporating Thick Granular Layers.Final Report, Contract F08635-87-C-0039, The Texas Transportation Institute, The Texas A&M University, College Station, Texas, 1990.

27. **Pezo, R.F.** A General Method of Reporting Resilient Modulus Tests of Soils - A Pavement Engineer's Point of View. Paper No: 93082, Transportation Research Board, 72nd Annual Meeting, National Research Council, Washington D. C., 1993.

28. **Lytton, R. L., J. Uzan, E.G. Fernando, R. Roque, D. Hiltunen, and S. M. Stoffels**. Development and validation of performance prediction models and specifications for asphalt binders and paving mixes. (Vol. 357). Washington, DC: Strategic Highway Research Program, 1993. http://onlinepubs.trb.org/onlinepubs/shrp/SHRP-A-357.pdf.

29. **Kolisoja, P.** Resilient Deformation Characteristics of Granular Materials. PhD dissertation, Tampere University of Technology, Publ. No. 223, Tampere, Finland, 1997.

30. **Ni, B., T.C. Hopkins, L. Sun, and T.L. Beckham**. Modeling the Resilient Modulus of Soils. Proceedings of the 6th International Conference on the Bearing Capacity of Roads, Railways, and Airfields. Vol. 2, A.A., Balkema Publishers, Rotterdam, the Netherlands, 2002, pp. 1131–1142.

31. **Ooi, P.S.K., A.R. Archilla, and K.G. Sandefur.** Resilient Modulus for Compacted Cohesive Soils. In Transportation Research Record: Journal of the Transportation Research Board, No. 1874, Transportation Research Board of the National Academies, Washington, D.C., 2004, pp. 115-124.

32. **Gupta, S., A. Ranaivoson, T. Edil, C. Benson, and A. Sawangsuriya**. Pavement Design Using Unsaturated Soil Technology. Minnesota DOT Report No. MN/RC-2007-11, University of Minnesota, Minneapolis, 2007.

33. **George, K. P.** Resilient Modulus Prediction Employing Soil Index Properties. Report No. FHWA/MS-DOT-RD-04-172, Mississippi Department of Transportation, Jackson, Mississippi, 2004.

34. **Mokwa, R., and M. Akin.** Measurement and Evaluation of Subgrade Soil Parameters: Phase I–Synthesis of Literature. Final Project Report No. FHWA/MT-09-006/8199, Montana Department of Transportation, 2009. https://trid.trb.org/view.aspx?id=907074.

35. **Rahim, A. M.** Subgrade soil index properties to estimate resilient modulus for pavement design. International Journal of Pavement Engineering, 6:3, 2005, 163-169, DOI: 10.1080/10298430500140891

36. **Jones, M. P. and M. W. Witczak.** Subgrade Modulus on the San Diego Test Road. In Transportation Research Record 641, Transportation Research Board, National Research Council, Washington, DC, 1977, pp. 1-6. https://trid.trb.org/view.aspx?id=73467

37. **Robnett, Q. L., and M. R. Thompson.** Resilient Properties of Subgrade Soils, Phase I – Development of Testing Procedure. Illinois Department of Transportation, Interim Report, 1973, pp. 45 https://apps.ict.illinois.edu/projects/getfile.asp?id=2830.

38. **Thompson, M. R., and Q. L. Robnett.** Resilient Properties of Subgrade Soils. Transportation Engineering Series No. 14. Illinois Cooperative Highway and Transportation Series, (160), 1976. http://www.ict.uiuc.edu/Publications/report%20files/TES-014.pdf

39. **Thompson, M. R., and Q. L. Robnett.** Resilient Properties of Subgrade Soils. Transportation Engineering Journal, ASCE, Vol. 105, No. TE1, Proc. Paper 14293, January 1979, pp. 71-89.

40. **Thompson, M. R., & T. G. LaGrow**. A Proposed Conventional Flexible Pavement Thickness Design Procedure. Report No. FHWA-IL-UI-223, Project IHR-510, Illinois Cooperative Highway and Transportation Research Program, 1988.

41. **Carmichael III, R. F., and E. Stuart**. Predicting resilient modulus: A study to determine the mechanical properties of subgrade soils (abridgment). Transportation Research Record, (1043), 1985. https://trid.trb.org/view.aspx?id=272214.

42. **Elliott, R. P., S. I. Thornton, K. Y. Foo, K. W. Siew, and R. Woodbridge.** Resilient Properties of Arkansas Subgrades. Final report, No. FHWA/AR-89/004, 1988. https://trid.trb.org/view.aspx?id=297506.

43. **Drumm, E.C., Y. Boateng-Poku, and T. J. Pierce.** Estimation of Subgrade Resilient Modulus from Standard Tests. Journal of Geotechnical Engineering, 116(5), 1990, pp. 774-789.

44. **Woolstrum, G. (1990).** Dynamic testing of Nebraska soils and aggregates. Transportation Research Record, (1278). https://trid.trb.org/view.aspx?id=352766

45. **Farrar, M. J., and Turner, J. P.** Resilient Modulus of Wyoming Subgrade Soils. Report No. 91-1, Mountain Plains Consortium, University of Wyoming, Laramie, Wyoming, 1991.

46. **Guthrie, W. S. and K. D. Jackson.** Laboratory Resilient Modulus Measurements of Aggregate Base Materials in Utah. Final Report, Publication UT-16.12, Research Division, Utah Department of Transportation and FHWA, U.S. Department of Transportation, 2015.

47. **Buu, T**. "Correlation of Resistance R-value and Resilient Modulus of Idaho Subgrade Soil." Idaho Department of Transportation, Division of Highways. Special Report No.ML-08-80-G, 1980.

48. **Yeh, S. T., & Su, C. K**. Resilient properties of Colorado soils. Colorado Department of Highways. 1989.

49. **Muench, S. T., J. P. Mahoney, and L. M. Pierce**. "WSDOT pavement guide." Washington State Department of Transportation, Internet Module [Online] Available http://training. ce. washington. edu/WSDOT/,(accessed April 30, 2009). 2006.

50. **Heukelom, W. and A. J. G. Klomp**. Dynamic Testing as a Means of Controlling Pavements During and After Construction. Proceedings International Conference on the Structural Design of Asphalt Pavements, Ann Arbor, Michigan, 1962, pp. 667-679. https://trid.trb.org/view.aspx?id=717112.

51. **Green, James L., and Jim W. Hall**. Nondestructive vibratory testing of airport pavements. Volume 1. Experimental test results and development of evaluation methodology and procedure. ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS, 1975.

52. **Powell, W. D., J. F. Potter, H. C. Mayhew, and M. E. Nunn**. The Structural Design of Bituminous Roads. Transport and Road Research Laboratory, TRRL Laboratory Report 1132, Department of Transport, Berkshire, United Kingdom, 1984. https://trid.trb.org/view.aspx?id=211702.

53. **Lofti, H. A., C. W. Schwartz, and M. W. Witczak**. Compaction Specification for the Control of Pavement Subgrade Rutting. In Transportation Research Record 1196, Transportation Research Board, National Research Council, Washington, DC, 1988, pp. 108-115. http://onlinepubs.trb.org/Onlinepubs/trr/1988/1196/1196-011.pdf.

54. **Ohio, DOT**. Pavement Design Manual. Ohio Department of Transportation, Office of Pavement Engineering, July 2008. https://www.dot.state.oh.us/Divisions/Engineering/Pavement/Archive%20PDM/Pavement_Design_Manual_2015-07-17_version.pdf.

55. **Paterson W. D. L. and J. H. Maree.** An Interim Mechanistic Procedure for the Structural Design of Asphalt Pavements. National Institute for Transport and Road Research, Pretoria, South Africa, 1978.

56. **Wambura, G.** Characterization of Unbound Pavement Materials for Mechanistic-Empirical Performance Prediction. MS Thesis, University of Maryland, College Park. 2003

57. **Taylor, A. J. and D. H. Timm.** Mechanistic Characterization of Resilient Moduli for Unbound Pavement Layer Materials. NCAT Report-09-06, National Center for Asphalt Technology, Auburn University, Alabama, 2009.

58. **Ceylan, H., and K. G. S. Kim.** Characterization of Unbound Materials (Soils / Aggregates) for Mechanistic-Empirical Pavement Design Guide. Center for Transportation Research and Education, Iowa State University: Iowa Department of Transportation, 2009.

59. **Elias, M. B., and H. H. Titi.** Evaluation of Resilient Modulus Model Parameters for Mechanistic–Empirical Pavement Design. In Transportation Research Record: Journal of the Transportation Research Board, No. 1967, Transportation Research Board of the National Academies, Washington, D.C., 2006, pp. 89–100.

60. **Hopkins, T. C., T. L. Beckham, and C. Sun.** Resilient Modulus of Compacted Crushed Stone Aggregate Bases. Research Report KTC-05-27/SPR-229-01-1F, Kentucky Transportation Center, College of Engineering, University of Kentucky, 2007, pp. 89.

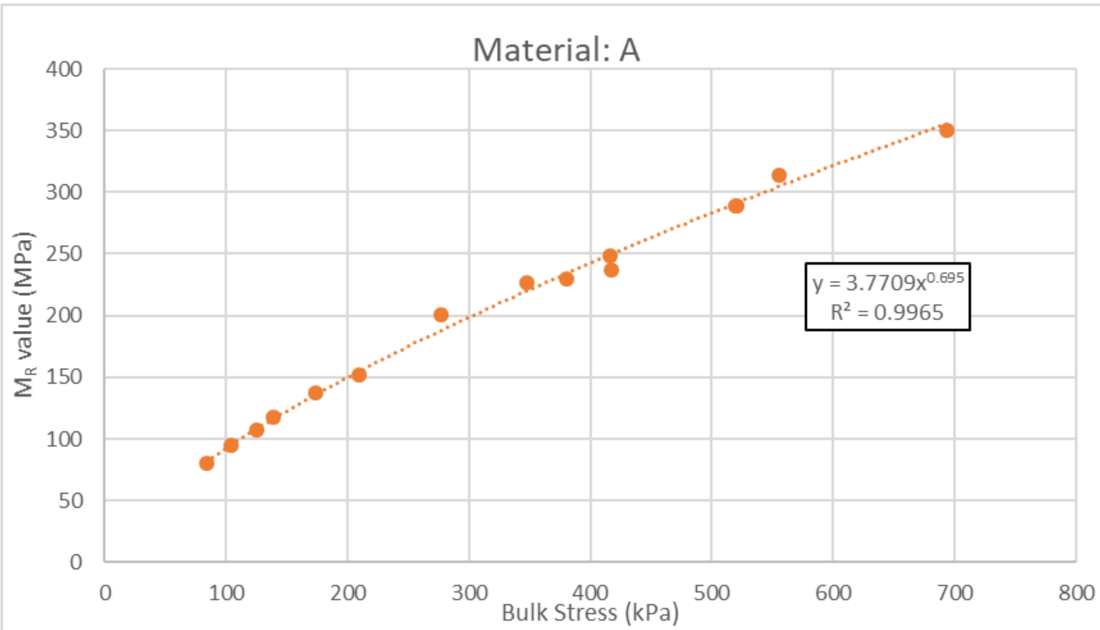
61. **Hossain, Z., M. Zaman, and C. Doiron.** Mechanistic Empirical Pavement Design Guide Input Parameters for Unbound Aggregates in Oklahoma. In Proceedings of the Second International Conference on Geotechnical and Earthquake Engineering, October 25-27, 2013, Chengdu, China, International Association of Chinese Geotechnical Engineers (IACGE), 2013, pp. 578–585.

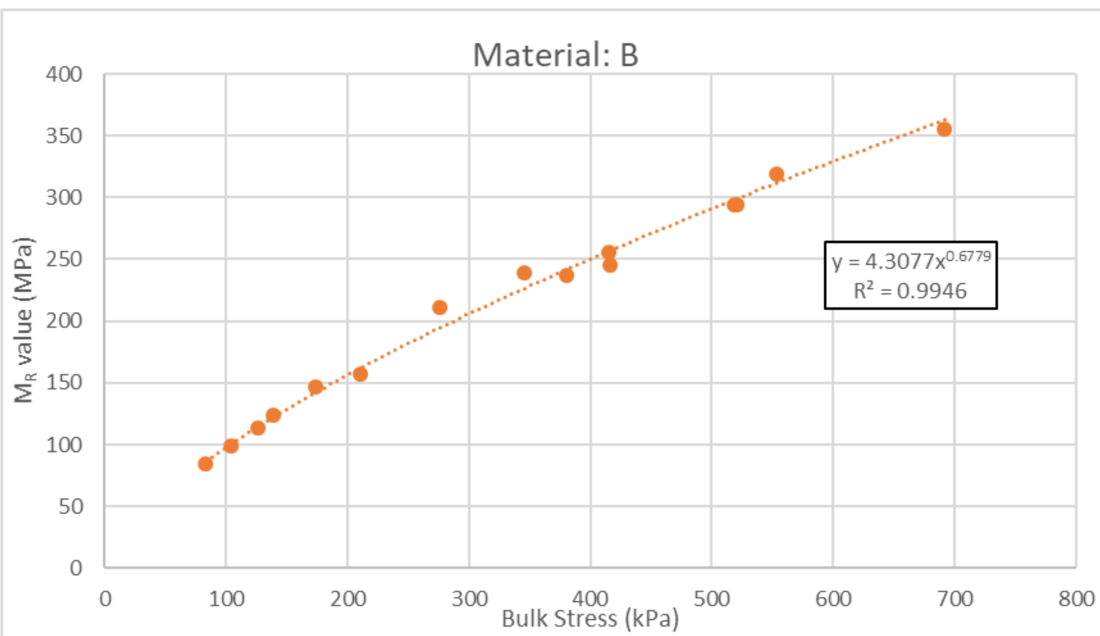
62. **Yau, A., and H. L. Von Quintus**. Study of LTPP Laboratory Resilient Modulus Test Data and Response Characteristics. Report FHWA-RD-02-051, Federal Highway Administration, Washington, DC, 2002.

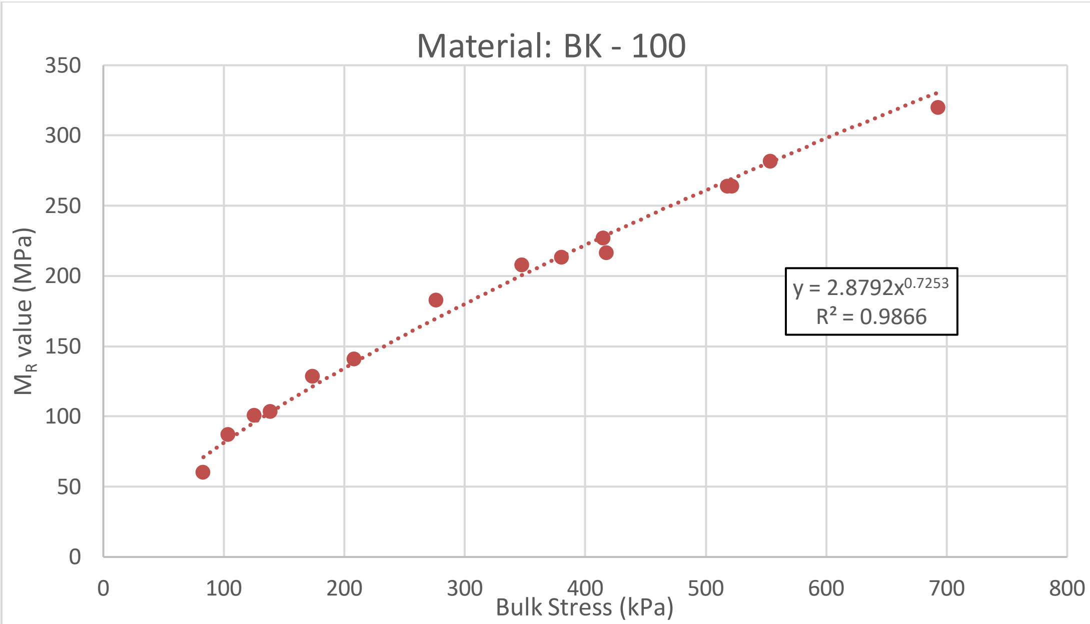
63. **Lee, W., N. C. Bohra, A. G. Altschaeffl, and T. D. White**. Resilient Modulus of Cohesive Soils. Journal of Geotechnical and Geoenvironmental Engineering, 123(2), 1997, 131-135.

Appendix A
Fitting Different Models to Aggregate Resilient Modulus Test Data

K-θ Model Parameters for Base Aggregates



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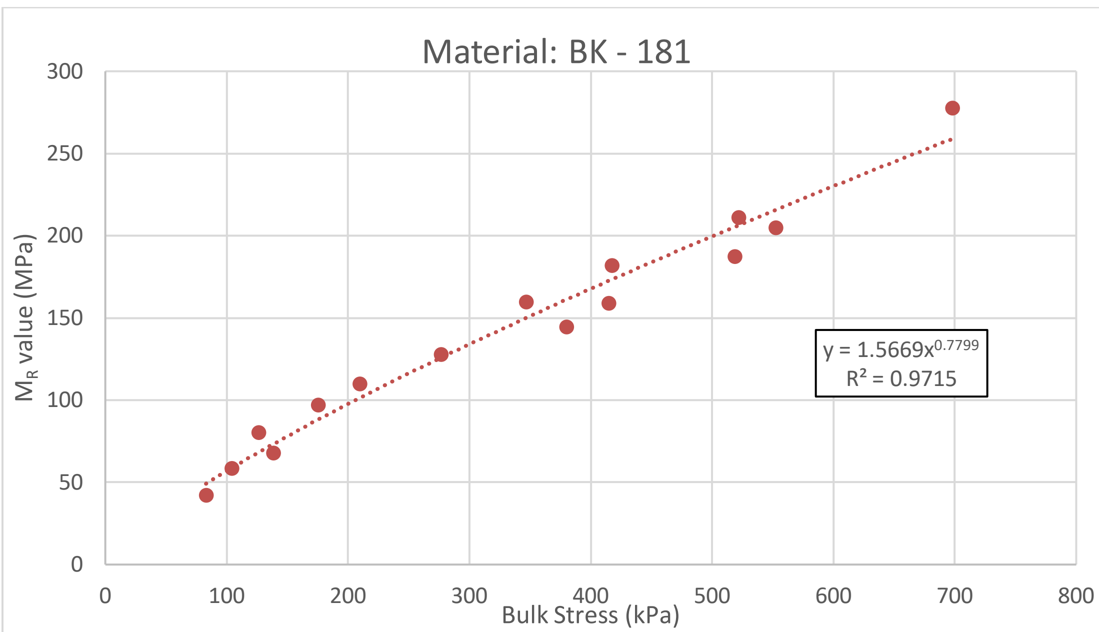
350
300
250
200
150
100
50
0
M_R value (MPa)
0
100
200
300
400
500
600
700
800
Bulk Stress (kPa)
y = 2.8792x^0.7253
R² = 0.9866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200
150
100
50
0
M_R value (MPa)
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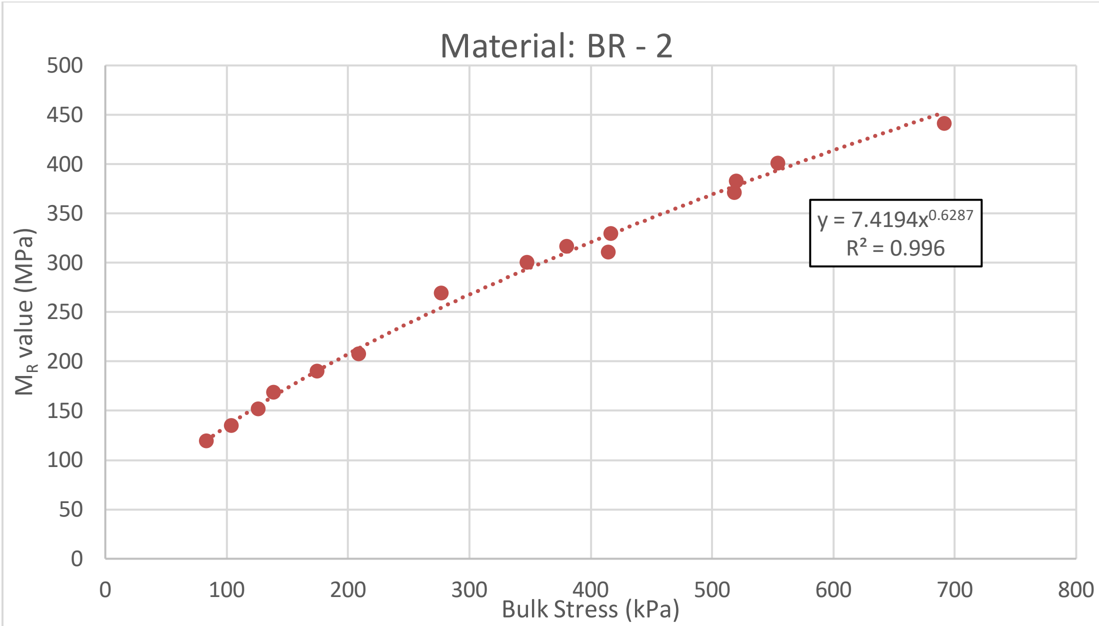
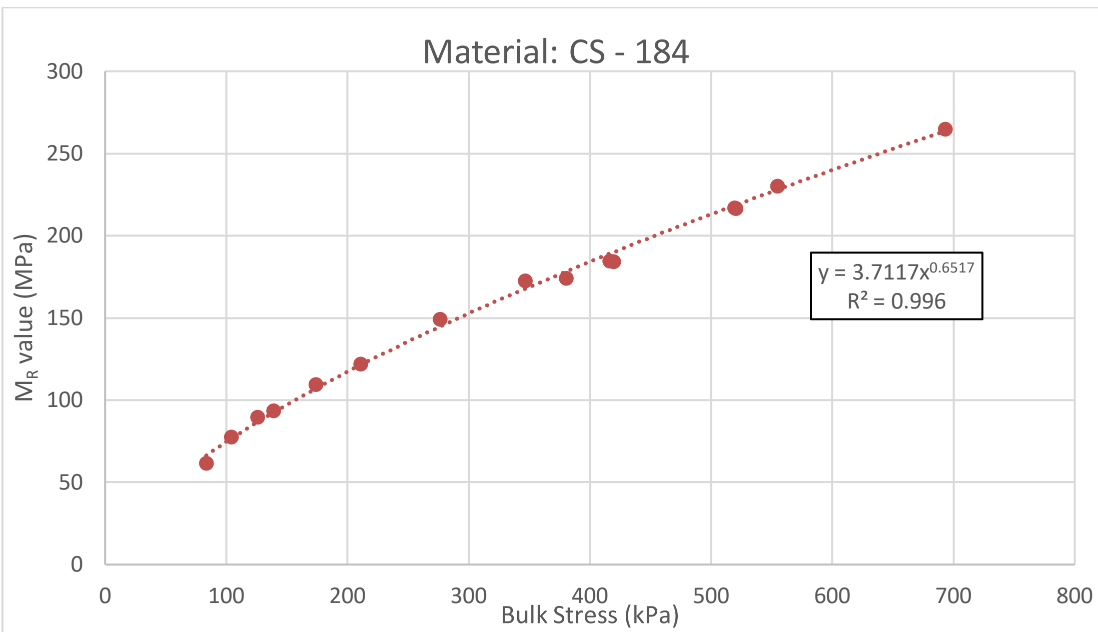
100
200
300
400
500
600
700
800
Bulk Stress (kPa)
y = 1.5669x^0.7799
R² = 0.9715


Material: BR - 2
500
450
400
350
300
250
200
150
100
50
0
M_R value (MPa)
0
100
200
300
400
500
600
700
800
Bulk Stress (kPa)
$y = 7.4194x^{0.6287}$
$R^2 = 0.996$


Material: CS - 184
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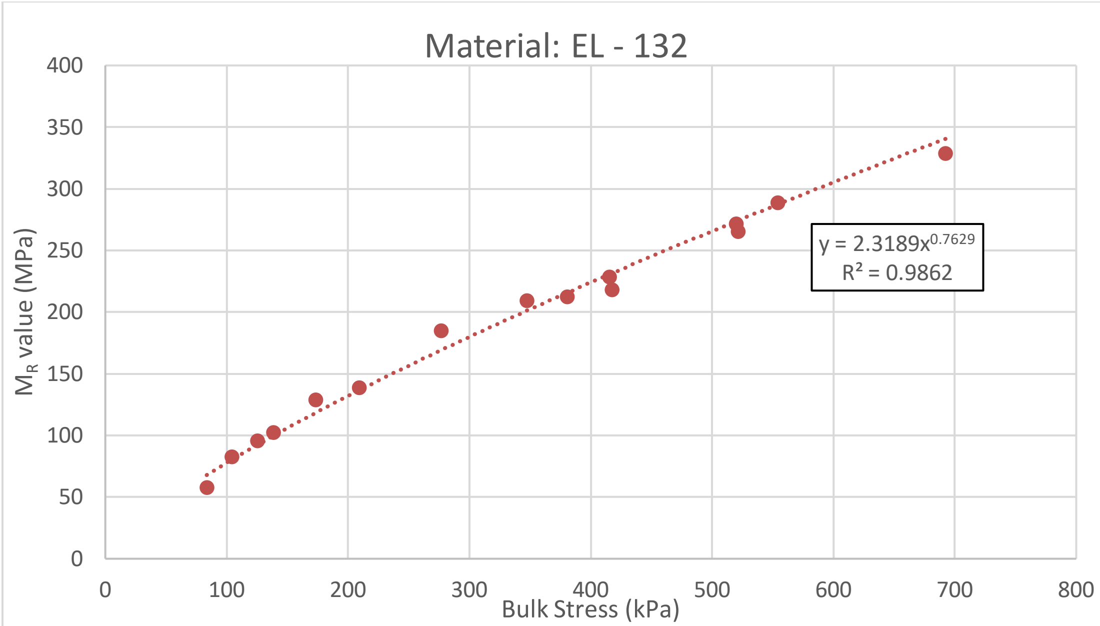
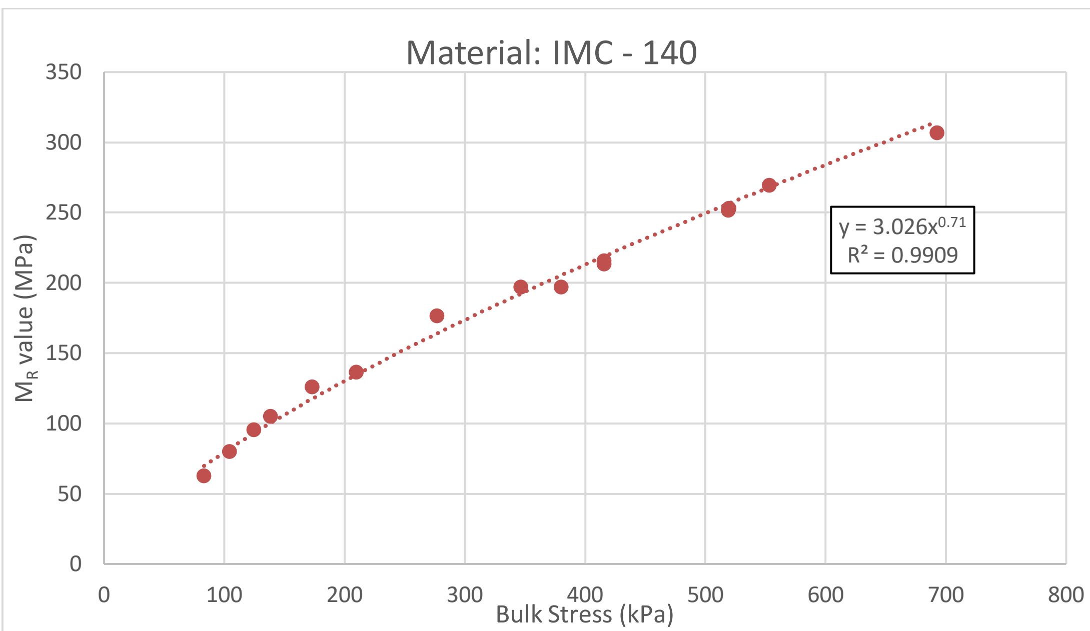
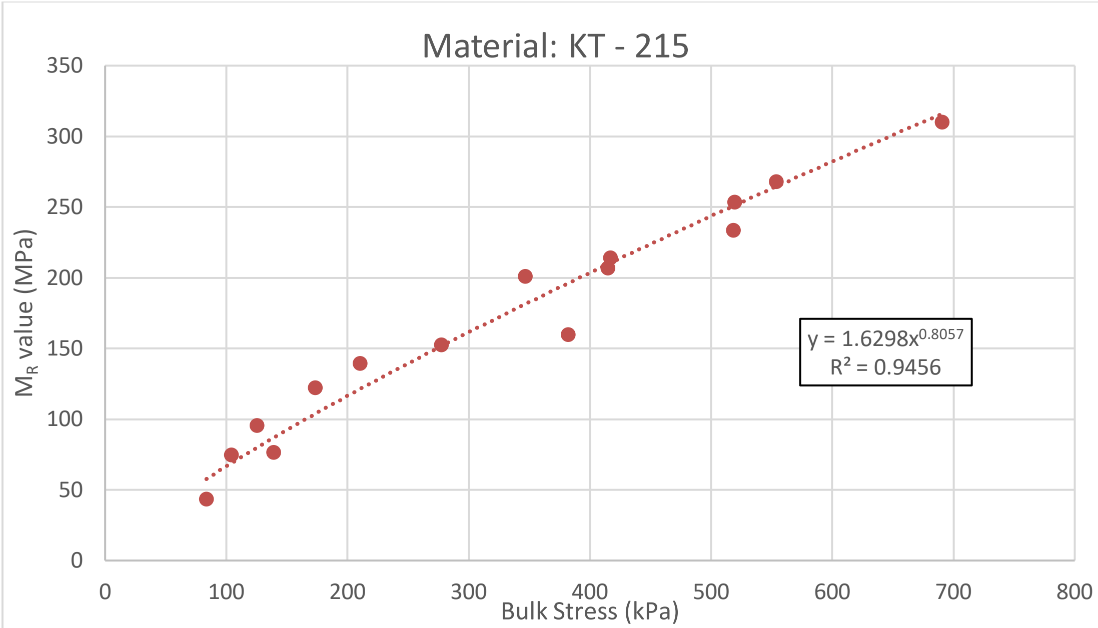
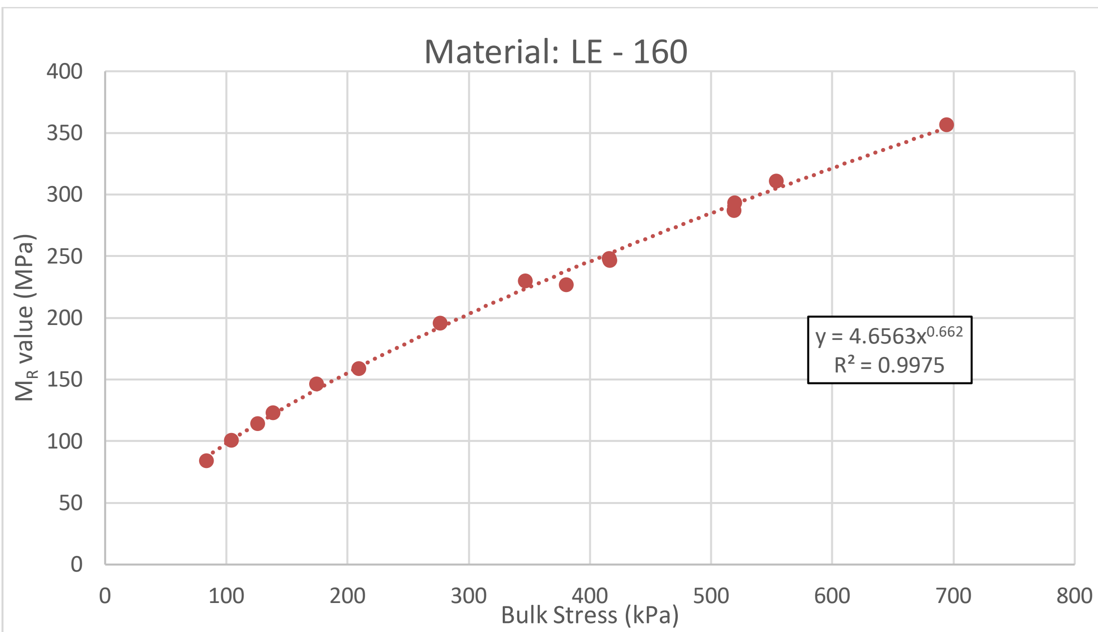
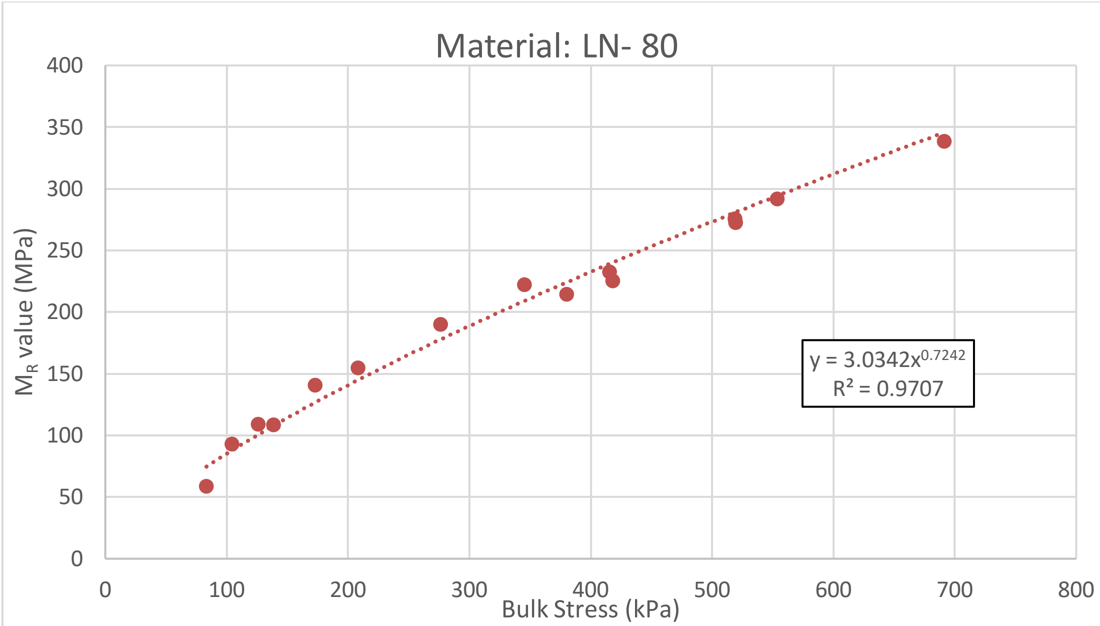
200
150
100
50
0
M_R value (MPa)
0
100
200
300
400
500
600
700
800
Bulk Stress (kPa)
$y = 3.7117x^{0.6517}$
$R^2 = 0.996$

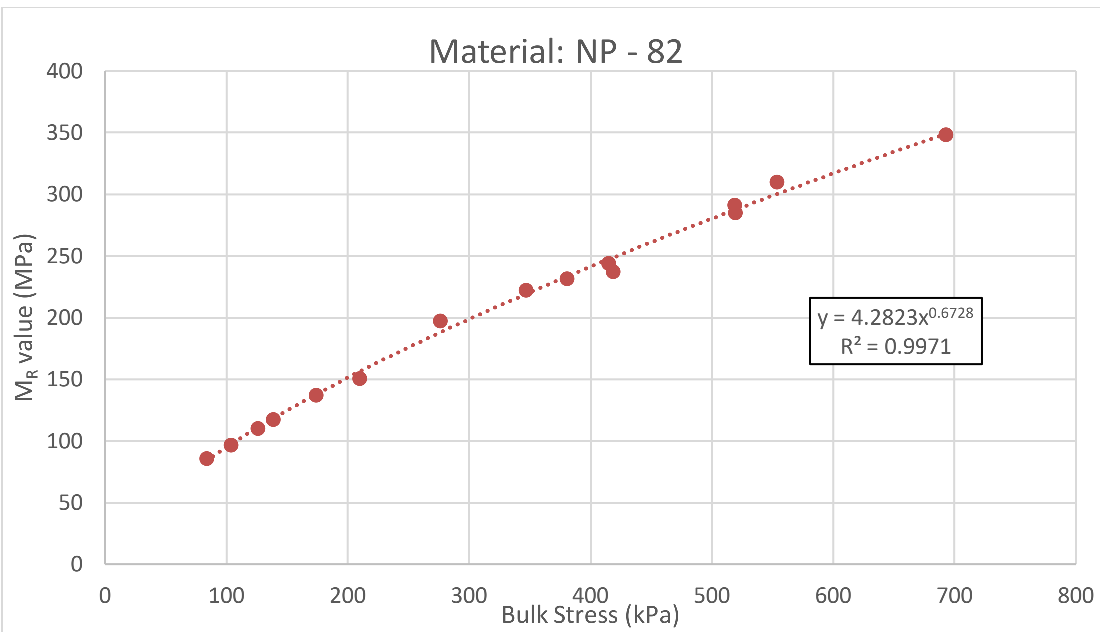

Material: EL - 132
M_R value (MPa)
400
350
300
250
200
150
100
50
0
0
100
200
300
400
500
600
700
800
Bulk Stress (kPa)
y = 2.3189x$^{0.7629}$
R^2 = 0.9862

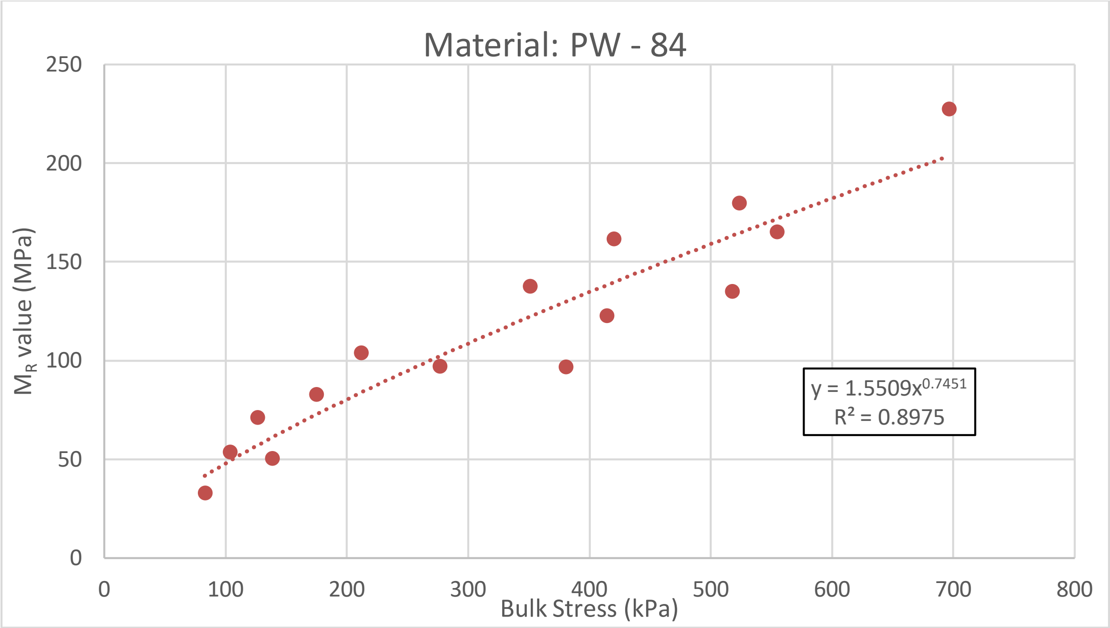

Material: IMC - 140
M_R value (MPa)
350
300
250
200
150
100
50
0
0
100
200
300
400
500
600
700
800
Bulk Stress (kPa)
y = 3.026x$^{0.71}$
R^2 = 0.9909

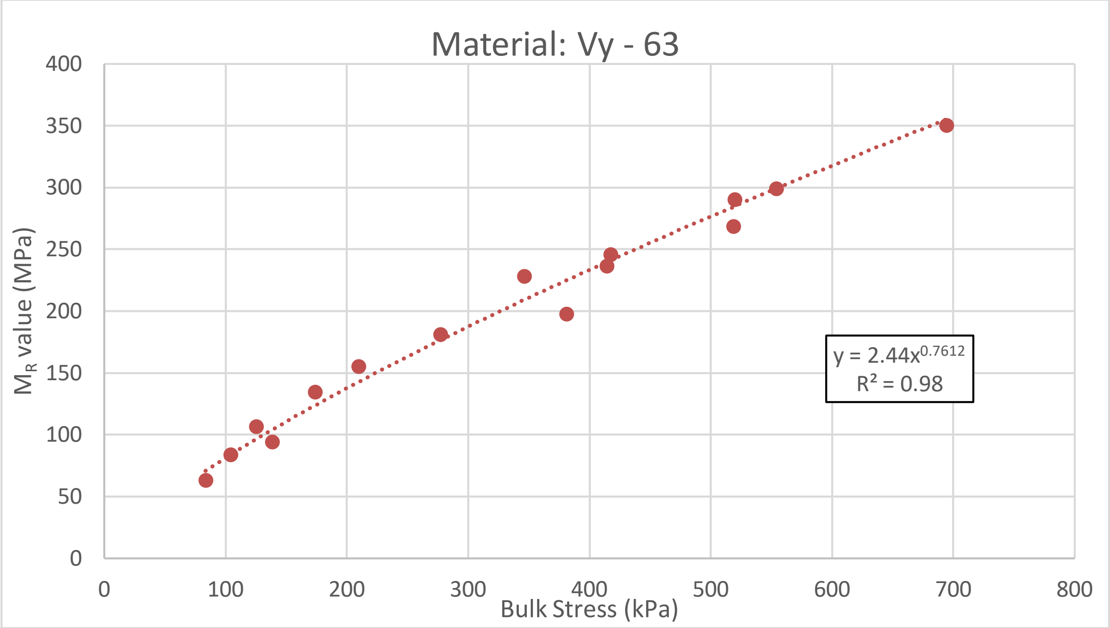

Material: KT - 215
M_R value (MPa)
Bulk Stress (kPa)
$y = 1.6298x^{0.8057}$
$R^2 = 0.9456$
0
50
100
150
200
250
300
350
0
100
200
300
400
500
600
700
800

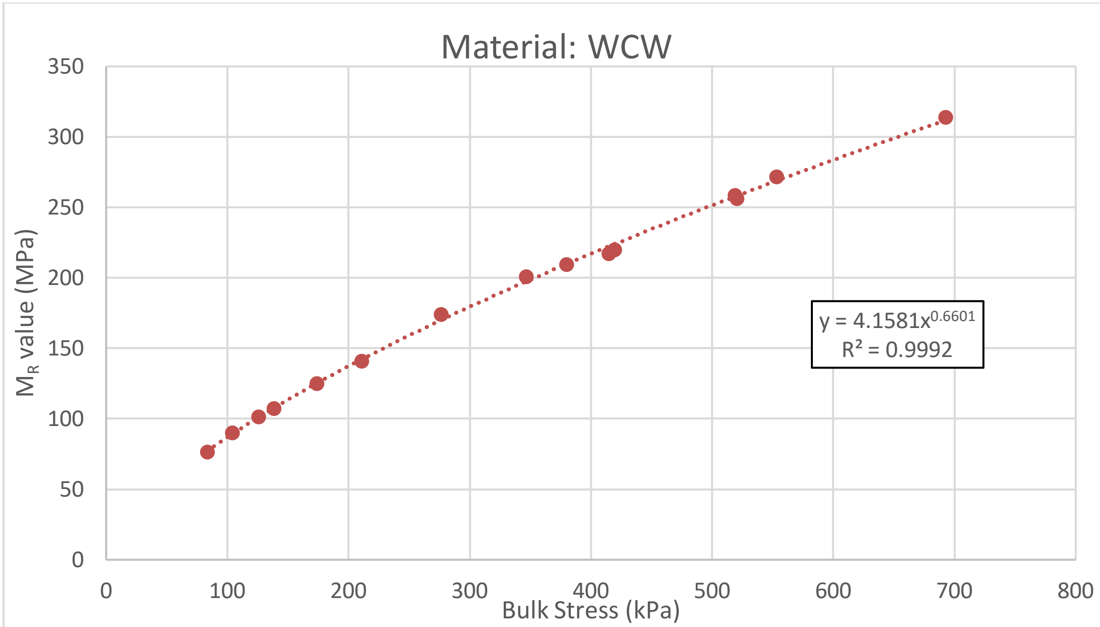

Material: LE - 160
M_R value (MPa)
Bulk Stress (kPa)
$y = 4.6563x^{0.662}$
$R^2 = 0.9975$
0
50
100
150
200
250
300
350
400
0
100
200
300
400
500
600
700
800

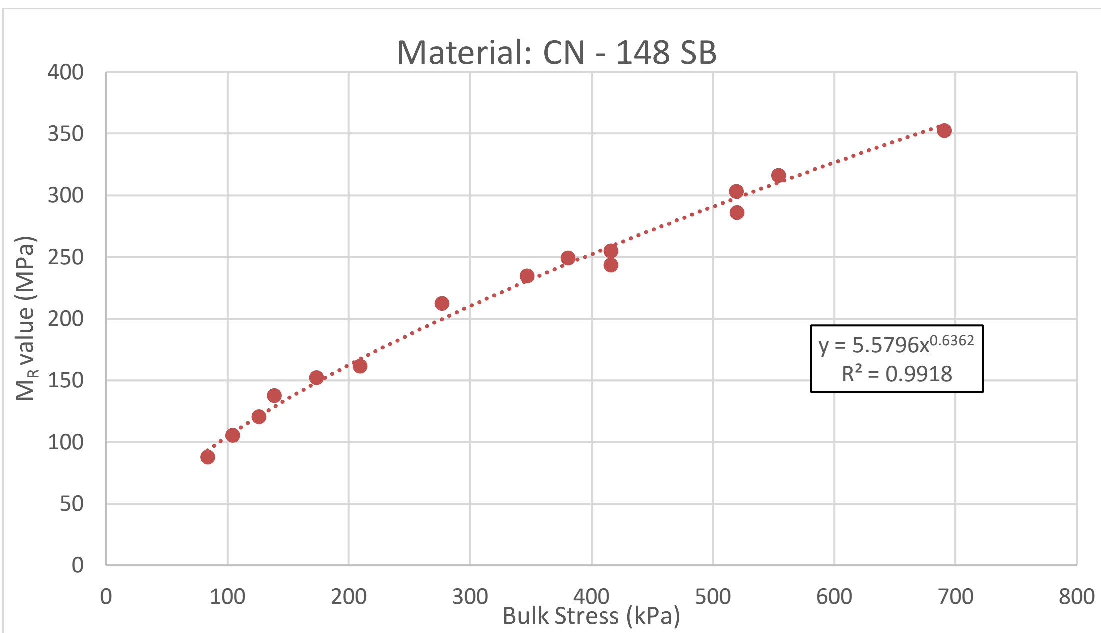

Material: LN- 80
400
350
300
250
200
150
100
50
0
0
100
200
300
400
500
600
700
800
Bulk Stress (kPa)
M_R value (MPa)
y = 3.0342x^0.7242
R² = 0.9707

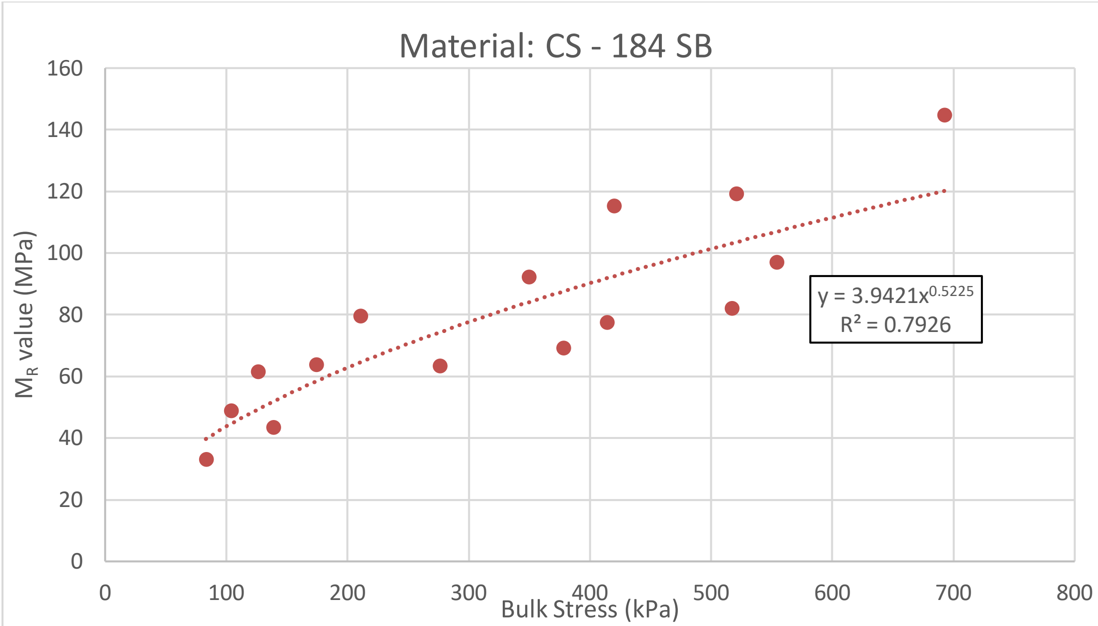

Material: NP - 82
400
350
300
250
200
150
100
50
0
0
100
200
300
400
500
600
700
800
Bulk Stress (kPa)
M_R value (MPa)
y = 4.2823x^0.6728
R² = 0.9971

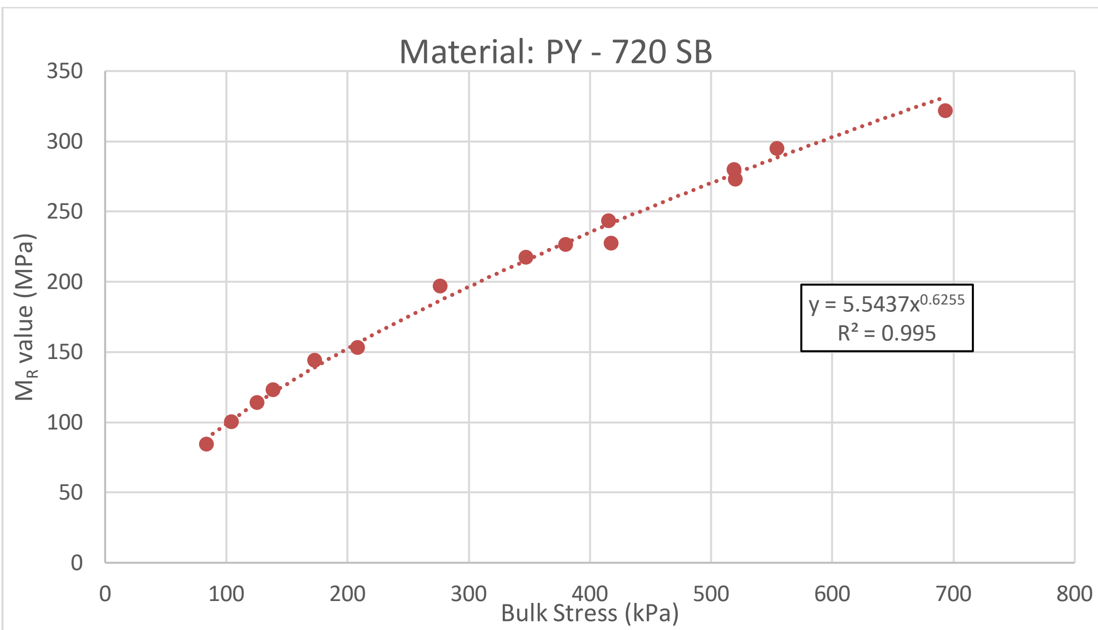

Material: PW - 84
M_R value (MPa)
Bulk Stress (kPa)
y = 1.5509x^0.7451
R² = 0.8975
0
50
100
150
200
250
0
100
200
300
400
500
600
700
800


Material: Vy - 63
M_R value (MPa)
Bulk Stress (kPa)
y = 2.44x^0.7612
R² = 0.98
0
50
100
150
200
250
300
350
400
0
100
200
300
400
500
600
700
800

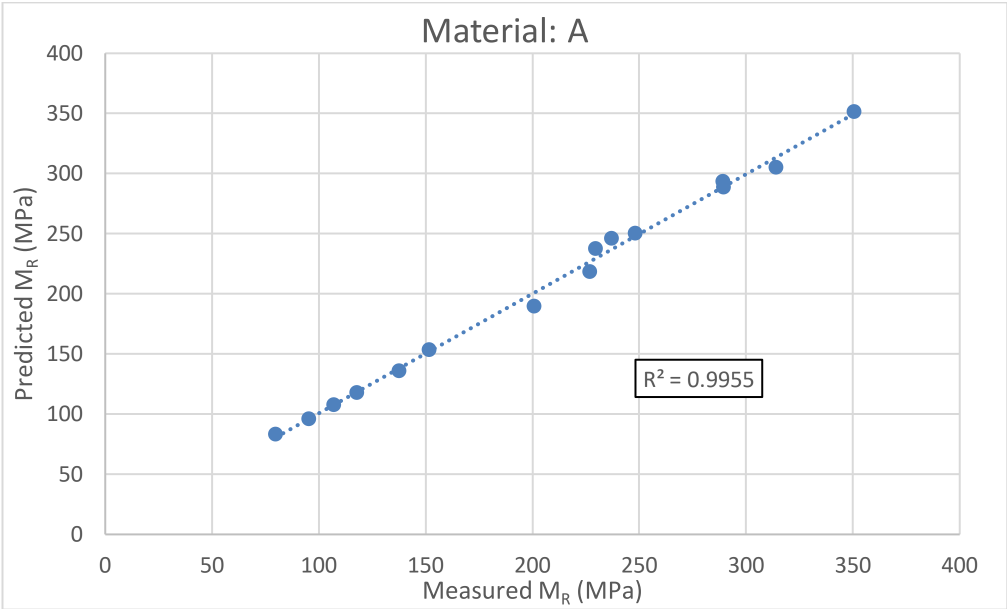

Material: WCW
M_R value (MPa)
Bulk Stress (kPa)
y = 4.1581x^0.6601
R² = 0.9992
350
300
250
200
150
100
50
0
0
100
200
300
400
500
600
700
800

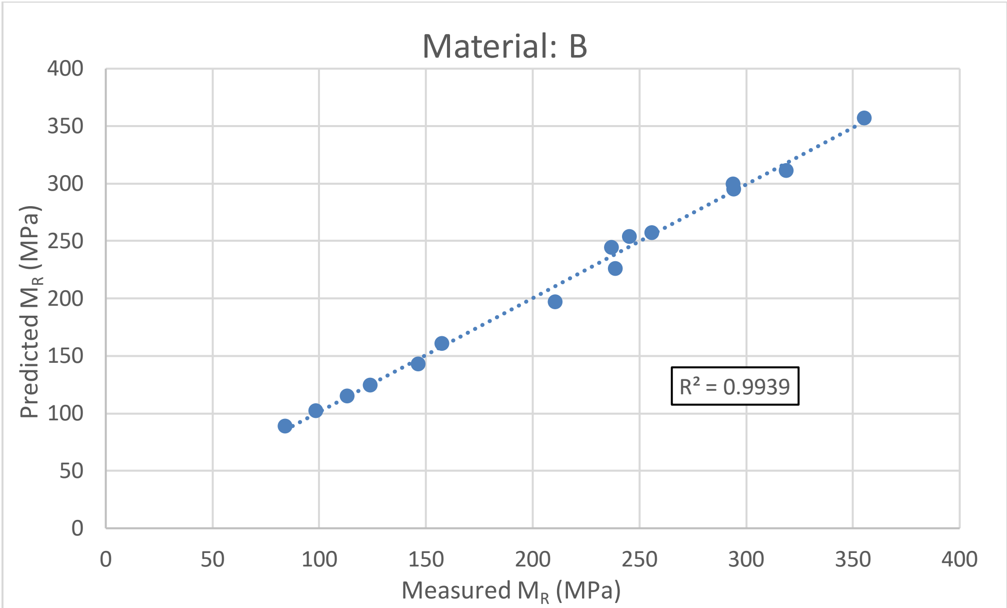

Material: CN - 148 SB
M_R value (MPa)
Bulk Stress (kPa)
y = 5.5796x^0.6362
R² = 0.9918
400
350
300
250
200
150
100
50
0
0
100
200
300
400
500
600
700
800

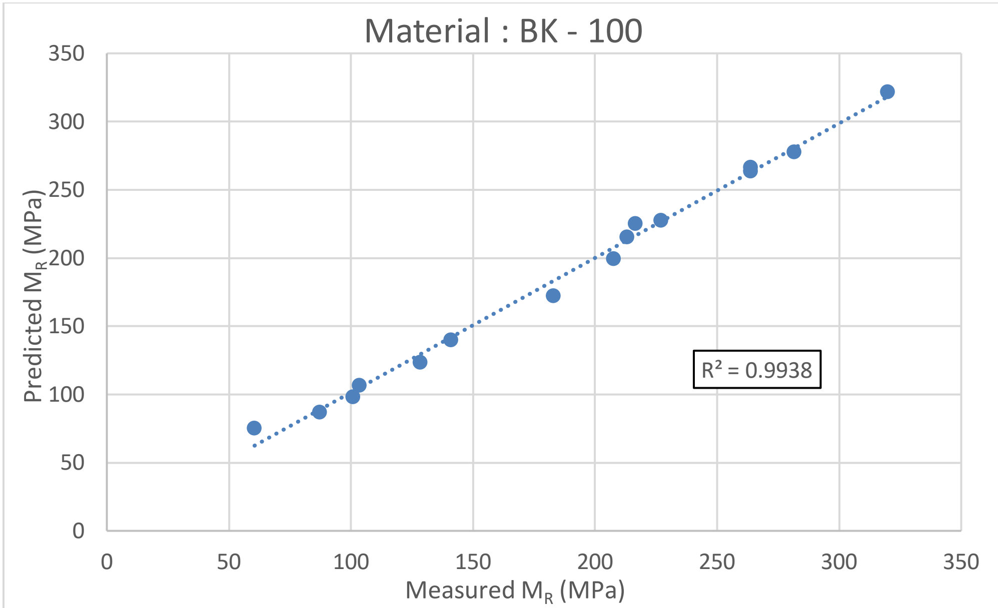

Material: CS - 184 SB
160
140
120
100
80
60
40
20
0
M_R value (MPa)
0
100
200
300
400
500
600
700
800
Bulk Stress (kPa)
$y = 3.9421x^{0.5225}$
$R^2 = 0.7926$

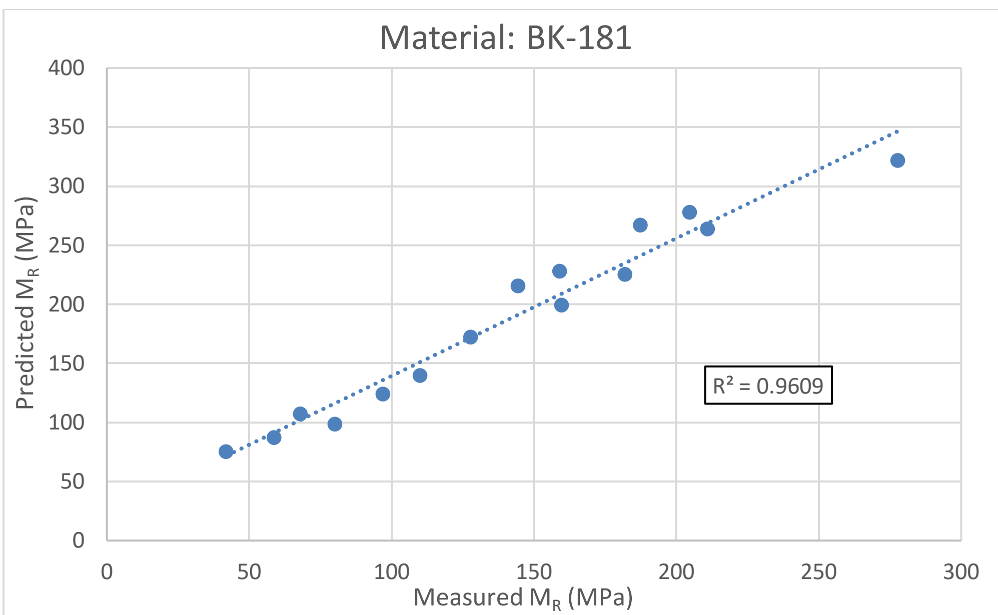

Material: PY - 720 SB
350
300
250
200
150
100
50
0
M_R value (MPa)
0
100
200
300
400
500
600
700
800
Bulk Stress (kPa)
$y = 5.5437x^{0.6255}$
$R^2 = 0.995$

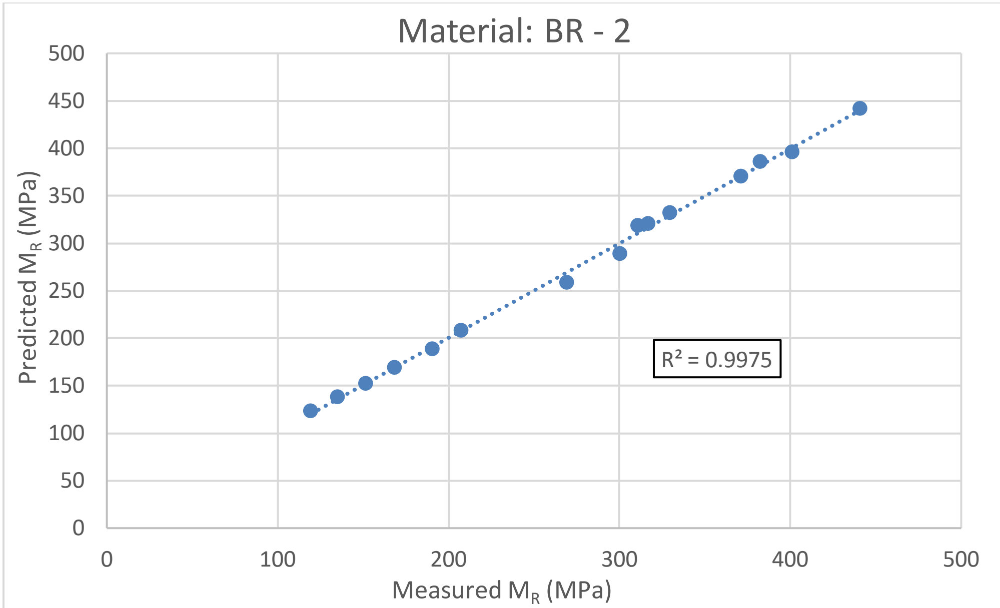
Uzan Model Predicted M_R and Laboratory Measured M_R Plots

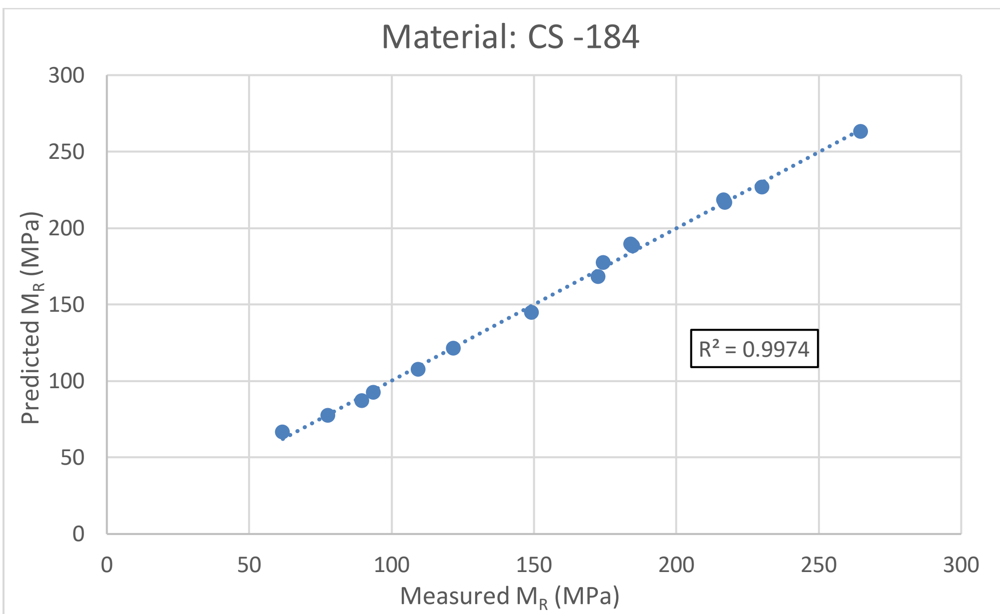


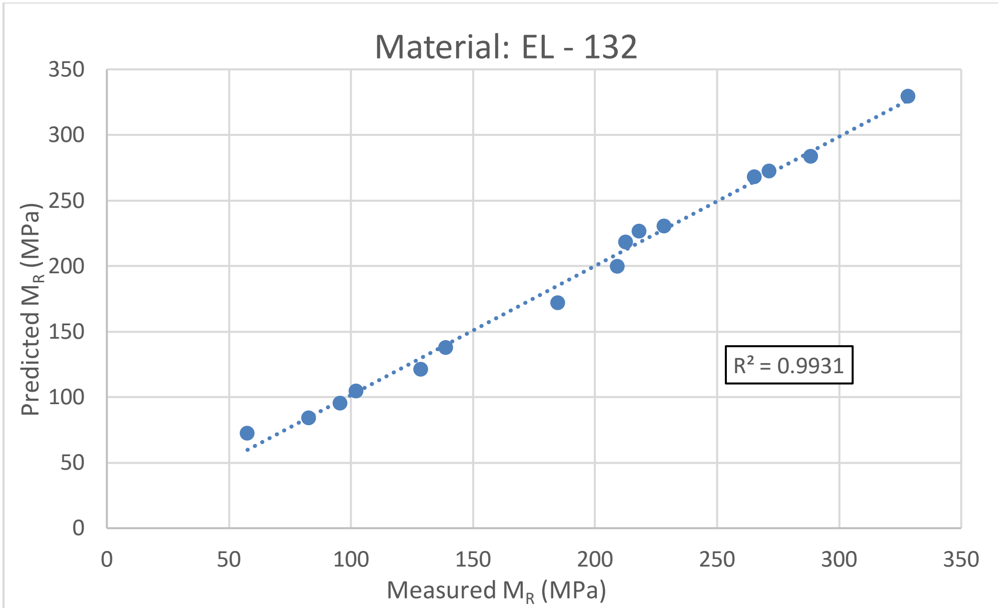


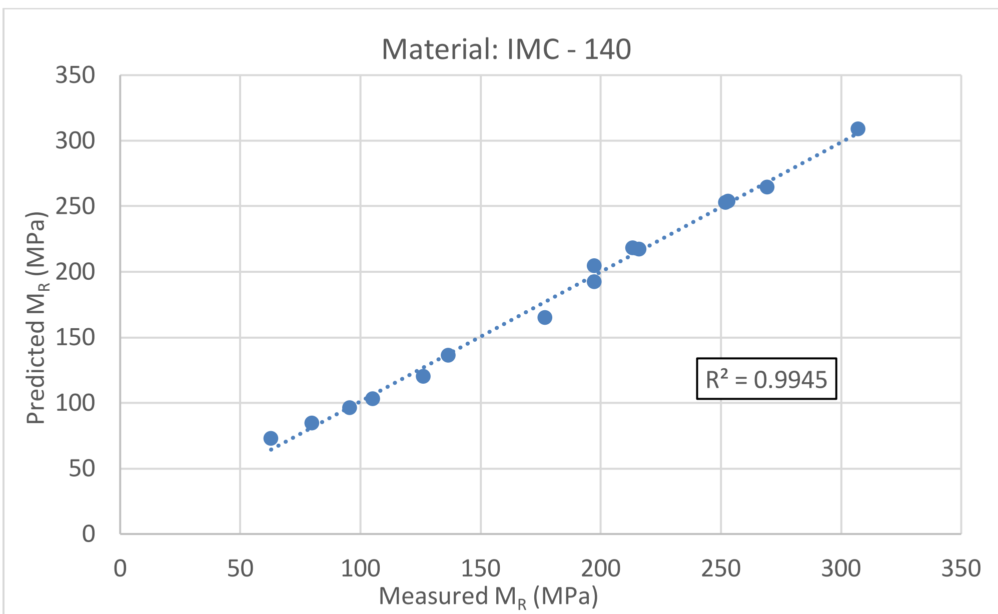

Material : BK - 100
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9938$
0
50
100
150
200
250
300
350

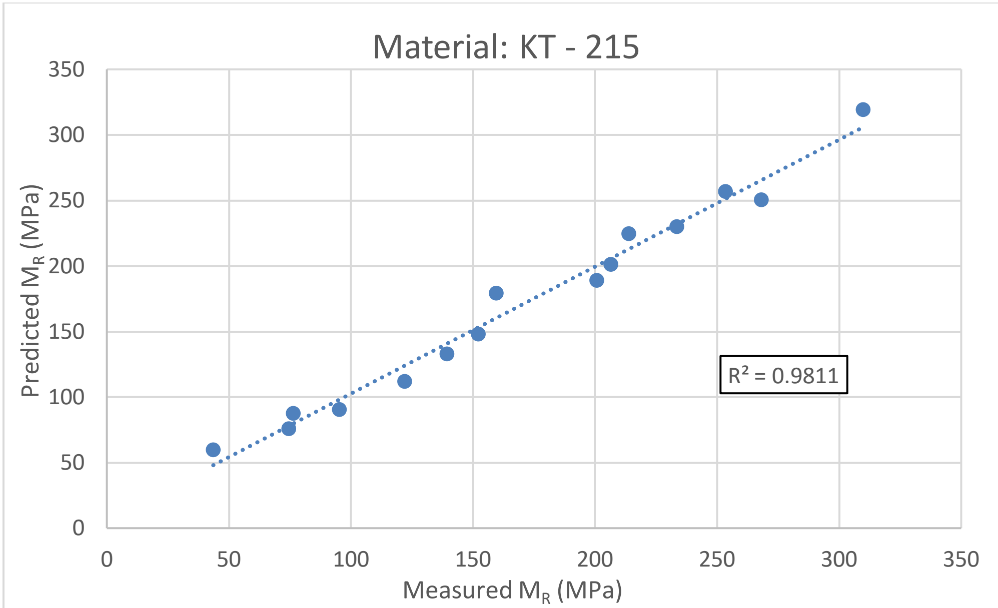

Material: BK-181
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9609$
0
50
100
150
200
250
300
350
400

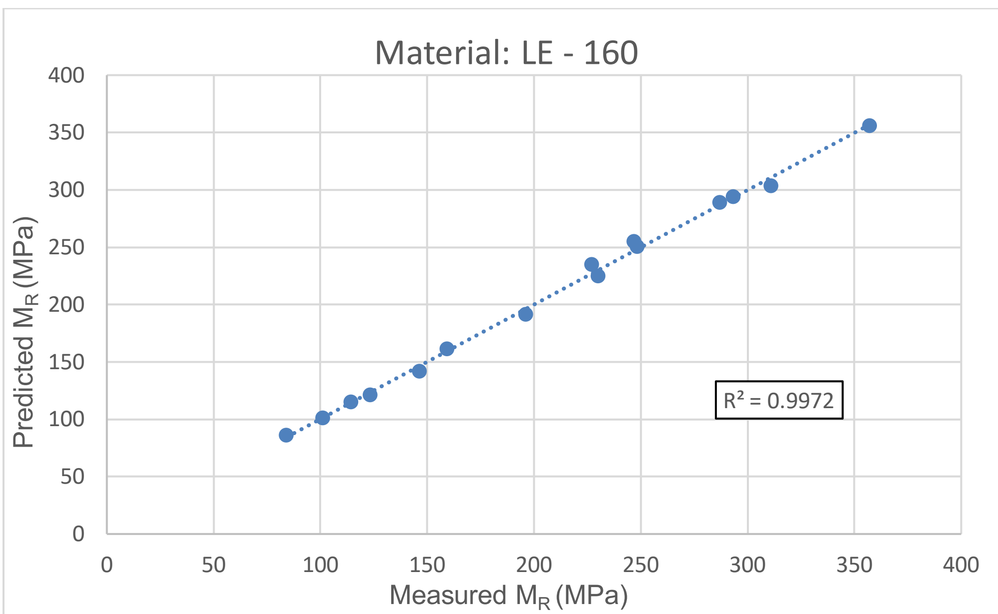

Material: BR - 2
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9975$
0
50
100
150
200
250
300
350
400
450
500
0
100
200
300
400
500

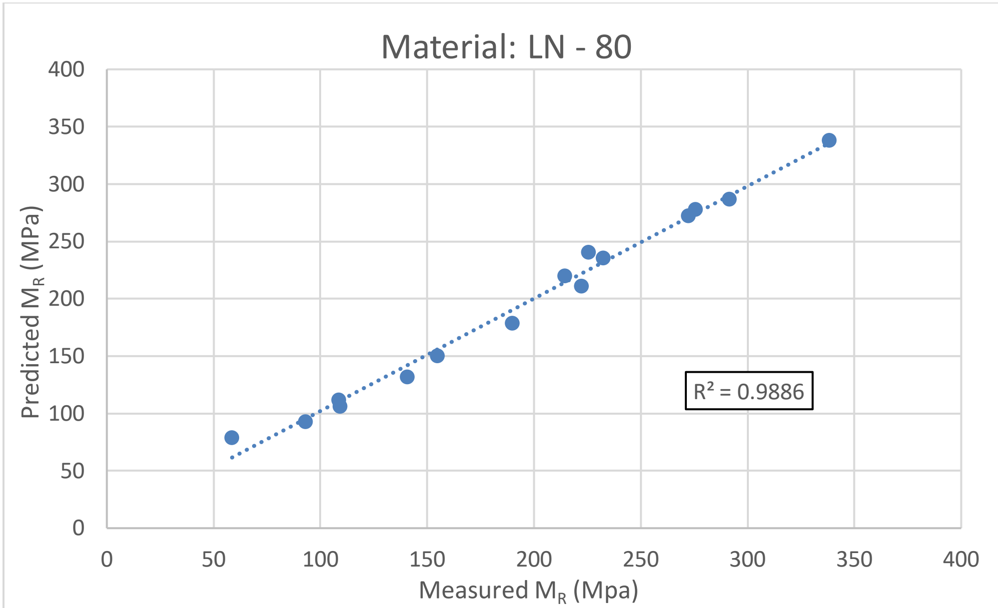

Material: CS -184
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9974$
0
50
100
150
200
250
300
0
50
100
150
200
250
300

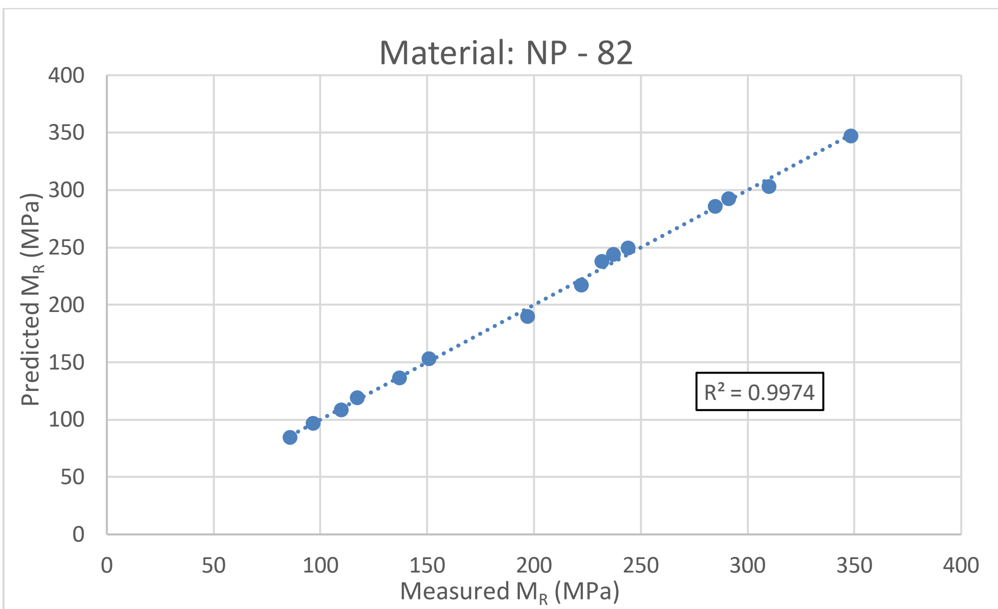

Material: EL - 132
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9931$
0
50
100
150
200
250
300
350

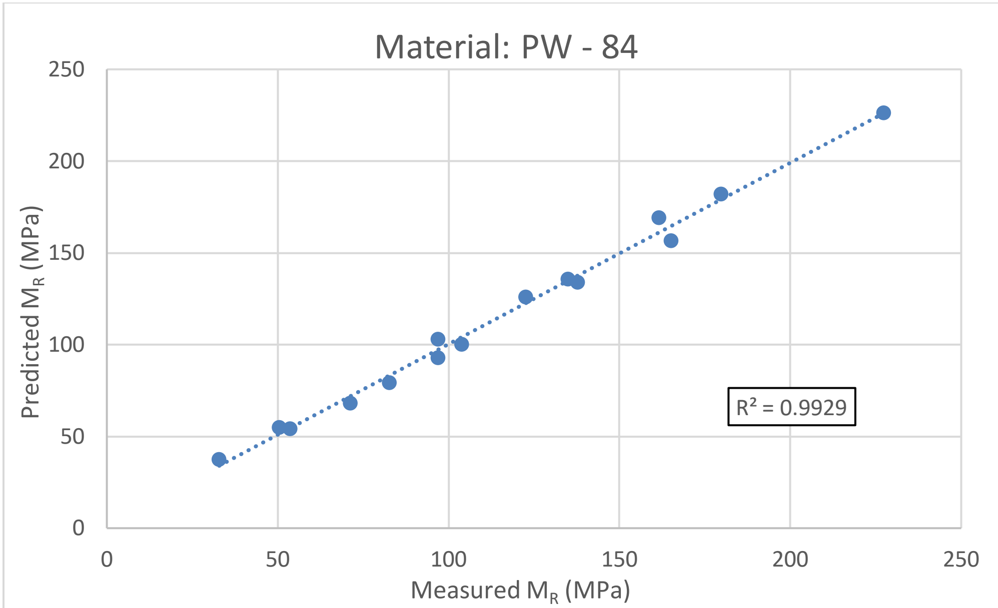

Material: IMC - 140
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9945$
0
50
100
150
200
250
300
350

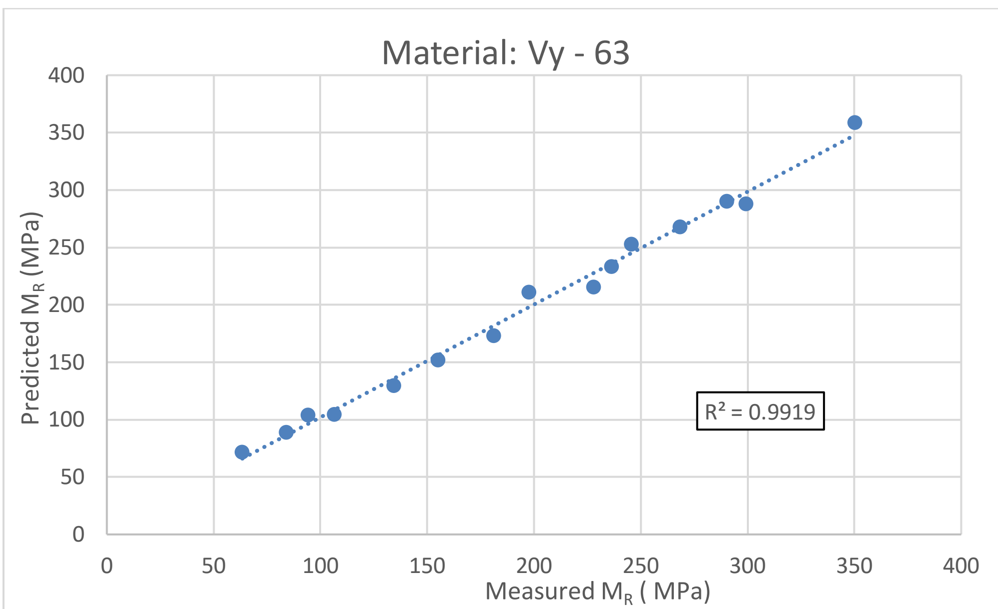

Material: KT - 215
Predicted M_R (MPa)
Measured M_R (MPa)
R² = 0.9811
0
50
100
150
200
250
300
350

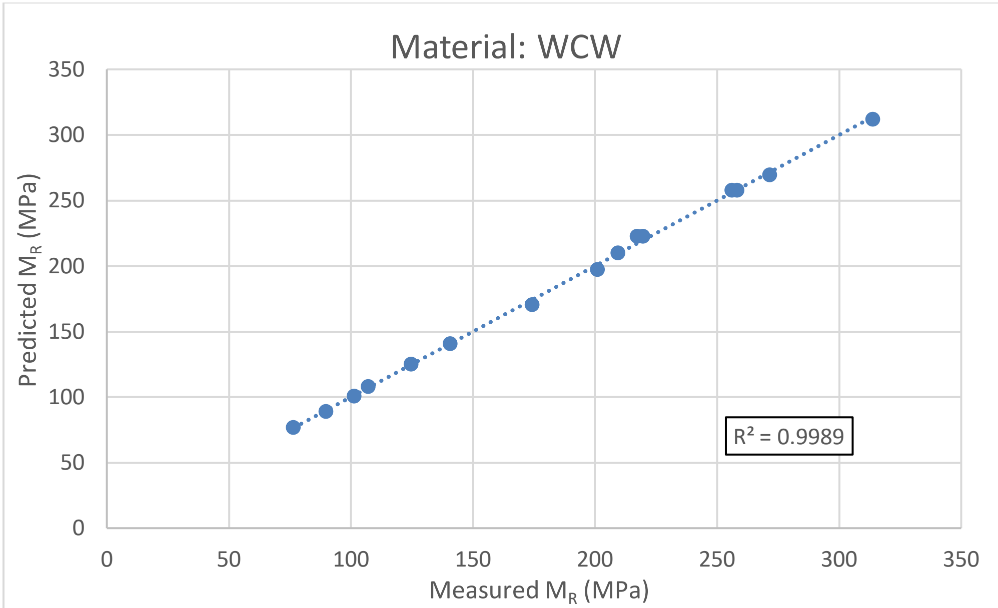

Material: LE - 160
Predicted M_R (MPa)
Measured M_R (MPa)
R² = 0.9972
0
50
100
150
200
250
300
350
400

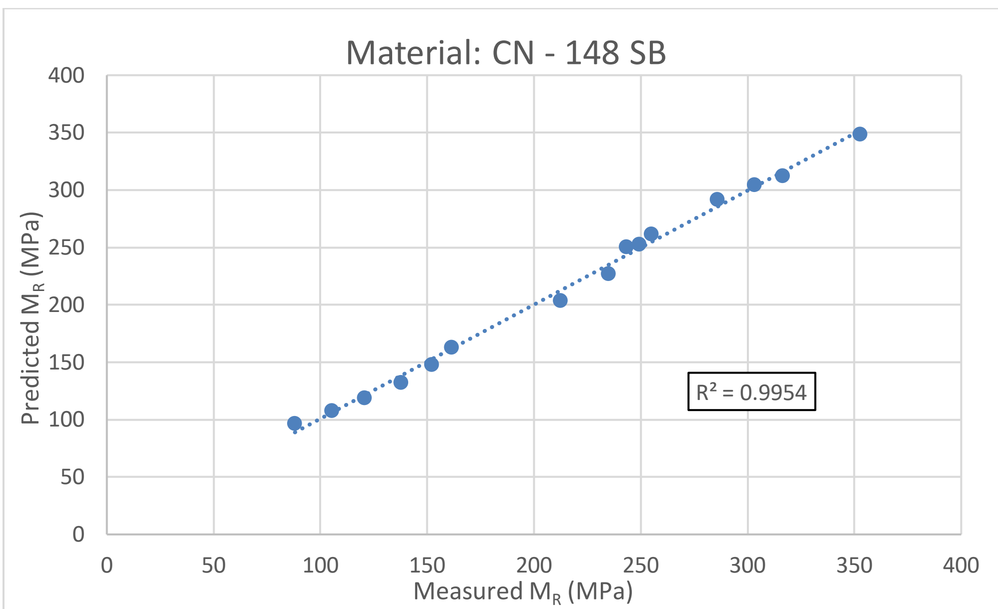

Material: LN - 80
Predicted M_R (MPa)
Measured M_R (Mpa)
R^2 = 0.9886
0
50
100
150
200
250
300
350
400

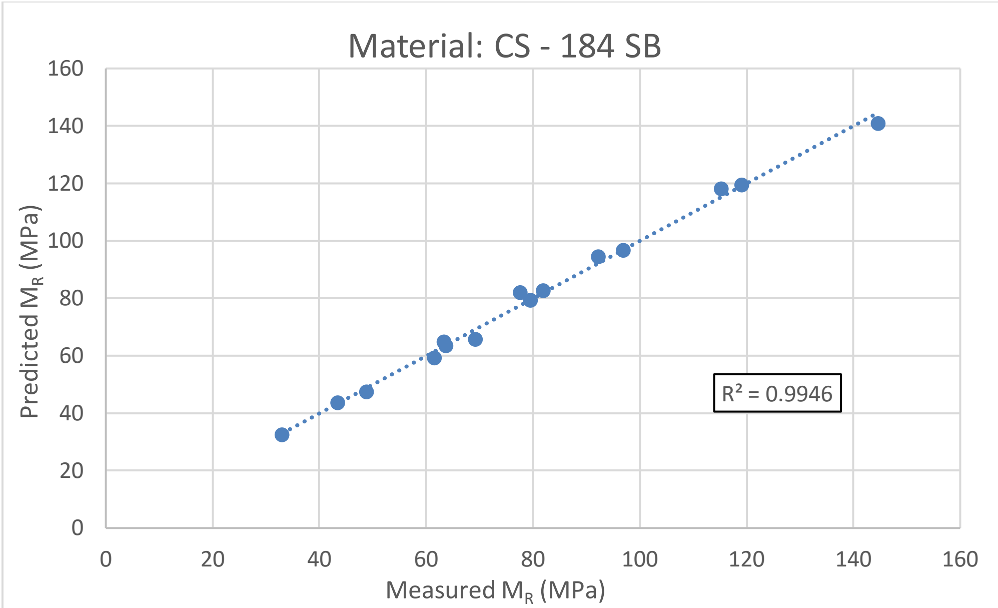

Material: NP - 82
Predicted M_R (MPa)
Measured M_R (MPa)
R^2 = 0.9974
0
50
100
150
200
250
300
350
400

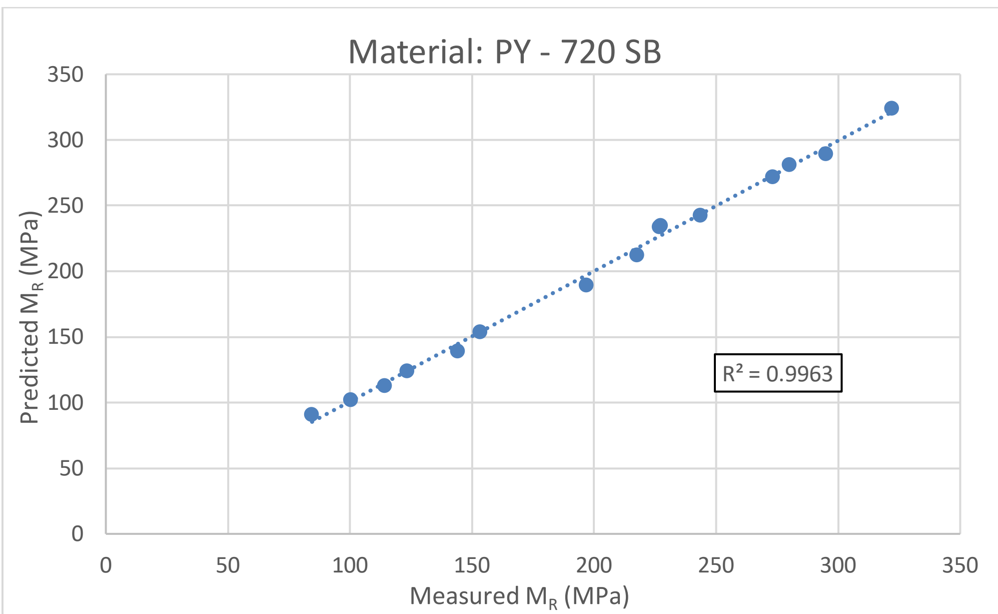

Material: PW - 84
Predicted M_R (MPa)
Measured M_R (MPa)
R^2 = 0.9929
0
50
100
150
200
250


Material: Vy - 63
Predicted M_R (MPa)
Measured M_R (MPa)
R^2 = 0.9919
0
50
100
150
200
250
300
350
400

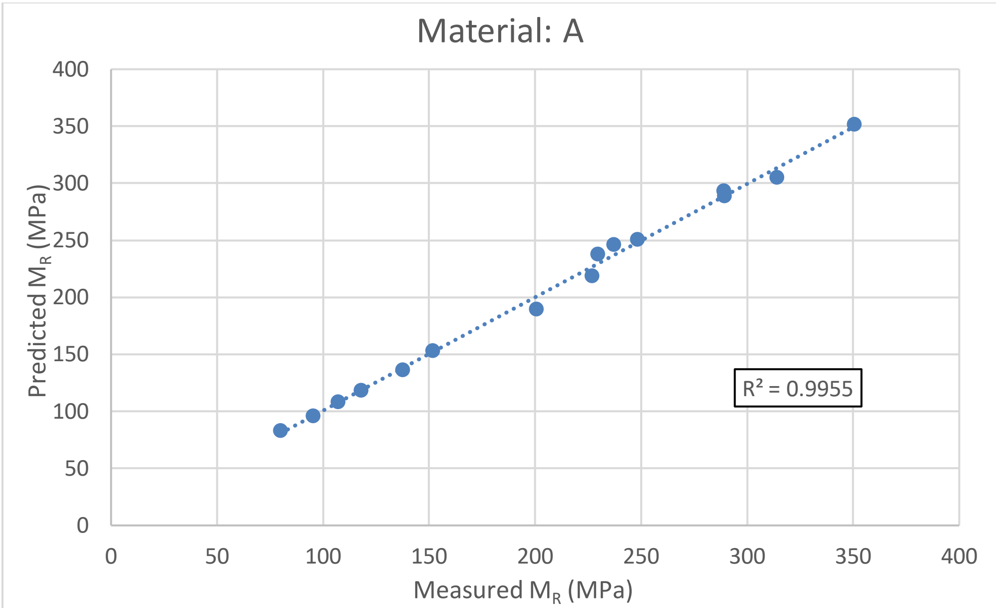

Material: WCW
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9989$
0
50
100
150
200
250
300
350

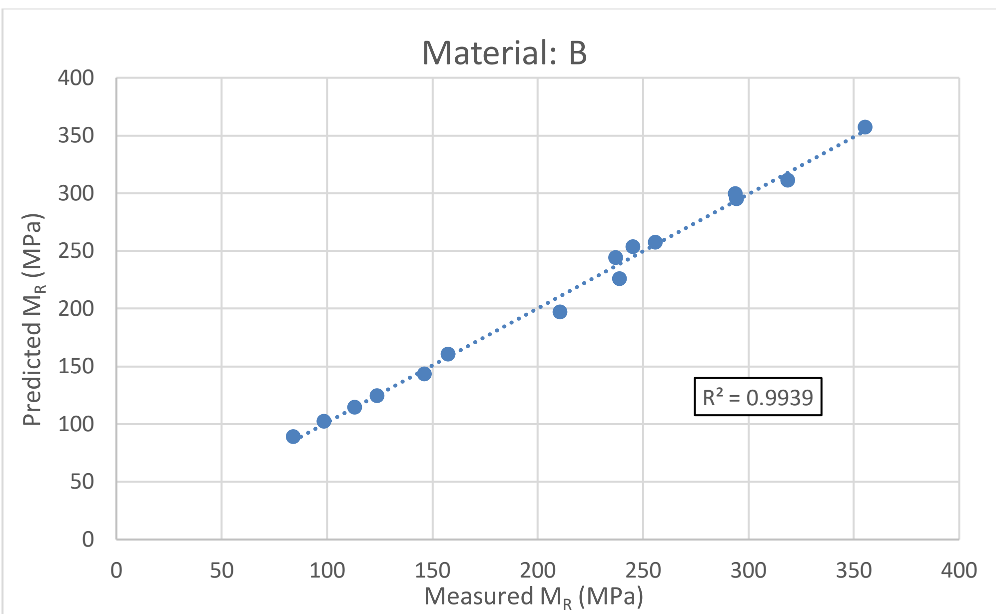

Material: CN - 148 SB
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9954$
0
50
100
150
200
250
300
350
400

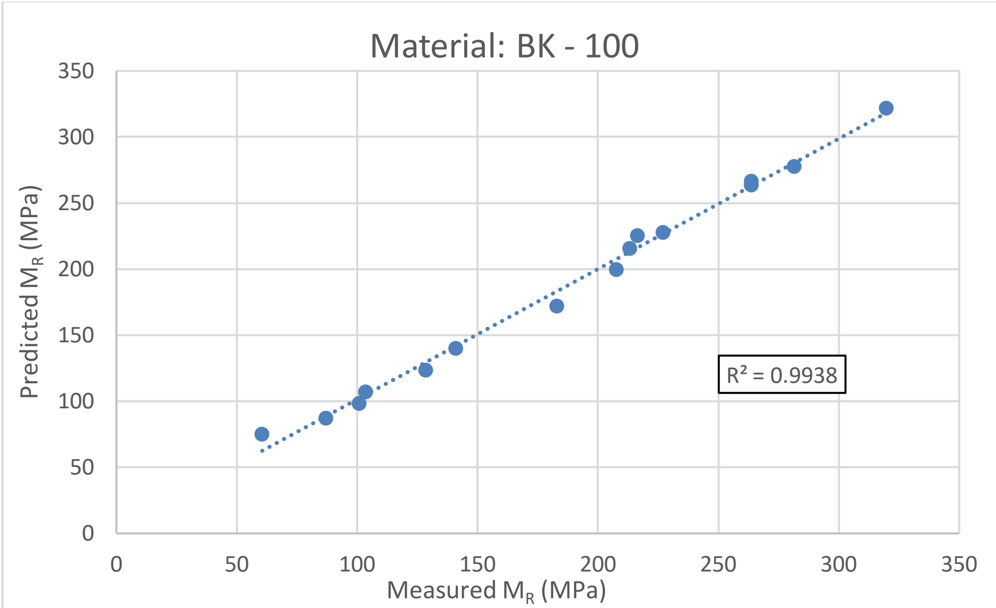

Material: CS - 184 SB
Predicted M_R (MPa)
Measured M_R (MPa)
R² = 0.9946
0
20
40
60
80
100
120
140
160

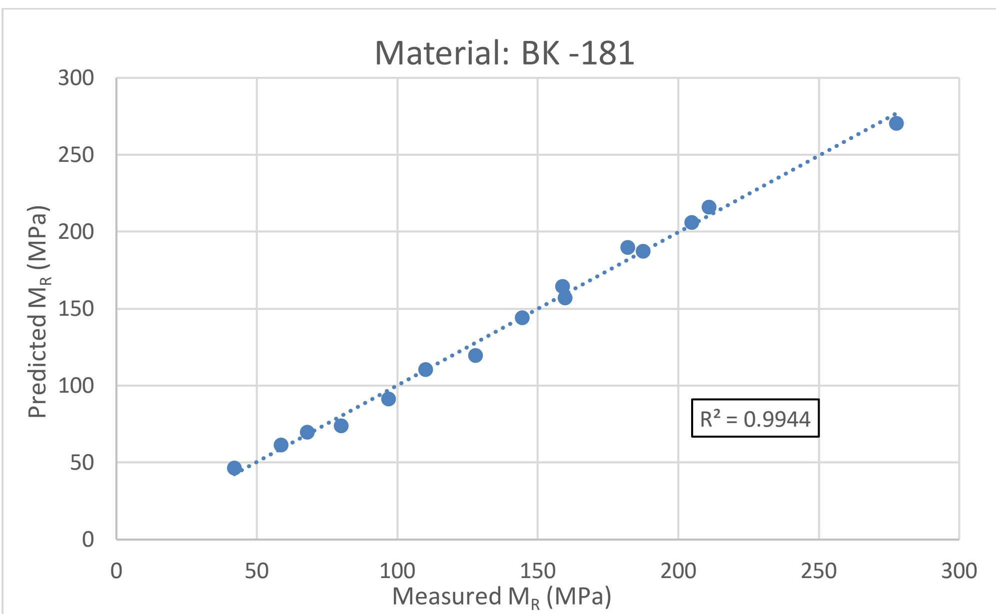

Material: PY - 720 SB
Predicted M_R (MPa)
Measured M_R (MPa)
R² = 0.9963
0
50
100
150
200
250
300
350

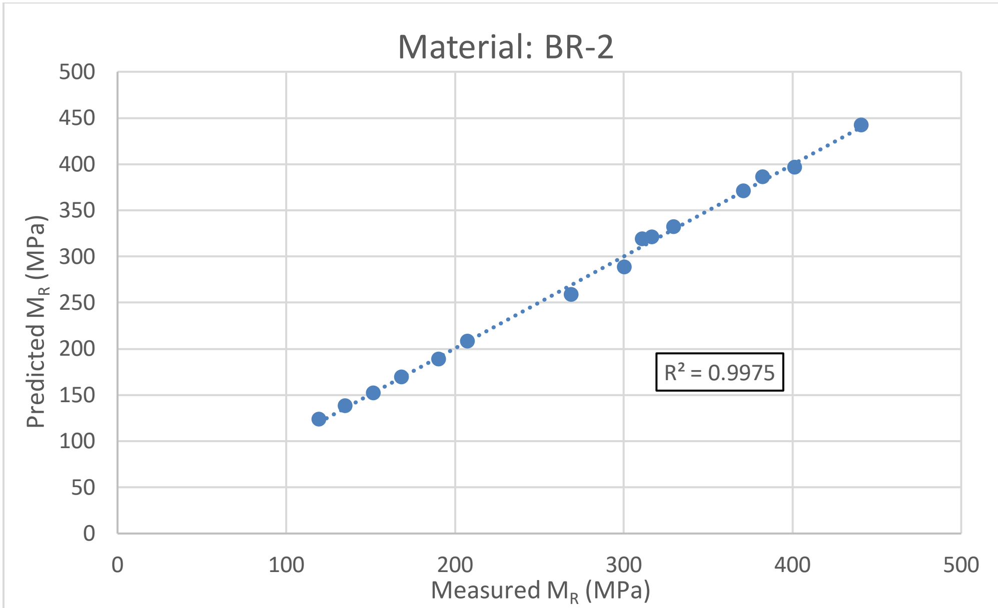
Modified Uzan Model Predicted M_R and Laboratory Measured M_R Plots

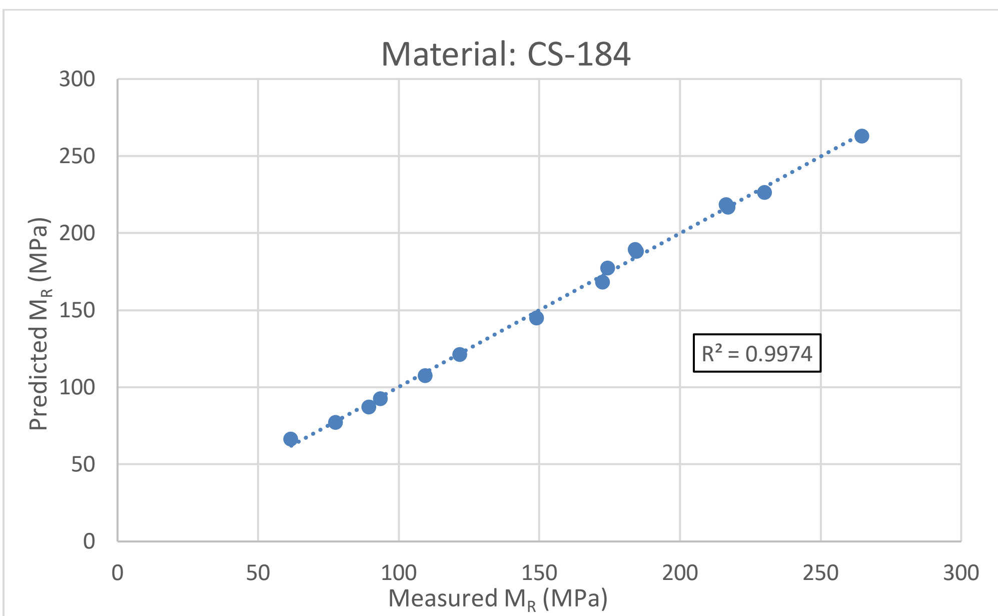


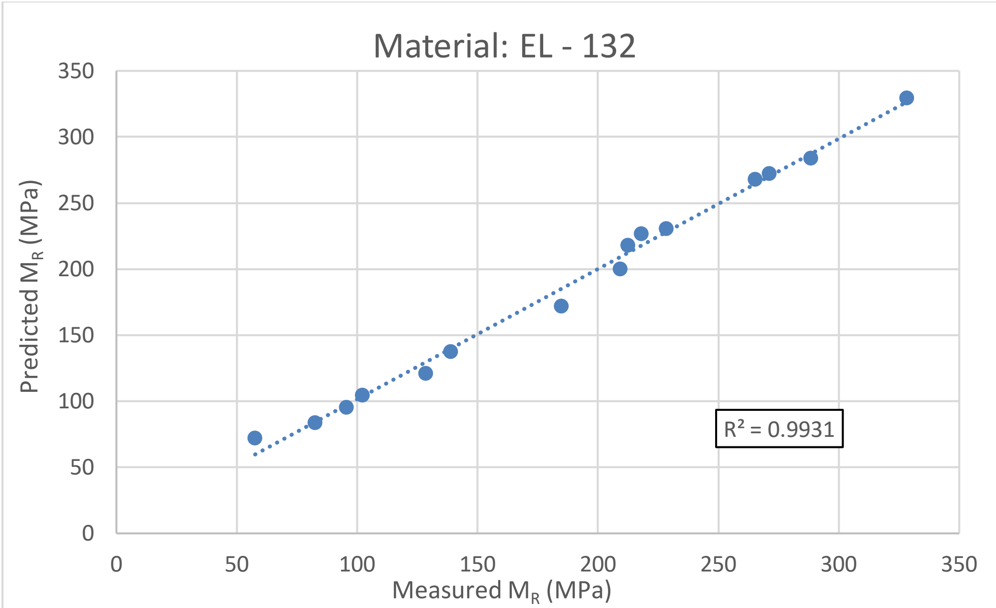


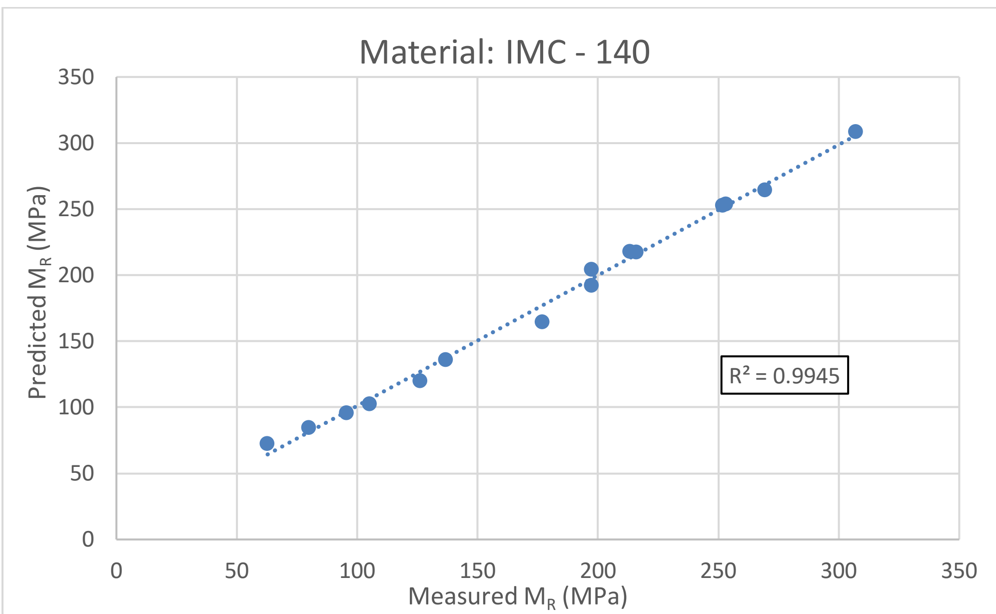

Material: BK - 100
Predicted M_R (MPa)
Measured M_R (MPa)
R² = 0.9938
0
50
100
150
200
250
300
350

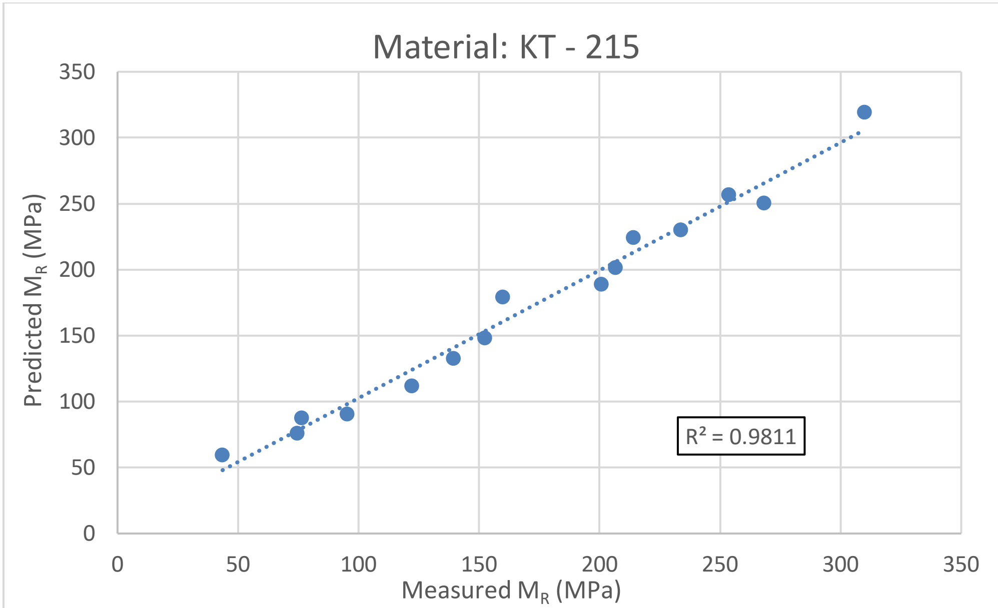

Material: BK -181
Predicted M_R (MPa)
Measured M_R (MPa)
R² = 0.9944
0
50
100
150
200
250
300

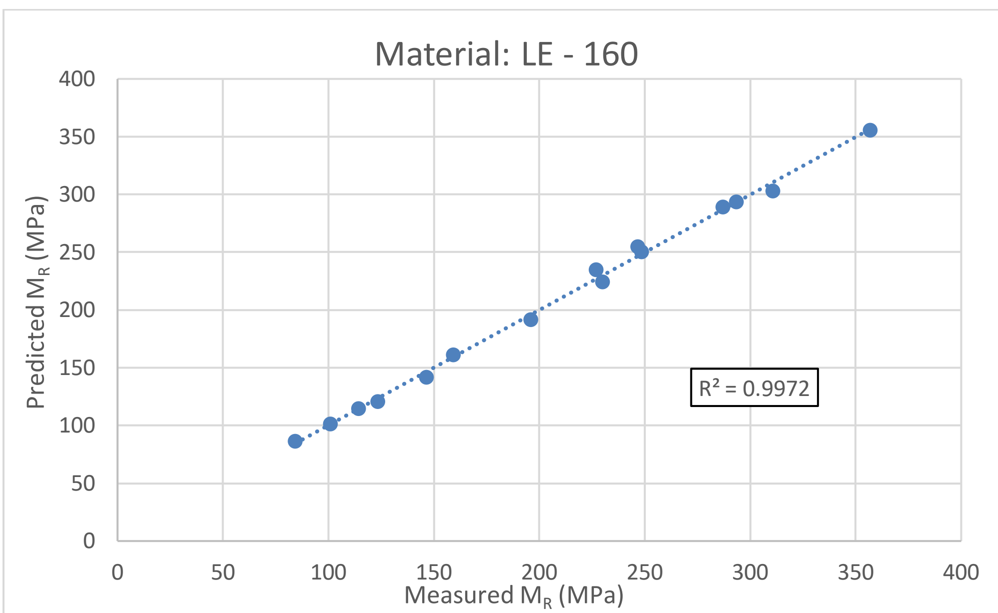

Material: BR-2
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9975$
0
50
100
150
200
250
300
350
400
450
500

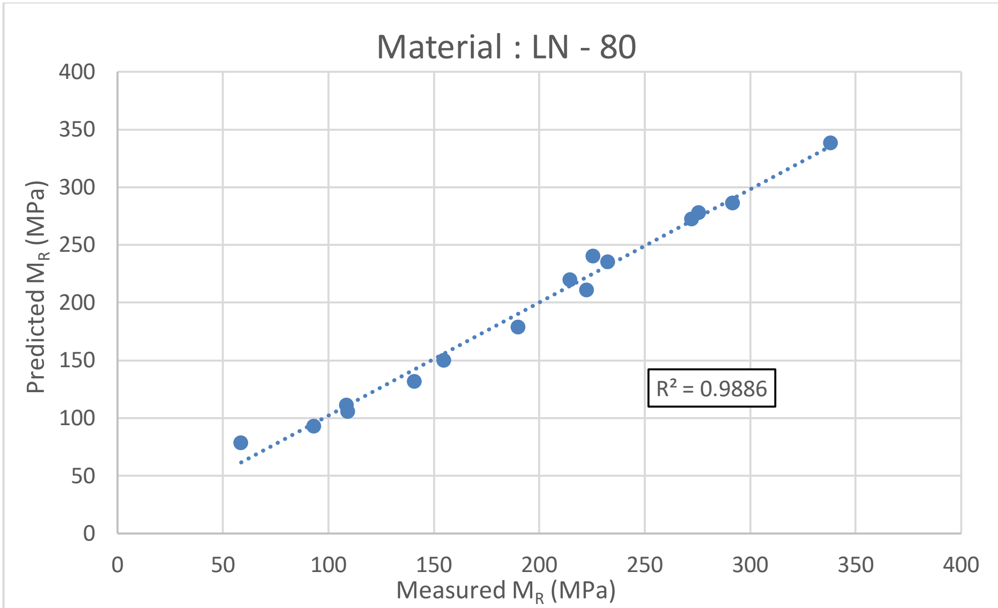

Material: CS-184
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9974$
0
50
100
150
200
250
300

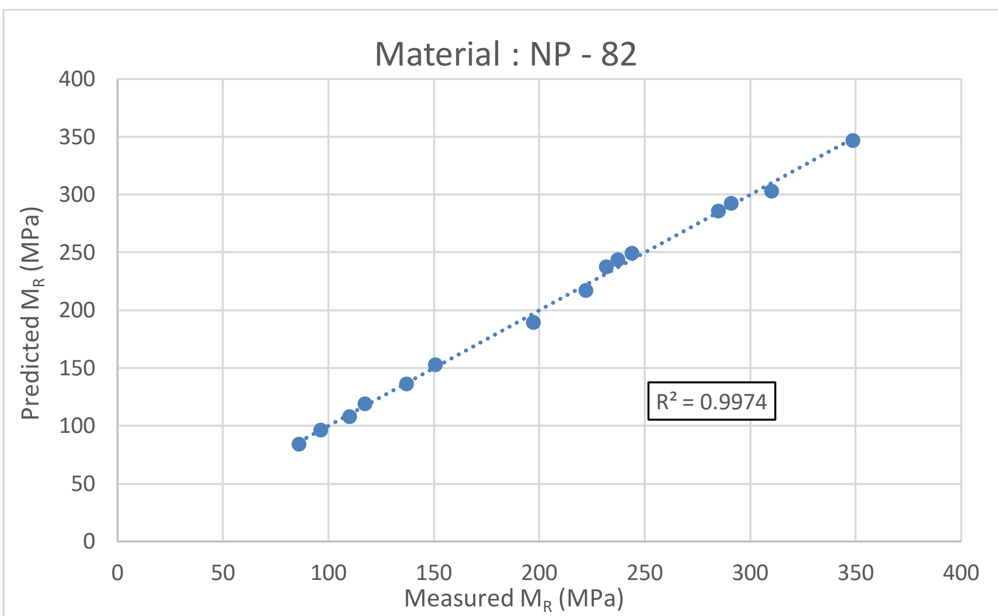

Material: EL - 132
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9931$
0
50
100
150
200
250
300
350

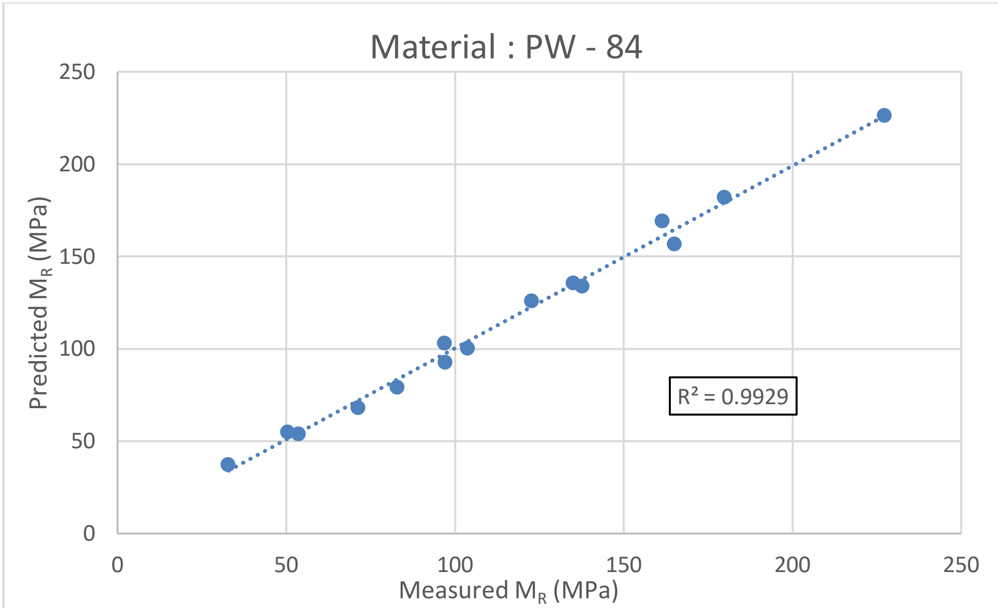

Material: IMC - 140
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9945$
0
50
100
150
200
250
300
350

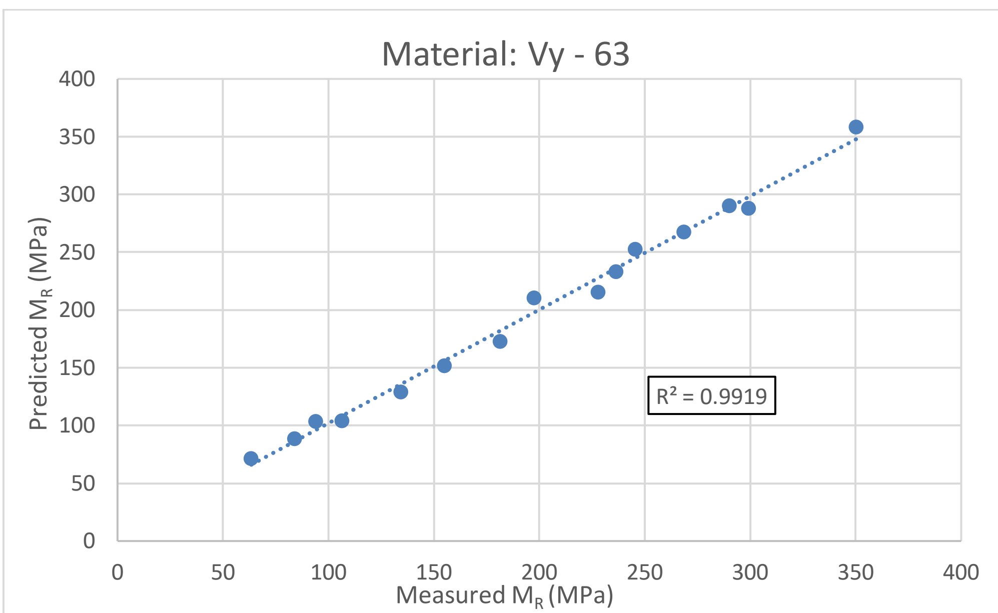

Material: KT - 215
Predicted M_R (MPa)
Measured M_R (MPa)
R² = 0.9811
0
50
100
150
200
250
300
350

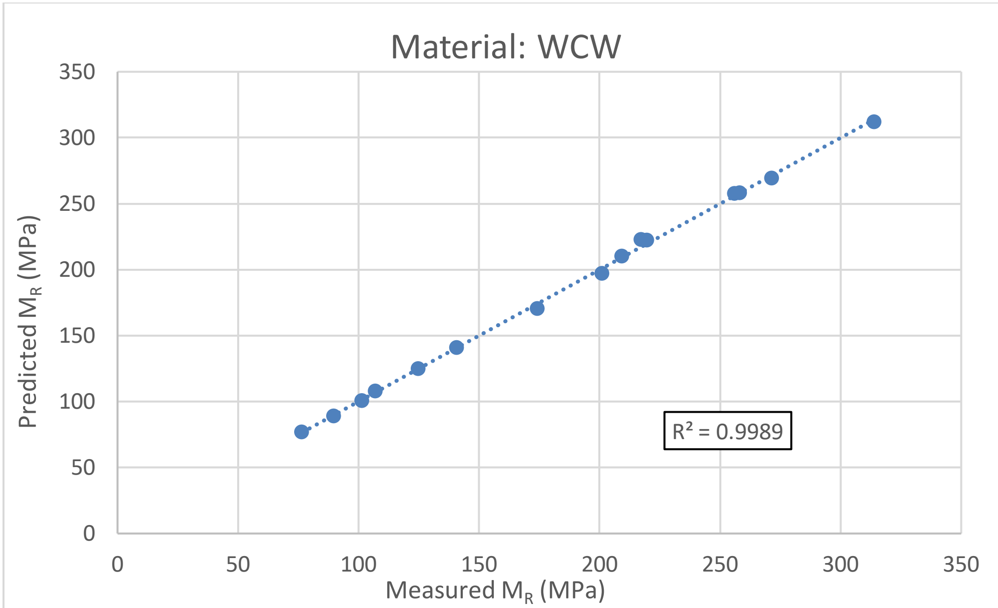

Material: LE - 160
Predicted M_R (MPa)
Measured M_R (MPa)
R² = 0.9972
0
50
100
150
200
250
300
350
400

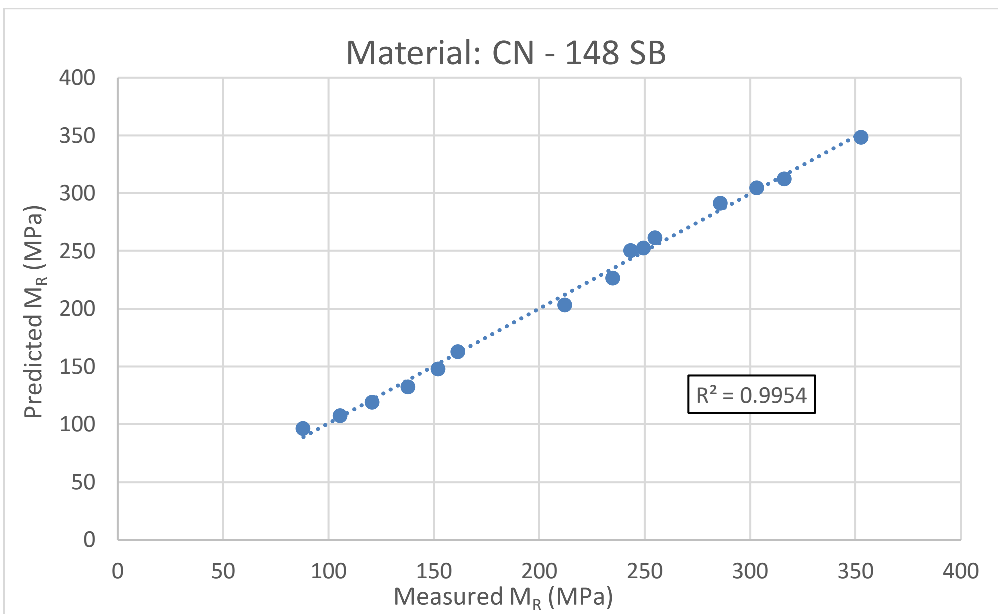

Material : LN - 80
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9886$
0
50
100
150
200
250
300
350
400

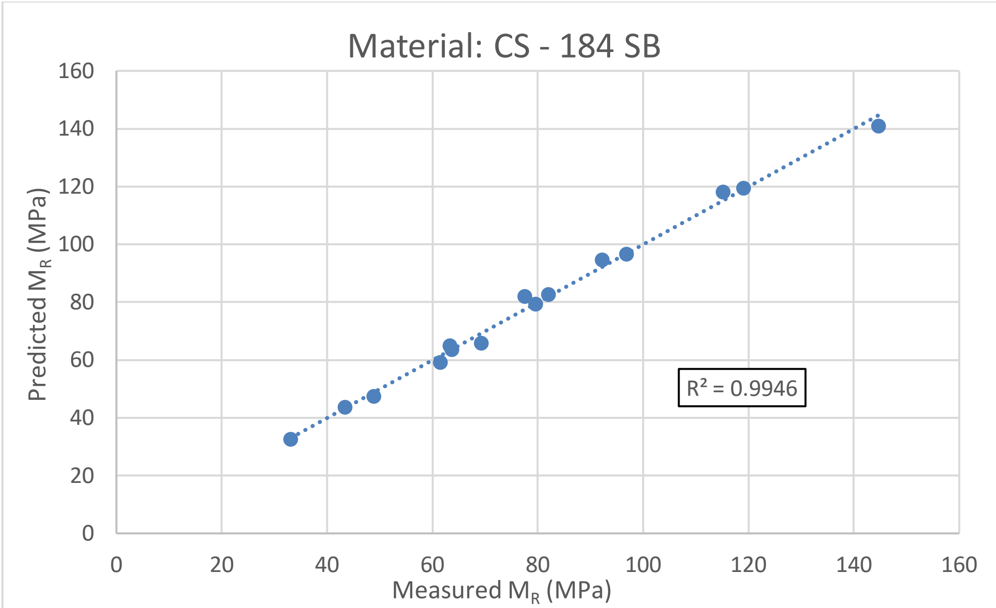

Material : NP - 82
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9974$
0
50
100
150
200
250
300
350
400

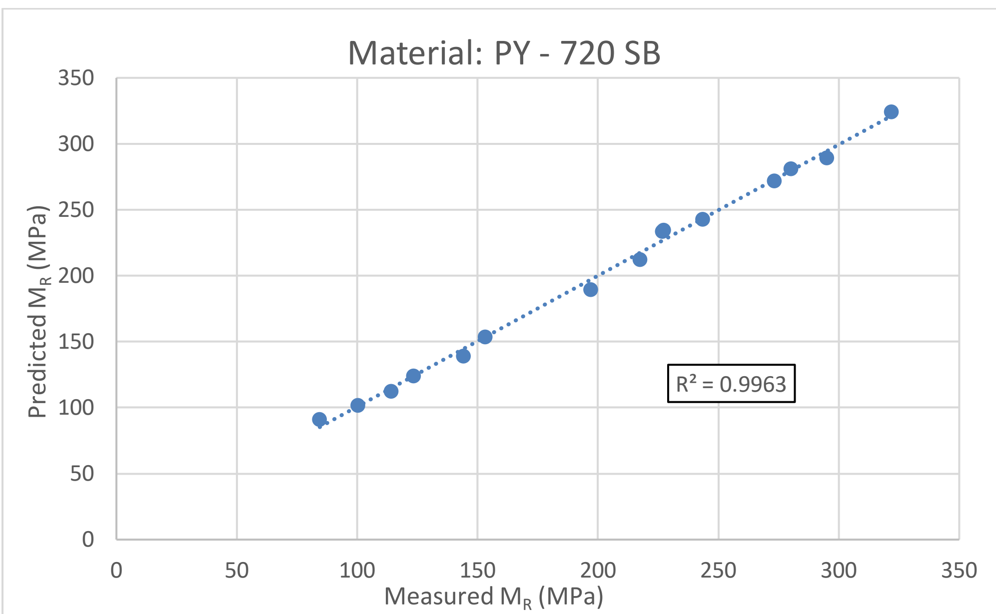

Material : PW - 84
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9929$
0
50
100
150
200
250


Material: Vy - 63
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9919$
0
50
100
150
200
250
300
350
400

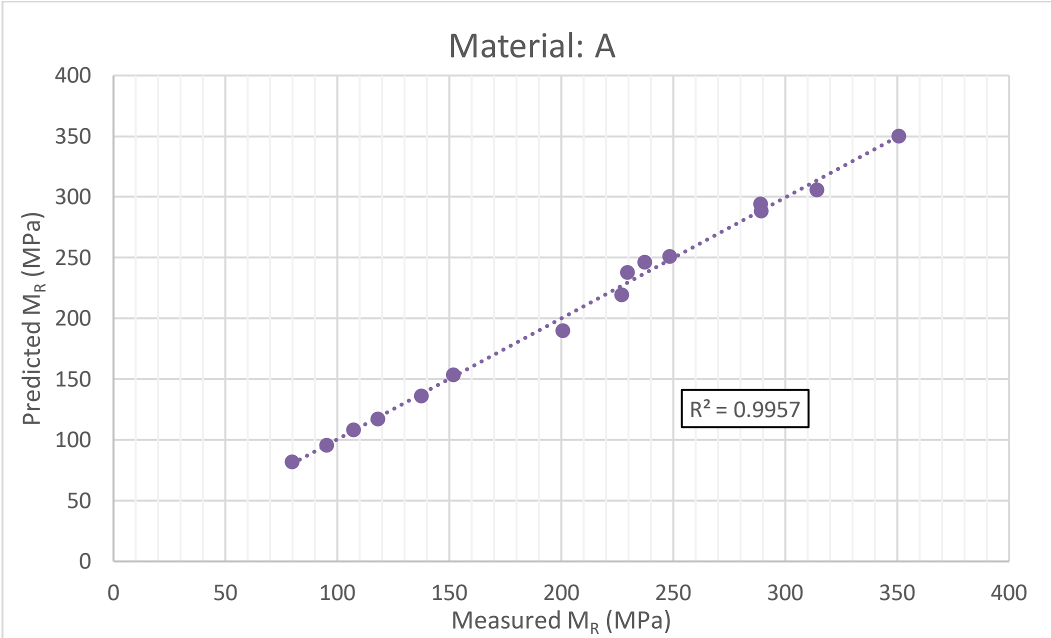

Material: WCW
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9989$
0
50
100
150
200
250
300
350

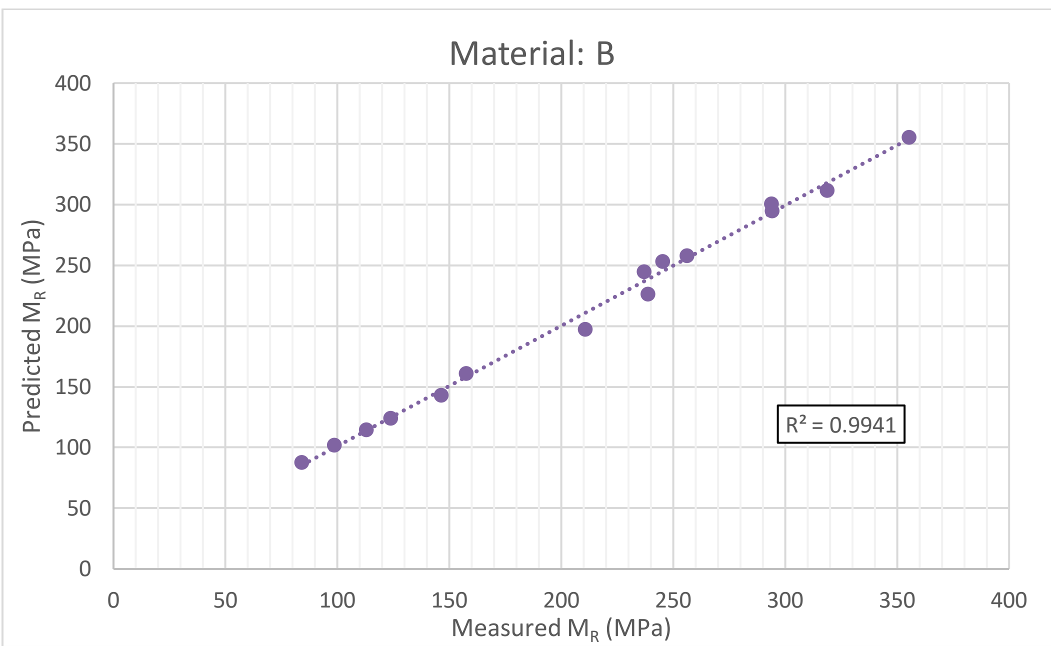

Material: CN - 148 SB
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9954$
0
50
100
150
200
250
300
350
400

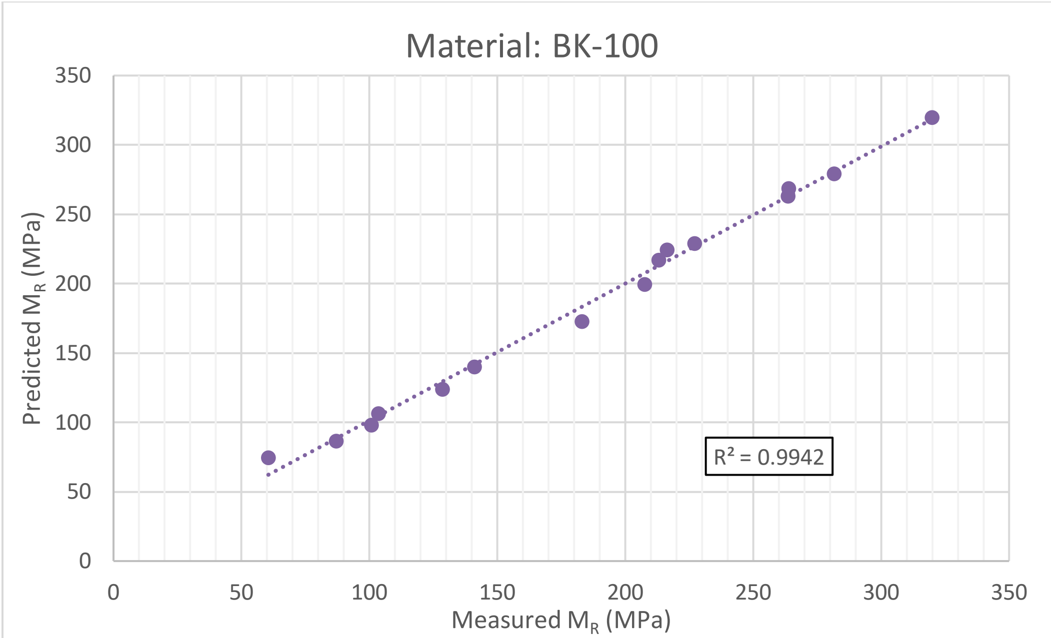

Material: CS - 184 SB
Predicted M_R (MPa)
0
20
40
60
80
100
120
140
160
Measured M_R (MPa)
$R^2 = 0.9946$

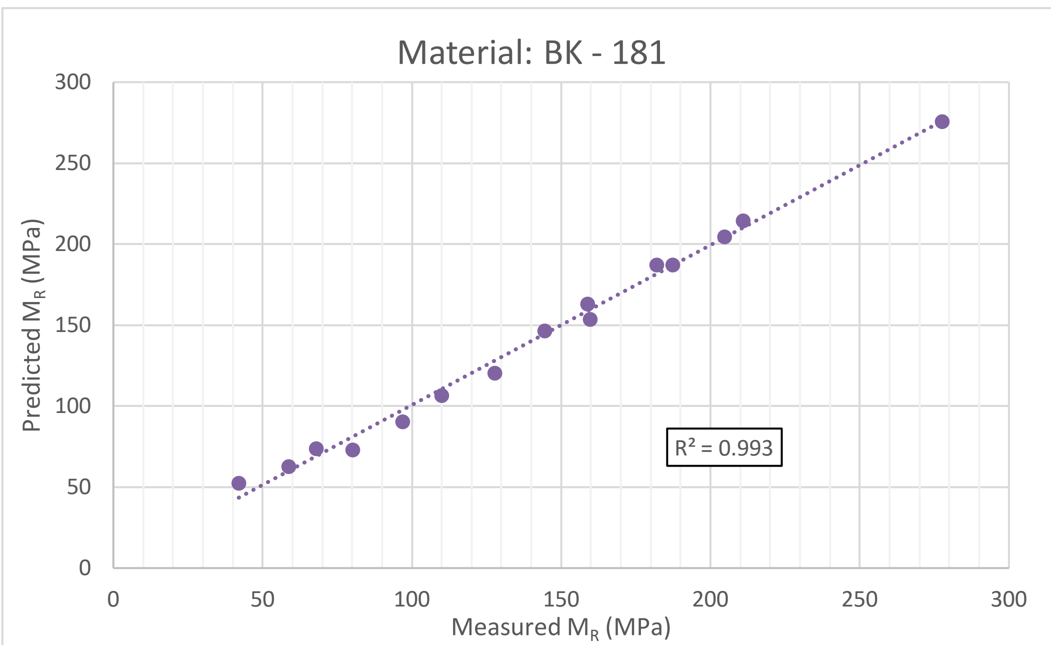

Material: PY - 720 SB
Predicted M_R (MPa)
0
50
100
150
200
250
300
350
Measured M_R (MPa)
$R^2 = 0.9963$

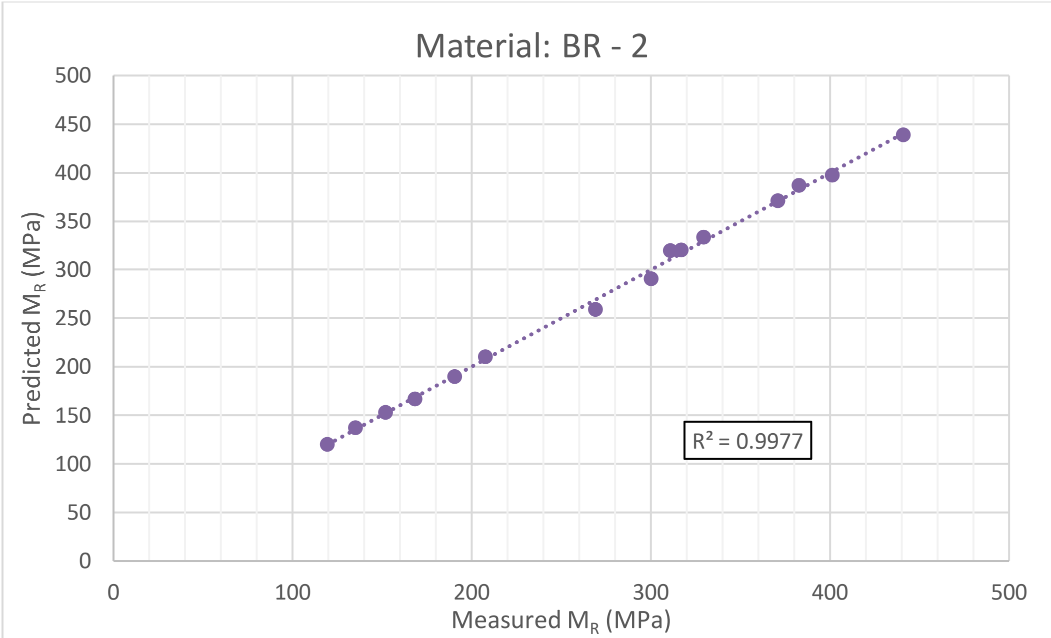
MEPDG Model Predicted M_R and Laboratory Measured M_R Plots

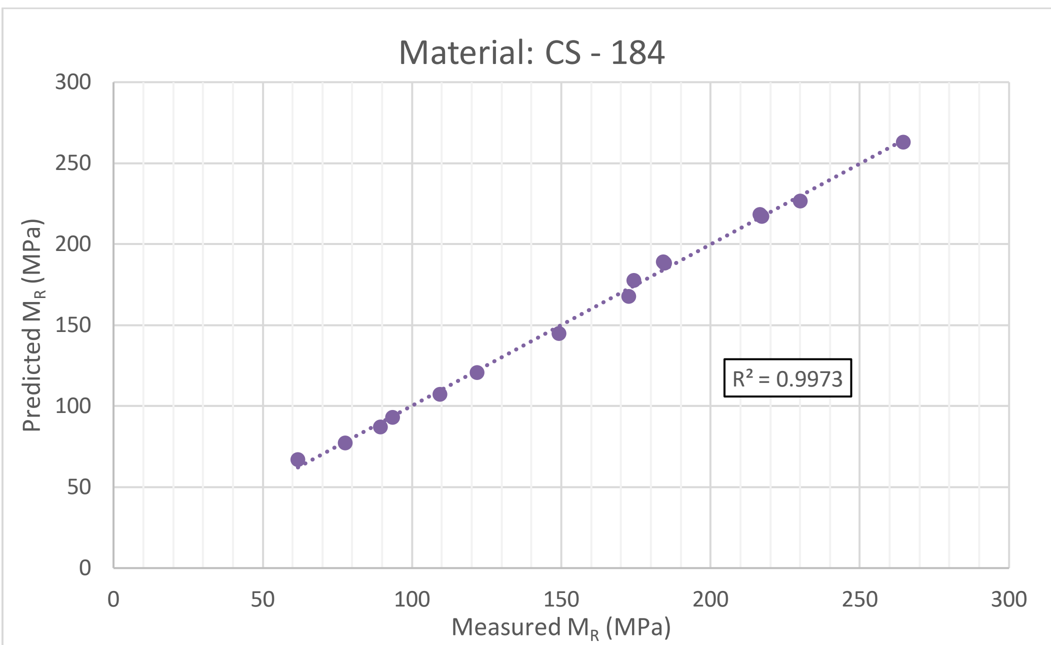


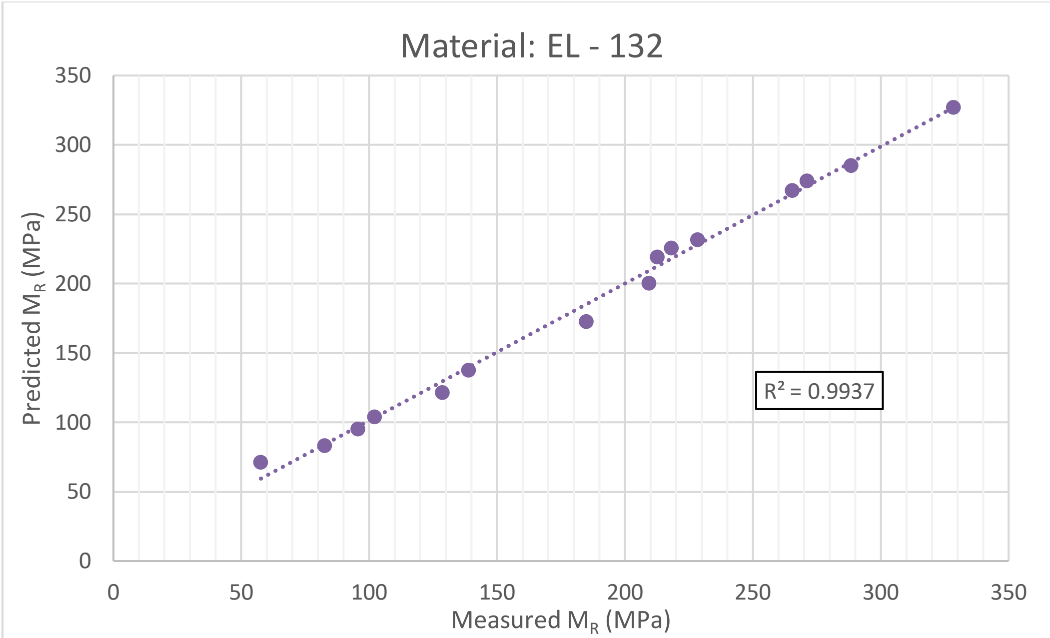


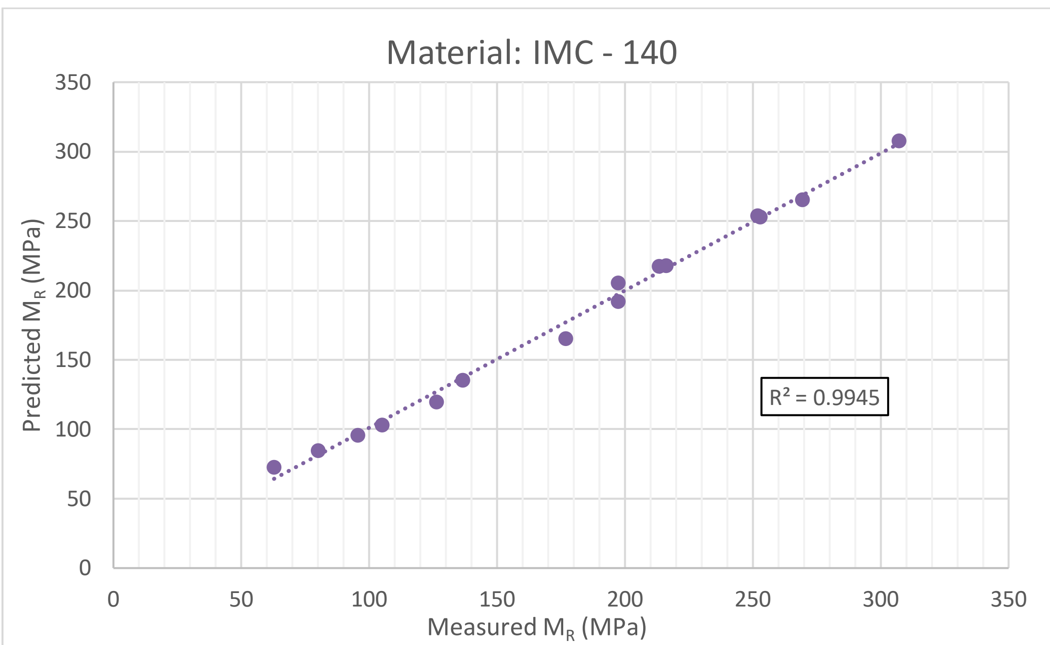

Material: BK-100
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9942$
0
50
100
150
200
250
300
350

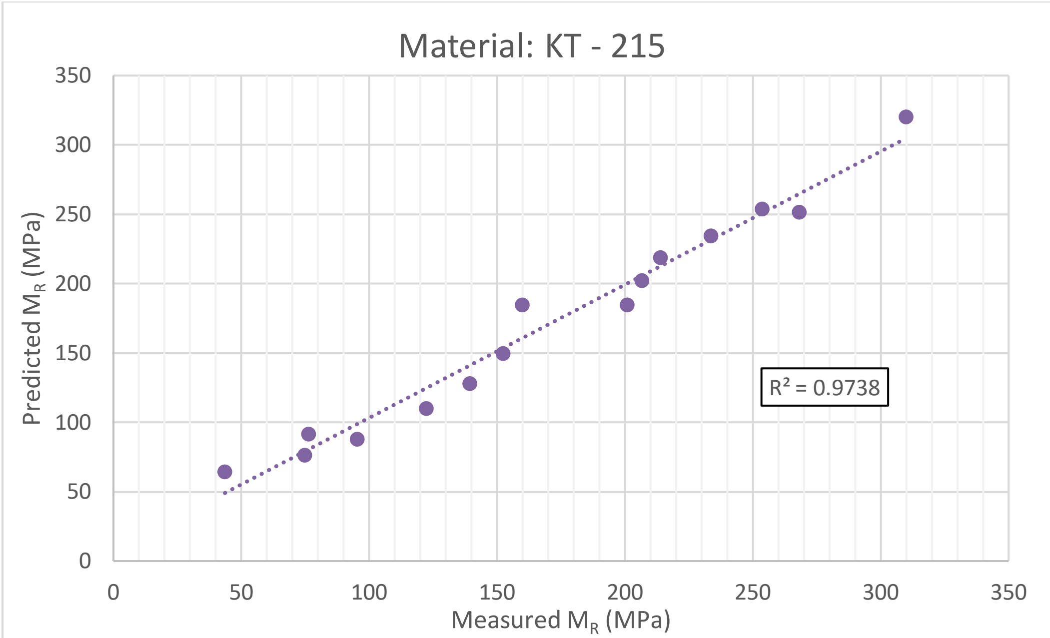

Material: BK - 181
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.993$
0
50
100
150
200
250
300

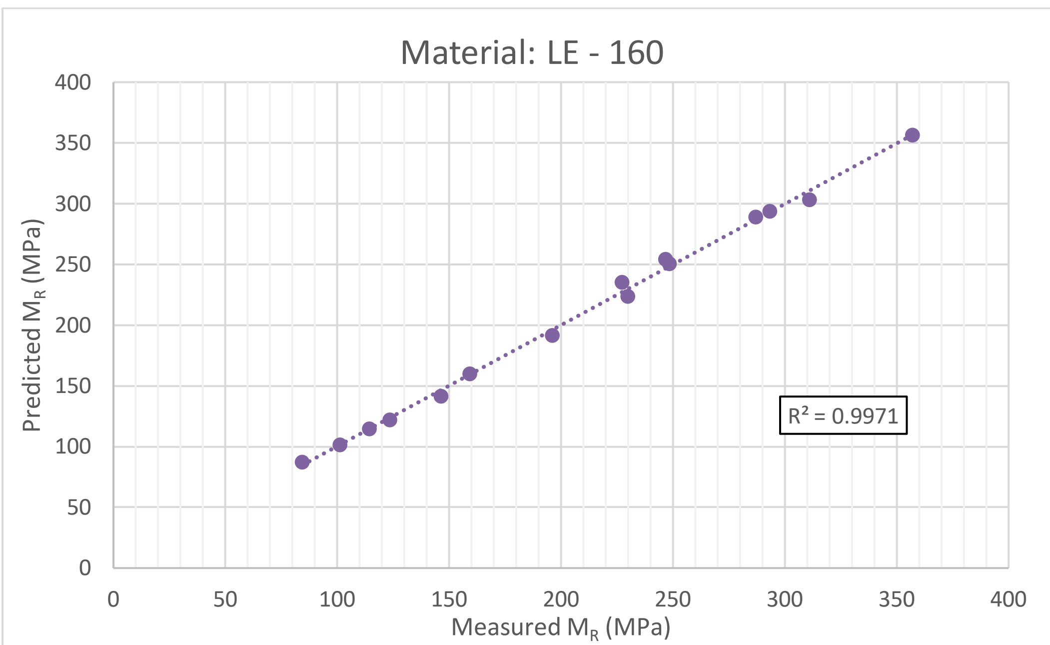

Material: BR - 2
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9977$
0
50
100
150
200
250
300
350
400
450
500

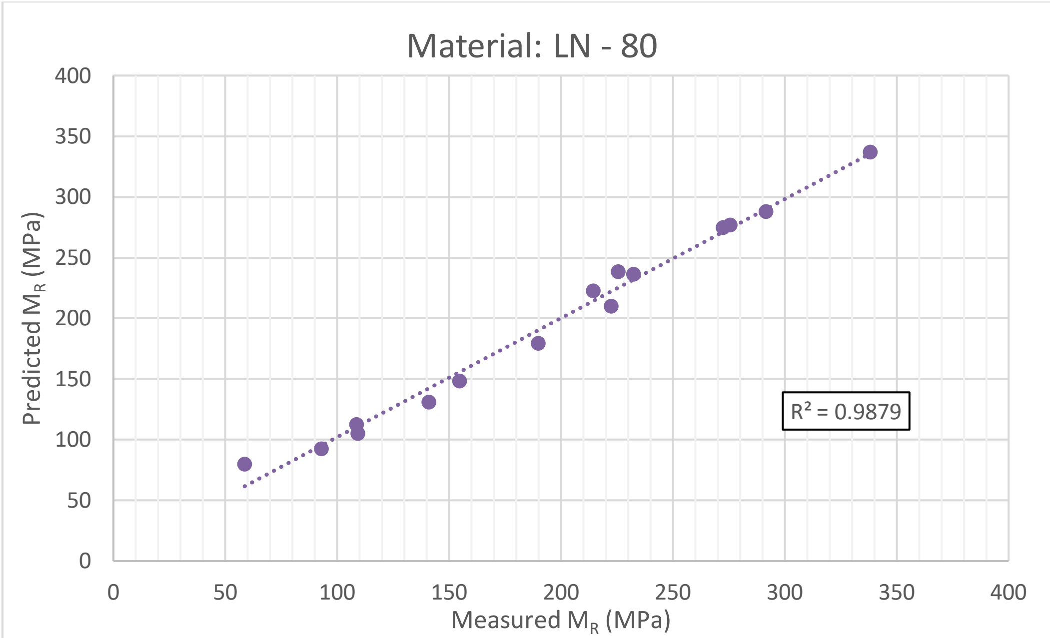

Material: CS - 184
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9973$
0
50
100
150
200
250
300

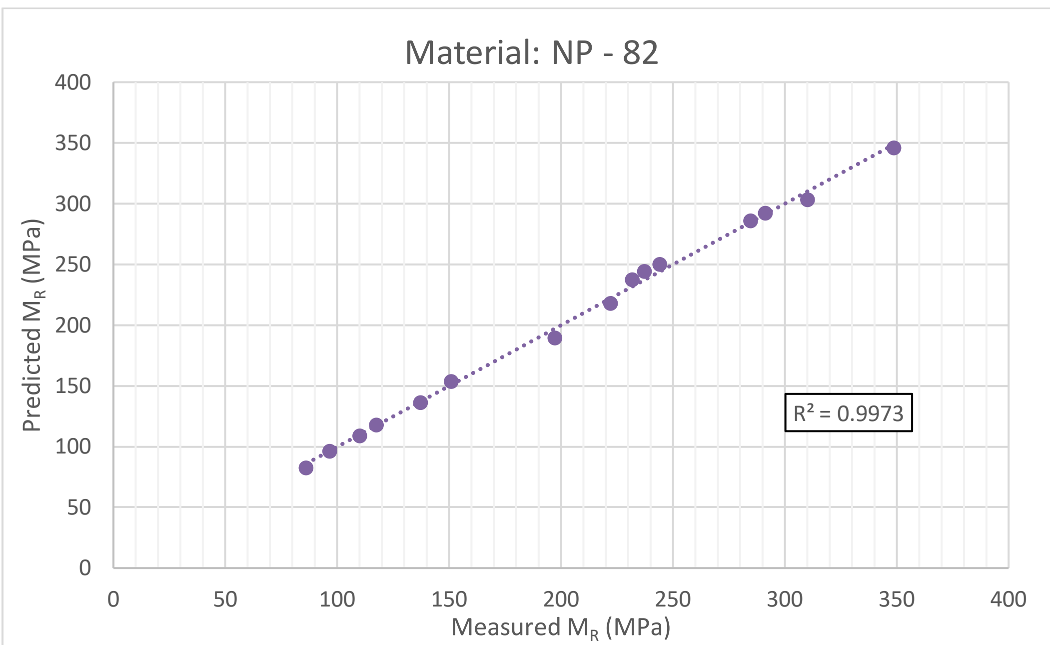

Material: EL - 132
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9937$
0
50
100
150
200
250
300
350

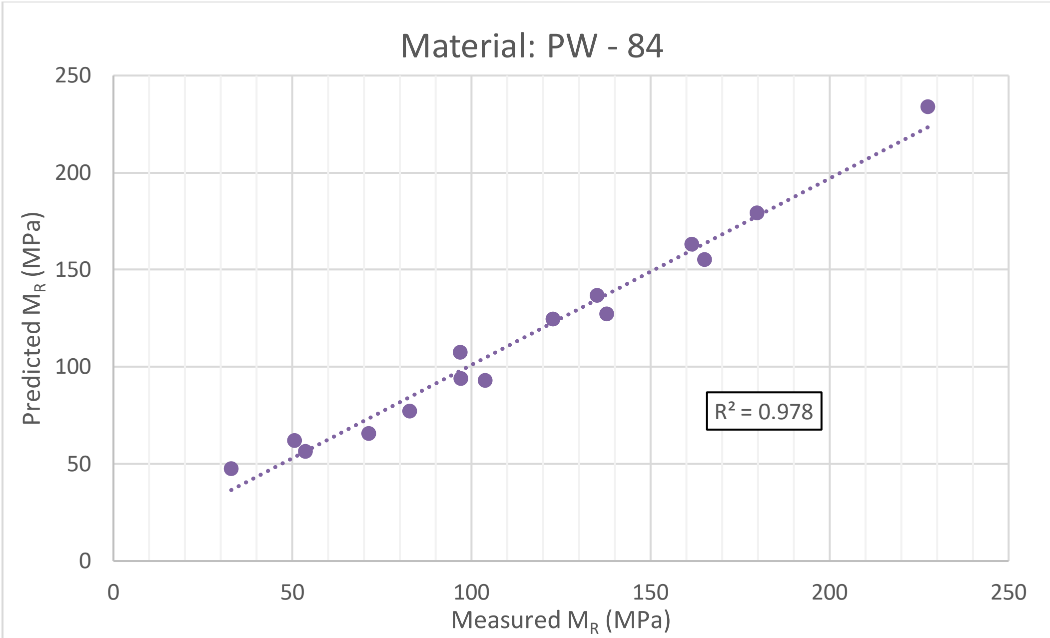

Material: IMC - 140
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9945$
0
50
100
150
200
250
300
350

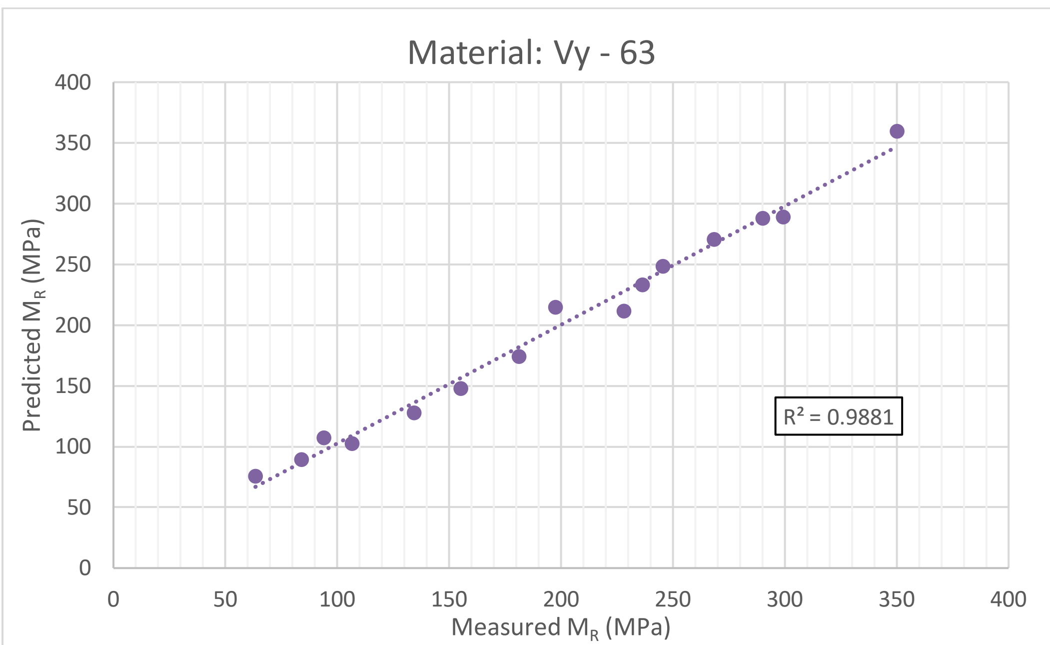

Material: KT - 215
Predicted M_R (MPa)
Measured M_R (MPa)
R² = 0.9738
0
50
100
150
200
250
300
350

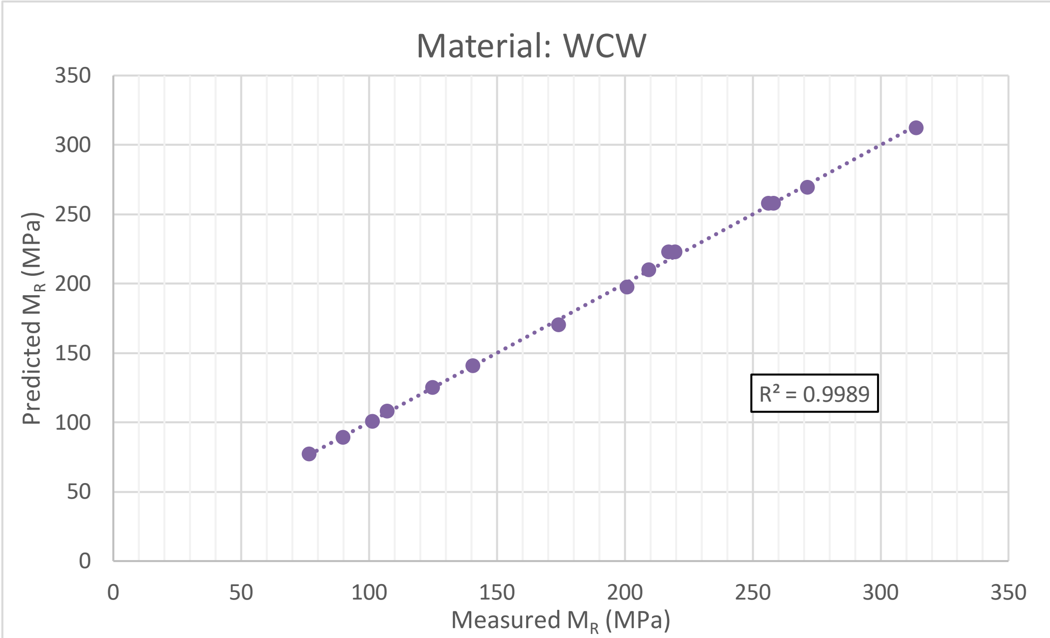

Material: LE - 160
Predicted M_R (MPa)
Measured M_R (MPa)
R² = 0.9971
0
50
100
150
200
250
300
350
400

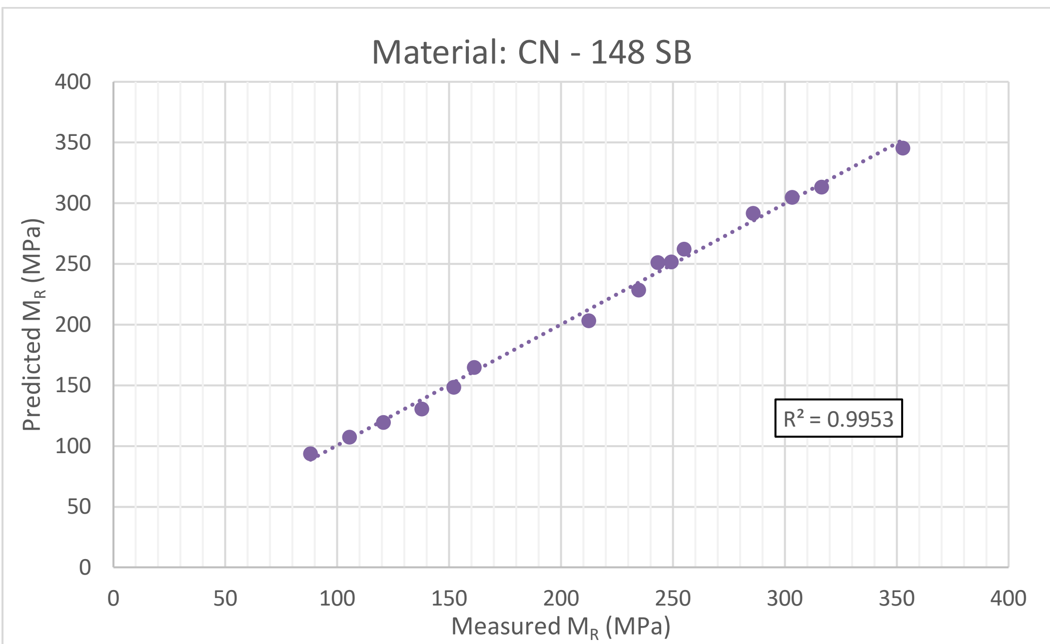

Material: LN - 80
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9879$
0
50
100
150
200
250
300
350
400

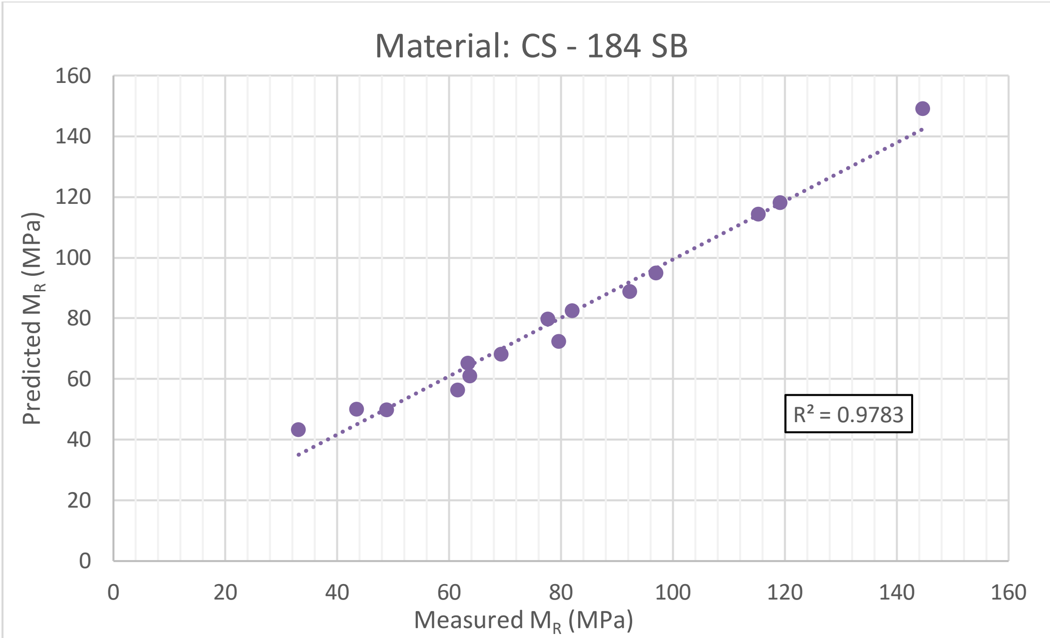

Material: NP - 82
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9973$
0
50
100
150
200
250
300
350
400

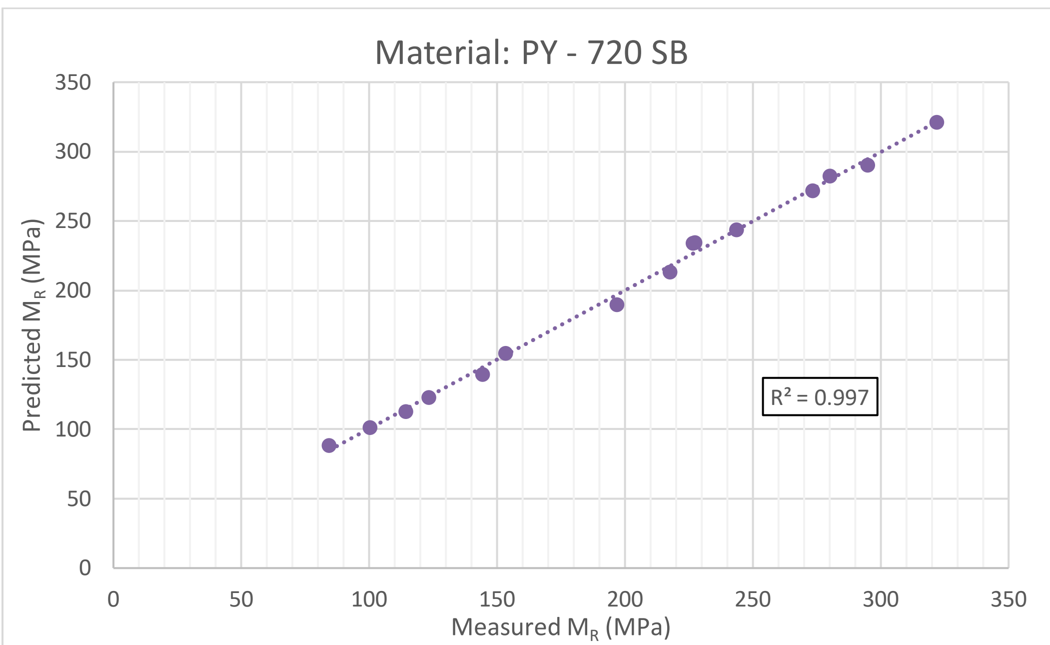

Material: PW - 84
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.978$
0
50
100
150
200
250


Material: Vy - 63
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9881$
0
50
100
150
200
250
300
350
400

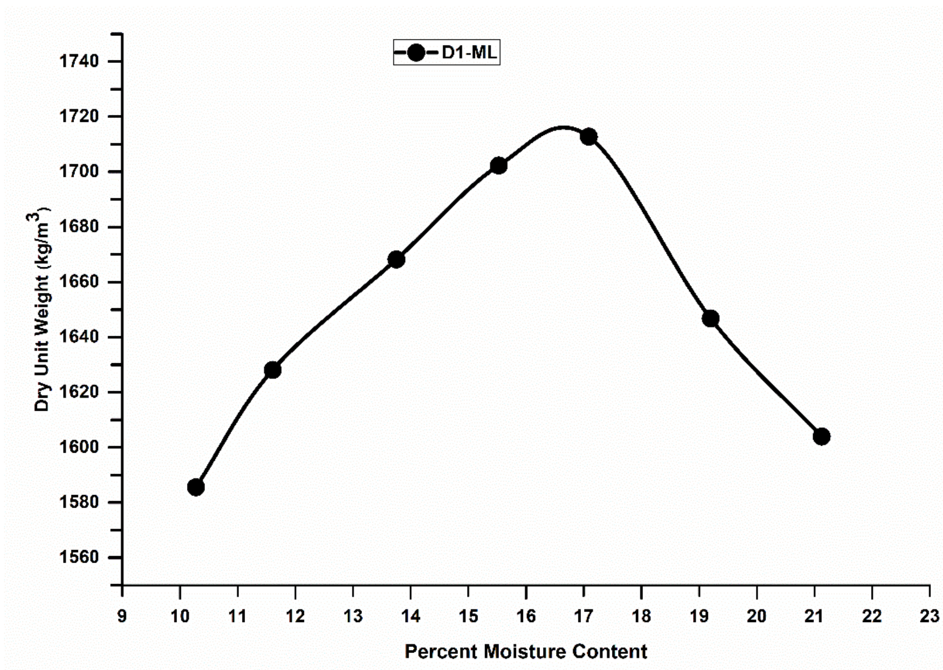

Material: WCW
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9989$
0
50
100
150
200
250
300
350

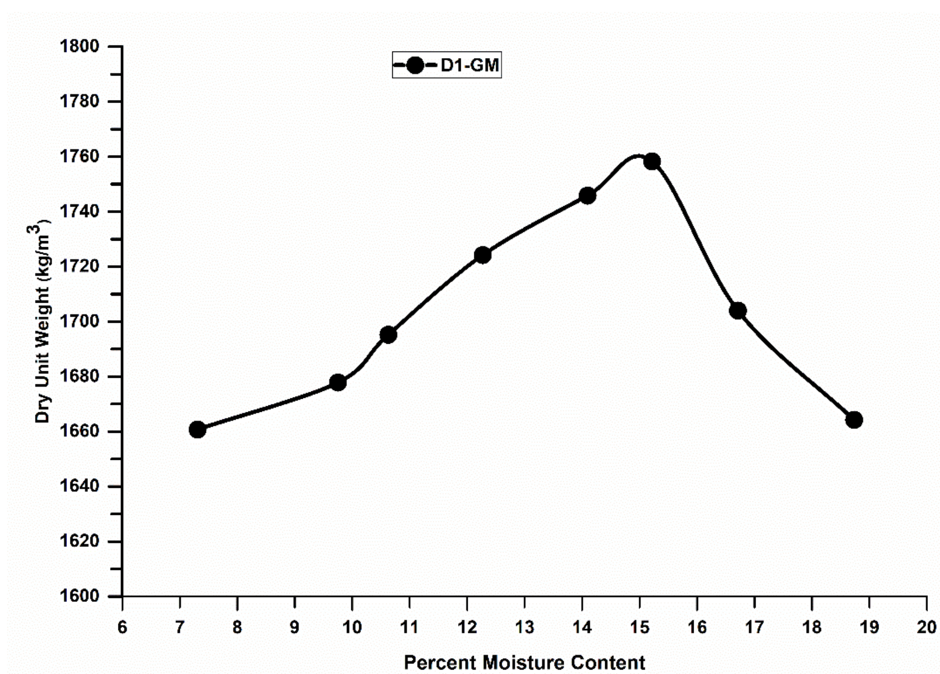

Material: CN - 148 SB
Predicted M_R (MPa)
Measured M_R (MPa)
$R^2 = 0.9953$
0
50
100
150
200
250
300
350
400

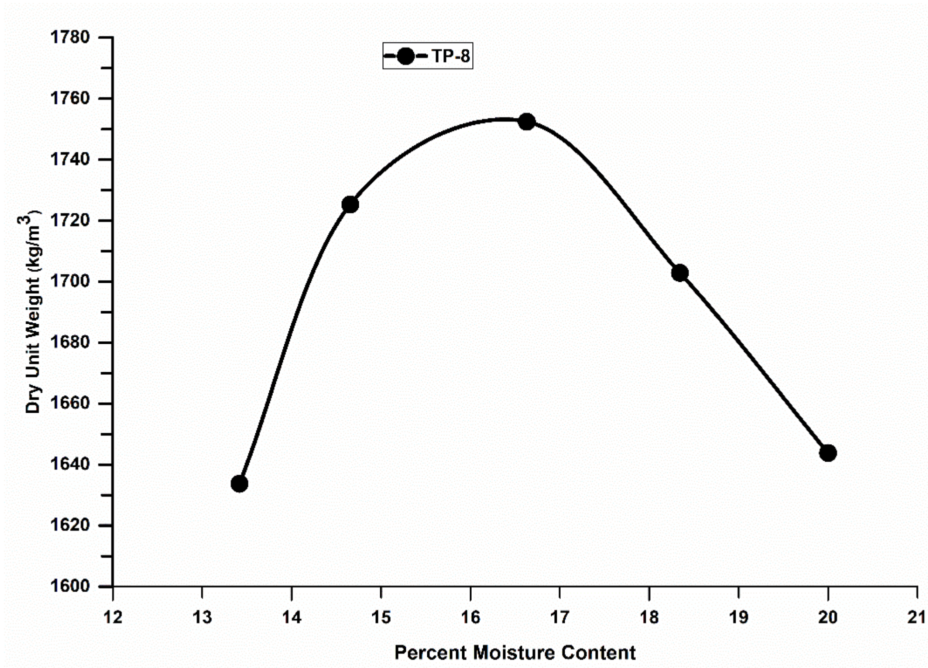

Material: CS - 184 SB
160
140
120
100
80
60
40
20
0
Predicted M_R (MPa)
R² = 0.9783
0
20
40
60
80
100
120
140
160
Measured M_R (MPa)

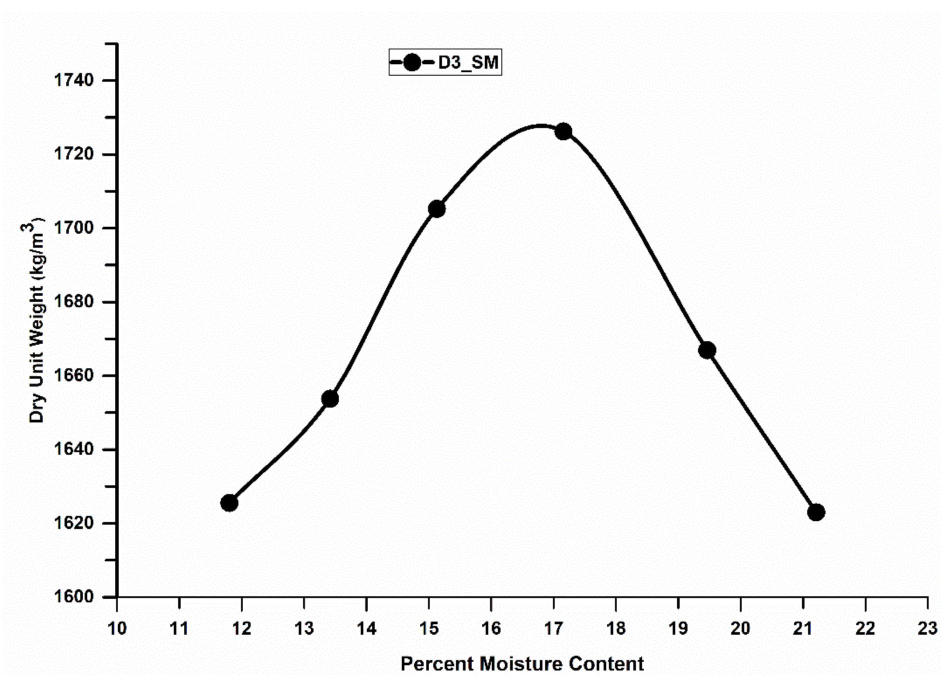

Material: PY - 720 SB
350
300
250
200
150
100
50
0
Predicted M_R (MPa)
R² = 0.997
0
50
100
150
200
250
300
350
Measured M_R (MPa)

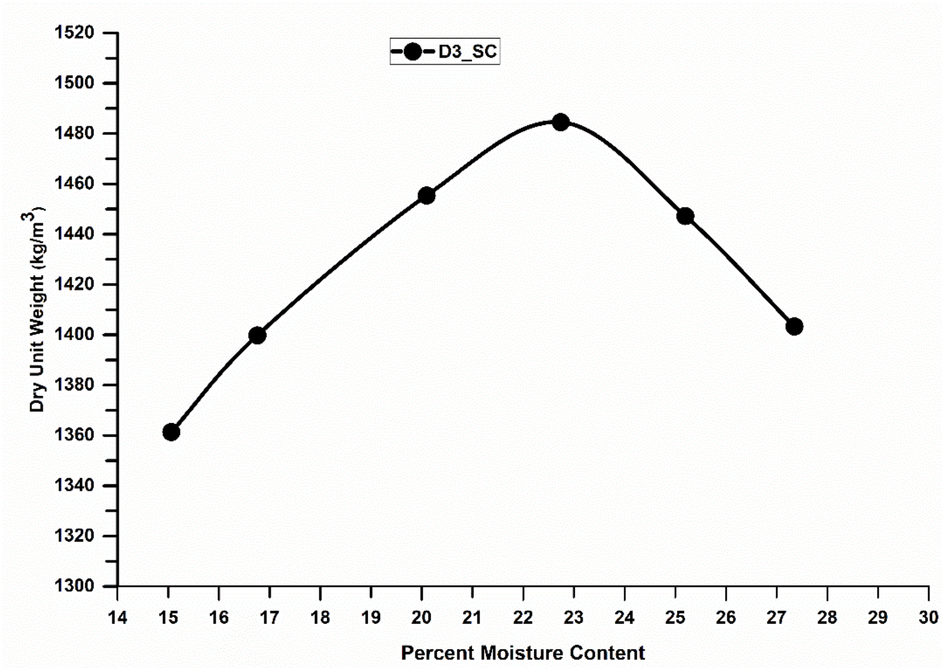
Appendix B
Moisture Density Plots for Subgrade Soils

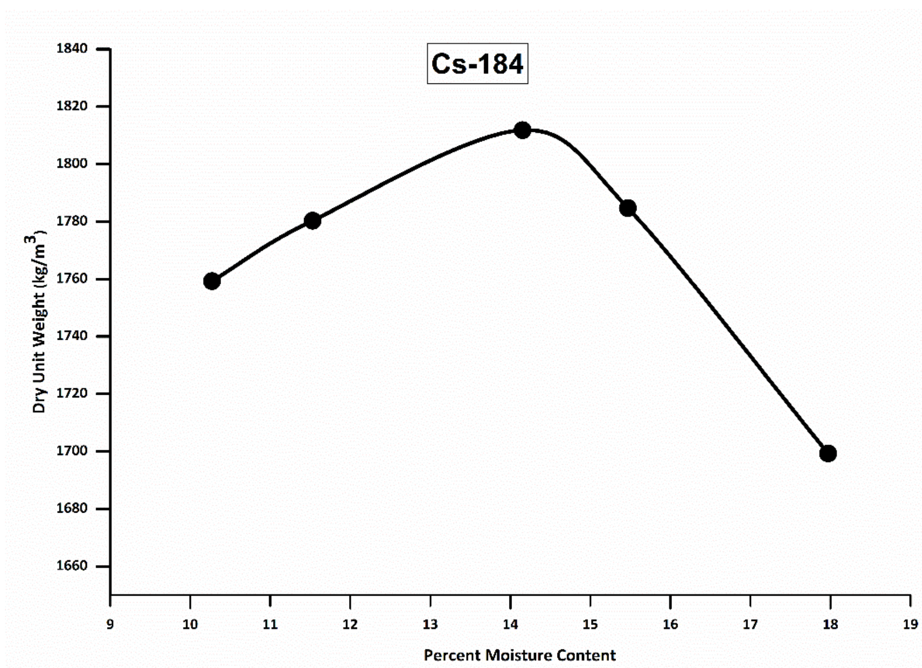


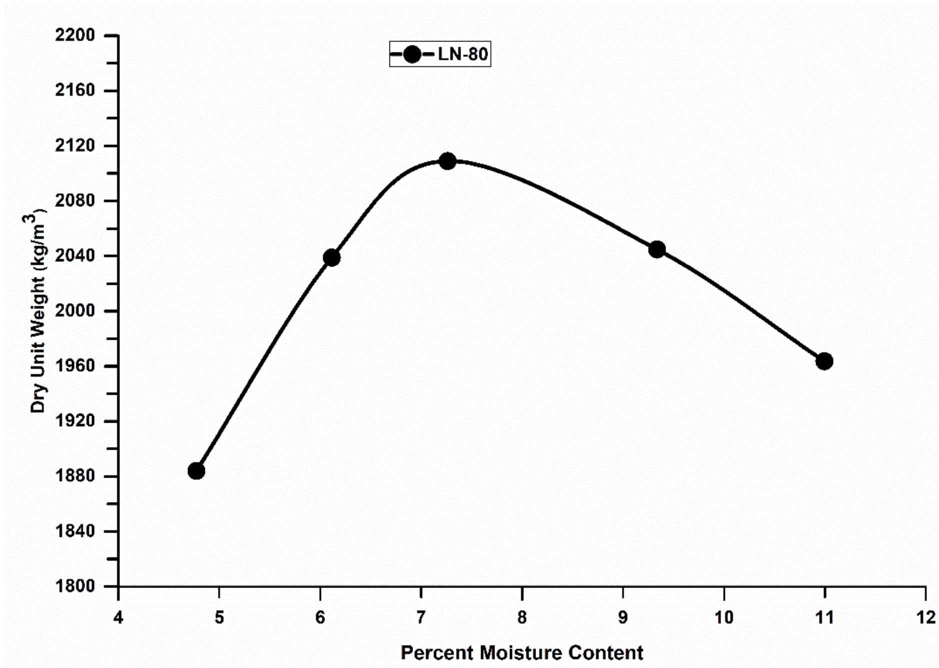


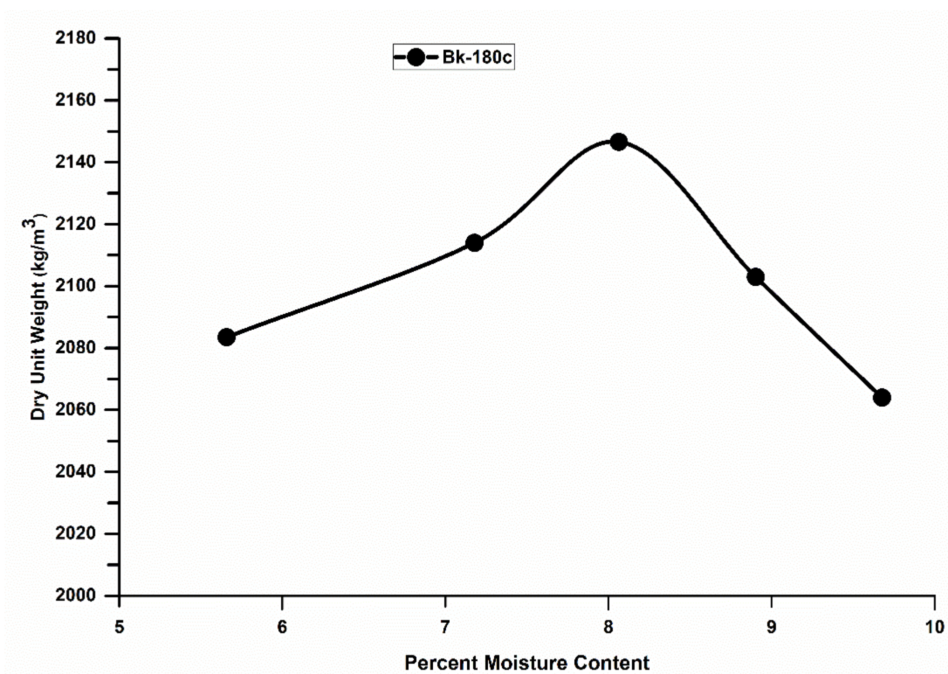

TP-8
Dry Unit Weight (kg/m3)
Percent Moisture Content
1600
1620
1640
1660
1680
1700
1720
1740
1760
1780
12
13
14
15
16
17
18
19
20
21

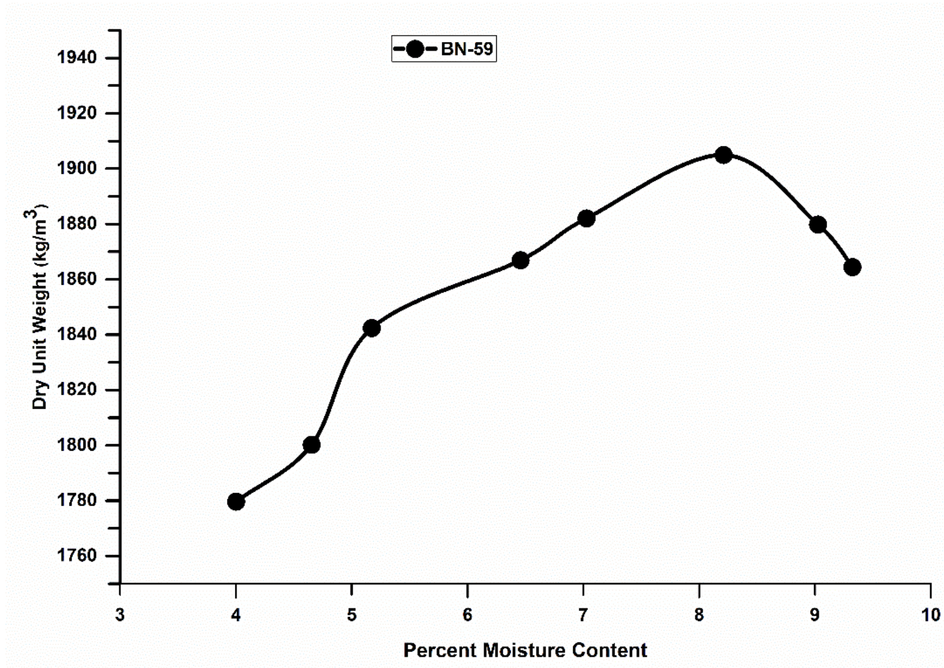

D3_SM
Dry Unit Weight (kg/m3)
Percent Moisture Content
1600
1620
1640
1660
1680
1700
1720
1740
10
11
12
13
14
15
16
17
18
19
20
21
22
23


D3_SC
Dry Unit Weight (kg/m3)
1300
1320
1340
1360
1380
1400
1420
1440
1460
1480
1500
1520
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
Percent Moisture Content

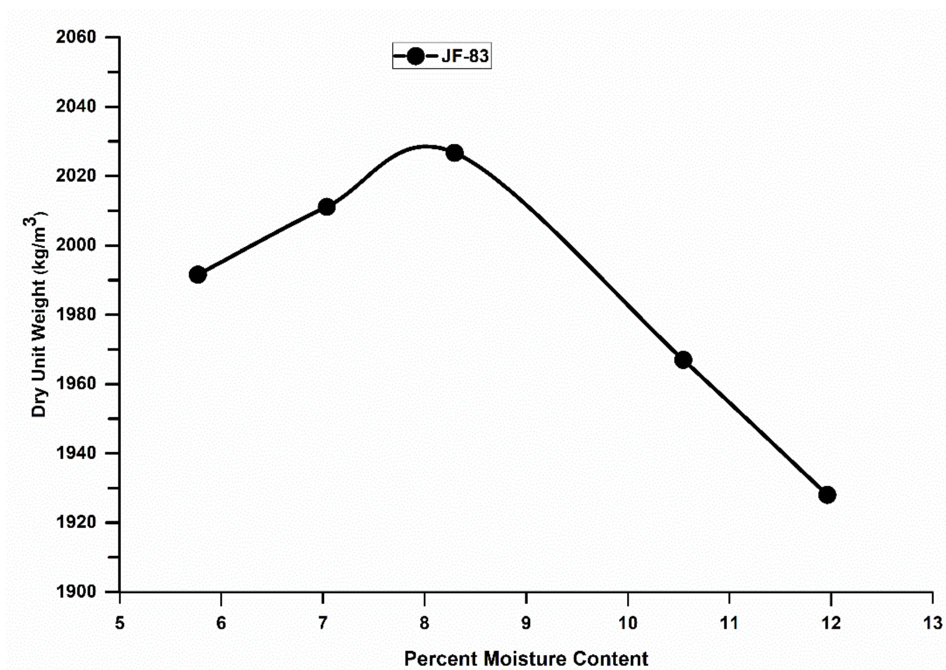

Cs-184
Dry Unit Weight (kg/m3)
1660
1680
1700
1720
1740
1760
1780
1800
1820
1840
9
10
11
12
13
14
15
16
17
18
19
Percent Moisture Content


LN-80
Dry Unit Weight (kg/m3)
2200
2160
2120
2080
2040
2000
1960
1920
1880
1840
1800
4
5
6
7
8
9
10
11
12
Percent Moisture Content

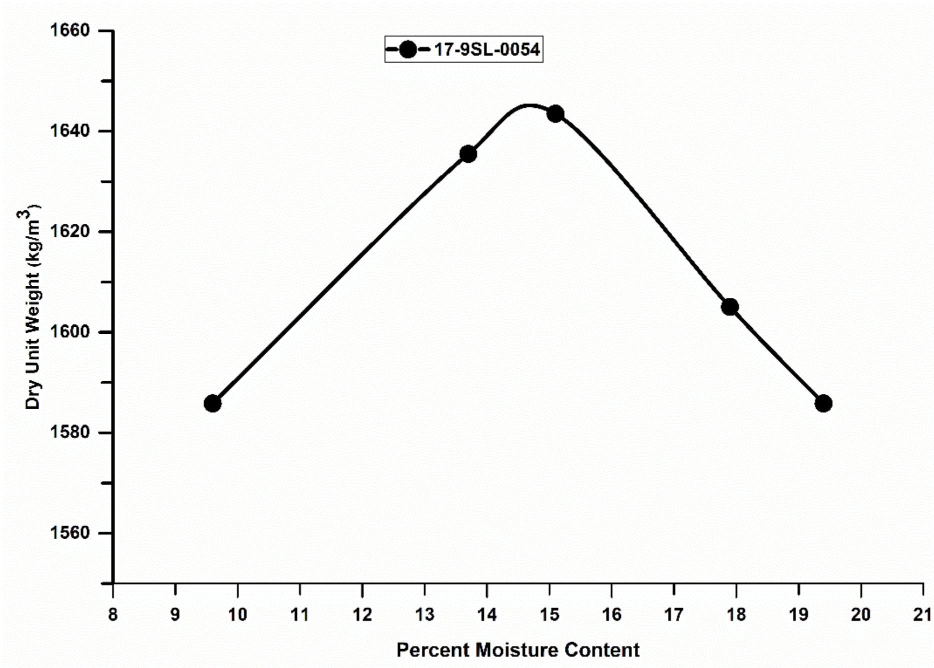

Bk-180c
Dry Unit Weight (kg/m3)
2180
2160
2140
2120
2100
2080
2060
2040
2020
2000
5
6
7
8
9
10
Percent Moisture Content

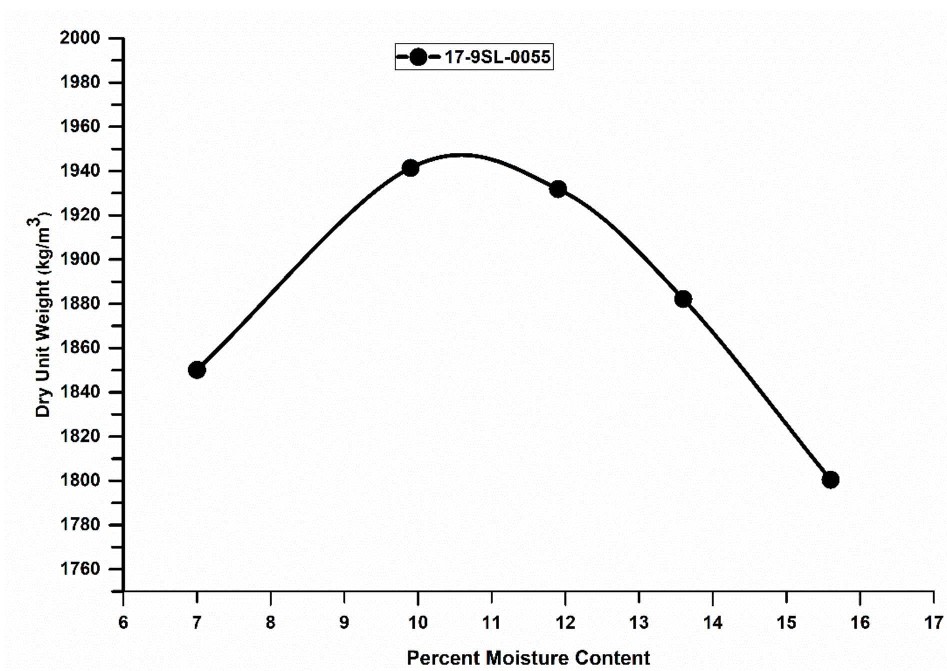


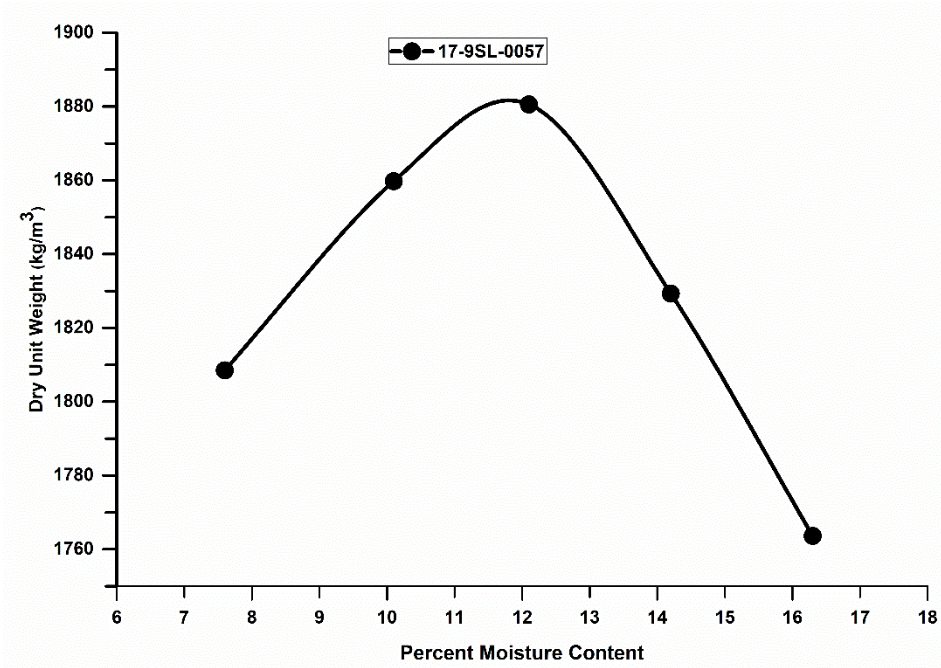
BN-59
Dry Unit Weight (kg/m^3)
1940
1920
1900
1880
1860
1840
1820
1800
1780
1760
3
4
5
6
7
8
9
10
Percent Moisture Content

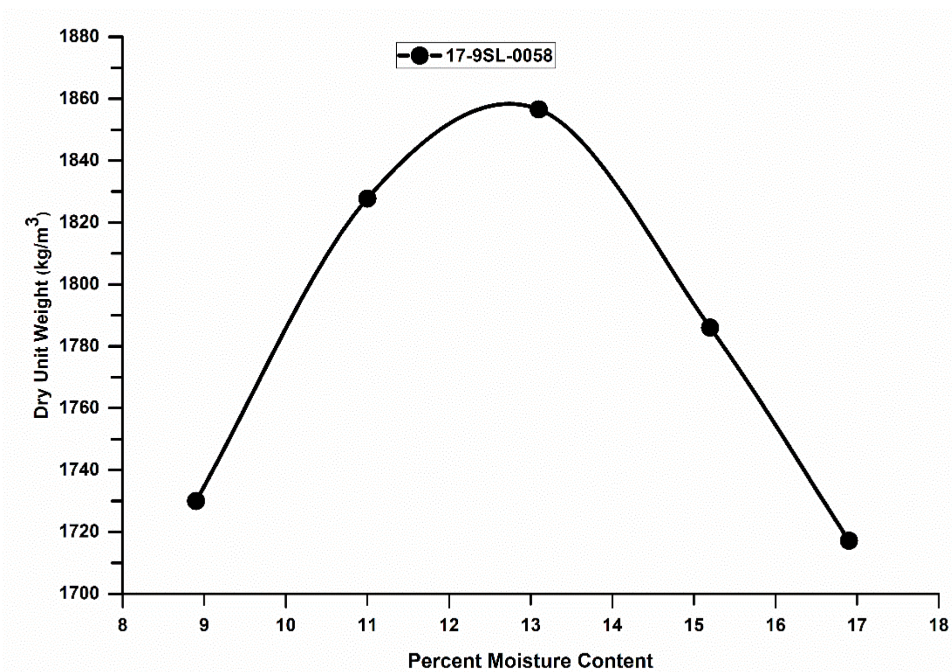


JF-83
Dry Unit Weight (kg/m^3)
2060
2040
2020
2000
1980
1960
1940
1920
1900
5
6
7
8
9
10
11
12
13
Percent Moisture Content

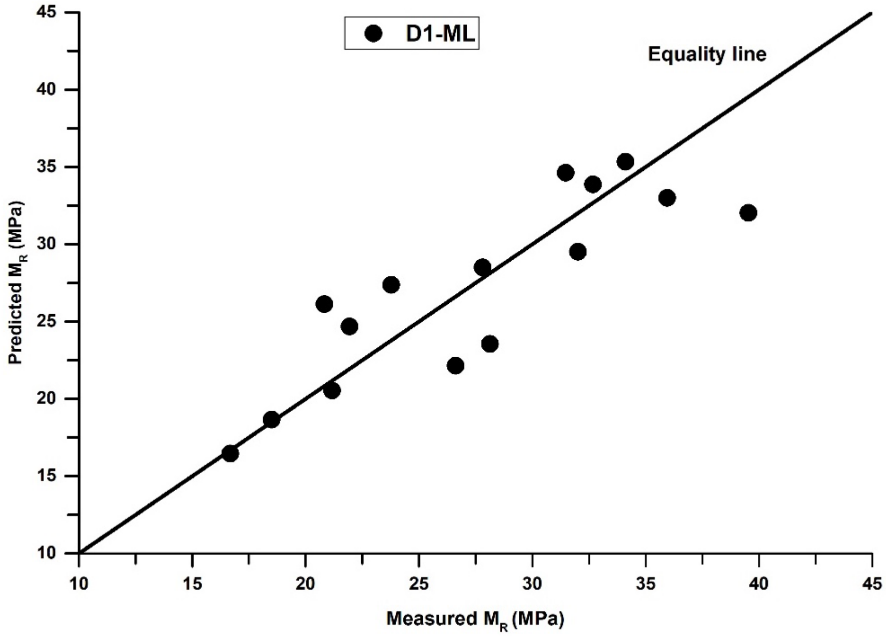

17-9SL-0054
Dry Unit Weight (kg/m3)
Percent Moisture Content
1660
1640
1620
1600
1580
1560
8
9
10
11
12
13
14
15
16
17
18
19
20
21

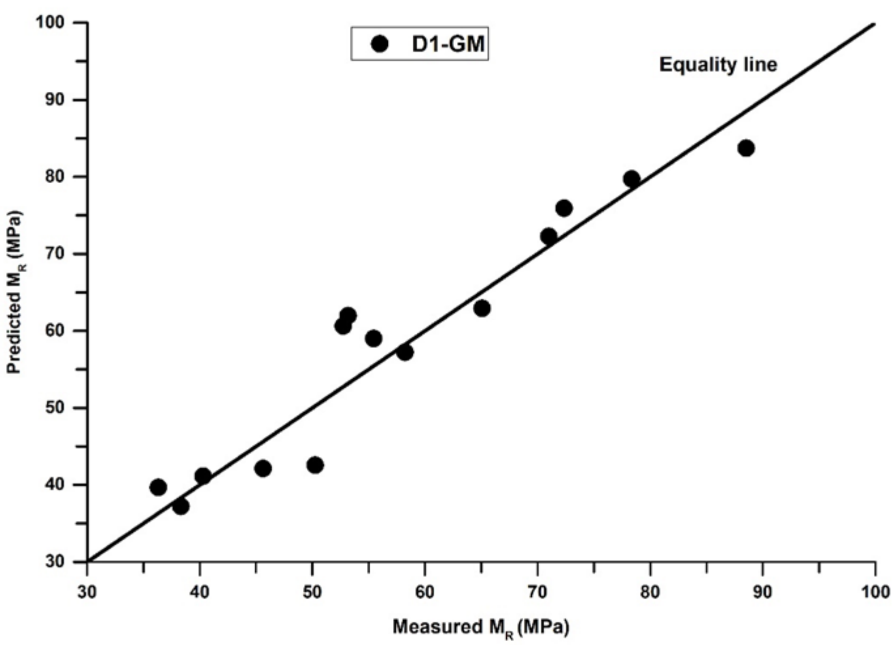

17-9SL-0055
Dry Unit Weight (kg/m3)
Percent Moisture Content
2000
1980
1960
1940
1920
1900
1880
1860
1840
1820
1800
1780
1760
6
7
8
9
10
11
12
13
14
15
16
17

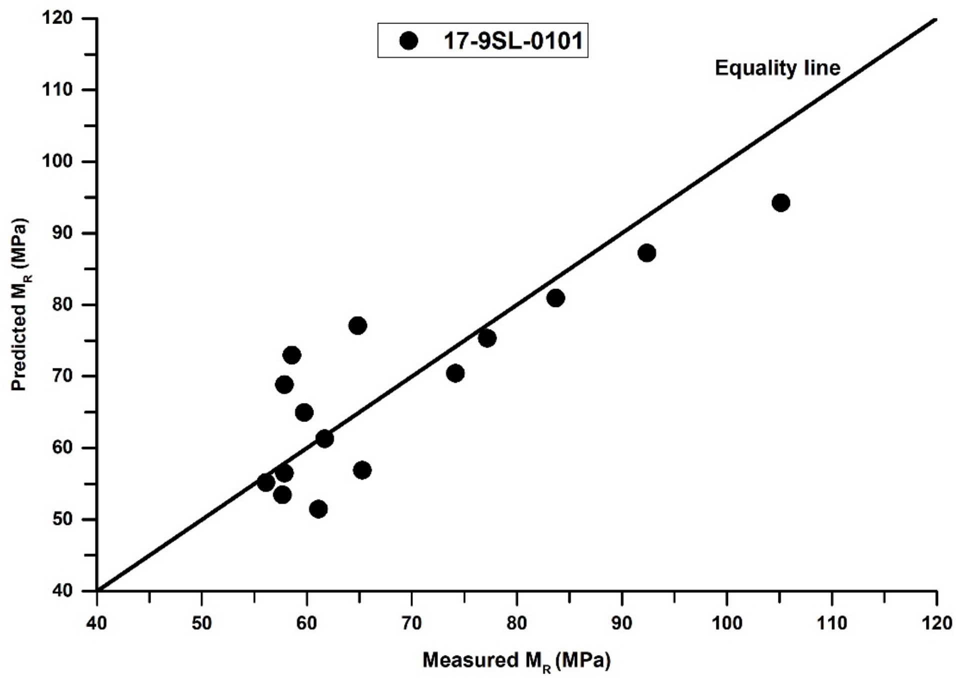

17-9SL-0057
Dry Unit Weight (kg/m^3)
Percent Moisture Content
1900
1880
1860
1840
1820
1800
1780
1760
6
7
8
9
10
11
12
13
14
15
16
17
18

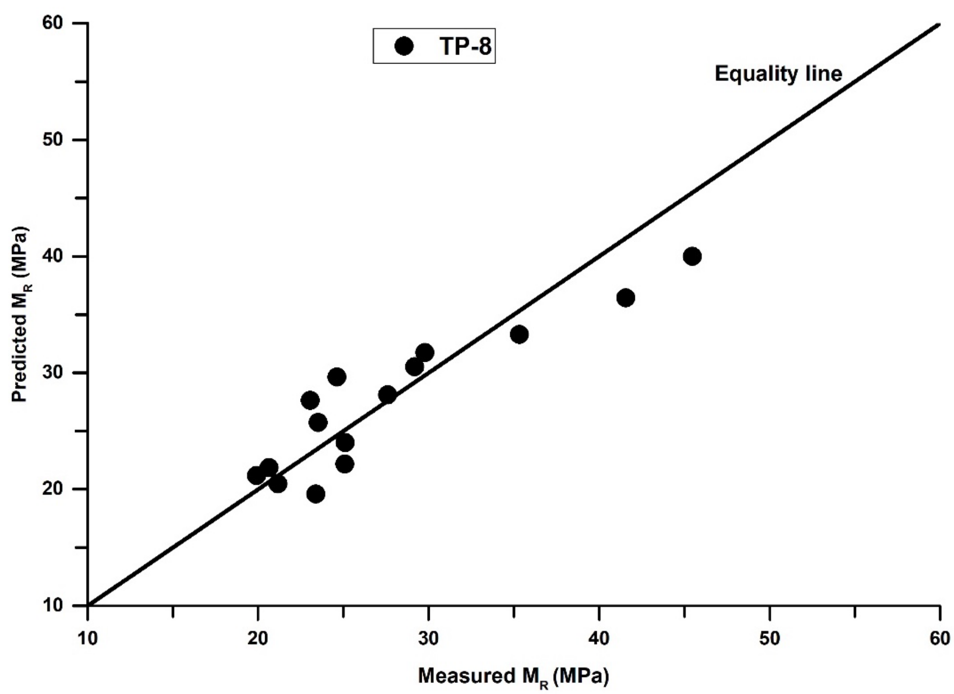

17-9SL-0058
Dry Unit Weight (kg/m^3)
Percent Moisture Content
1880
1860
1840
1820
1800
1780
1760
1740
1720
1700
8
9
10
11
12
13
14
15
16
17
18

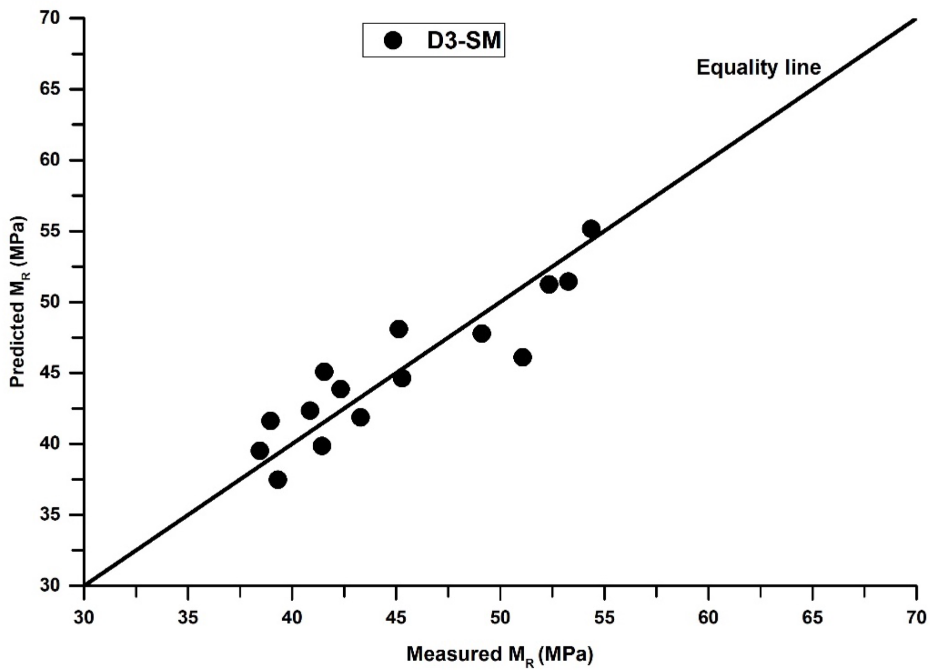
Appendix C
Fitting the MEPDG Model to Subgrade Resilient Modulus Test Data

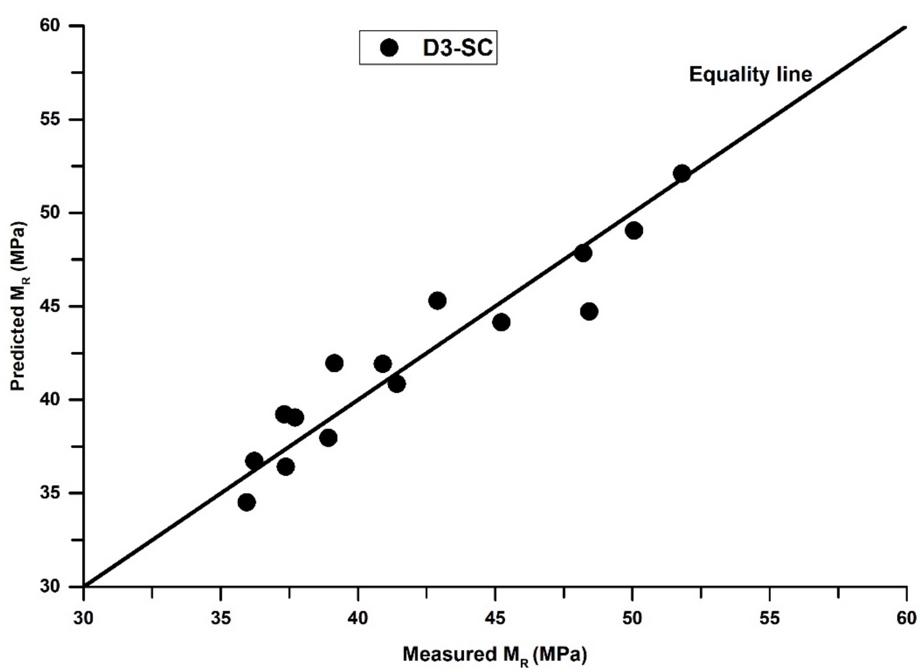


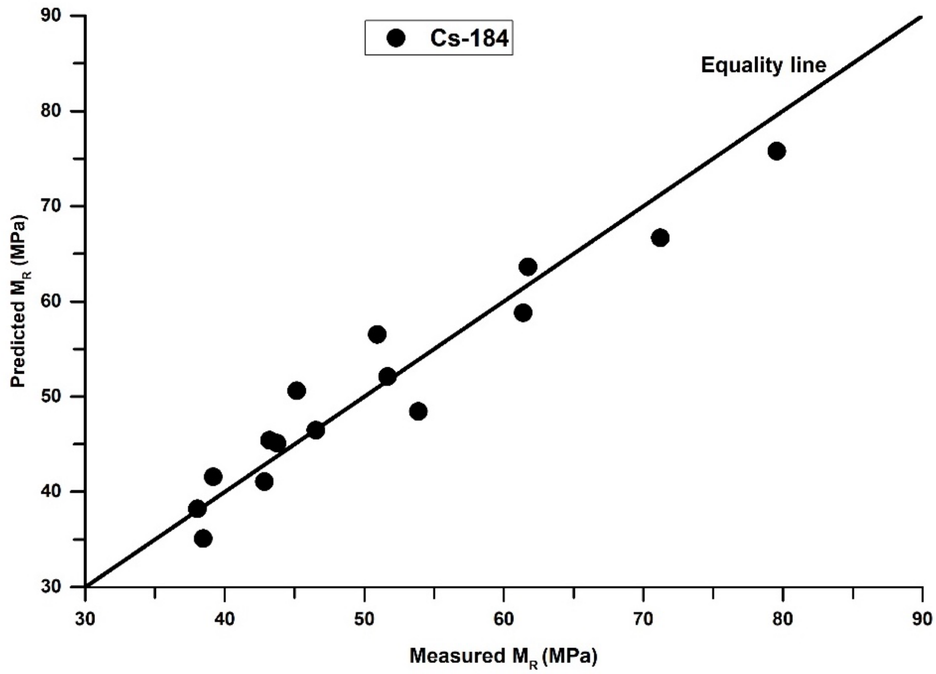


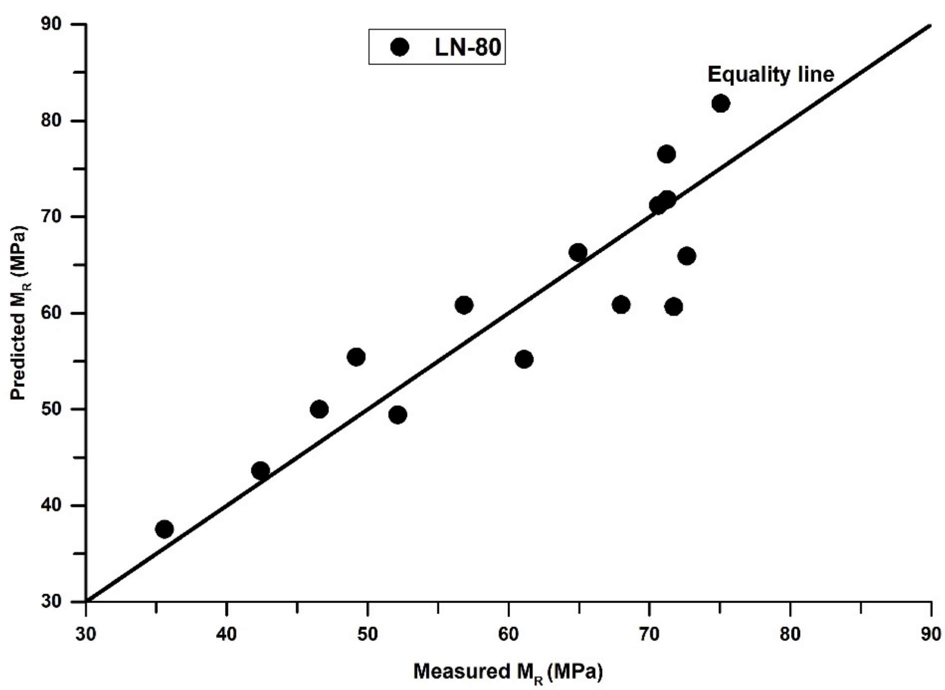

17-9SL-0101
Equality line
Predicted M_R (MPa)
Measured M_R (MPa)
40
50
60
70
80
90
100
110
120

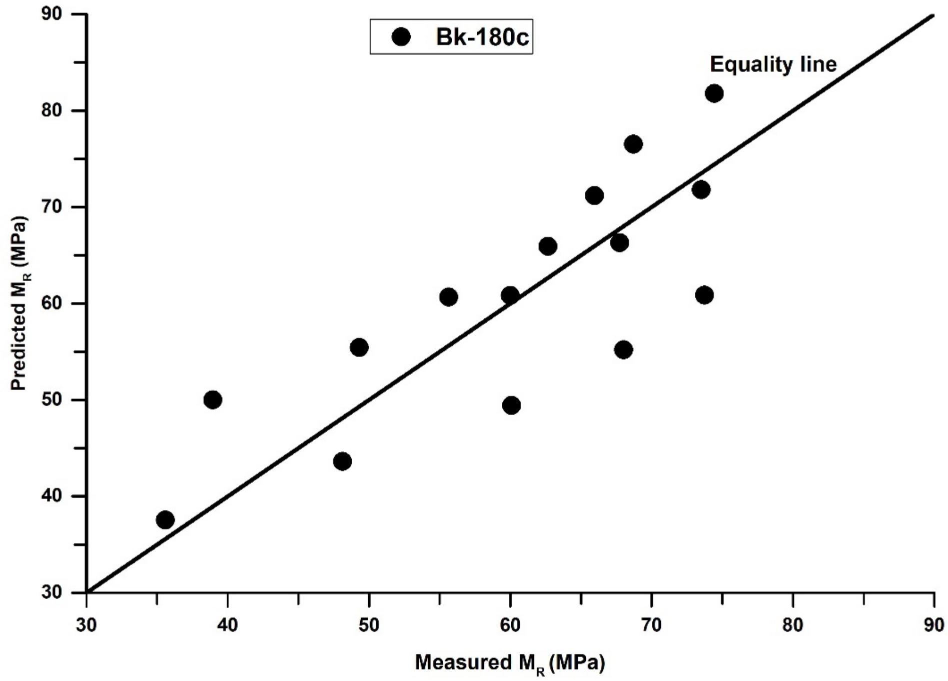

TP-8
Equality line
Predicted M_R (MPa)
Measured M_R (MPa)
10
20
30
40
50
60

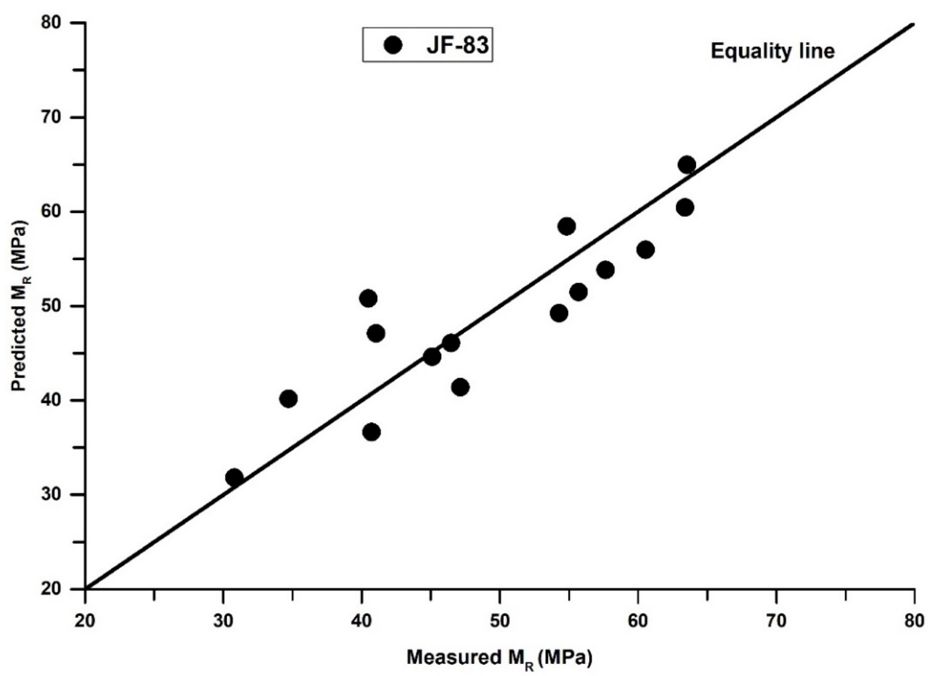

D3-SM
Equality line
Predicted M_R (MPa)
Measured M_R (MPa)
30
35
40
45
50
55
60
65
70

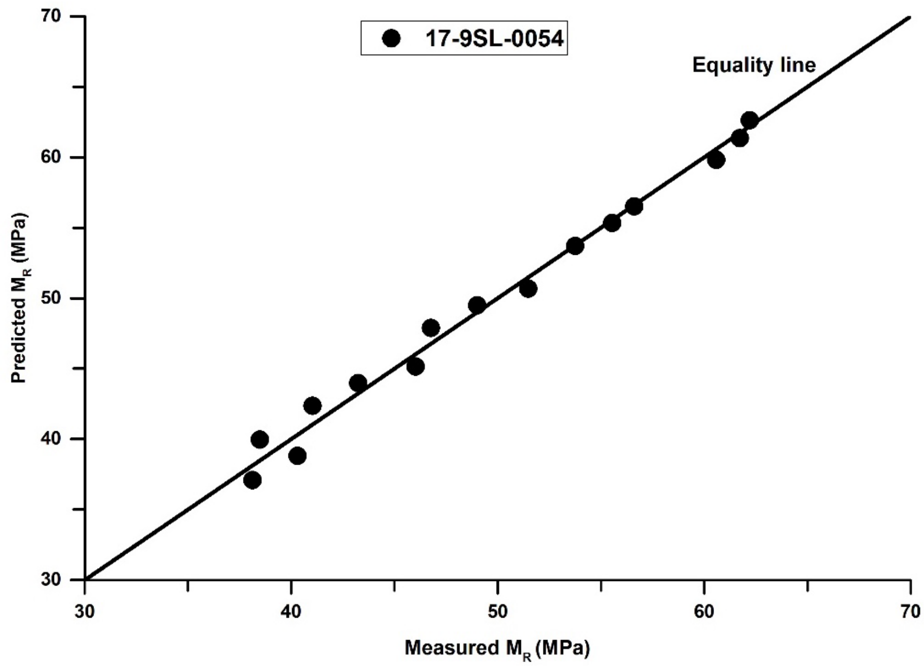

D3-SC
Equality line
Predicted M_R (MPa)
Measured M_R (MPa)
30
35
40
45
50
55
60

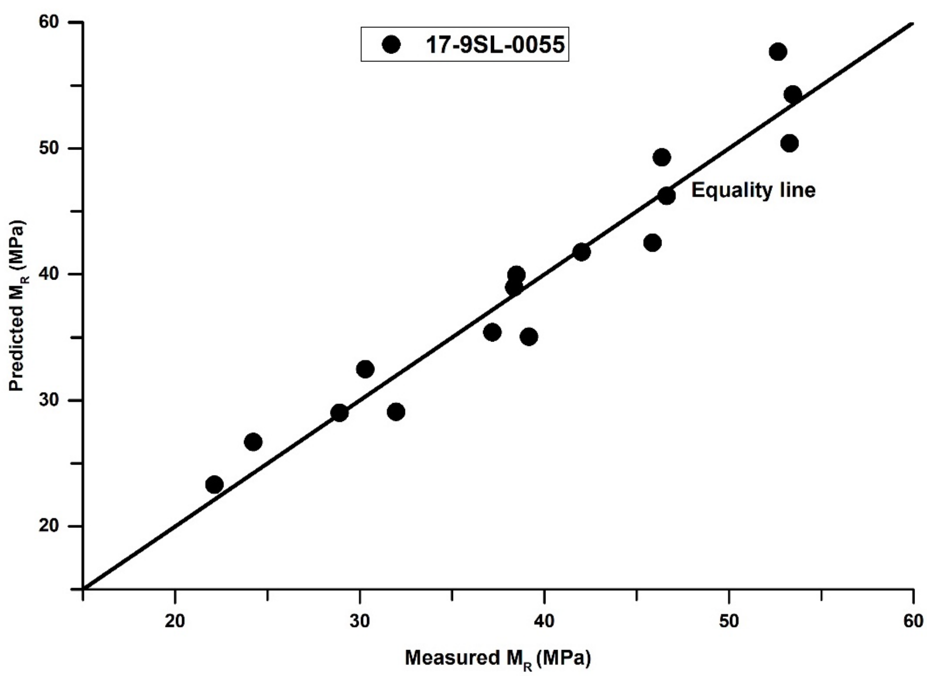

Cs-184
Equality line
Predicted M_R (MPa)
Measured M_R (MPa)
30
40
50
60
70
80
90

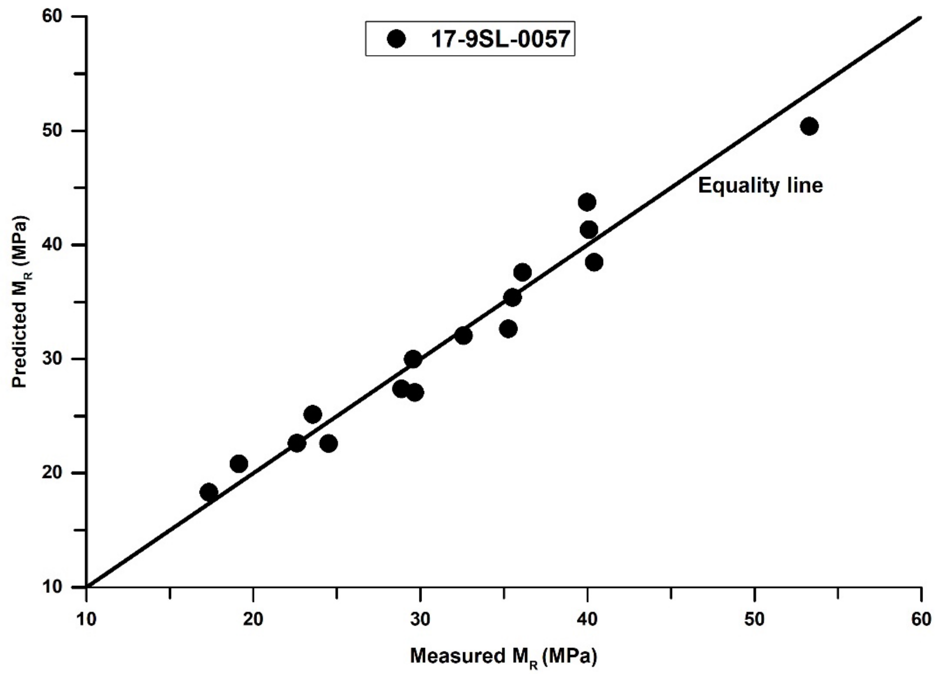

LN-80
Equality line
Predicted M_R (MPa)
Measured M_R (MPa)
30
40
50
60
70
80
90

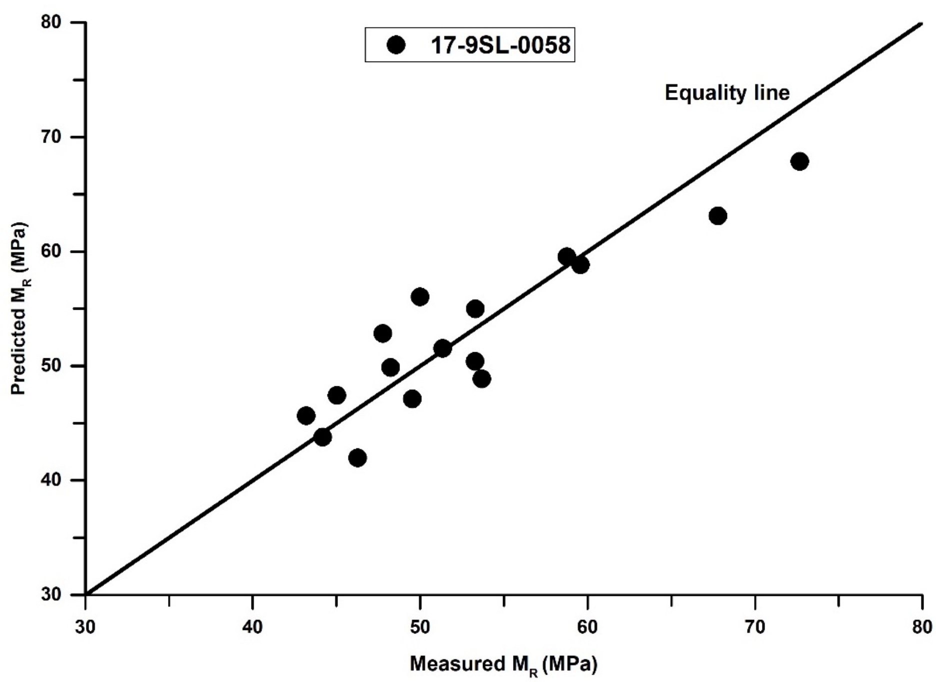

Bk-180c
Equality line
Predicted M_R (MPa)
Measured M_R (MPa)
30
40
50
60
70
80
90


JF-83
Equality line
Predicted M_R (MPa)
Measured M_R (MPa)
20
30
40
50
60
70
80


17-9SL-0054
Equality line
Predicted M_R (MPa)
Measured M_R (MPa)
30
40
50
60
70


17-9SL-0055
Equality line
Predicted M_R (MPa)
Measured M_R (MPa)
20
30
40
50
60


17-9SL-0057
Equality line
Predicted M_R (MPa)
Measured M_R (MPa)
10
20
30
40
50
60


17-9SL-0058
Equality line
Predicted M_R (MPa)
Measured M_R (MPa)
30
40
50
60
70
80