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16. Abstract

Field and laboratory testing programs were conducted to develop models that predict the resilient modulus of subgrade soils from the test results of DCP, CIMCPT, FWD, Dynaflect, and soil properties. The field testing program included DCP, CIMCPT, FWD, and Dynaflect testing, whereas the laboratory program included repeated load triaxial resilient modulus tests and physical properties and compaction tests. Nine overlay rehabilitation pavement projects in Louisiana were selected. A total of four soil types (A-4, A-6, A-7-5, and A-7-6) were considered at different moisture-dry unit weight levels. The results of the laboratory and field testing programs were analyzed and critically evaluated. A comprehensive statistical analysis was conducted on the collected data. The results showed a good agreement between the predicted and measured resilient modulus from the various field test methods considered. The DCP and CIMCPT models were enhanced when the soil moisture content and dry unit weight were incorporated. The results also showed that, among all backcalculated FWD moduli, those backcalculated using ELMOD 5.1.69 software had the best correlation with the measured M_r. Finally, the M_r values estimated using the approach currently adopted by the LADOTD were found to correlate poorly with the measured M_r values.

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Comparative Evaluation of Subgrade Resilient Modulus from Non-destructive, In-situ, and Laboratory Methods

(Final Report)

by

Louay N. Mohammad, Ph.D.
Professor of Civil and Environmental Engineering
Director, Engineering Materials Characterization Research Facility

Kevin Gaspard, P.E.
Research Engineer

Ananda Herath, Ph.D., P.E.
Postdoctoral Researcher

And
Munir D. Nazzal, Ph.D.
Research Associate

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ABSTRACT

The resilient modulus (M_r) is a fundamental engineering material property that describes the non-linear, stress-strain behavior of pavement materials under repeated loading. M_r attribute has been recognized widely for characterizing materials in pavement design and evaluation. The 1986 AASHTO guide for design of pavement structures has incorporated the M_r of subgrade material into the design process. Considerable attention has also been given to it in the design and evaluation of pavement structures in the Mechanistic-Empirical Pavement Design Guide (MEPDG).

Field and laboratory testing programs were conducted to develop models that predict the resilient modulus of subgrade soils from the test results of various test devices, namely, Falling Weight Deflectometer (FWD), Dynamic Deflection Determination (Dynaflect), Continuous Intrusion Miniature Cone Penetrometer (CIMCPT), and Dynamic Cone Penetrometer (DCP). The field testing program included DCP, CIMCPT, FWD, and Dynaflect testing, whereas the laboratory program included repeated load triaxial resilient modulus tests, and physical properties and compaction tests. Nine overlay rehabilitation pavement projects in Louisiana were selected. A total of four soil types (A-4, A-6, A-7-5, and A-7-6) were considered at different moisture-dry unit weight levels. The results of the laboratory and field testing programs were analyzed and critically evaluated. Subsequently, statistical models for predicting the resilient modulus were developed. The results showed a good agreement between the predicted and measured resilient modulus from the various field test methods considered. Two models were developed for the DCP and CIMCPT, namely, a direct model that includes the measurements of these devices and a soil property model that includes the measurements of these devices as well as the physical properties of tested soils. It was noted that the soil property models had a better prediction than the direct models. The results also showed that, among all backcalculated FWD moduli, those backcalculated using ELMOD 5.1.69 software had the best correlation with the measured M_r. Finally, no significant correlation was found between the M_r values estimated using the approach currently adopted by the LADOTD and those measured in the laboratory.

ACKNOWLEDGMENTS

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The effort of William T. Tierney, research specialist/LTRC in conducting the cone penetration tests and soil sampling is appreciated. Shawn Elisar, Glen Gore, Gary Keel, and Mitch Terrel are greatly appreciated for conducting the falling weight deflectometer and Dynaflect tests. The assistance of Amar Raghavendra in getting the MTS system operational for the resilient modulus tests is acknowledged. The help of the geotechnical laboratory staff in conducting various soil tests is also appreciated.

IMPLEMENTATION IN PAVEMENT DESIGN

This report presents the results of a study conducted to develop resilient modulus prediction models of subgrade soils from different in-situ tests, including: FWD, Dynaflect, CIMCPT, and DCP.

The devices considered in this study can be utilized for design, construction, maintenance, research, quality control/quality assurance, and forensic analysis. Each device and method has its assets and liabilities. Practically speaking, the DCP will probably be utilized more by the design, maintenance, and construction sections, simply because of its cost (< $2,500), versatility, maintenance, and ease of use. Currently, only the LADOTD research section, LTRC, owns and operates an FWD, a Dynaflect, and a CIMCPT. It is noted, as of this writing, that the purchasing of a brand new FWD, Dynaflect, and CIMCPT would cost $250,000, $80,000, and $100,000, respectively. The following sections provide a description of the possible implementation of the considered in-situ test devices in the pavement design and analysis procedures.

Dynamic Cone Penetrometer (DCP)

1) **Design of New and Rehabilitated Pavements.** LADOTD currently utilizes the 1993 AASHTO method to design its pavement. One of the factors used to determine the pavement thicknesses is the subgrade resilient modulus. Instead of using the current method, which utilizes an average value for each parish, the subgrade resilient modulus could be determined by testing with the DCP. This would assure that the resilient modulus would be accurately represented for the project. Furthermore, the new Mechanistic-Empirical Pavement Design Guide requires that testing be conducted to utilize level II data for design.
2) **Forensic Analysis of Pavement Failures.** This tool can be utilized to determine the in place soil conditions (resilient modulus or Dynamic Cone Penetrometer Index (mm/blow)) in areas in which pavement failures have occurred. With this information, the design, construction, or maintenance engineer can make an accurate assessment of the soil conditions and develop an appropriate rehabilitation strategy.

Falling Weight Deflectometer (FWD)

The FWD can be utilized with confidence in the design of rehabilitated pavements, as well as for forensic analysis, due to good correlation with laboratory tests provided by this study. It is not a good tool for quality control because it is subject to inaccuracies when testing is conducted directly on soils or unbound base courses, such as stone. It does have the advantage of being able to assess the pavement structure quickly without having to drill holes through the pavement structure, as is required with the DCP.

Dynamic Deflection Determination (Dynaflect)

The Dynaflect can be utilized with confidence in the design of rehabilitated pavements, forensic analysis, and quality control due to good correlation with laboratory tests provided by this study. Unlike the FWD, it can be used for quality control, but the DCP would be a better choice, for reasons previously mentioned.

Continuous Intrusion Miniature Cone Penetrometer (CIMCPT)

CIMCPT can be used in similar situations as the DCP. It is less labor intensive and quicker than the DCP. It has the advantage of being able to go deeper (greater than 25 feet) into the subgrade than the DCP. The CIMCPT is suitable for the site conditions that require a cut. However, it is mounted to a vehicle and thus less versatile and more costly to purchase and maintain than the DCP.

Implementation Presentation and Guidelines

An implementation presentation can be developed and presented to each district to familiarize personnel with the capabilities of each tool. Furthermore, a pavement analysis guideline can be published and distributed within LADOTD. It is recommended that the Engineering Directives and Standard Memo (EDSM) and pavement design manual of LADOTD be revised to incorporate the use of these devices.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

The resilient modulus of pavement materials and subgrades is an important input parameter for the design of pavement structures. Therefore, an accurate measurement of M_r is needed to ensure the efficiency and accuracy of the pavement design. Many studies that were conducted to demonstrate the effects of pavement materials' M_r on the design of pavements showed that the input value of M_r has a dramatic effect on the designed thickness of the base course and asphalt layers.

The resilient modulus of pavement materials is typically determined using the RLT test. However, this test requires well trained personnel and expensive laboratory equipment. In addition, it is considered to be relatively time consuming. Therefore, highway agencies tried to seek different alternatives. Various empirical correlations have been used to determine resilient modulus in the last three decades. The resilient modulus of subgrade soils is related to several parameters, such as the soil support value (SSV), the R-value, the California bearing ratio (CBR), and the Texas triaxial classification value. However, these parameters do not represent the dynamic load behavior under moving vehicles.

To overcome the disadvantages in the subgrade M_r estimation procedures, different in-situ techniques were proposed to determine the M_r of different pavement materials. These techniques are characterized by the ease of operation and their ability to assess the structural integrity and estimate the elastic moduli of in situ pavement layers. They have an additional advantage of being able to assess the pavement structure without destroying it.

This study was initiated to evaluate the use of different in situ testing devices as an alternative for determining the pavement materials M_r through laboratory triaxial tests. For this purpose, field and laboratory testing programs were performed. The field program included conducting CIMCPT, FWD, Dynaflect, and DCP tests on nine pavement projects. In addition to the laboratory repeated load triaxial resilient modulus, physical soil properties tests were performed on samples from the tested sections. Statistical analyses were performed to develop models that predict the resilient modulus measured in the laboratory based on the results obtained from the different in situ testing devices considered.

BACKGROUND

The resilient modulus is a fundamental engineering material property that describes the non-linear stress-strain behavior of pavement materials under repeated loading. It is defined as the ratio of the maximum cyclic stress (σ_{cyc}) to the recoverable resilient (elastic) strain (ε_r) in a repeated dynamic loading. The American Association of State Highway Transportation Officials (AASHTO) 1993 and the MEPDG have adopted the use of resilient modulus of subgrade soils as a material property in characterizing pavements for their structural analysis and design. The MEPDG provided three different levels of input as a means for obtaining the resilient modulus of subgrade materials. The levels are presented in Table 1.

The M_r is typically determined in the laboratory through conducting the Repeated Load Triaxial (RTL) test on representative material samples. Generally, the RLT test requires well trained personnel and expensive laboratory equipment; it is also considered relatively time consuming. Therefore, different state agencies were hesitant to conduct it, and instead used different approaches to estimate the M_r. One of these approaches is the use of empirical correlations with physical properties of tested soils. During the last three decades, various empirical correlations have been proposed and used to predict M_r. Van Til et al [1] related M_r of subgrade soils to the soil support value (SSV) employed in the earlier AASHTO design equation. They also made a correlation chart in which the values of M_r can be determined by the internal friction of the R-value, the CBR, and the Texas triaxial classification value. Many other correlations between M_r, the CBR, the R-value, and soil support values were also developed [2]. The Louisiana Department of Transportation and Development (LADOTD) has historically estimated the M_r of subgrade soils based on the soil support value (SSV) using the following equation:

$$M_r = 1500 + 450\left(\left(\frac{53}{5}\right)(SSV - 2)\right) - 2.5\left(\left(\frac{53}{5}\right)(SSV - 2)\right)^2 \tag{1}$$

where

M_r = resilient modulus and

SSV = soil support value.

The SSV is obtained from a database based on the parish system in Louisiana. Currently, the LADOTD uses a typical M_r value for each parish instead of obtaining subgrade M_r values for each project. This can lead to inaccuracies in the pavement design, since the subgrade M_r can vary from site to site within the parish as well as seasonally. Thus, the use of M_r based on a typical parish value can result in an under design of pavement structure leading to premature pavement failures.

Table 1
Input levels for the M-E design guide *[3]*

Material	Input Level 1	Input Level II	Input Level III
Granular Materials	Measured M_r in laboratory	Estimated M_r from correlations	Default M_r
Cohesive Materials	Measured M_r in laboratory	Estimated M_r from correlations	Default M_r

Another alternative for estimating the M_r of subgrade soils is the use of in situ test devices. Different devices have been proposed and used during the last few decades. The following sections give a brief background of the in situ devices investigated in this study.

CIMCPT Test Device

The CIMCPT is a simple and economical test that provides rapid, continuous, and reliable measurements of the soil physical and strength properties. As shown in Figures 1 and 2, the CIMCPT device consists of a continuous push device, hydraulic motor, miniature cone penetrometer, and data acquisition system. The cone is attached to a coiled push rod, which allows a continuous penetration, and is mechanically straightened as the cone is pushed into the soil. As the miniature cone penetrates into the ground, the tip resistance (q_c) and sleeve friction (f_s) readings are recorded. The penetration resistance is related to the strength of the soil. The tip resistance depends on the size of the cone tip, rate of penetration, types of soil, density, and moisture content.

During the last few decades, the CIMCPT test has gained popularity among other in situ tests in the characterization of subgrade soil, the construction control of embankments, the assessment of the effectiveness of ground modification, and other shallow depth (upper 5 to 10 m) applications *[4]*. Mohammad et al. developed different models for predicting the resilient modulus of coarse and fine soils from the CIMCPT test results *[5-13]*. A summary of these models is presented in Table 2.

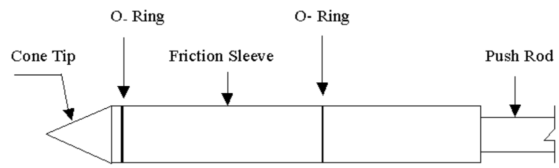


Figure 1
A typical friction cone penetrometer

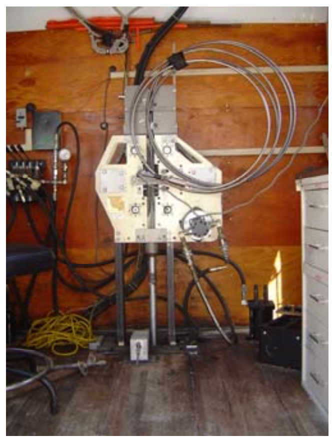


Figure 2
Continuous intrusion miniature cone penetration

FWD Test Device

Based on early work in France during the 1960s, the Technical University of Denmark, the Danish Road Institute, and the Dynatest Group have gradually developed and employed the FWD for use as nondestructive testing of highway and airfield pavements. The FWD is a trailer mounted device that delivers an impulse load to the pavement, as shown in Figure 3. The equipment automatically lifts a weight to a given height. The weight is dropped onto a 300 mm circular load plate with a thin rubber pad mounted underneath. A load cell measures the force or load applied to the pavement under the plate, and the deflections caused by the impulse load are measured by sensors placed at different distances from the center of the load plate. Based on the measured load and deflections of the elastic moduli of the tested pavement, layers can be backcalculated using one of the different softwares available, such as MODULUS, ELMOD or EVERCALC.

Because of its versatility and ease of use, the FWD is becoming the device of choice of highway agencies. The Florida Department of Transportation conducted a survey of the 50 states and three Canadian provinces to assess the current practices of using FWD [14]. Their results indicate that 70 percent of the surveyed agencies use the modulus determined from the FWD data to estimate subgrade strength.

The relation between the moduli obtained from FWD and the laboratory measured resilient modulus was examined in previous studies. Rahim et al [15] suggested that, for different types of cohesive and granular soils, the FWD moduli backcalculated using MODULUS 5.0 software was, on average, identical to the laboratory measured M_r.

Table 2
Summary of CIMPT models developed by Mohammad et al. [10, 11]

Correlation	Comment
$\frac{M_r}{\sigma_c^{0.55}} = \frac{1}{\sigma_v}\left(31.79 q_c + 74.81\frac{f_s}{w}\right) + 4.08\frac{\gamma_d}{\gamma_w}$	Fine grained soil based on the in situ stresses
$\frac{M_r}{\sigma_c^{0.55}} = 6.66\frac{(q_c\sigma_b)}{\sigma_v^{\ 2}} - 32.99\frac{f_s}{q_c} + 0.52\frac{\gamma_d}{(w\gamma_w)}$	Coarse grained soil based on the in situ stresses
$\frac{M_r}{\sigma_3^{0.55}} = 47.03\frac{q_c}{\sigma_1} + 170.40\frac{f_s}{\sigma_1 w} + 1.67\frac{\gamma_d}{\gamma_w}$	Fine grained soil based on the traffic and in situ stresses
$\frac{M_r}{\sigma_3^{0.55}} = 18.95\frac{q_c\sigma_b}{\sigma_1^2} + 0.41\frac{\gamma_d}{\gamma_w w}$	Coarse grained soil based on the traffic and in situ stresses

Note:
M_r- resilient modulus (MPa),
σ_3- minor principal stress (σ_c- confining) (kPa),
σ_1- major principal stress (σ_v- vertical stress) (kPa),
q_c - tip resistance(MPa),
f_s- sleeve friction (MPa),
w- water content (as a decimal),
γ_d- dry unit weight (kN/m^3), and
γ_w- unit weight of water (kN/m^3)
σ_b - bulk stress

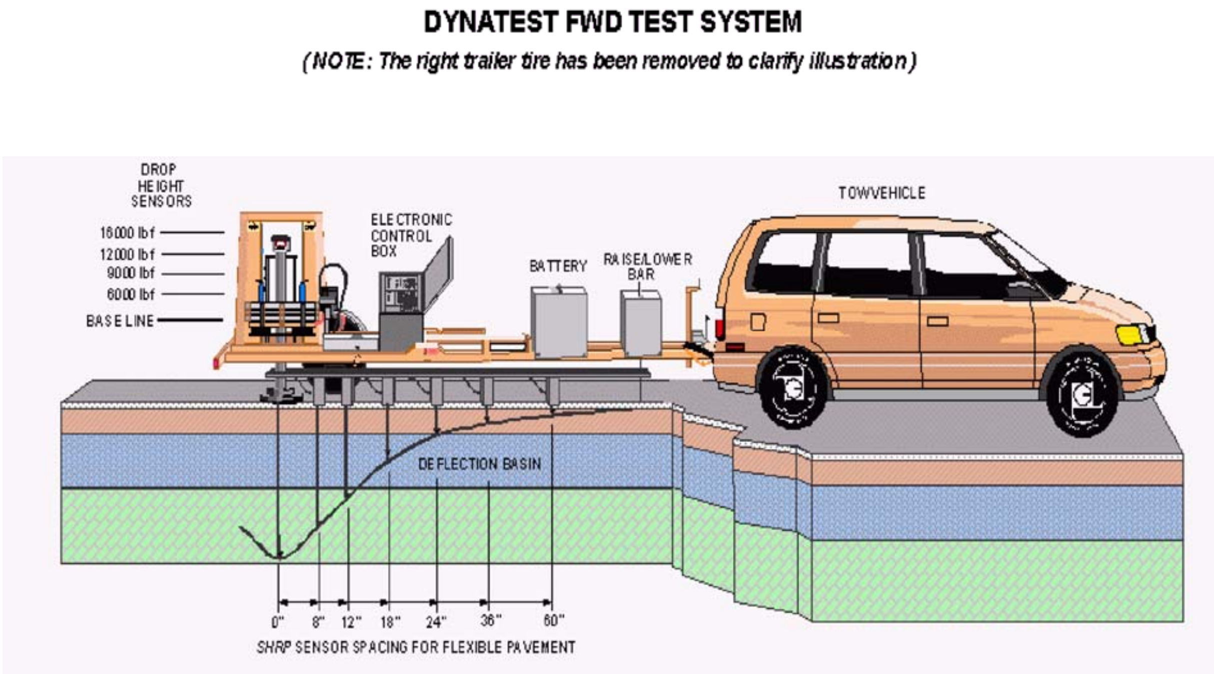


Figure 3
Dynatest Model 8000 (FWD)

Dynaflect Test Device

The Dynamic Deflection Determination (Dynaflect) is an electromagnetic system for measuring the dynamic deflection of a surface or structure caused by an oscillatory load. Measurements are independent of a fixed surface reference. The deflections measured on flexible pavements by the Dynaflect system have been correlated to those obtained by the Benkleman Beam by a number of research groups in highway departments and universities.
The Dynaflect induces a dynamic load on the pavement and measures the resulting deflections using geophones, usually five, spaced under the trailer at approximately 300 mm (1 foot) intervals from the application of the load. The pavement is subjected to 4.45 kN (1000 lbf) of dynamic load at a frequency of 8 Hz, which is produced by two counterrotating, unbalanced flywheels. The cyclic force is transmitted vertically to the pavement through two steel wheels, spaced 508 mm (20 inches) from center to center. The dynamic force during each rotation of the flywheels varies from 4.9 to 9.3 kN (1100 to 2100 lbf).

Figure 4 shows a typical Dynaflect deflection basin. The Dynaflect measures only half of the deflection bowl, while the other half is assumed to be a mirror image of the measured portion. In Figure 4, the measurement W_1 is the maximum depth of the deflection bowl and occurs near the force wheels. The terms W_2, W_3, W_4, and W_5 are the deflections at geophones 2, 3, 4, and 5, respectively.

The maximum deflection, W_1 provides an indication of the relative strength of the total road section. The surface curvature index, SCI (W_1-W_2), provides an indication of the relative strength of the upper (pavement) layers. The base curvature index, BCI (W_4-W_5), and the fifth sensor value W_5 provide a measure of the relative strength of the foundation. For all four parameters, W_1, SCI, BCI, and W_5, lower values indicate greater strength.

To the knowledge of the authors, no research was conducted to correlate the Dynaflect test measurements to the resilient modulus of subgrade soils.

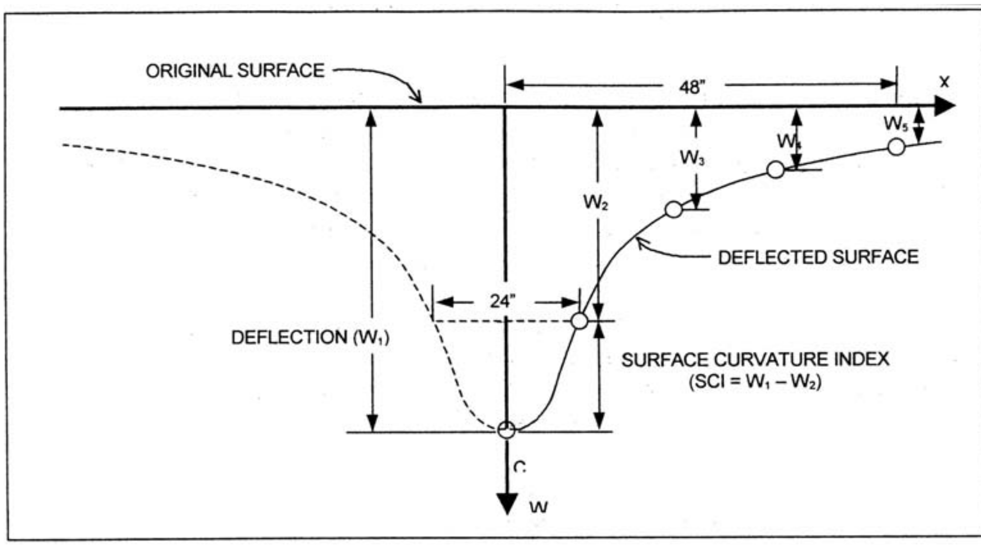


Figure 4
Typical DYNAFLECT deflection basin

DCP Test Device

DCP is a portable instrument that consists of an 8 kg sliding hammer, an anvil, a pushing rod (diameter 16 mm), and a steel cone tip, as shown in Figure 5a. The cone tip angle is 60 degrees, and its diameter is 20 mm. The diameter of the pushing rod is less than that of the cone base. This design assists in reducing the frictional forces along the wall of the cone penetrometer. The DCP test consists of pushing a conical tip, attached to the bottom of the pushing rod, into the soil layer and measuring the resistance to penetration.

DCP tests are designed to estimate the structural capacity of pavement layers and embankments. The DCP has the ability to verify both the level and the uniformity of compaction, which makes it an excellent tool for the quality control of pavement construction. In addition, it can also be used to determine the tested pavement's layer thickness.

During the past decades, the DCP measurement has been correlated to many engineering properties, such as the CBR, shear strength, and elastic modulus. In addition, different models were developed to predict the laboratory measured M_r using DCP test results. A summary of these models is presented in Table 3. The MEPDG software also used the DCP results to estimate the M_r values of different pavement layers by first computing the California bearing ratio (CBR) using the CBR-DCP relation proposed by Webster *[16]* (Equation(2)) and then predicting M_r based on the M_r-CBR relation suggested by Powell et al. *[17]* (Equation(3)). However, since the CBR is estimated using a static test, these types of correlations do not take into account the dynamic behavior of pavements under moving vehicles.

$$CBR = \frac{292}{DCPI^{1.12}} \tag{2}$$

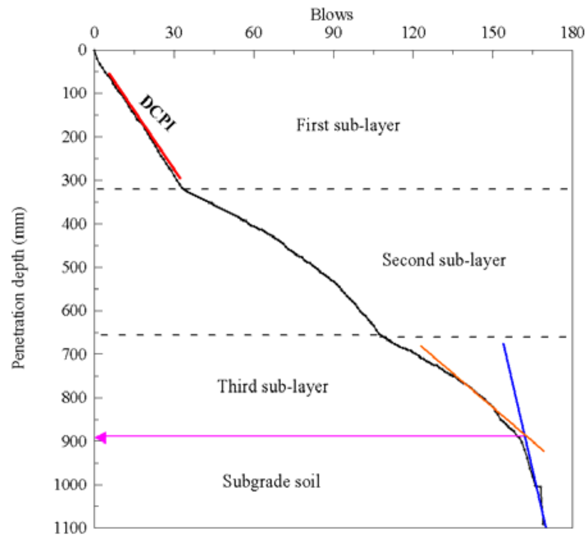
$$M_r = 17.58\,(CBR)^{0.64} \tag{3}$$

where

M_r = resilient modulus in MPa, and

DCPI = penetration index, mm/blow

(a)



(b)

Figure 5
(a) The DCP test (b) A typical DCP profile

Table 3
Mr-DCP correlations reported in Literature

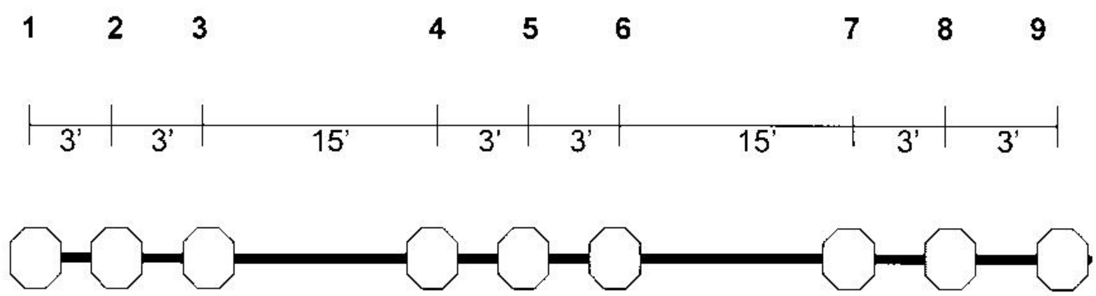
Study	Correlation	Soil type	Comment
Hasan *[18]*	$M_r = 7013.065 - 2040.783\ln(DCPI)$	Cohesive	M_r in psi, DCPI in in/blow
George et al. *[19]*	$M_r = a_o\left(DCPI\right)^{a1}\left(\gamma_{dr}{}^{a2} + \left(LL/w_c\right)^{a3}\right)$	Cohesive	Mr in psi, DCPI in in/blow; W_c is moisture content; LL is Liquid limit ; c_u is coefficient of uniformity; $w_{cr} = \frac{\text{field moisture}}{\text{optimum moisture}}$; and $\gamma_{dr} = \frac{\text{field } \gamma_d}{\text{maximum } \gamma_d}$ a_o,a_1,a_2 and a_3 model coefficients.
	$M_r = a_o(DCPI/\log c_u)^{a1}(w_{cr}{}^{a2} + \gamma_{dr}{}^{a3})$	Granular	

OBJECTIVE

The objective of this research is to develop models that predict the resilient modulus of subgrade soils from the test results of various in situ test devices, namely, DCP, CIMCPT, FWD, and Dynaflect, along with properties of tested soils. The study also evaluates the advantages and limitations for the different in situ devices considered. The results of this study will be used to develop guidelines for the implementation of the measurements of the considered in situ test devices in pavement design procedures including the new Mechanistic-Empirical pavement design method.

SCOPE

Nine pavement projects in Louisiana were selected for field FWD, Dynaflect, CIMCPT, and DCP tests. These projects were LA333, LA347, US171, LA991, LA22, LA28, LA344, LA182, and LA652. Three sets (A, B, and C) of tests were conducted at each pavement project site, as shown in Figure 6. Each testing set was approximately 500-ft apart, unless field conditions dictated otherwise. Each set contained nine points (1 to 9). A total of four soil types (classified as A-4, A-6, A-7-5, and A-7-6, according to the AASHTO soil classification) were considered at different moisture-dry unit weight levels. The DCP tests were performed at points 1, 4, and 7 in a set. The FWD and Dynaflect tests were performed at all nine points in a set. The CIMCPT tests were performed at points 3, 6, and 9 in a set. The field experimental program also included obtaining Shelby tube soil sampling at points 2, 5, and 8. Once testing was completed, subgrade material was augered out of points 2, 3, 5, 6, 8, and 9 and used to perform classification tests. The laboratory experimental program consisted of repeated load triaxial resilient modulus on the Shelby tube specimens. In addition, test results from recently completed research projects were also incorporated in the model development *[10,21]*.



Points 1, 4, 7	DCP tests
Points 2, 5, 8	Shelby tubes
Points 3, 6, 9	Minicone tests
All points	Dynaflect & FWD

Figure 6
Field-testing layout for each set

METHODOLOGY

Field and laboratory testing programs were performed on soils of nine pavement projects in Louisiana. Field testing consisted of conducting FWD, Dynaflect, CIMCPT, and DCP tests. Furthermore, the laboratory program included conducting repeated load triaxial resilient modulus tests and physical properties and compaction tests. Laboratory tests consisted of the determination of resilient modulus and properties of investigated soils. A typical layout of the field testing program is shown in Figure 6. Table 4 presents the test factorial of this study.

Field Testing Program

The following sections present a description of the sites considered in this study. A brief description of the in situ tests and the testing procedures pursued in this study is also provided.

Descriptions of Testing Sites

Both the LADOTD headquarters pavement and geotechnical design engineer and LADOTD district design and water resources engineer sections were consulted to obtain the location of projects that were currently in the design or construction process. These projects encompassed various pavement typical sections and soil conditions and thus allowed representative samples of the soils typically encountered in Louisiana highway construction to be evaluated. Figure 7 presents the locations of each testing site, while the pavement for the projects selected is shown in Figure 8. A brief description of each site is provided below.

Route LA 333. This project is located in Vermillion Parish, and testing was conducted in the northbound lane. Site testing was conducted at locations with minimal cracking to reduce errors in the data collection process, though such locations were difficult to locate. The pavement typical section consisted of 6 inch thick asphalt concrete pavement, 8.5 inch thick soil cement base course, and a clay embankment with a plastic index (PI) ranging from 22 to 26.

Route LA 347. This project is located in St. Landry Parish, and testing was conducted in the southbound lane. Site testing was conducted at locations with minimal cracking to reduce errors in the data collection process. The typical pavement section consisted of 5 inch thick asphaltic concrete pavement, 8.5 inch thick soil cement base course, and a clay subgrade with a PI ranging from 27 to 38.

Route US 171. This project is located in Beauregard Parish, and testing was conducted in the northbound lane. Since the wearing course was scheduled to be placed later, the typical pavement section that was tested consisted of 5 inch thick asphaltic concrete binder course, 10-inch thick crushed stone base course, 12 inch thick cement treated subbase, and a clay subgrade with a PI ranging from 12 to 29.

Table 4
Test Factorial

Project	Site	Lab. M_r (test points)	FWD (test points)	DCP (test points)	CIMCPT (test points)	Dynaflect (test points)	Shelby tubes (test points)
LA333	A	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
	B	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
	C	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
LA347	A	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
	B	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
	C	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
US171	A	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
	B	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
	C	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
LA991	A	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
	B	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
	C	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
LA22	A	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
	B	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
	C	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
LA28	A	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
	B	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
LA344	A	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
	B	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
	C	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
LA182	A	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
	B	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
	C	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
LA652	A	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
	B	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8
	C	2,5,8	1 to 9	1,4,7	3,6,9	1 to 9	2,5,8

Legend: FWD- Falling weight deflectometer, DCP- Dynamic cone penetration, CIMCPT- Continuous intrusion miniature cone penetration test, Lab. M_r -Laboratory measured resilient modulus

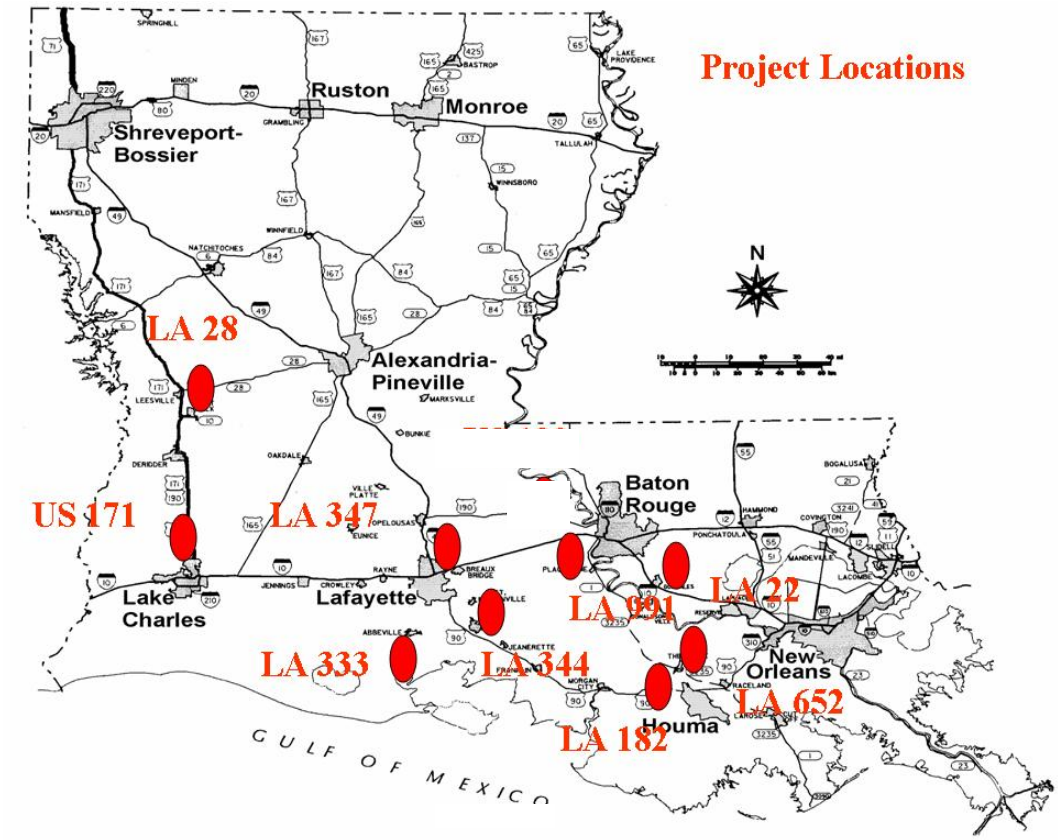


Figure 7
Locations of the pavement projects

Route LA 991. This project is located in Iberville Parish, and testing was conducted on the westbound lane. The typical pavement section consisted of 4 inch thick asphaltic concrete pavement, 12 inch thick soil cement base course, and a clay subgrade with a PI ranging from 13 to 26.

Route LA 22. This project is located in Ascension parish, and testing was conducted on the eastbound lane. The section selected for testing had received a maintenance overlay to repair failed pavement areas. The typical pavement section varied. For site A, the asphaltic concrete was 17inches thick. Sites B and C had an asphaltic concrete pavement thickness of 13 inches. The asphalt concrete thicknesses for each site includes the thickness of the asphaltic concrete wearing, binder, and base course. Each site had a clay subgrade with a PI ranging from 20 to 24.

Route LA 28. This project is located in Vernon Parish, and testing was conducted on the eastbound outside shoulder. The pavement shoulder typical section consisted of 5 inch thick asphaltic concrete pavement and 10.75 inch thick crushed stone. Each site had a clay subgrade with a PI ranging from 43 to 61.

Route LA 344. This project is located in Iberia Parish, and testing was conducted on the eastbound lane. The pavement section consisted of 7.25 to 6 inch thick asphaltic concrete pavement and 7.5 to 7.0 inch thick soil cement base course. Sites A and B had a heavy clay subgrade with a PI ranging from 34 to 39, and Site C had a lean silt subgrade.

Route LA 182. This project is located in Lafourche Parish, and testing was conducted on the eastbound shoulder. The shoulder section was less than two years old and showed no signs of distress. The asphalt pavement thickness varied from 2 to 3 inches, and soil cement base course varied from 8 to 8.25 inches. Each site had a lean clay subgrade with an average PI of 23.

Route LA 652. This project is located in Lafourche Parish, and testing was conducted on the eastbound lane. The asphalt pavement ranged from 3.3 to 3.9 inches, and the soil cement base course ranged from 8.9 to 9.4 inches. Each site had a heavy clay subgrade with a PI ranging from 46 to 50.

Description of Field Tests

A visual survey of each of the tested sites was conducted prior to performing the different field tests. Based on this survey, a testing layout was established. The field testing included using different in situ test devices. A brief description of those tests is presented in the following sections.

FWD Tests

FWD tests were conducted on all nine points for each testing set, as presented in Figure 6. The Dynatest Model 8000 was used in this study to conduct all FWD tests. This device applies a transient load (approximately a half-sinusoidal wave with a loading time between 25 and 40 milliseconds) to the pavement layer by dropping a weight from a specified height on a 300 mm circular loading plate with a thin rubber pad mounted underneath. Different load magnitudes can be generated by varying the mass of weight and drop height. A 9,000-pound load level was used in this study. The pavement deformation induced by the applied load is obtained using sensors (geophones) located at different distances from the center of the load plate. In this study, the deformation was obtained using nine sensors. Based on the measured load and deflections, the elastic moduli of the different tested pavement layers were backcalculated using the different softwares and methods described below.

Florida Equation. The Florida Department of Transportation (FDOT) developed the following equation, known as Florida equation, to determine the subgrade resilient modulus *[21]*:

$$E_{FWD} = 0.03764\left(\frac{P}{d_r}\right)^{0.898}, \tag{4}$$

where

E_{FWD} = subgrade resilient modulus estimated from the FWD results (psi),

P = applied load (pounds), and

d_r = sensor deflection at 36 inches from the load plate [thousands of an inch (mils)].

ELMOD software version 5.1.69 *[22]*. This software was developed by Dynatest International, and it uses the Microsoft Access database for storing data from the field acquisition and backcalculation results. Different input values influence the backcalculated layer moduli values; these include: layer thickness, seed values, max depth to rigid layer, linear, non-linear, radius of curvature fit, and deflection basin fit.

MODULUS software version 6.0 *[23]*.This software was developed by the Texas Transportation Institute (TTI). It is a friendly program that has built in references to assist in the backcalculation process. The backcalculations were performed with semi-infinite subgrade and finite subgrade depths to bedrock models.

EVERCALC software version 5.0 *[24]*. This software was developed by the Washington Department of Transportation (WSDOT). The program uses the WESLEA layered elastic analysis program for forward analysis and a modified Augmented Gauss-Newton algorithm for optimization. It can handle up to 5 layers, 10 sensors, and 12 drops per station.

Dynaflect Tests. Dynaflect tests were conducted at each of the nine points of each tested site. Since the Dynaflect deflections should be corrected for the temperature as well as for other variables, the procedure for determining Dynaflect deflection correction factors, developed by Southgate *[25]*, was utilized to adjust the Dynaflect deflections to a standard temperature of 60^{O} F. The fact that the applicability of the procedure used to the conditions and construction materials in Louisiana was verified in a previous study is worth noting *[26]*.

DCP Tests. DCP tests were conducted on three points in each testing set, as presented in Figure 6. To perform the DCP tests, a one inch diameter hole was first drilled through the asphalt concrete pavement and base course with a Dewalt Rotary hammer drill. The DCP cone was then lowered through the hole and placed on the subgrade. The depth of penetration into the subgrade varied from approximately 24 to 36 inches, depending on site conditions. The field DCP tests were performed according to the American Society for Testing and Materials (ASTM) test procedure, D6951. During a typical DCP test, the penetration depth of DCP for each hammer drop (blow) was recorded and used to plot the DCP profile (blows vs. depth) for the tested soil. The DCPI value was then determined as the slope of that profile.

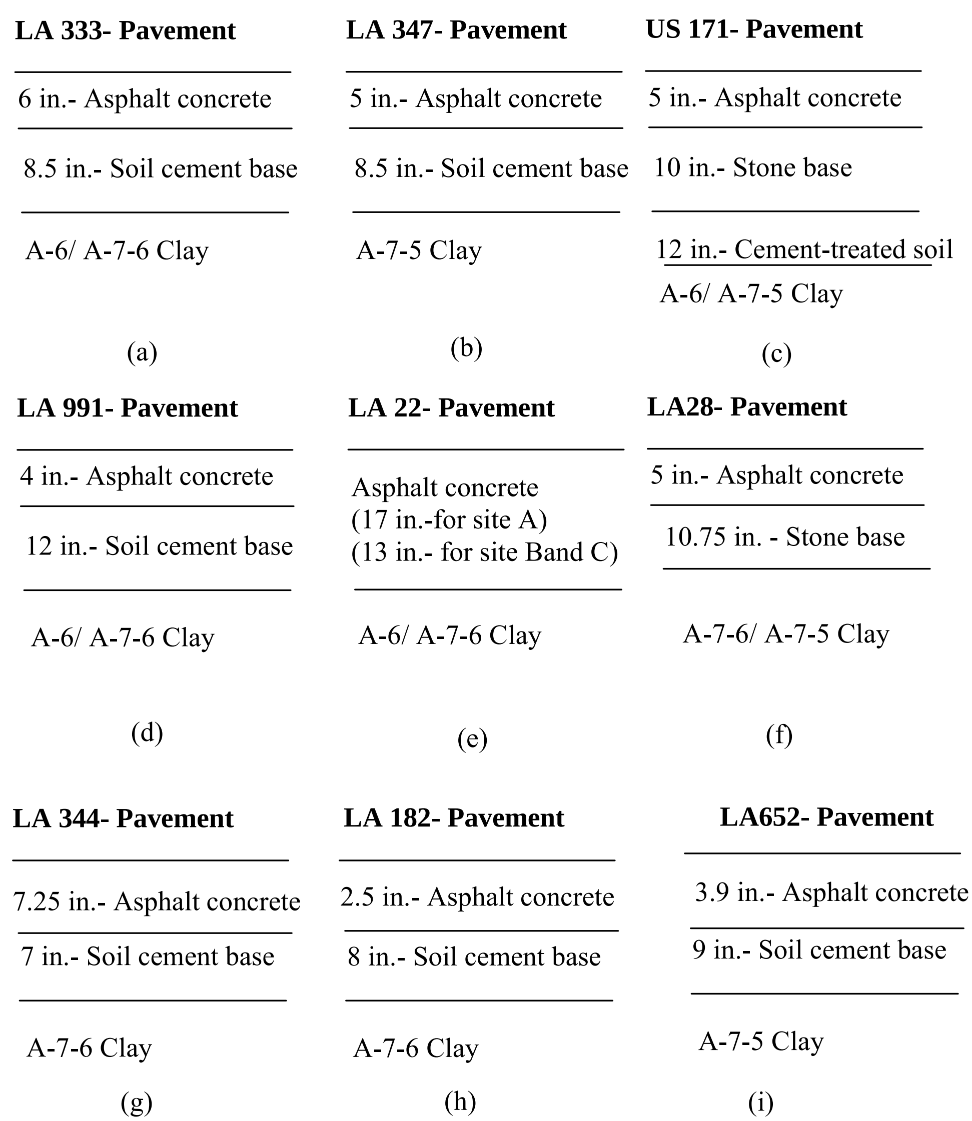


Figure 8
Pavement structures

CIMCPT Tests. CIMCPT tests were conducted on three points in each testing set, as illustrated in Figure 6. The miniature cone penetrometer used in this study had a cross sectional area of 2 cm^2, a friction sleeve area of 40 cm^2, and a cone apex angle of 60 degrees and was attached to a coiled push rod, which replaces the segmental push rods in the standard cones. Prior to conducting the CIMCPT tests, a six inch diameter hole was augured through the asphaltic concrete pavement and base course with a core rig. The six inch diameter hole was augured approximately six inches into the subgrade to ensure that any loose aggregate from the asphaltic concrete or base course was removed from the hole. Once the hole was augered, the cone was advanced into the ground at a rate of 2 cm/sec to a depth of approximately nine feet below the base course with continuous measurements of the tip resistance (q_c) and sleeve friction (f_s).

Shelby Tube Samples. Shelby tube samples were obtained at three points for each test section, as shown in Figure 6. To obtain Shelby tube samples, a six inch diameter hole was first augured with a core rig through the asphaltic concrete layer, and the base course layer and six inches into the subgrade. The core rig was then used to shove the three inch diameter Shelby tube into the subgrade. Although the Shelby tubes were 30 inches long and were fully pushed into the subgrade, only a 5.8-inch long specimen could be obtained from the tube. The obtained specimen was representative of the subgrade soil layer within 6 to 18 inches from the base course, as shown in Figure 9.

Once the tube was removed from the ground, the soil specimen was extracted from the tube using the extrusion device mounted on the truck. The soil specimens were then trimmed and wrapped in plastic and aluminum foil. They were then stored in Styrofoam containers and transported to the LTRC laboratory. The samples were kept in a 95 percent relative humidity-controlled room until they were tested.

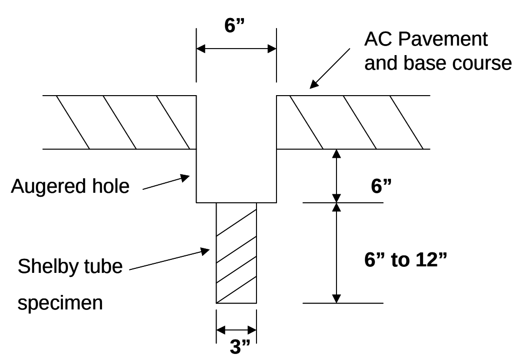


Figure 9
Shelby tube specimen location

Laboratory Testing Program

The laboratory testing program in this study consisted of conducting RLT resilient modulus tests and tests to determine the physical properties of tested soils. The following sections provide a description of these tests.

RLT Resilient Modulus Test

RLT M_r tests were conducted on the 5.6 inches high and 2.8 inches wide specimens obtained from Shelby tube samples collected in the field. All tests were performed using the Material Testing System (MTS) 810 machine with a closed loop and a servo hydraulic loading system. The applied load was measured using a load cell installed inside the triaxial cell. Placing the load cell inside the triaxial chamber eliminates the push-rod seal friction and pressure area errors and results in a reduction in the testing equipment error. An external load cell is affected by changes in confining pressure and load rod friction, and the internal load cell, therefore, gives more accurate readings. The capacity of the load cell used was ± 22.25 kN (±5000 lbf.). The axial displacement measurements were made using two linearly variable differential transducers (LVDT) placed between the top platen and base of the cell to reduce the amount of extraneous axial deformation measured compared to external LVDTs. Air was used as the confining fluid to the specimens. Figure 10 depicts a picture of the testing setup used in this study.

Resilient modulus tests were performed in accordance with AASHTO procedure T 294-94 *[27]* standard method. In this test method, the samples are first conditioned by applying 1,000 load cycles to remove most irregularities on the top and bottom surfaces of the test sample and to suppress most of the initial stage of permanent deformation. The conditioning of the samples is followed by a series of steps consisting of different levels of cyclic deviatoric stress, such that the resilient modulus is measured at varying normal and shear stress levels. The cyclic loading consists of repeated cycles of a haversine shaped load pulse. These load pulses have a 0.1 sec load duration and a 0.9 sec rest period.

Results obtained from the resilient modulus test were used to determine the non-linear elastic coefficients of the generalized constitutive model shown in Equation 5, which were used to determine the resilient modulus values at a field representative stress state.

$$\frac{M_r}{P_a} = k_1 \left(\frac{\theta}{P_a} \right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1 \right)^{k_3}, \tag{5}$$

where

M_r = resilient modulus,

$\theta = \sigma_1 + \sigma_2 + \sigma_3$ = bulk stress,

σ_1 = major principal stress,

σ_2 = intermediate principal stress,

σ_3 = minor principal stress/ confining pressure,

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

P_a = normalizing stress (atmospheric pressure) = 101.35 kPa (14.7 psi), and

k_1, k_2, k_3 = material constants.

Physical Property Tests

Soil property tests were also performed on the Shelby tube samples in accordance with the AASHTO and LADOTD standard test procedures. The tests included: determining moisture-unit weight (standard Proctor curve), Atterberg limits, hydrometer, sieve analysis, and soil classification of soils tested. Table 5 presents a summary of the designation of standard tests that were performed. The in situ dry unit weight (γ_d) and moisture content (w) of tested soils are presented in Table 6. Table 7 shows the physical properties of the soils.

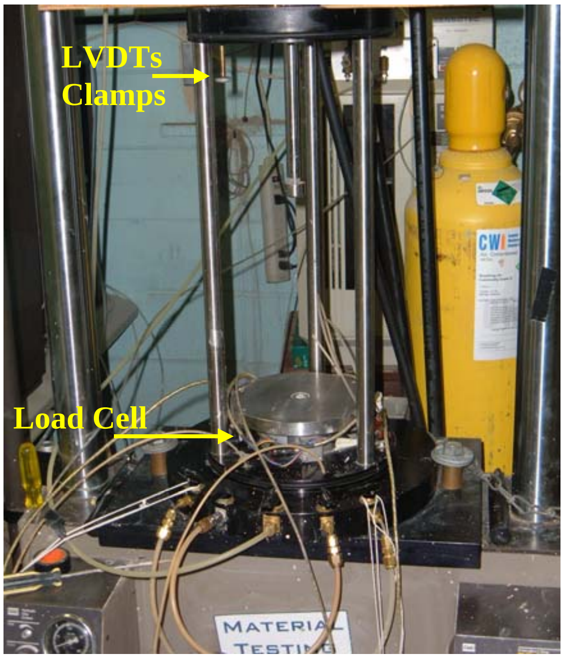


Figure 10
MTS Triaxial Testing Machine

Table 5
Soil classification test procedures

Test	LADOTD	AASHTO
Sample Preparation	TR 411M/411-95	T87-86
Hydrometer	TR 407-89	T88-00
Atterberg Limits	TR 428-67	T89-02, T90-00
Moisture/Density Curves	TR 418-93	T-99-01
Sieve Analysis	TR 113-75	T88-00
Organic Content	TR 413-71	T194-97
Moisture Content	TR 403-92	T 265

Table 6
Dry unit weights and moisture contents of soil tested

Project	Site/Soil ID	Test Point	γ_d (pcf)	w (%)	Project	Site/Soil ID	Test Point	γ_d (pcf)	w (%)
LA333	A	2	97.0	23.3	LA347	A	2	86.3	31.7
		5	94.5	25.2			5	85.7	32.4
		8	102.1	17.8			8	84.4	36.2
	B	2	96.4	21.7		B	2	88.8	30.1
		5	85.7	32.5			5	88.8	30.6
		8	83.8	34.8			8	87.0	32.1
	C	2	93.9	25.0		C	2	80.7	35.9
		5	90.7	23.0			5	79.4	35.9
		8	104.6	17.6			8	76.9	36.6
US171	A	2	93.9	33.6	LA991	A	2	101.4	26.6
		5	95.1	30.9			5	102.1	24.2
		8	101.4	21.8			8	102.7	25.3
	B	2	97.7	25.1		B	2	102.7	25.3
		5	102.7	24.7			5	102.7	26.1
		8	99.6	27.3			8	102.7	25.2
	C	2	114.7	16.9		C	2	102.7	25.1
		5	107.1	16.9			5	102.1	25.4
		8	112.8	15.5			8	102.1	25.6
LA22	A	2	104.0	25.4	LA182	A	2	80.0	32.6
		5	110.3	21.2			5	66.2	47.2
		8	103.3	24.3			8	72.5	31.7
	B	2	102.7	24.5		B	2	76.2	30.9
		5	107.7	20.5			5	112.8	30.8
		8	104.0	25.3			8	66.2	57.7
	C	2	110.3	19.1		C	2	78.1	31.7
		5	104.6	21.8			5	94.5	28.3
		8	107.7	21.0			8	58.0	58.0
LA344	A	2	95.1	23.2	LA652	A	2	80.0	32.6
		5	94.5	27.3			5	66.3	47.2
		8	99.6	24.8			8	72.3	31.7
	B	2	80.7	30.8		B	2	91.0	29.8
		5	95.8	31.0			5	66.0	48.3
		8	84.4	32.6			8	63.1	49.8
	C	2	104.6	24.8		C	2	94.4	28.3
		5	94.5	27.3			5	61.5	54.9
		8	87.0	33.0			8	56.8	60.3
LA28	A	2	106.5	22.0					
		5	97.7	21.3					
		8	107.1	21.6					
	B	2	102.1	21.3					
		5	102.1	21.2					
		8	102.1	20.7					

Legend: w - moisture content, γ_d - dry unit weight

Table 7
Physical properties of soils tested

Site	Passing #200 (%)	Silt (%)	Clay (%)	LL (%)	PI (%)	γ_{dmax} (pcf)	w_{opt} (%)	Soil Classification		
								USCS	AASHTO	Soil Type
LA333 A	95	63	32	37	15	105.0	20.0	CL	A-6	Lean clay
LA333 B	97	58	39	42	20	101.5	20.5	CL	A-7-6	Lean clay
LA333 C	94	55	39	41	15	109.0	18.5	CL	A-7-6	Lean clay
LA347 A	96	53	43	69	38	86.0	28.0	CH	A-7-5	Heavy clay
LA347 B	93	62	31	52	27	93.5	20.2	CH	A-7-5	Heavy clay
LA347 C	95	58	37	67	37	92.0	24.0	CH	A-7-5	Heavy clay
US171 A	72	18	54	46	28	114.0	19.0	CL	A-7-5	Lean clay
US171 B	84	57	27	46	29	115.0	18.5	CL	A-7-5	Lean clay
US171 C	53	30	23	27	12	119.0	12.0	CL	A-6	Lean clay
LA991 A	80	72	8	38	13	105.0	22.0	CL	A-6	Lean clay
LA991 B	89	59	30	39	16	104.0	20.0	CL	A-6	Lean clay
LA991 C	68	24	44	51	26	100.0	21.0	CL	A-7-6	Lean clay
LA22 A	80	50	30	40	23	110.0	17.5	CL	A-6	Lean clay
LA22 B	82	50	32	43	24	109.0	17.0	CL	A-7-6	Lean clay
LA22 C	87	55	32	39	20	109.0	17.0	CL	A-6	Lean clay
LA28 A	76	23	53	62	43	104.3	21.0	CH	A-7-6	Heavy clay
LA28 B	95	9	86	98	61	94.2	27.0	CH	A-7-5	Heavy clay
LA344 A	93	45	48	57	34	97.7	22.7	CH	A-7-6	Heavy clay
LA344 B	95	47	48	52	39	98.5	22.1	CH	A-7-6	Heavy clay
LA344 C	94	56	38	20	3	101.3	21.8	ML	A-4	Lean silt
LA182 A	86	52	34	41	23	105.4	19.1	CL	A-7-6	Lean clay
LA182 B	83	47	36	42	23	107.3	17.1	CL	A-7-6	Lean clay
LA182 C	93	53	40	46	22	104.3	18.4	CL	A-7-6	Lean clay
LA652 A	95	15	80	99	49	86.4	32.8	CH	A-7-5	Heavy clay
LA652 B	96	24	72	91	46	78.5	36.7	CH	A-7-5	Heavy clay
LA652 C	97	15	82	87	50	76.0	36.5	CH	A-7-5	Heavy clay

Legend: AASHTO- American Association of State Highway and Transportation Officials, LL- Liquid limit, PI- Plastic index, USCS- Unified soil classification system, w_{opt}-Optimum moisture content, γ_{dmax} -Maximum dry unit weight

DISCUSSION OF RESULTS

The main focus of this study was to develop models that predict the resilient modulus of subgrade soils from the results of the CIMCPT, DCP, FWD, and Dynaflect test data and predict the physical properties of soil tested. Prior to the development of models, a field representative M_r value was defined.

A comprehensive statistical analysis was conducted using the Statistical Analysis System (SAS) program to develop models that predict the resilient modulus of subgrade soils from the results of various in situ tests devices considered in this study (CIMCPT, DCP, FWD, and Dynaflect test). Direct models that only consider the results from the different types of test devices were developed. In addition, multiple regression models were used to correlate M_r with the measurements obtained from each DCP and CIMCPT test and to determine the physical properties of tested soils.

The development of multiple regression models includes several steps. In the first step, scatter plots between the dependent variable and the independent variables are examined for possible linear correlations. The significance of the linear correlations between any two variables is measured using the Pearson product-moment coefficient of correlation (r). If the value of r is zero or near zero, such indicates that no evidence of an apparent linear correlation is present. If the value of r is positive or negative one, a perfect linear correlation does exist. Based on the results of this step, all possible variables that showed good linear correlation with the dependent variable are examined.

The second step of the development of multiple regression models includes choosing the best model with least number of dependent variables. Different methods are available in selecting the best model. In this study, the stepwise selection method was used. This method fits all possible simple linear models and chooses the best one with the largest F-test statistical value. Then, all possible two-variable models that include the first variable are compared, and so on. The significance of each variable included is rechecked at each step along the way and removed if it falls below the significance threshold.

Based on the results of the variable selection analysis, multiple regression analysis is conducted on the best model selected. To check for its adequacy, examine the significance of independent variables, and detect any multicolinearity (possible correlations among the independent variables) or heteroscedasticity (unequal error variance) problems. The adequacy of the model is assessed using the F-test. The probability associated with the F-test is designated as Pr> F or p-value. A small p-value (less than 0.05) implies that the model is significant in explaining the variation in the dependent variable. The t-test is utilized to examine the significance of each of the independent variables used in the model. Similar to that of the F-test, the probability associated with the t-test is designated with a p-value. A p-value that is less than 0.05 indicates that, at a 95 percent confidence level, the independent variable is significant in explaining the variation of the dependent variable. The multicolinearity is detected using the variance inflation factor (VIF). A VIF factor greater than 10 indicates that weak dependencies may be starting to affect the regression estimates. Finally, the residual plot is used to check for heteroscedasticity by examining whether the data has a certain pattern.

A Field Representative Resilient Modulus Value of Subgrade Soils

A field representative stress condition for subgrade soils consisted of a vertical stress level of 41.3 kPa (6 lbf/in.^{2}) that included a cyclic stress level of 37.2 kPa (5.4 lbf/in.^{2}) and a contact stress level of 4.1 kPa (0.6 lbf/in.^{2}). A confining stress level of 14.0 kPa (2 lbf/in.^{2}) was also considered. These stress levels were selected based on a stress analysis conducted to compute a field representative stress condition in the subgrade layer *[15,18]*. The interpolated M_r was considered as the laboratory measured M_r from the repeated load triaxial test. This stress level also corresponds to the "resilient modulus at the break point" proposed by Thompson et al. *[28]*.

Development of M_r Prediction Models for DCP Test Results

Tables 8 and 9 present the combined DCP and M_r results that were used in developing regression models that predict the laboratory measured M_r from the DCP test results. The fact that Table 9 includes DCP test results from a recently completed project at the LTRC is noted *[20]*. The ranges of variables used in the regression analysis are presented in Table 10. In order to determine the independent variables that should be included in the multiple regression analysis, possible linear correlations between the dependent variable M_r and DCPI, Log (DCPI), 1/DCPI, dry unit weight (γ_d), water content (w), and γ_d/w were first considered. Figures 11 through 16 present the scatter plots between the dependent variable and independent variables. The fact that as the M_r decreases the DCPI increases is noted. Such implies that soil stiffness decreases as the DCPI increases. Therefore, there may be a good linear correlation between the inverse of DCPI and M_r. Figures 14 and 15 demonstrate that the laboratory measured M_r increases with the increase in the dry unit weight and the decrease in the water content. Finally, Figure 16 shows the variation of M_r with the γ_d/w. The fact that M_r increases with a decreasing slope as the γ_d/w increases is noted.
Tables 11 and 12 present the correlation coefficient matrix of all variables for this study. The

fact that the best correlation was found between the M_r and 1/DCPI ($r = 0.87$, p-value <0.001)is noted. In addition, γ_d, w, and γ_d/w were also found to have a significant relation to M_r. Based on this result, the 1/DCPI, γ_d, w, and γ_d/w variables were further used in the stepwise selection analysis.

Table 8

DCP and laboratory M_r test results (this study)

Project	Site/Soil ID	Test Point	Lab. M_r (ksi)	DCPI (mm/blow)	Project	Site/Soil ID	Test Point	Lab. M_r (ksi)	DCPI (mm/blow)
LA333	A	2	6.3	18.8	LA347	A	2	9.0	13.7
		5	4.5	21.5			5	12.7	9.9
		8	5.8	20.7			8	9.1	12.5
	B	2	5.7	21.0		B	2	12.0	11.0
		5	3.8	24.4			5	10.5	12.0
		8	2.7	21.6			8	10.7	11.6
	C	2	3.9	20.0		C	2	8.1	14.0
		5	3.3	24.4			5	7.6	17.8
		8	6.0	18.9			8	8.4	13.9
US171	A	2	2.2	34.4	LA991	A	2	4.4	27.2
		5	3.4	30.5			5	4.3	27.9
		8	3.5	30.8			8	4.4	24.8
	B	2	3.5	30.0		B	2	4.3	25.9
		5	7.2	17.2			5	4.5	26.0
		8	4.5	26.8			8	4.5	26.0
	C	2	13.3	9.6		C	2	3.8	22.0
		5	10.2	12.1			5	3.7	26.9
		8	9.3	12.9			8	3.5	23.0
LA22	A	2	5.8	20.0	LA182	A	2	3.8	34.1
		5	5.7	19.0			5	3.6	38.0
		8	5.6	23.0			8	4.6	28.9
	B	2	5.7	18.0		B	2	3.8	30.1
		5	7.8	14.9			5	5.1	23.4
		8	8.6	13.0			8	4.1	36.8
	C	2	5.6	21.0		C	2	2.8	30.0
		5	5.9	20.0			5	3.4	35.1
		8	5.6	23.0			8	2.7	53.3
LA344	A	2	4.4	21.0	LA652	A	2	1.9	53.4
		5	4.2	24.5			5	1.1	65.2
		8	4.3	24.5			8	2.6	47.0
	B	2	4.5	18.9		B	2	3.1	40.0
		5	4.6	21.4			5	2.7	30.0
		8	4.6	31.3			8	5.6	28.1
	C	2	5.7	18.2		C	2	1.6	60.0
		5	5.5	19.3			5	2.6	42.3
		8	6.0	18.6			8	2.2	46.0
LA28	A	2	4.8	35.3					
		5	4.0	41.0					
		8	4.9	37.0					
	B	2	12.6	9.0					
		5	10.3	12.0					
		8	10.5	13.0					

Legend: DCPI- DCP penetration index, Lab. M_r – Laboratory resilient modulus measured at a cyclic stress level of 37.2 kPa (5.4 $lbf/in.^2$), contact stress level of 4.1 kPa (0.6 $lbf/in.^2$), and confining pressure of 14 kPa (2 $lbf/in.^2$)

Table 9
DCP and laboratory M_r test results [20]

Type of Material	Soil ID	Location	γ_d (pcf)	W (%)	Lab. M_r (ksi)	DCPI (mm/blow)
Clay	Clay-1	Lab	110.9	11.0	10.4	17.0
	Clay-2	Lab	117.8	12.5	12.0	16.7
	Clay-3	Lab	104.6	14.6	8.3	23.0
	Clay-4	Lab	117.2	13.9	12.1	13.0
	Clay-5	Lab	95.8	8.4	9.7	18.4
	Clay-6	Lab	106.5	9.4	10.1	15.0
	Clay-7	Lab	109.6	13.3	10.2	22.5
Clayey Silt	Clayey Silt-1	Lab	101.4	19.0	7.0	26.1
	Clayey Silt-2	Lab	100.2	15.4	9.7	18.8
	Clayey Silt-3	Lab	100.8	20.1	7.2	27.0
	Clayey Silt(ALF)	Field	104.0	18.5	6.2	29.0
Clay	LA-182	Field	100.2	21.1	5.6	36.0
	US-61	Field	100.8	15.6	9.0	10.2
*Clay	ALF 4	Field	102.1	23.6	5.3	24.2

Legend: DCPI- DCP penetration index, Lab. M_r – Laboratory resilient modulus measured at a cyclic stress level of 37.2 kPa (5.4 lbf/in.^{2}), contact stress level of 4.1 kPa (0.6 lbf/in.^{2}), and confining pressure of 14 kPa (2 lbf/in.^{2}), w - moisture content, γ_d - dry unit weig

Table 10
Ranges of variables of subgrade materials used in DCP model development

Property	Range for A-4 soils	Range for A-6 soils	Range for A-7-5 soils	Range for A-7-6 soils
No. of samples	6	26	45	15
M_r (ksi)	5-10	4-14	1-14	3-9
DCPI (mm/blow)	19-36	10-28	9-65	13-41
PI (%)	4-6	12-23	27-61	15-43
γ_d (pcf)	100-104	96-118	57-113	84-108
w (%)	15-24	8-27	21-60	18-35
LL (%)	22-28	27-40	46-98	41-62
Sand (%)	7-58	11-35	4-28	3-32
Silt (%)	28-72	37-72	9-62	23-58
Clay (%)	14-23	8-32	27-86	32-53
Passing sieve #200 (%)	42-93	65-89	72-96	68-97

Legend: M_r – Resilient modulus, DCPI- DCP penetration index, PI- Plasticity index, w- Water content, LL-Liquid limit, Silt- Percentage of silt, Clay- Percentage of clay, γ_d- Dry unit weight

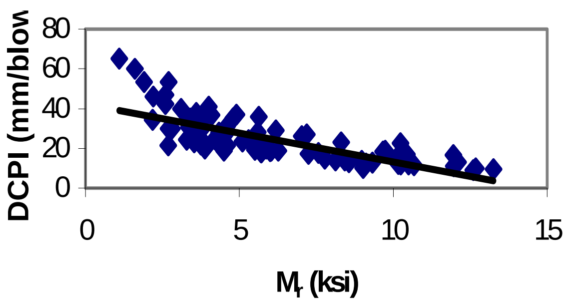


Figure 11
Variation of M_r with DCPI

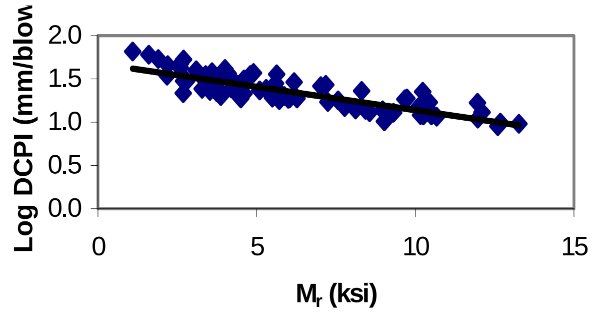


Figure 12
Variation of M_r with Log (DCPI)

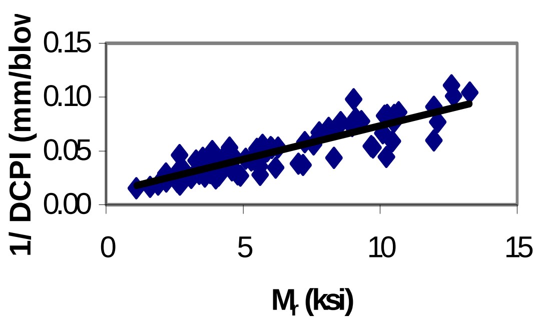


Figure 13
Variation of M_r with 1/DCPI

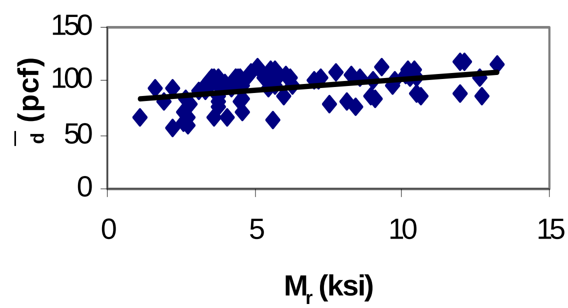


Figure 14
Variation of M_r with γ_d

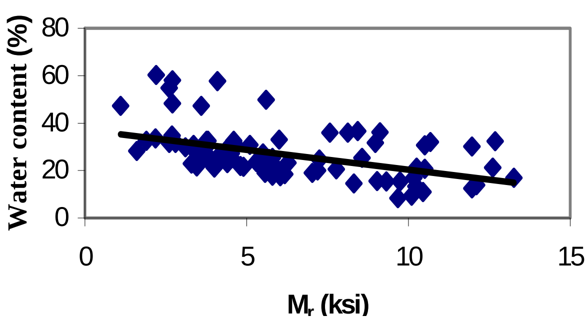


Figure 15 Variation of M_r with water content

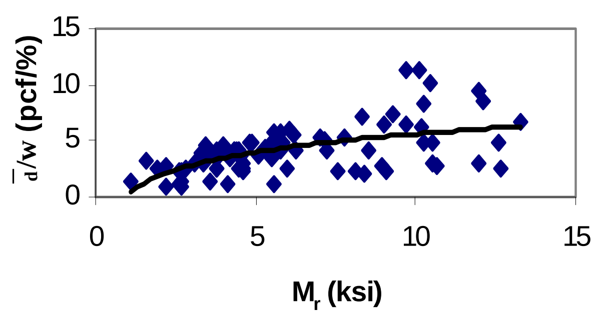


Figure 16 Variation of M_r with γ_d/w

Table 11
A correlation matrix for the DCP test results (p-value)

	γ_d	w	M_r	DCPI	γ_d /w	#200	%Silt	%Clay	LL	PI	Log (DCPI)	1/DCPI
γ_d	-	<0.001	<0.001	<0.001	<0.001	<0.001	0.32	<0.001	<0.001	<0.001	<0.001	<0.001
w	<0.001	-	<0.001	<0.001	<0.001	<0.001	0.28	<0.001	<0.001	<0.001	<0.001	<0.001
M_r	<0.001	<0.001	-	<0.001	<0.001	0.24	0.44	0.009	0.09	0.21	<0.001	<0.001
DCPI	<0.001	<0.001	<0.001	-	<0.001	0.15	0.98	0.40	0.05	0.004	<0.001	<0.001
γ_d /w	<0.001	<0.001	<0.001	<0.001	-	<0.001	0.81	<0.001	<0.001	<0.001	<0.001	<0.001
-# 200	<0.001	<0.001	0.24	0.15	<0.001	-	0.006	<0.001	<0.001	<0.001	0.19	0.22
%Silt	0.32	0.28	0.44	0.98	0.81	0.006	-	<0.001	<0.001	<0.001	0.03	0.38
%Clay	<0.001	<0.001	0.009	0.40	<0.001	<0.001	<0.001	-	<0.001	<0.001	0.003	0.10
LL	<0.001	<0.001	0.09	0.05	<0.001	<0.001	<0.001	<0.001	-	<0.001	0.03	0.042
PI	<0.001	<0.001	0.21	0.004	<0.001	<0.001	<0.001	<0.001	<0.001	-	0.10	0.68
Log (DCPI)	<0.001	<0.001	<0.001	<0.001	<0.001	0.19	0.03	0.003	0.03	0.10	-	<0.001
1/DCPI	<0.001	<0.001	<0.001	<0.001	<0.001	0.22	0.38	0.10	0.42	0.68	<0.001	-

Legend: DCPI- Dynamic cone penetration index, γ_d- Dry unit weight, w- water content, PI- Plasticity index, LL- Liquid limit, #200- Percent passing #200 sieve, %Silt- Percentage of silt, and %Clay- Percentage of clay

Table 12
A correlation matrix for the DCP test results (r-value)

	γ_d	w	M_r	DCPI	γ_d /w	#200	%Silt	%Clay	LL	PI	Log (DCPI)	1/DCPI
γ_d	1.00	-0.89	0.42	-0.49	0.75	-0.52	0.10	-0.45	-0.49	-0.42	-0.43	0.34
w	-0.89	1.00	-0.48	0.50	-0.86	0.49	-0.11	0.44	0.48	0.43	0.45	0.36
M_r	0.42	-0.48	1.00	-0.76	0.56	-0.14	0.08	-0.27	-0.18	-0.13	-0.85	0.87
DCPI	-0.49	0.50	-0.76	1.00	-0.42	0.15	-0.004	-0.10	-0.24	0.29	0.96	-0.85
γ_d /w	0.75	-0.86	0.56	-0.42	1.00	-0.62	-0.03	-0.40	-0.47	-0.42	-0.39	0.33
-# 200	-0.52	0.49	-0.14	0.15	-0.62	1.00	0.29	0.40	0.46	0.37	0.14	-0.13
%Silt	0.10	-0.11	0.08	-0.004	-0.03	0.29	1.00	-0.76	-0.60	-0.64	-0.22	0.09
%Clay	-0.45	0.44	-0.27	-0.10	-0.40	0.40	-0.76	1.00	0.88	0.86	-0.31	-0.17
LL	-0.49	0.48	-0.18	-0.24	-0.47	0.46	-0.60	0.88	1.00	0.95	0.23	-0.09
PI	-0.42	0.43	-0.13	0.29	-0.42	0.37	-0.64	0.86	0.95	1.00	0.17	-0.04
Log (DCPI)	-0.43	0.45	-0.85	0.96	-0.39	0.14	-0.22	0.31	0.23	0.17	1.00	-0.97
1/DCPI	0.34	0.36	0.87	-0.85	0.33	-0.13	0.09	-0.17	-0.09	-0.04	-0.97	1.00

Legend: DCPI- Dynamic cone penetration index, γ_d- Dry unit weight, w- water content, PI- Plasticity index, LL- Liquid limit, #200- Percent passing #200 sieve, %Silt- Percentage of silt, and %Clay- Percentage of clay

Table 13 presents a summary of the results of the analysis. The fact that the best prediction model should include only 1/DCPI and γ_d/w variables can be noted. In addition, the 1/DCPI variable had a much higher partial R-square than the γ_d/w variable, which suggests that it has a greater influence on the model prediction. In an effort to demonstrate the effectiveness of the selection analysis, a multiple regression analysis was conducted on a model that includes 1/DCPI, γ_d, w, and γ_d/w as independent variables. Table 14 presents the results of this analysis. The fact that the 1/DCPI and γ_d/w are the only significant variables ($Pt<0.05$); these are compatible with the results of the variable selection analysis can be noted.

A simple linear regression analysis was conducted in an effort to develop a model that directly predicts the laboratory measured M_r from the 1/DCPI value. The results of this analysis yielded the model shown in Equation 6, which will be referred to as the direct model. The model had a coefficient of determination, R^2, value of 0.91 and root square error, RMSE, value of 0.88 ksi. Figure 17 illustrates the results of regression analysis. The fact that the proposed model fits the data may be observed. Figure 17 also shows the 95 percent prediction interval. The 95 percent prediction interval is considered as a measure of the accuracy of the M_r values predicted using the model developed. The fact that 95 percent of the data points fall within the boundaries of this interval may be noted.

$$M_r = \frac{151.8}{(DCPI)^{1.096}} \tag{6}$$

where
M_r = Resilient modulus (ksi), and

DCPI = Dynamic cone penetration index (mm/blow).

In the absence of uniform soil properties along a soil layer, a direct relationship between the resilient modulus and DCPI is useful. A correlation among resilient modulus, soil properties, and DCPI may also be useful in examining the effect of soil properties on the DCPI predicted M_r values. Therefore, a multiple regression analysis was also conducted to develop a model that predicts laboratory measured M_r from the 1/DCPI and the physical properties of the tested soils, which will hereafter be referred to as the soil-property model. The independent variables that were used in the multiple regression analysis were 1/DCPI and γ_d/w, which were selected based on the stepwise selection analysis (Table 13). Table 15 shows the results of the multiple regression analysis. It is noted that both variables (1/DCPI and γ_d/w) are significant at a 95 percent confidence level. In addition, those variables have a VIF value close to 1, which indicates that these variables are not collinear. Figure 18 presents the residual plot of the DCP- soil property model. There is no distinct pattern among the residuals; this rules out the model heteroscedasticity.

Table 13
Summary of stepwise selection

Variable Entered	Variable Removed	Number of Variables In	Partial R-Square	Model R-Square	F Value	Pr > F
1/DCPI		1	0.794	0.794	338.98	<.0001
γ_d/w		2	0.082	0.876	56.74	<.0001

Table 14
Summary of multiple regression analysis for variable selection

Variable	Parameter Estimate	t Value	Pr > |t|
Intercept	0.62	0.27	0.7857
1/ DCPI	220.63	21.30	<.0001
γ_d	0.024	-1.48	0.1422
w	-0.027	0.93	0.3528
γ_d/w	0.66	6.57	<.0001

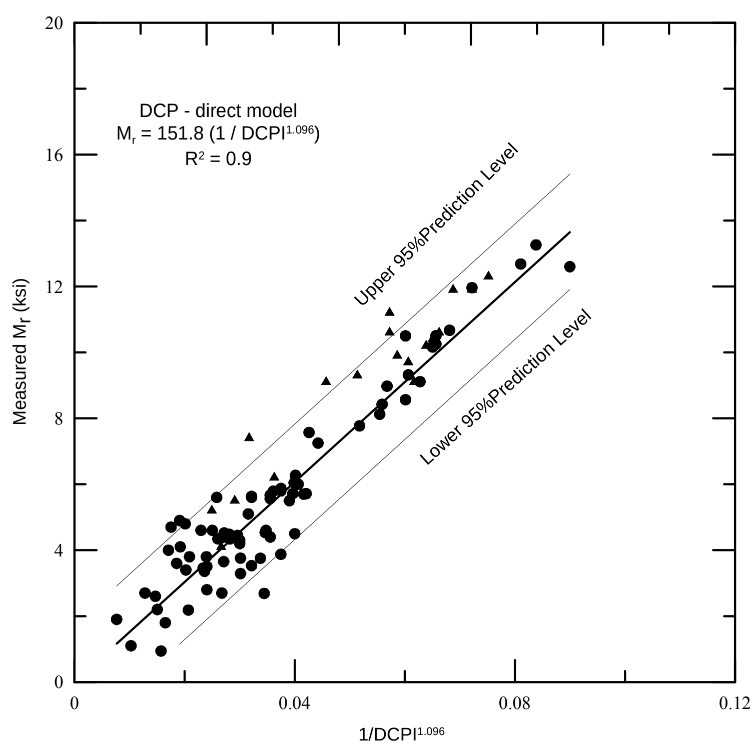


Figure 17
Variation of laboratory measured M_r with 1/DCPI

Table 15
Results of Analysis of DCP- Soil Property Model

Variable	DF	Parameter Estimate	t Value	Pr > |t|	Standardized Estimate	VIF
$1/DCPI^{1.147}$	1	165.5	17.56	<.0001	0.77	1.12
$\left(\frac{\gamma_d}{w}\right)$	1	0.0966	6.89	<.0001	0.30	1.12

$$M_r = 165.5\left(\frac{1}{DCPI^{1.147}}\right) + 0.0966\left(\frac{\gamma_d}{w}\right)$$

where,
M_r –Resilient modulus (ksi),
DCPI – Dynamic cone penetration index (mm/blow),
γ_d –Dry unit weight (pcf), and
w – Water content (%).

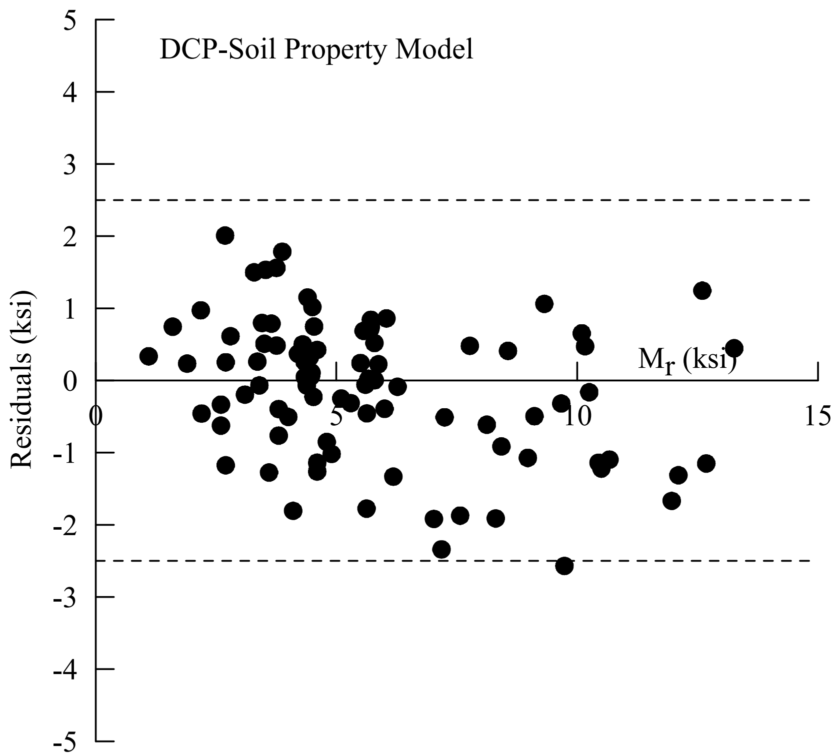


Figure 18
Residuals from DCP-Soil Property Model

Figure 19 shows the M_r predicted by the DCP soil property model versus the M_r measured in the laboratory. The fact that a good agreement was obtained between the predicted and measured values with (R^2=0.92 and RMSE=0.86) may be observed. Furthermore, the model was able to provide a good prediction of the data obtained from a study reported by George et al. *[11]* (Appendix A, Table A1) that was not used in the development of the model.

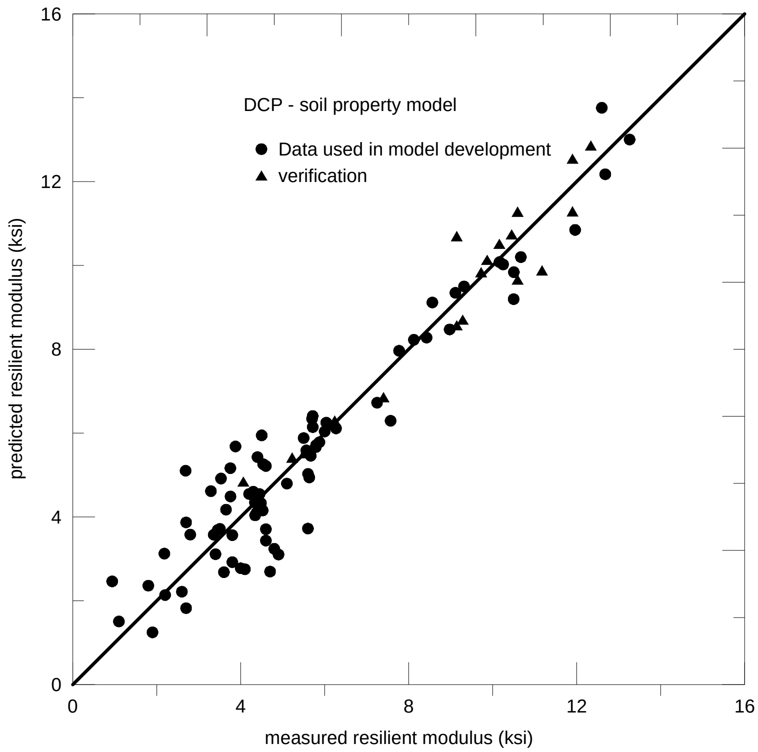


Figure 19
Laboratory measured M_r vs. values predicted from DCP-soil property model

Development of M_r Prediction Models for CIMCPT Test Results

A statistical analysis was performed on the CIMCPT and M_r test results shown in Tables 16 and 17 to develop models that predict the M_r from the CIMCPT test results. The models were developed for fine grained soils using test results of LA333, LA347, US71, LA991, LA22, LA28, LA344 and data from a previous LTRC project *[10]*. The CIMCPT and M_r test results from the field test were used to develop the models. The ranges of variables are presented in Table 18. The variation of the dependent variable M_r and tip resistance (q_c), sleeve friction (f_s), γ_d, w, γ_d/w, plasticity index (PI), liquid limit (LL), percent passing #200 sieve (#200), percentage of silt (%Silt), and percentage of clay (%Clay) are presented in figures 22 through 26.

Table 16
CIMCPT and Laboratory M_r test results for this study (this study)

Project	Site	Test Point	Lab. M_r (ksi)	q_c (ksi)	f_s (ksi)	Project	Site	Test Point	Lab. M_r (ksi)	q_c (ksi)	f_s (ksi)
LA333	A	2	6.3	0.7025	0.0022	LA347	A	2	9.0	1.5791	0.0421
		5	4.5	1.0450	0.0058			5	12.7	1.2322	0.0464
		8	5.8	0.9289	0.0131			8	9.1	1.3803	0.0377
	B	2	5.7	0.2525	0.0102		B	2	12.0	2.6372	0.0058
		5	3.8	0.2308	0.0029			5	10.5	1.0726	0.0639
		8	2.7	0.4340	0.0087			8	10.7	1.4020	0..0276
	C	2	3.9	0.5225	0.0169		C	2	8.1	1.3208	0.0581
		5	3.3	0.3324	0.0203			5	7.6	1.4804	0.0377
		8	6.0	0.6894	0.0305			8	8.4	1.5530	0.0479
US171	A	2	2.2	0.1829	0.0131	LA991	A	2	4.4	0.0871	0.0087
		5	3.4	0.2322	0.0087			5	4.3	0.0929	0.0087
		8	3.5	0.2119	0.0160			8	4.4	0.1248	0.0087
	B	2	3.5	0.2627	0.0160		B	2	4.3	0.1176	0.0087
		5	7.2	0.2671	0.0160			5	4.5	0.1205	0.0102
		8	4.5	0.2656	0.0174			8	4.5	0.1089	0.0102
	C	2	13.3	1.9013	0.0581		C	2	3.8	0.1176	0.0116
		5	10.2	1.4340	0.0377			5	3.7	0.0987	0.0102
		8	9.3	1.2627	0.0348			8	3.5	0.1350	0.0131
LA22	A	2	5.8	0.4296	0.0189	LA344	A	2	4.4	0.3512	0.0290
		5	5.7	0.6168	0.0145			5	4.2	0.3672	0.0290
		8	5.6	0.7983	0.0203			8	4.3	0.3643	0.0174
	B	2	5.7	0.8520	0.0247		B	2	4.5	0.3614	0.0363
		5	7.8	1.0015	0.0348			5	4.6	0.4165	0.0203
		8	8.6	1.3716	0.0435			8	4.6	0.6430	0.0261
	C	2	5.6	0.4296	0.0189		C	2	5.7	0.2743	0.0392
		5	5.9	1.0552	0.0160			5	5.5	0.8665	0.0290
		8	5.6	0.7663	0.0174			8	6.0	1.1248	0.0044
LA28	A	2	4.8	0.5065	0.0174						
		5	4.0	0.4049	0.0116						
		8	4.9	0.4383	0.0145						
	B	2	12.6	1.3077	0.0305						
		5	10.3	1.3077	0.0305						
		8	10.5	1.7605	0.0421						

Legend: f_s- Sleeve friction, Lab. M_r – Laboratory resilient modulus measured at a cyclic stress level of 37.2 kPa (5.4 lbf/in.^{2}), contact stress level of 4.1 kPa (0.6 lbf/in.^{2}), and confining pressure of 14 kPa (2 lbf/in.^{2}), q_c- Tip resistance

Table 17
CIMCPT and Laboratory M_r test results *[10]*

Site	Soil ID	γ_d (pcf)	W (%)	Lab. M_r (ksi)	q_c (ksi)	f_s (ksi)
PRF-silty clay	1	100.2	25.4	4.0	0.3628	0.0096
	2	104.0	23.0	4.3	0.4644	0.0104
	3	105.9	20.8	4.4	0.3904	0.0131
	4	106.5	23.2	4.5	0.4093	0.0106
	5	107.1	21.5	5.5	0.4572	0.0134
PRF-heavy clay	1	62.4	61.6	0.6	0.0406	0.0027
	2	62.4	65.1	0.6	0.0450	0.0029
	3	64.3	60.4	0.8	0.0464	0.0033
	4	63.0	62.5	1.5	0.0581	0.0033
	5	64.3	59.0	0.9	0.0566	0.0027
	6	64.9	59.5	1.4	0.0552	0.0026
I-10/ LA-42 clay	1	106.5	21.5	4.2	0.3019	0.0151
	2	108.4	19.6	3.4	0.2729	0.0163
	3	104.0	23.0	1.9	0.1640	0.0081
	4	102.7	21.4	2.9	0.2917	0.0173
	5	105.9	20.8	3.4	0.2642	0.0137
	6	103.3	22.5	1.8	0.1800	0.0090
LA-15 clay	1	109.0	24.1	6.9	0.4136	0.0219
	2	102.1	23.0	4.7	0.3019	0.0166
	3	105.9	28.4	6.5	0.3004	0.0179
	4	96.4	27.3	5.2	0.3106	0.0140
	5	112.2	18.8	8.8	0.4456	0.0195
	6	96.8	31.4	3.6	0.2975	0.0159
LA-89 clay	1	114.0	24.9	4.8	0.2525	0.0144
	2	101.4	26.8	2.3	0.1974	0.0156
	3	100.2	28.6	1.4	0.0726	0.0090
	4	107.7	24.6	2.8	0.2598	0.0151
Siegen Lane clay	1	115.3	9.5	8.5	0.4499	0.0180
	2	107.7	22.5	3.9	0.1916	0.0226
	3	107.7	16.7	10.3	0.4877	0.0165
	4	97.0	23.1	3.6	0.2337	0.0152

Legend: f_s- Sleeve friction, Lab. M_r – Laboratory resilient modulus measured at a cyclic stress level of 37.2 kPa (5.4 lbf/in.^{2}), contact stress level of 4.1 kPa (0.6 lbf/in.^{2}), and confining pressure of 14 kPa (2 lbf/in.^{2}), q_c- Tip resistance, w - moisture content, γ_d - dry unit weight

Table 18
Ranges of variables of subgrade materials used in CIMCPT model development

Property	Range for A-4 soils	Range for A-6 soils	Range for A-7-5 soils	Range for A-7-6 soils
No. of samples	8	26	18	39
Lab. M_r (ksi)	6-8	2-14	2-14	1-11
q_c (ksi)	0.4-0.5	0.1-1.9	0.2-2.6	0.04-1.4
f_s (ksi)	0.0096-0.0134	0.0022-0.0581	0.0058-0.0639	0.0026-0.0435
PI (%)	<6	11-23	27-61	15-66
γ_d (pcf)	100-107	94-115	77-103	62-112
w (%)	21-25	9-29	21-37	18-65
LL (%)	28	27-40	46-98	41-93
Sand (%)	7	11-35	4-28	2-32
Silt (%)	70	30-72	9-62	14-58
Clay (%)	23	8-32	27-86	32-84
Passing sieve #200 (%)	93	65-89	72-96	68-98

Legend: Lab. M_r – Laboratory resilient modulus measured at a cyclic stress level of 37.2 kPa (5.4 lbf/in.^{2}), contact stress level of 4.1 kPa (0.6 lbf/in.^{2}), and confining pressure of 14 kPa (2 lbf/in.^{2}), PI- Plasticity index, w- Water content, LL- Liquid limit, Silt- Percentage of silt, Clay- Percentage of clay, γ_d- Dry unit weight, q_c - Tip resistance, f_s - Sleeve friction

Figures 20 and 21 show the variation of M_r with the tip resistance and sleeve friction, respectively. As the tip resistance and sleeve friction increase, the resilient modulus of subgrade soils increases. This implies that soil stiffness increases as the tip resistance and sleeve friction increase. Furthermore, this also indicates that there may be a good correlation between M_r and both the tip resistance and sleeve friction. Figure 22 shows the variation of M_r with the γ_d/w. As the γ_d/w increases, the M_r increases with a decreasing slope. Therefore, there may be a correlation between the γ_d/w and M_r.

The correlation coefficient matrix of different variables is presented in Tables 19 and 20. A good linear correlation between M_r and (q_c) tip resistance and M_r and (f_s) sleeve friction is observed with r = 0.82 and r = 0.70, respectively. Such is expected, as the cone's tip resistance and sleeve friction measure the shear strength and frictional resistance of soils, respectively, both of which are known to significantly affect the soil stiffness.

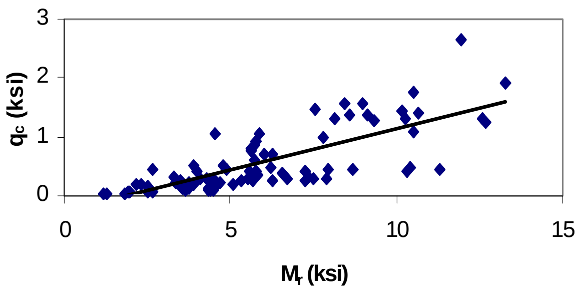


Figure 20
Variation of M_r with q_c

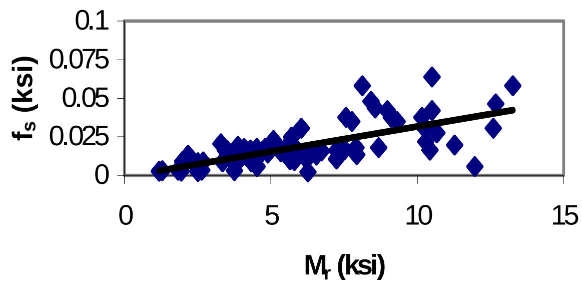


Figure 21
Variation of M_r with f_s

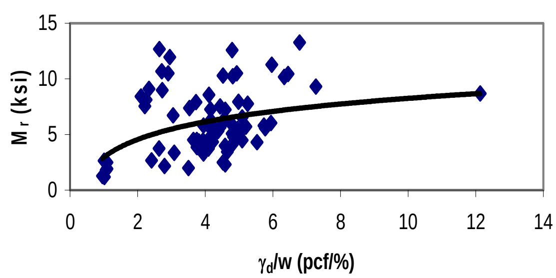


Figure 22
Variation of M_r with γ_d/w

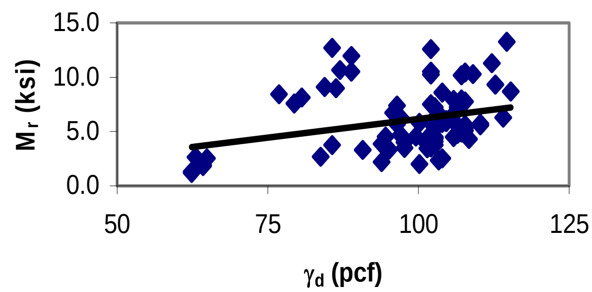


Figure 23
Variation of M_r with γ_d

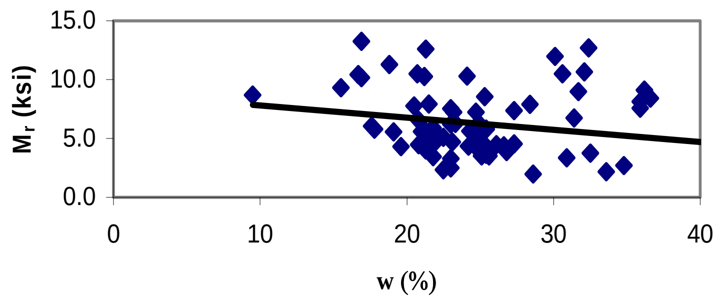


Figure 24
Variation of M_r with w

Table 19
A correlation matrix for the CIMCPT test results (p-value)

	γ_d	w	M_r	q_c	f_s	γ_d/w	#200	%Silt	%Clay	LL	PI
γ_d	-	<0.0001	0.01	0.95	0.42	<0.0001	<0.0001	0.0013	<0.0001	<0.0001	<0.0001
w	<0.0001	-	0.0001	0.11	0.04	<0.0001	0.0016	<0.0001	<0.0001	<0.0001	<0.0001
M_r	0.01	0.0001	-	<0.0001	<0.0001	0.001	0.26	0.39	0.12	0.92	0.38
q_c	0.95	0.11	<0.0001	-	<0.0001	0.46	0.38	0.91	0.53	0.48	0.89
f_s	0.42	0.04	<0.0001	<0.0001	-	0.26	0.23	0.93	0.42	0.64	0.91
γ_d/w	<0.0001	<0.0001	0.001	0.46	0.26	-	0.002	0.05	<0.0001	<0.0001	<0.0001
-#200	<0.0001	0.0016	0.26	0.38	0.23	0.002	-	0.004	0.008	0.006	0.06
%Silt	0.0013	<0.0001	0.39	0.91	0.93	0.05	0.004	-	<0.0001	<0.0001	<0.0001
%Clay	<0.0001	<0.0001	0.12	0.53	0.42	<0.0001	0.008	<0.0001	-	<0.0001	<0.0001
LL	<0.0001	<0.0001	0.92	0.48	0.64	<0.0001	0.006	<0.0001	<0.0001	-	<0.0001
PI	<0.0001	<0.0001	0.38	0.89	0.91	<0.0001	0.06	<0.0001	<0.0001	<0.0001	-

Legend: q_c- Tip resistance, f_s- Sleeve friction, γ_d- Dry unit weight, w- water content, PI- Plasticity index, LL- Liquid limit, #200- Percent passing #200 sieve, %Silt- Percentage of silt, and %Clay- Percentage of clay

Table 20
A correlation matrix for the CIMCPT test results (r-value)

	γ_d	w	M_r	q_c	f_s	γ_d/w	#200	%Silt	%Clay	LL	PI
γ_d	1.00	-0.93	0.27	0.007	0.09	0.83	-0.40	0.33	-0.57	-0.63	-0.62
w	-0.92	1.00	-0.39	-0.17	-0.22	-0.83	0.33	-0.40	0.60	0.63	0.63
M_r	0.27	-0.39	1.00	0.82	0.70	0.33	-0.12	0.09	-0.16	-0.01	-0.09
q_c	0.007	-0.17	0.82	1.00	0.63	0.08	-0.09	0.01	-0.07	0.07	0.01
f_s	0.09	-0.22	0.70	0.63	1.00	0.12	-0.13	0.01	-0.09	0.05	0.01
γ_d/w	0.83	-0.83	0.33	0.08	0.12	1.00	-0.32	0.21	-0.40	-0.48	-0.47
-#200	-0.40	0.33	-0.12	-0.09	-0.13	-0.32	1.00	0.31	0.28	0.29	0.20
%Silt	0.33	-0.40	0.09	0.01	0.01	0.21	0.31	1.00	-0.83	-0.69	-0.75
%Clay	-0.57	0.60	-0.16	-0.07	-0.09	-0.40	0.28	-0.83	1.00	0.87	0.88
LL	-0.63	0.63	-0.01	0.07	0.05	-0.48	0.29	-0.69	0.87	1.00	0.97
PI	-0.62	0.63	-0.09	0.01	0.01	-0.47	0.20	-0.75	0.88	0.97	1.00

Legend: q_c- Tip resistance, f_s- Sleeve friction, γ_d- Dry unit weight, w- water content, PI- Plasticity index, LL- Liquid limit, #200- Percent passing #200 sieve, %Silt- Percentage of silt, and %Clay- Percentage of clay

Tables 19 and 20 also show that q_c, f_s, γ_d, w, and γ_d/w are the only variables that have a significant relation to M_r, and hence, they should be included in the variable stepwise selection analysis. Table 21 presents a summary of the results of the stepwise selection analysis. The fact that the best model includes q_c, f_s, and γ_d/w can be noted. The f_s variable had the greatest influence on the prediction of the model, as is indicated by the partial R^2.

Regression analyses were conducted on the CIMCPT-M_r data to develop two models. The first model, the direct model, relates the laboratory measured M_r directly to the f_s and q_c, while the second model, the soil-property model, predicts laboratory measured M_r from f_s, q_c, and the physical properties of the tested soil. The results of the first regression analysis yielded the direct model shown in Equation 7. The direct model had R^2 and RMSE values of

0.77 and 1.34, respectively. Figure 25 shows the variation of M_r predicted by the direct model and the M_r measured in the laboratory. The results indicate that the model was effective in predicting the M_r of subgrade soils from the results of the CIMCPT.

$$M_r = 2.12 + 3.44q_c + 63.15f_s \tag{7}$$

where
M_r = resilient modulus (ksi),
q_c = tip resistance (ksi), and
f_s=sleeve friction (ksi)

Table 22 presents the results of regression analyses that were conducted to develop the soil-property model. The results show that the model had R^2 of 0.86 and an RMSE of 0.96. Furthermore, q_c and γ_d/w had a more significant effect on the prediction of the model than f_s, as is indicated by the t-value. In addition, all three variables have VIF values less than five, which indicates that these variables are not collinear. To test for any possible heteroscedasticity of the CIMCPT soil-property model, the residuals are plotted against the resilient modulus value as shown in Figure 26. The figure illustrates very little evidence of heteroscedasticity in the model.

Figure 27 shows M_r predicted by the CIMCPT soil-property model and those measured in the laboratory. It is observed that the model predicted M_r values were comparable with M_r measured values. Such indicates that the model was effective in predicting the M_r values for the soil tested.

Typical variation of tip resistance, sleeve friction, and predicted M_r with depth is presented in Appendix A, Figure A1.

Table 21
Results of the Variable selection for CIMCPT-M_r model

Variable Entered	Variable Removed	Number Vars In	Partial R-Square	Model R-Square	C(p)	F Value	Pr > F
q_c		1	0.6745	0.675	47.4290	184.44	<.0001
γ_d/w		2	0.0760	0.751	18.0526	26.79	<.0001
f_s		3	0.0412	0.792	3.0173	17.23	<.0001

Table 22
Results of the Multiple Regression for CIMCPT-M_r model

Variable	DF	Parameter Estimate	t-value	Pr > |t|	Standardized Estimate	Variance Inflation
fs	1	3.547	13.19	<.0001	0.47	3.52
qc	1	52.886	5.15	<.0001	0.21	4.74
γ_d / w	1	0.517	12.33	<.0001	0.38	2.74

$$M_r = 3.55q_c + 52.88f_s + 0.52(\frac{\gamma_d}{w})$$

where,
M_r –Resilient modulus (ksi),
q_c –Tip resistance (ksi),
f_s – Sleeve friction (ksi),
γ_d –Dry unit weight (pcf), and
w – Water content (%).

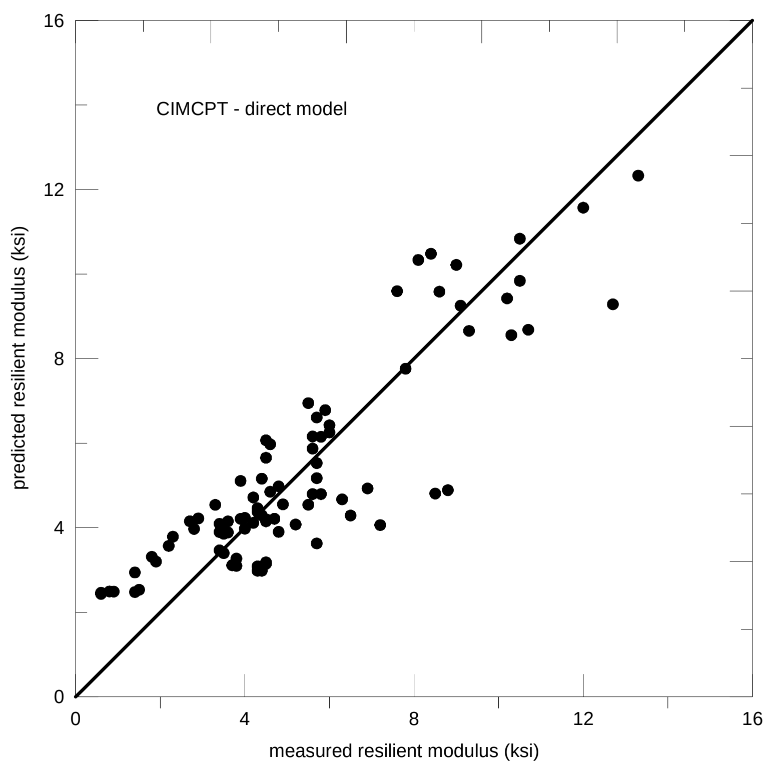


Figure 25
Predictions from the CIMCPT-Direct Model

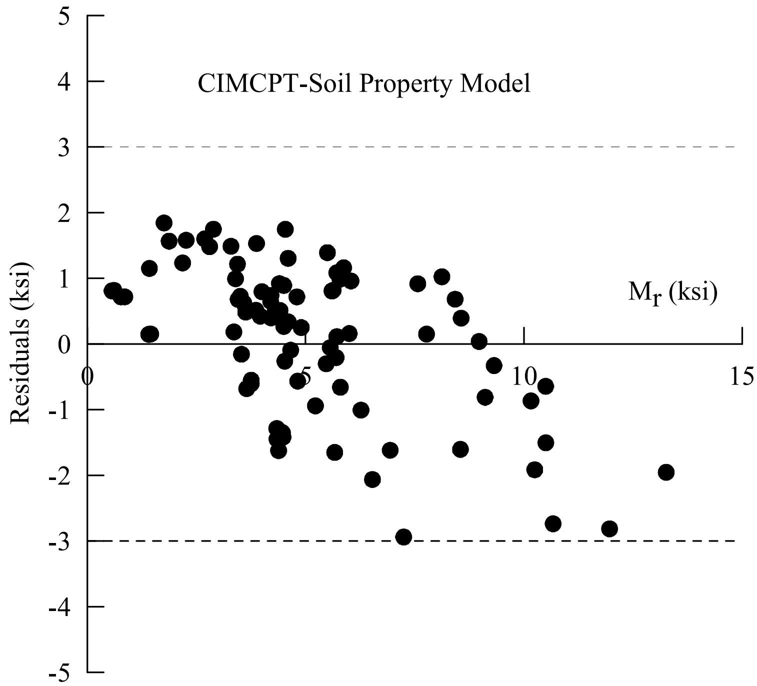


Figure 26
Residuals from CIMCPT-Soil Property Model

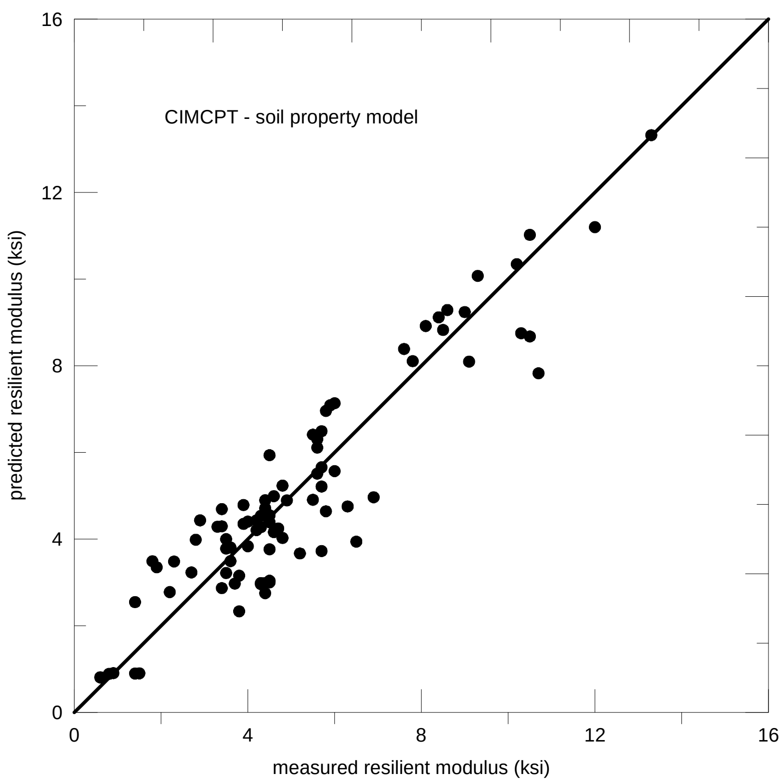


Figure 27
Predictions from the CIMCPT-Soil Property Model

Development of M_r Prediction Models for FWD Test Results

Three backcalculation software packages were used to interpret the FWD data, namely, ELMOD 5.1.69, MODULUS 6, and EVERCALC 5.0. The Florida equation was also used for comparison. During the testing process, there were three readings taken at a load of 9,000 lbs. The results used in the statistical analysis reflect the averages of the three readings. Since MODULUS 6 only uses readings from seven sensors and the data were collected with nine sensors, the files were modified to accommodate the MODULUS 6 software.

Results of ELMOD 5.1.69 Software Backcalculation

Linear backcalculation models were used in this study. Seed values refer to modulus input values for layers prior to the beginning of the backcalculation process. The seed values used for this study were taken from a previous study *[29]*.

Four types of linear backcalculation models were used to backcalculate the FWD moduli. The first two models used seven and nine sensors with no seed values. The third model used nine sensors by inputting seed values in the backcalculation process. Finally, the fourth model used was the one recommended by Dynatest Consulting, Inc. Further information on the models used can be found in the ELMOD 5.1.69 manual. The fact that, in all backcalculation analyses, the maximum depth of the rigid layer was fixed at 240 inches is worth noting. The results of the FWD moduli backcalculation analyses using the four models considered are presented in Table 23.

Linear regression analyses were conducted to develop models that predict the laboratory measured M_r from the FWD moduli that were backcalculated using the previously mentioned analyses. The results of the regression analyses yielded the models shown in Table 24. Figures 28 through 31 illustrate the prediction of these models. The fact that among the four backcalculation models evaluated in this study, models without seed values had better correlation (R^2=0.71 and RMSE=1.32ksi), while the model recommended by the Dynatest had lower R^2 value of 0.61 and higher RMSE value of 1.53 ksi, is noted.

Results of MODULUS 6 Backcalculation

MODULUS 6 backcalculation analyses were performed using semi-infinite and finite depth to bedrock models. For the finite depth to bedrock model, the software provides a ratio called E4/stiff layer to account for the stiffness relationship between the subgrade and bedrock layers. In most cases, the software recommends the use of 100 for the E4/stiff layer ratio; however, for a stiff subgrade layer, a value of five or less should be considered. Therefore, three

Table 23
Results of FWD Backcalculation Using ELMOD Software

Project	Site/ Soil ID	Test Point	No seed 7-sensor	No seed 9-sensor	seed 9-sensor	Cal=2 9-sensor	Project	Site/ Soil ID	Test Point	No seed 7-sensor	No seed 9-sensor	seed 9-sensor	Cal=2 9-sensor
			Backcalculated M_r (ksi)							Backcalculated M_r (ksi)			
LA333	A	2	14.8	14.8	14.7	12.1	LA347	A	2	15.1	14.9	14.8	12.1
		5	14.1	14.1	13.9	11.4			5	14.9	14.7	14.4	11.9
		8	14.2	14.2	14.0	11.4			8	14.6	14.7	14.6	11.9
	B	2	10.4	8.5	10.1	8.4		B	2	16.0	16.5	15.7	13.3
		5	11.6	9.0	11.1	8.9			5	15.0	14.9	14.8	12.3
		8	12.2	10.6	13.0	10.3			8	14.9	15.0	14.8	12.2
	C	2	11.9	11.9	11.9	9.8		C	2	14.9	15.0	14.6	12.1
		5	12.3	12.6	12.8	10.4			5	15.0	15.2	15.0	12.3
		8	11.2	11.2	11.3	9.0			8	15.6	15.5	15.4	12.8
ksi US171	A	2	11.9	11.9	12.1	9.3	LA991	A	2	9.7	9.4	9.6	7.6
		5	11.2	11.7	11.5	8.9			5	8.6	8.5	8.6	6.7
		8	11.5	11.6	11.6	8.9			8	7.8	7.8	7.8	5.9
	B	2	12.1	11.8	11.8	9.2		B	2	7.8	7.7	7.6	6.2
		5	12.8	12.8	12.7	10.1			5	7.9	7.9	7.8	6.5
		8	12.7	12.8	12.8	10.0			8	9.4	9.4	9.3	7.6
	C	2	24.1	23.9	23.5	18.0		C	2	9.8	9.8	9.3	8.0
		5	24.7	25.3	24.4	18.7			5	9.7	9.7	9.7	7.9
		8	27.2	27.6	26.3	20.3			8	10.4	10.5	10.1	8.4
LA22	A	2	15.0	14.7	14.6	15.7	LA182	A	2	6.9	7.0	6.9	5.4
		5	15.4	15.6	15.4	16.4			5	7.2	7.3	7.3	5.7
		8	14.4	14.7	14.7	15.3			8	7.8	8.0	7.9	6.3
	B	2	16.5	16.5	16.5	17.5		B	2	7.7	8.0	8.0	6.7
		5	18.4	18.8	18.7	19.4			5	7.4	7.5	7.3	6.3
		8	17.8	17.6	17.7	18.6			8	7.8	7.8	8.0	6.5
	C	2	14.8	14.8	14.8	15.6		C	2	8.4	8.7	9.2	7.3
		5	14.6	14.7	14.5	15.5			5	8.5	8.5	9.0	7.1
		8	16.2	16.2	16.3	16.9			8	8.4	8.7	8.8	7.0
LA344	A	2	8.6	8.7	8.8	7.1	LA652	A	2	4.2	4.1	4.0	3.5
		5	8.9	8.9	9.0	7.4			5	4.2	4.1	4.2	3.5
		8	9.1	9.1	9.4	7.5			8	4.5	4.5	4.5	3.8
	B	2	10.2	10.2	10.2	8.5		B	2	6.7	6.7	6.8	5.5
		5	6.7	6.6	6.4	5.5			5	4.9	4.9	5.0	4.1
		8	6.0	5.9	5.8	4.7			8	4.2	4.2	4.2	3.5
	C	2	10.7	10.8	10.8	8.7		C	2	4.6	4.6	4.5	3.5
		5	11.4	11.3	11.5	9.3			5	4.8	4.8	4.8	3.9
		8	11.0	11.1	11.5	9.2			8	4.6	4.5	4.5	3.7
LA28	A	2	14.9	15.6	16.3	12.9	Legend: Cal- Calibration, M_r –Resilient modulus						
		5	13.7	14.0	13.8	11.2							
		8	12.6	12.9	13.1	10.6							
	B	2	25.0	26.1	26.1	20.8							
		5	25.1	26.2	26.2	20.8							
		8	26.2	27.1	27.0	21.9							

Table 24
Results of statistical analysis for M_r-FWD (ELMOD 5.1.69) model

ELMOD 5.1.69	Model	R^2	RMSE
7-sensor (no seed)	$M_r = 0.40E_{fwd} + 0.49$	0.71	1.32
9-sensor (no seed)	$M_r = 0.39E_{fwd} + 0.64$	0.70	1.32
9-sensor (seed)	$M_r = 0.39E_{fwd} + 0.61$	0.69	1.36
9-sensor (Cal=2)	$M_r = 0.40E_{fwd} + 1.13$	0.61	1.53

Legend: E_{fwd}- Backcalculated modulus from FWD (ksi), M_r- Resilient modulus (ksi), R^2- Coefficient of determination, RMSE- Root mean square for error

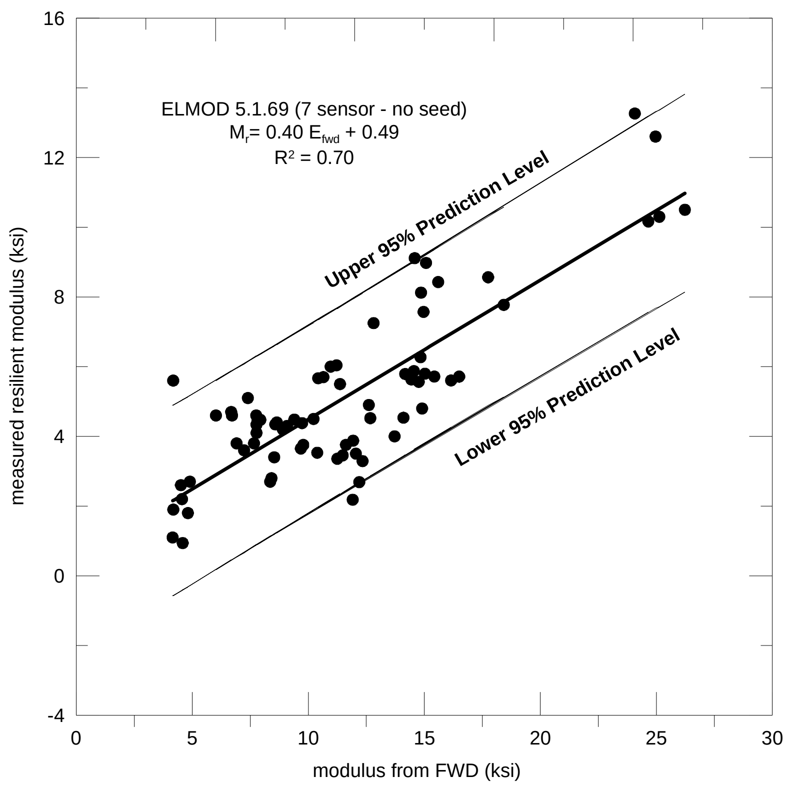


Figure 28
M_r versus FWD modulus backcalculated ELMOD 5.1.69 (7-sensor with no seed values)

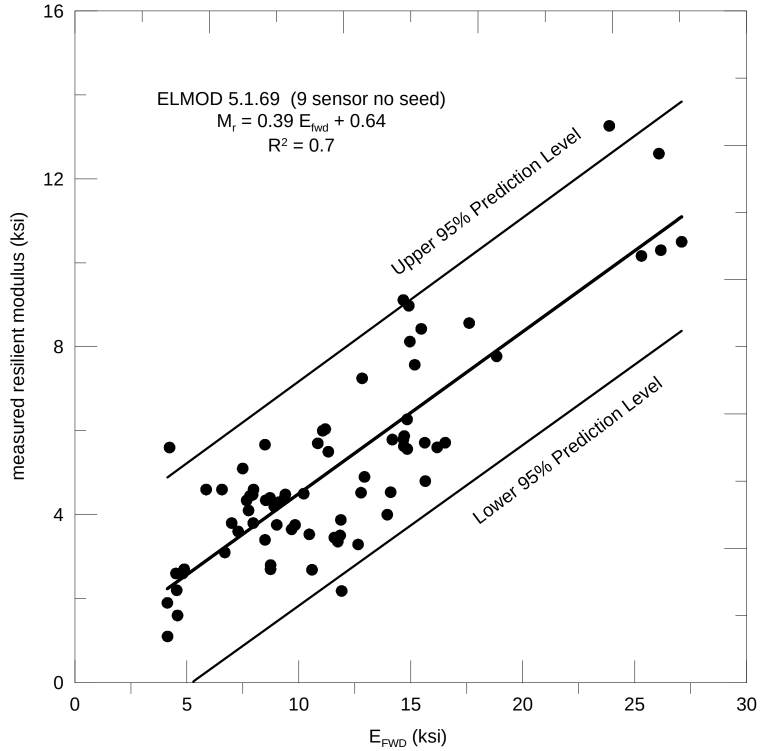


Figure 29
M_r vs. FWD moduli backcalculated using ELMOD 5.1.69 (9-sensor with no seed values)

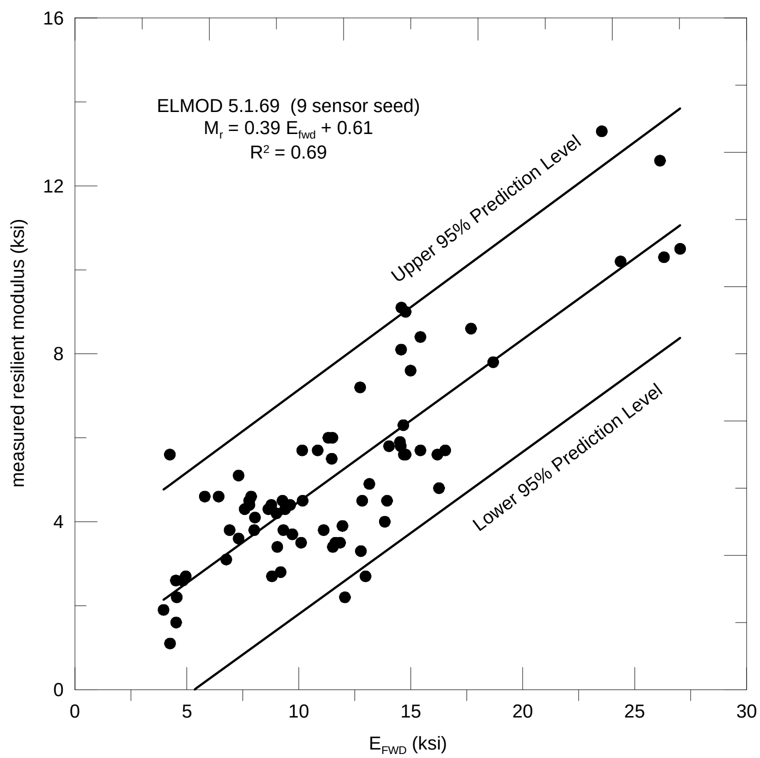


Figure 30
M_r vs. FWD moduli backcalculated using ELMOD 5.1.69 (9-sensor with seed values)

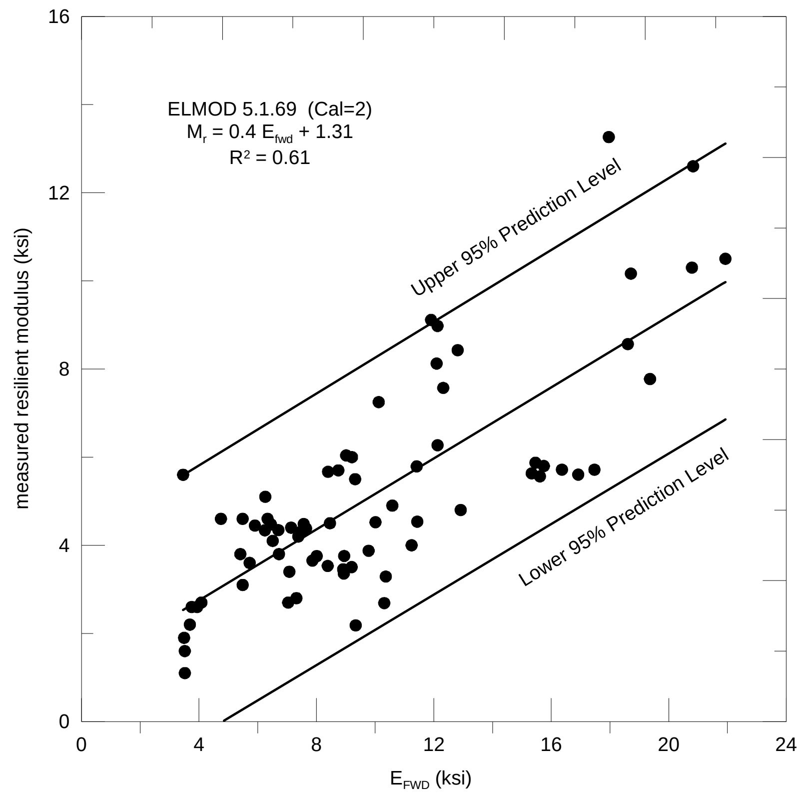


Figure 31
M_r vs. FWD moduli backcalculated using ELMOD 5.1.69 (Calibration = 2)

MODULUS 6 backcalculation analyses were conducted using finite depth to bedrock models for E4/stiff layer ratio values of 100, 5, and 3. Based on the results of these analyses, the FWD backcalculated moduli values closest to the laboratory measured M_r were selected.

Regression analyses were conducted to correlate the laboratory measured M_r from the FWD moduli backcalculated using the semi-infinite and finite depth analyses shown in Table 25. Based on the results of the regression analyses, the models shown in Table 26 were developed. Figures 32 and 33 illustrate the two models, respectively. The fact that the regression model developed using the semi-infinite analysis was better than that developed using the finite depth analyses that were obtained when using the FWD moduli backcalculated from an analysis that did not utilize seed values is noted. However, both models had a relatively low R^2 (0.46 and 0.54) and high RMSE value (1.7 ksi and 1 ksi), which indicates that a poor correlation exists between the M_r and the FWD moduli backcalculated using MODULUS 6 software. Such is also observed in Figures 34 and 35, where data points were widely scattered about the model line.

Table 25
Results of FWD backcalculation analysis using MODULUS 6 software

Project	Site/Soil ID	Test Point	Semi-infinite	Finite Depth	Project	Site/Soil ID	Test Point	Semi-infinite	Finite Depth
			Backcalculated M_r (ksi)					Backcalculated M_r (ksi)	
LA333	A	2	18.4	11.5	LA347	A	2	17.3	8.3
		5	17.5	10.8			5	17.5	11.2
		8	17.0	10.9			8	17.1	10
	B	2	13.3	6.6		B	2	18.1	12.3
		5	15.3	7.1			5	17.0	9.8
		8	16.6	8			8	17.4	11.2
	C	2	13.9	9.8		C	2	17.7	9.8
		5	15.1	10.4			5	17.7	9.8
		8	14.4	9			8	16.9	10.3
US171	A	2	14.8	7.8	LA991	A	2	12.9	7.6
		5	13.5	7.1			5	12.5	6.7
		8	13.8	7.3			8	12.5	5.9
	B	2	13.7	7.9		B	2	9.1	6.2
		5	16.4	8.1			5	9.5	6.5
		8	17.0	8.4			8	11.3	7.6
	C	2	28.0	14		C	2	11.5	8
		5	29.4	14.9			5	11.7	7.9
		8	31.4	15.9			8	12.3	8.4
LA22	A	2	26.1	15.7	LA182	A	2	11.9	5.4
		5	27.9	16.4			5	12.1	5.7
		8	28.3	15.3			8	11.4	6.3
	B	2	27.4	17.5		B	2	9.4	6.7
		5	27.3	19.4			5	8.2	6.3
		8	25.9	18.6			8	10.6	6.5
	C	2	24.1	15.6		C	2	10.7	7.3
		5	24.6	15.5			5	10.9	7.1
		8	24.6	16.9			8	11.3	7
LA344	A	2	12.5	6	LA652	A	2	9.2	3.5
		5	12.0	6.2			5	10.0	3.5
		8	14.0	6.4			8	10.6	3.8
	B	2	12.8	6.3		B	2	16.4	5.5
		5	12.8	8.5			5	12.1	4.1
		8	11.0	5.5			8	11.1	3.5
	C	2	14.8	8.2		C	2	8.4	3.5
		5	15.8	8.7			5	7.5	3.9
		8	14.9	8.6			8	7.3	3.7
LA28	A	2	15.9	11.7	Legend: M_r –Resilient modulus, SL- Stiff layer				
		5	14.4	11.1					
		8	13.5	10.3					
	B	2	26.2	18.5					
		5	26.6	18.8					
		8	27.5	19.6					

Table 26
Results of statistical analysis for M_r-FWD (MODULUS 6) model

MODULUS 6	Model	R^2	RMSE
Semi Infinite	$M_r = 0.25E_{fwd} + 1.02$	0.54	1.38
Finite Depth	$M_r = 0.40E_{fwd} + 0.90$	0.46	1.7

E_{fwd}- Backcalculated modulus from FWD (ksi), M_r- Resilient modulus (ksi), R^2- Coefficient of determination, RMSE- Root mean square for error (ksi)

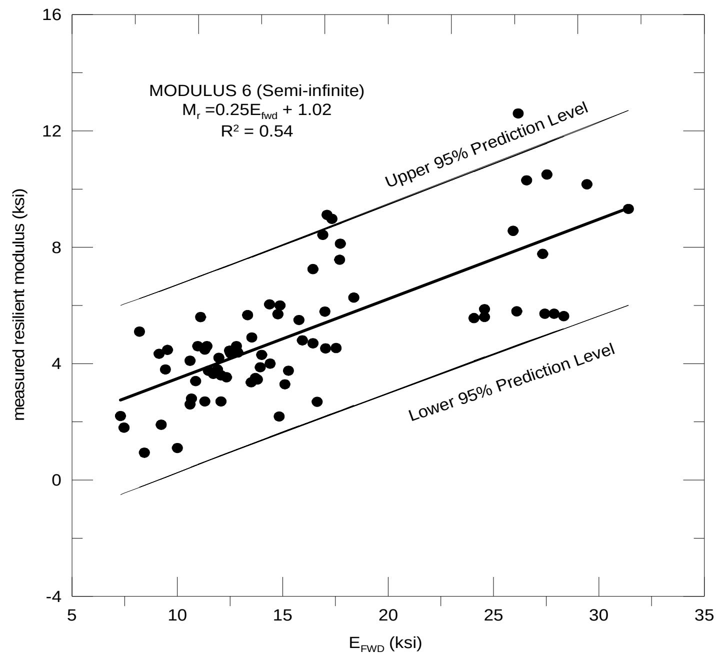


Figure 32
M_r vs. FWD moduli backcalculated using MODULUS 6 (semi infinite subgrade layer)

Results of EVERCALC 5.0

Table 27 shows the results of the FWD moduli backcalculation using EVERCALC 5.0 software. Regression analysis was performed on the M_r and the FWD moduli backcalculated using EVERCALC 5.0 software. The results of this analysis yielded the model shown in Equation 19. The model had R^2 and RMSE values of 0.51 and 1.62, respectively. Figure 36 presents the results from the statistical analysis. The fact that poor correlation exists between the FWD moduli backcalculated using EVERCALC 5.0 and the M_r measured in the laboratory is noted.

$$M_r = 0.26E_{fwd} + 1.19 \qquad (8)$$

where

M_r = resilient modulus (ksi), E_{fwd}= backcalculated modulus from FWD (ksi).

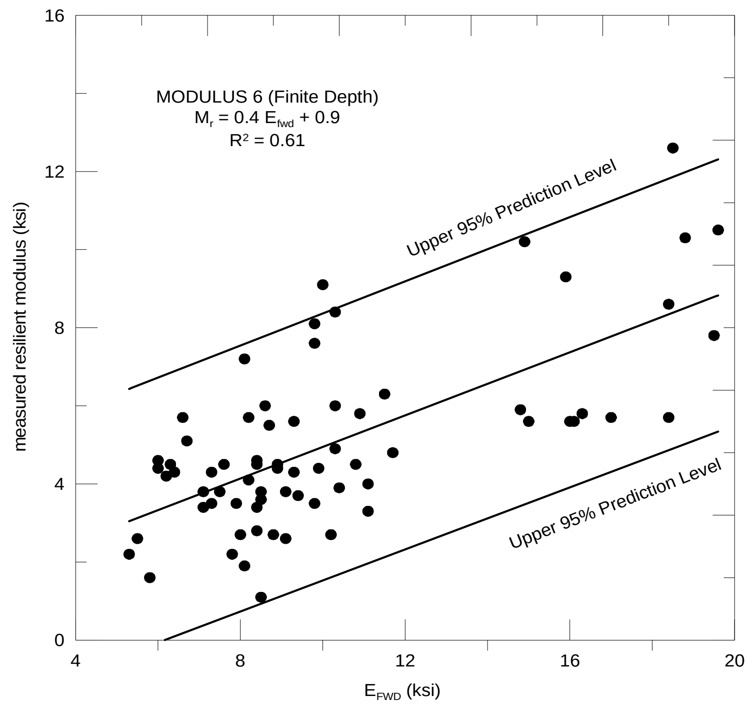


Figure 33
M_r vs. FWD moduli backcalculated using MODULUS 6 (finite depth)

Results of Florida Equation

The FWD moduli backcalculated using the Florida equation is shown in Table 27. Equation 9 presents the correlation between the FWD moduli backcalculated using the Florida equation and M_r measured in the laboratory. While Figure 35 illustrates this correlation, The fact that the correlation is poor and has a low R^2 value of 0.49 is noted.

$$M_r = 0.24E_{fwd} + 0.94 \tag{9}$$

where
Mr = Resilient modulus (ksi),
E_{fwd}= Backcalculated modulus from FWD (ksi).

Development of M_r Prediction Models for Dynaflect Test Results

LADOTD developed a chart to determine the subgrade modulus and structural number based upon deflection readings taken with the Dynaflect. This chart was used to obtain the subgrade moduli E_d from the Dynaflect test results (Table 28). Equation 10 and Figure 36 present the result of the regression analysis that was conducted to correlate the backcalculation results with the laboratory measured M_r. The correlation had an R^2 value of 0.73 and an RMSE value of 1.46.

$$M_r = 0.41E_d + 2.26 \tag{10}$$

where
Mr = Resilient modulus (ksi),
E_{fwd}= Backcalculated modulus from FWD (ksi).

Table 27
Results of FWD backcalculation using EVERCALC 5.0 and Florida equation

Project	Site	ID	Lab	Ever	Fl	Project	Site	ID	Lab	Ever	Fl
			M_r (ksi)						M_r (ksi)		
LA333	A	2	6.3	18.3	19.4	LA347	A	2	9.0	15.8	19.0
		5	4.5	17.4	18.8			5	12.7	15.8	18.3
		8	5.8	16.9	18.3			8	9.1	16.2	18.2
	B	2	5.7	13.0	14.2		B	2	12.0	17.2	19.4
		5	3.8	15.2	16.8			5	10.5	16.3	18.7
		8	2.7	16.3	18.4			8	10.7	16.6	18.6
	C	2	3.9	13.6	15.2		C	2	8.1	17.1	18.4
		5	3.3	14.9	16.5			5	7.6	16.7	18.4
		8	6.0	14.0	15.9			8	8.4	16.5	19.5
US171	A	2	2.2	14.5	16.9	LA991	A	2	4.4	12.5	14.5
		5	3.4	13.4	15.4			5	4.3	12.3	15.7
		8	3.5	13.4	15.5			8	4.4	12.7	16.5
	B	2	3.5	13.5	14.9		B	2	4.3	8.9	10.5
		5	7.2	15.4	17.0			5	4.5	9.4	11.1
		8	4.5	15.6	17.1			8	4.5	11.0	13.4
	C	2	13.3	27.0	29.8		C	2	3.8	11.0	13.5
		5	10.2	28.8	32.5			5	3.7	11.5	14.0
		8	9.3	29.9	33.7			8	3.5	11.7	14.5
LA22	A	2	5.8	26.2	27.8	LA182	A	2	3.8	13.0	15.0
		5	5.7	27.9	30.2			5	3.6	12.7	15.1
		8	5.6	28.6	32.5			8	4.6	11.3	13.6
	B	2	5.7	27.8	28.6		B	2	3.8	9.1	10.3
		5	7.8	26.9	27.6			5	5.1	8.0	9.4
		8	8.6	25.5	25.9			8	4.1	10.1	12.0
	C	2	5.6	24.6	24.4		C	2	2.8	10.3	12.1
		5	5.9	25.2	25.3			5	3.4	10.6	12.5
		8	5.6	24.8	25.8			8	2.7	11.0	13.1
LA344	A	2	4.4	13.0	14.4	LA652	A	2	1.9	5.1	6.3
		5	4.2	12.4	13.4			5	1.1	5.7	7.2
		8	4.3	14.1	16.5			8	2.6	6.1	7.5
	B	2	4.5	12.0	13.1		B	2	3.1	9.8	11.5
		5	4.6	9.7	8.5			5	2.7	7.1	8.7
		8	4.6	8.0	7.9			8	5.6	6.6	8.6
	C	2	5.7	14.9	16.2		C	2	1.6	8.1	11.6
		5	5.5	15.6	17.7			5	2.6	7.5	9.5
		8	6.0	14.9	16.4			8	2.2	7.4	9.5
LA28	A	2	4.8	15.6	17.3	Legend: Elm- Ever- EVERCALC, Fl- Florida equation, Lab- Laboratory,					
		5	4.0	14.4	16.1						
		8	4.9	13.6	15.0						
	B	2	12.6	25.0	27.7						
		5	10.3	25.0	28.0						
		8	10.5	25.0	29.5						

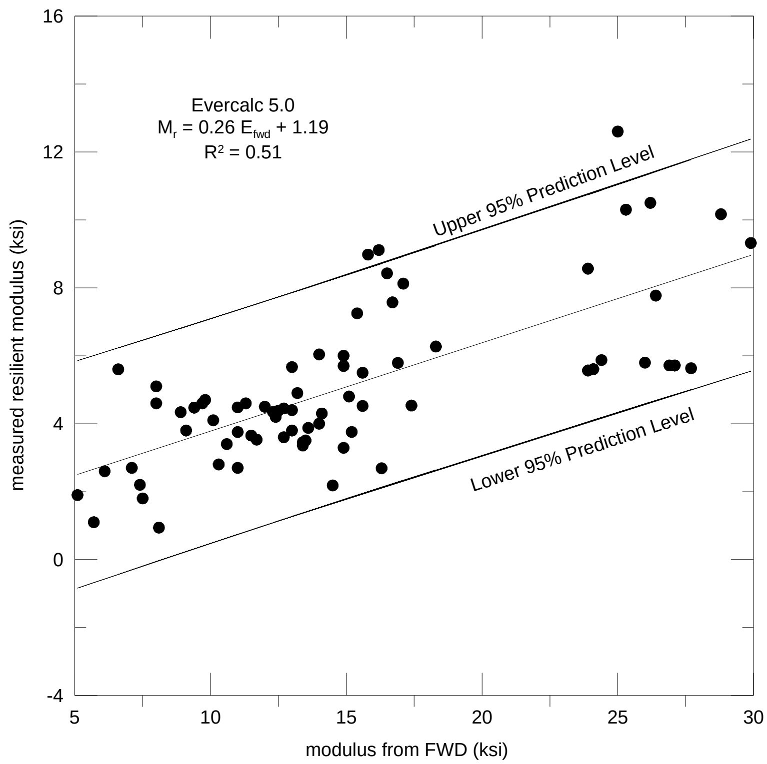


Figure 34
M_r vs. FWD moduli backcalculated using EVERCALC 5.0

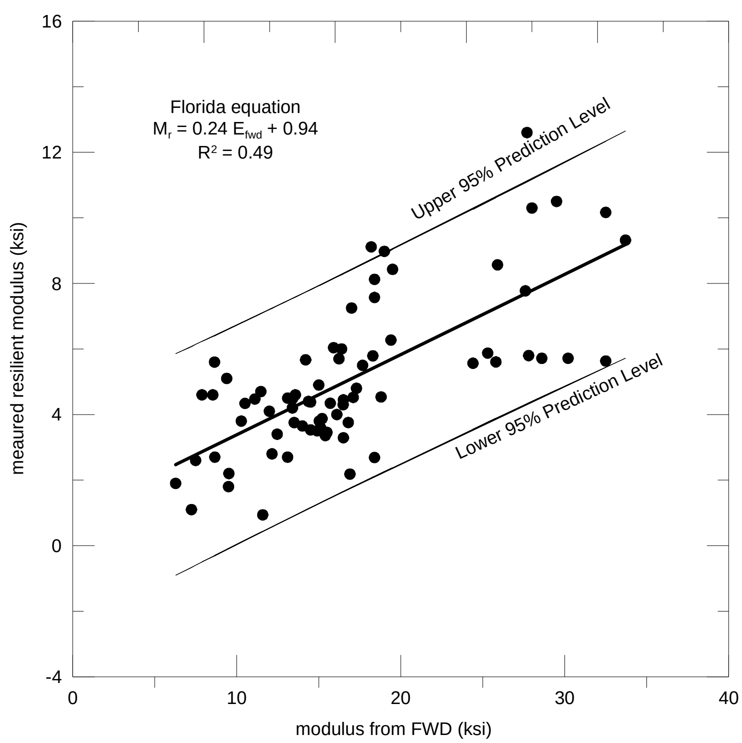


Figure 35
M_r vs. FWD moduli backcalculated using ELMOD 5.1.69 Florida equation

Table 28
Dynaflect test results

Project	Site/Soil ID	Test Point	Lab. M_r (ksi)	Dynaflect moduli (ksi)	Project	Site/Soil ID	Test Point	Lab-M_r (ksi)	Dynaflect moduli (ksi)
LA333	A	2	6.3	8.2	LA347	A	2	9.0	19.0
		5	4.5	7.7			5	12.7	18.3
		8	5.8	7.9			8	9.1	18.2
	B	2	5.7	7.1		B	2	12.0	19.4
		5	3.8	7.8			5	10.5	18.7
		8	2.7	8.7			8	10.7	18.6
	C	2	3.9	5.8		C	2	8.1	18.4
		5	3.3	5.9			5	7.6	18.4
		8	6.0	5.6			8	8.4	19.5
US171	A	2	2.2	7.0	LA991	A	2	4.4	4.2
		5	3.4	6.5			5	4.3	4.1
		8	3.5	6.5			8	4.4	4.2
	B	2	3.5	6.7		B	2	4.3	3.5
		5	7.2	7.6			5	4.5	3.7
		8	4.5	7.5			8	4.5	4.0
	C	2	13.3	16.7		C	2	3.8	3.8
		5	10.2	15.8			5	3.7	3.7
		8	9.3	14.7			8	3.5	3.8
LA22	A	2	5.8	6.9	LA182	A	2	3.8	4.2
		5	5.7	7.0			5	3.6	4.3
		8	5.6	7.3			8	4.6	4.1
	B	2	5.7	8.0		B	2	3.8	3.9
		5	7.8	8.4			5	5.1	4.0
		8	8.6	7.8			8	4.1	3.8
	C	2	5.6	6.2		C	2	2.8	3.8
		5	5.9	6.2			5	3.4	4.1
		8	5.6	6.3			8	2.7	4.1
LA344	A	2	4.4	3.8	LA652	A	2	1.9	2.4
		5	4.2	4.0			5	1.1	2.7
		8	4.3	4.3			8	2.6	2.9
	B	2	4.5	4.3		B	2	3.1	4.2
		5	4.6	3.2			5	2.7	2.7
		8	4.6	3.3			8	5.6	2.4
	C	2	5.7	4.3		C	2	1.6	3.7
		5	5.5	4.4			5	2.6	3.2
		8	6.0	4.3			8	2.2	3.3
LA28	A	2	4.8	9.0	Legend: Lab. M_r – Laboratory resilient modulus measured at a cyclic stress level of 37.2 kPa (5.4 lbf/in.^{2}), contact stress level of 4.1 kPa (0.6 lbf/in.^{2}), and confining pressure of 14 kPa (2 lbf/in.^{2})				
		5	4.0	9.7					
		8	4.9	9.8					
	B	2	12.6	23.5					
		5	10.3	23.5					
		8	10.5	24.0					

Results of the LADOTD Method

Figure 37 shows the results of comparing the LADOTD resilient modulus values obtained from the soil support values (SSV) that are assigned for each parish (Appendix A, Table A2) with those obtained from laboratory testing. The range of resilient modulus values for the locations tested using the LADOTD method was from 7.6 to 9.2 ksi, while the laboratory resilient modulus values ranged from 1 to 14 ksi. Most of the LADOTD method estimated that M_r values are not comparable with the laboratory measured values. These results are acceptable, as the LADOTD uses a typical average SSV value for the emitter parish; however, the M_r value can vary from site to site within the parish.

Limitations of the Models

The prediction models developed in this study are valid for cohesive subgrade soils with M_r values from 1 to 14 ksi, PI values from 3 to 66 percent, LL values from 20 to 99, and other soil properties, as presented in Table 7.

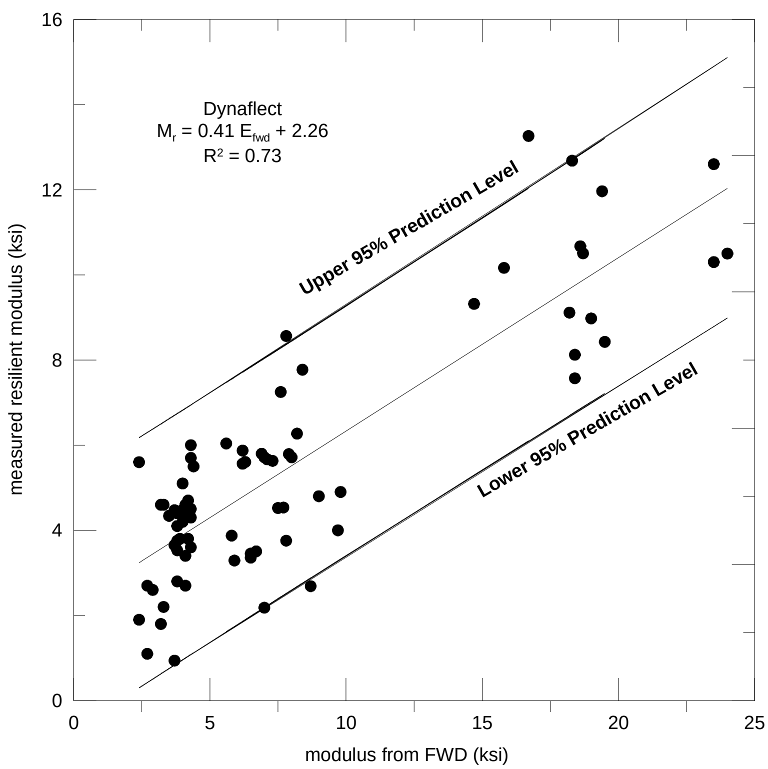


Figure 36
Dynaflect statistical results

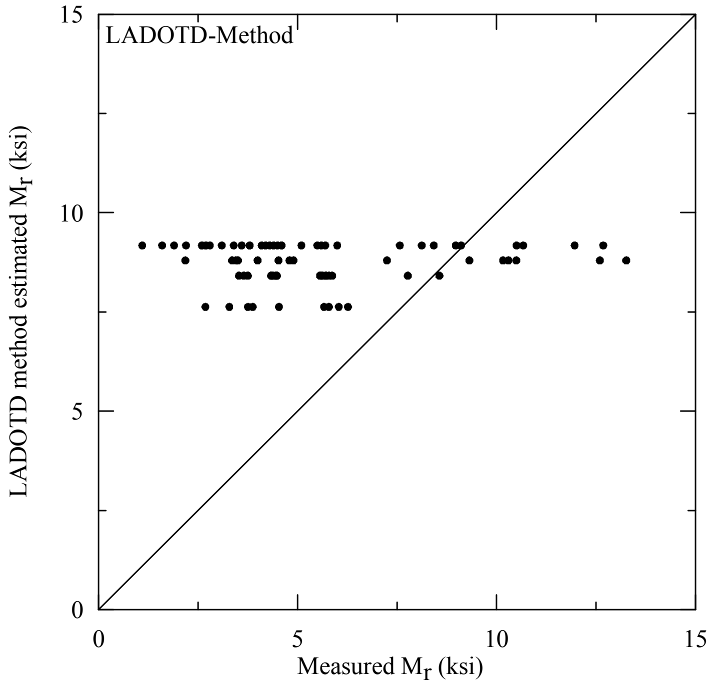


Figure 37
LADOTD method estimated resilient modulus

Table 29
Summary of the analysis

Method	Model	R^2	RMSE (ksi)	Comments
DCP-Soil Property Model	$M_r = 165.5\left(\frac{1}{DCPI^{1.147}}\right) + 0.0966\left(\frac{\gamma_d}{w}\right)$	0.92	0.88	Subgrade soils: $1<M_r<14$ ksi
DCP-Direct Model	$M_r = \frac{151.8}{(DCPI)^{1.096}}$	0.91	0.88	Subgrade soils: $1<M_r<14$ ksi
CIMCPT- Soil Property Model	$M_r = 3.55q_c + 52.88f_s + 0.52(\frac{\gamma_d}{w})$	0.86	0.96	Subgrade soils: $1<M_r<14$ ksi
CIMCPT-Direct Model	$M_r = 2.12 + 3.44q_c + 63.15f_s$	0.77	1.34	Subgrade soils: $1<M_r<14$ ksi
Dynaflect	$M_r = 0.41E_d + 2.26$	0.73	1.46	Nomographs and temperature correction
ELMOD 5.1.69	$M_r = 0.40E_{fwd} + 0.49$	0.71	1.32	7-Sensor no seed value
MODULUS 6	$M_r = 0.27E_{fwd} + 0.82$	0.52	1.60	Semi infinite subgrade
EVERCALC 5.0	$M_r = 0.26E_{fwd} + 1.19$	0.51	1.62	Subgrade soils $1<M_r<14$ ksi
Florida Equation	$M_r = 0.24E_{fwd} + 0.94$	0.49	1.65	Subgrade soils $1<M_r<14$ ksi
LADOTD Method	No correlation established	N/A	N/A	N/A

Legend: DCPI – Dynamic cone penetration index (mm/blow), E_d- Modulus from Dynaflect (ksi), E_{fwd}- Modulus from FWD (ksi), LADOTD- Louisiana Department of Transportation and Development, N/A- Not applicable, M_r –Resilient modulus (ksi) at a cyclic stress level of 37.2 kPa (5.4 lbf/in.^{2}), contact stress level of 4.1 kPa (0.6 lbf/in.^{2}), and confining pressure of 14 kPa (2 lbf/in.^{2}), q_c –Tip resistance (ksi), f_s – Sleeve friction (ksi), RMSE- Root mean square for error (ksi), γ_d –Dry unit weight (pcf), w – Water content (%)

CONCLUSIONS

This report presents the development of models in an effort to predict the resilient modulus of subgrade soils from the test results of DCP, CIMCPT, FWD, Dynaflect, and soil properties of subgrade soils. Field and laboratory testing programs were conducted. The field testing program included DCP, CIMCPT, FWD, and Dynaflect testing, whereas the laboratory program included repeated load triaxial resilient modulus tests and physical properties and compaction tests. Comprehensive statistical analyses were conducted on the laboratory and field test results of subgrade soils. Based on the results of this study, the following conclusions can be drawn:

- The DCP soil-property model ranked the best for the prediction of resilient modulus of subgrade soils, followed by the DCP direct model, the CIMCPT soil-property model, the CIMCPT direct model, Dynaflect, ELMOD 5.1.69, MODULUS 6, EVERCALC 5.0, the Florida equation, and the current DOTD method.

- A good agreement was obtained between the M_r predicted using DCPI and those measured using repeated load triaxial tests.

- The predicted M_r values obtained from the CIMCPT direct model, which included CIMCPT tip resistance and sleeve friction as independent variables, matched the measured M_r values. This demonstrates the applicability of the CIMCPT test results in predicting the M_r of pavement subgrade cohesive soils.

- The DCP and CIMCPT test results are influenced by the soil properties, and the two models were enhanced when moisture content and dry unit were incorporated.

- Among all backcalculated FWD moduli, those backcalculated using ELMOD 5.1.69 software had the best correlation with M_r measured in the laboratory repeated loading triaxial tests.

- From a practical standpoint, the subgrade modulus, as determined from the DCP-soil property model, DCP-direct model, CIMCPT soil-property model, CIMCPT direct model, Dynaflect, or FWD utilizing ELMOD 5.1.69 backcalculation software, may be used with the same confidence, considering the ranges of the coefficient of determination.

- The coefficients of determination (R^2) for models predicting M_r of subgrade soils using the MODULUS 6, EVERCALC 5.0, and the Florida equation were the lowest among the models developed.

- The M_r values estimated using the approach currently adopted by the LADOTD were found to correlate poorly with the laboratory measured M_r values.

RECOMMENDATIONS

This report presents the results of a study conducted in an effort to develop resilient modulus prediction models of subgrade soils from different in situ tests such as FWD, Dynaflect, CIMCPT, and DCP. The approach of predicting M_r used in this study is an improvement over the current procedure used by LADOTD in pavement design application. The fact that the models are mainly applicable to cohesive soils with PI values from 3 to 66 percent, LL values from 20 to 99, and other soil properties, as presented in Table 7 is noted.

The following initiatives are recommended in order to facilitate the implementation of this study:

1) Implement the results of this study into the design manual for use by LADOTD engineers.

2) Establish an implementation and verification study through field projects. Selected projects should incorporate various types of cohesive soils.

3) The proposed study should incorporate granular soils in order to facilitate the development of generalized M_r prediction models for all soils encountered during construction of roadways in Louisiana, as the models in this study were developed for cohesive soils and may not be capable of predicting M_r values of granular soils.

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Appendix A

Figure A1: Typical Profile of Tip Resistance (q_c), Sleeve Friction (fs), and Predicted M_r (LA333 Site, Test Point C8)

Table A1: Test Results for Verification of DCP Models

Table A2: M_r Estimated From LADOTD Method

Table A1
Test Results for Verification of DCP Models

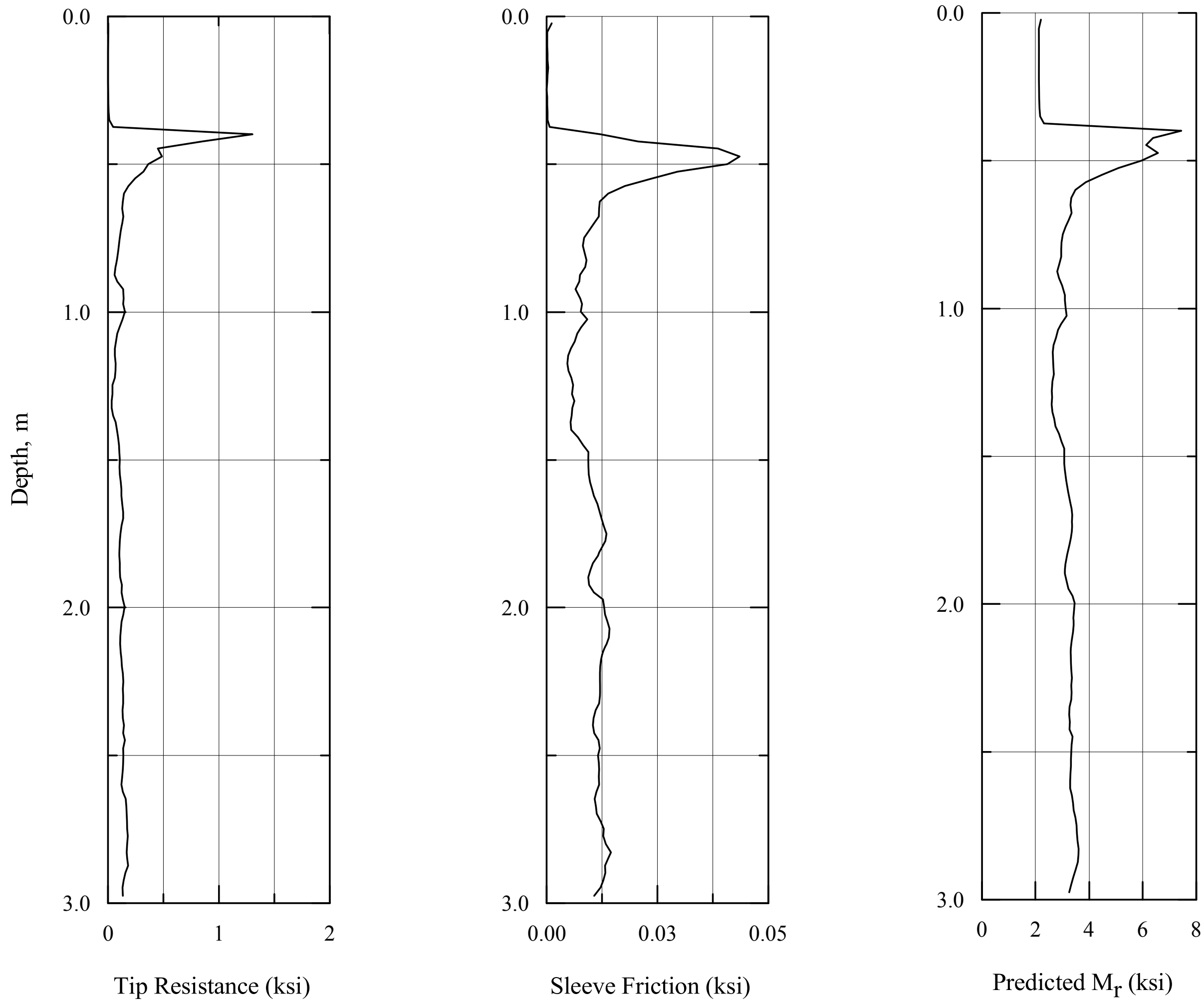
Location and Site	Test Point	γ_d (pcf)	w (%)	Lab. M_r (ksi)	DCPI (mm/blow)
Mississippi (George et al. *[19]*)	1591+00	117.2	14.6	7.4	23.3
	1347+00	117.2	15.4	9.1	16.7
	1595+00	107.1	18.2	10.2	12.3
	1596+00	110.3	16.1	9.9	13.3
	88+00	108.4	14.8	12.3	10.6
	90+00	110.9	17.8	10.6	13.6
	94+00	113.4	16.8	11.2	13.6
	96+00	115.9	15.1	11.9	11.0
	108+00	108.4	18.1	9.3	15.0
	114+00	106.5	22.0	4.1	27.3
	116+00	107.7	18.9	5.5	25.2
	172+00	115.3	16.2	9.1	12.7
	176+00	115.9	17.3	5.2	29.0
	178+00	109.6	20.7	6.2	20.6
	262+62	104.0	19.1	9.7	12.9
	264+50	103.3	17.2	10.4	12.1
	266+00	110.3	18.5	11.9	11.5
	670+00	108.4	15.8	10.6	11.9

Legend: DCPI- DCP Index, V- Verification data from another study [19], w- Moisture content, γd- Dry unit weight, Lab. M_r – Laboratory resilient modulus measured at a cyclic stress level of 37.2 kPa (5.4 lbf/in.^{2}) and confining pressure of 14 kPa (2 lbf/in.^{2})

Table A2
M_r Estimated From LADOTD Method

Parish	Soil Support Value	Resilient Modulus (psi)	Parish	Soil support value	Resilient Modulus (psi)
Acadia	3.7	8797	Madison	3.8	9176
Allen	3.6	8413	Morehouse	3.8	9176
Ascension	3.6	8413	Natchitoches	4.0	9916
Assumption	3.5	8023	Orleans	3.4	7627
Avoyelles	3.8	9176	Ouachita	4.0	9916
Beauregard	3.7	8797	Plaquemines	4.0	9916
Bienville	4.0	9916	Pointe Coupee	3.8	9176
Bossier	3.7	8797	Rapides	4.0	9916
Caddo	4.1	10278	Red River	4.1	10278
Calcasieu	3.8	9176	Richland	3.9	9549
Caldwell	4.0	9916	Sabine	3.9	9549
Cameron	3.8	9176	St. Bernard	3.5	8023
Catahoula	3.7	8797	St. Charles	3.4	7627
Claiborne	4.1	10278	St. Helena	3.9	9549
Concordia	3.6	8413	St. James	3.5	8023
Desoto	3.8	9176	St. John	3.4	7627
East Baton Rouge	3.6	8413	St. Landry	3.8	9176
East Carroll	3.8	9176	St. Martin	3.5	8023
East Feliciana	4.4	11330	St. Mary	3.7	8797
Evangeline	3.9	9549	St. Tammany	3.8	9176
Franklin	4.0	9916	Tangipahoa	4.2	10634
Grant	4.0	9916	Tensas	3.8	9176
Iberia	3.8	9176	Terrebonne	3.7	8797
Iberville	3.6	8413	Union	4.1	10278
Jackson	3.8	9176	Vermillion	3.4	7627
Jefferson	3.5	6023	Vernon	3.7	8797
Jefferson Davis	3.6	8413	Washington	3.8	9176
Lafayette	4.0	9916	Webster	3.9	9549
Lafourche	3.8	9176	West Baton Rouge	3.8	9176
Lasalle	3.8	9176	West Carroll	3.9	9549
Lincoln	4.1	10278	West Feliciana	4.2	10634
Livingston	3.9	9549	Winn	4.0	9916

Figure A1
Typical Profile of Tip Resistance (q_c), Sleeve Friction (fs), and Predicted M_r (LA333 Site, Test Point C8)



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