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MAINE DEPARTMENT OF TRANSPORTATION

DETERMINATION OF RESILIENT MODULUS FOR MAINE ROADWAY SOILS

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16. Abstract (limit 200 words)

The Maine Department of Transportation commissioned this study to examine methods of obtaining resilient modulus for use in pavement design. Resilient modulus is a measure of soil layer stiffness and is highly subjective to density, moisture content, soil fabric structure, compaction method, laboratory equipment compliance, and technician skill. As a result, several alternative test methods have been proposed. These alternative test methods include resilient modulus correlation to results from torsional shear and resonant column tests, a modified gyratory test machine normally used for testing asphalt concrete specimens, and a small-scale falling weight deflectometer device.

The study used resilient modulus test data of fourteen Maine soils published by Law Engineering (1992). Soil index property data and falling weight deflectometer (FWD) data was obtained from the Strategic Highway Research Program's (SHRP) Long Term Pavement Performance (LTPP) database. Three methods for determining resilient modulus were examined: (1) backcalculation of resilient modulus using computer software, (2) determination of the K_n constants for various constitutive resilient modulus equations by linear regression analysis, and (3) correlations between resilient modulus and soil property data and stress state. Computer backcalculation was done using MODCOMP 4 and MODULUS 5.1. The backcalculated resilient moduli did not compare well with the laboratory moduli when the programs automatically estimated the depth to hard layer and outliers were neglected. The K_n constants for seven common constitutive relationships were developed for fourteen Maine soils using linear regression. Two equations correlating resilient modulus to dry density, water content, grain size distribution and stress state were also generated from linear regression techniques. California bearing ratio (CBR) does not correlate well with resilient modulus, therefore, no correlations involving CBR were examined.

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DETERMINATION OF RESILIENT MODULUS FOR MAINE ROADWAY SOILS

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EXECUTIVE SUMMARY

The Maine Department of Transportation (MDOT) uses the 1993 AASHTO Guide for Design of Pavement Structures (AASHTO Guide) as a design aid to determine the thickness of asphalt cement concrete paving, base, and subbase layers. The AASHTO Guide bases its design methods on resilient modulus (M_R) rather than California bearing ratio (CBR). MDOT has pneumatically actuated resilient modulus testing equipment that may now be obsolete. MDOT commissioned this study to determine a practical method for determining resilient modulus for use in design.

There are several factors that influence the magnitude of resilient modulus. Changes in dry density, moisture content, soil fabric, and stress state are in-situ factors that must be duplicated in the laboratory to obtain accurate resilient moduli for design purposes. System compliance factors such as strain measurement and loading pulse shape are also influential in laboratory resilient modulus testing.

A study of 13 independent agencies with resilient modulus test equipment was conducted to examine the variability of laboratory resilient modulus. Three synthetic specimens were tested using the AASHTO T294-92 resilient modulus specification. An individual laboratory could produce repeatable results with an average variation in resilient modulus of only six percent. However when viewed collectively the average variation for two of the specimens was 22% and 40% for the third. Therefore the resilient modulus is partially a function of the laboratory that does the test.

Alternative resilient modulus test methods have been proposed. Determining resilient modulus from the shearing strain measured by torsional shear (TS) and resonant column (RC) tests have been examined. A modified gyratory test machine (GTM) normally used for testing asphalt concrete specimens has also been used to measure resilient modulus of soil specimens. The GTM may be a good alternative to standard resilient modulus testing because it is a combination kneading, dynamic consolidation, and shear testing machine. The GTM also simulates shear stress reversals inherent to moving loads. However, additional research is needed for the GTM to become a practical method. Drumm et al. (1996) developed an alternative test method (ATM) resembling a small-scale FWD device. The ATM apparatus consists of a standard Proctor mold, an instrumented drop hammer, and an oscilloscope.

Several constitutive relationships have been used to represent the nonlinear behavior of resilient modulus. These equations relate resilient modulus to stress conditions for use in numerical models for pavement performance (Equations 1 through 7). The constitutive relationships are often soil type dependent. Type 1 soils are most affected by bulk stress (θ), which is the sum of the deviator stress (σ_d) and three times the confining pressure (σ_3). Equations 1 through 4 are used for type 1 soils. Type 2 soils are most affected by the deviator stress. Equations 5 through 7 are used for type 2 soils. Good correlation between the K_n constants aids designers in choosing compatible pairs of K_n constants. For type 1 soils the best K_n constant correlation was found using Equation 1. For type 2 soils the best K_n constant correlation was found using Equation 6. The atmospheric pressure term (P_a) in Equations 2, 3, and 6 makes these relationships unit independent. Therefore they can be used with SI (kPa) or English (psi) units.

$$M_R = K_1(\theta)^{K_2} \qquad \text{(Type 1 soil)} \qquad \text{(Eq. 1)}$$

$$M_R = K_3 P_a \left(\frac{\theta}{P_a}\right)^{K_4} \qquad \text{(Type 1 soil)} \qquad \text{(Eq. 2)}$$

$$M_R = K_5 P_a \left(\frac{\theta}{P_a}\right)^{K_6} \left(\frac{\sigma_d}{P_a}\right)^{K_7} \qquad \text{(Type 1 or 2 soil)} \qquad \text{(Eq. 3)}$$

$$M_R = K_8(\sigma_{cyclic})^{K_9} K_{10}(1+\sigma_3)^{K_{11}} \qquad \text{(Type 1 soil)} \qquad \text{(Eq. 4)}$$

$$M_R = K_{12}(\sigma_d)^{K_{13}} \qquad \text{(Type 2 soil)} \qquad \text{(Eq. 5)}$$

$$M_R = K_{14} P_a \left(\frac{\sigma_d}{P_a}\right)^{K_{15}} \qquad \text{(Type 2 soil)} \qquad \text{(Eq. 6)}$$

$$M_R = K_{16}(\sigma_d)^{K_{17}} K_{18}(1+\sigma_3)^{K_{19}} \qquad \text{(Type 2 soil)} \qquad \text{(Eq. 7)}$$

Correlation between resilient modulus and soil index properties, strength parameters, and CBR has been examined. Fairly accurate equations for predicting resilient modulus can be developed using statistical regression analysis if enough data is available. Drumm et al. (1993) identified possible soil index properties that could be used in regression analyses (Table 1). Drumm et al. (1993) determined that the best correlation (Equation 8, R^2=0.70) was developed using deviator stress (σ_d), deviation from maximum dry density ($\Delta\gamma_{dmax}$), liquidity index (*LI*), percent saturation (*S*), classification (*class*), deviation from optimum water content (ΔW_{opt}), confining pressure (σ_3), and plasticity index (*PL*). Laguros et al. (1993) correlated cohesion (*C*), major principle stress (σ_1), internal friction angle (ϕ), bulk stress (θ), and elasticity (E) to resilient modulus with reasonable accuracy (Equation 9, R^2=0.7336, and Equation 0, R^2 not reported).

$$\log M_R(psi) = 46.93 + 0.0188\sigma_d + 0.0333\Delta\gamma_{d\max} - 0.1143LI + 0.4680S$$
$$+ 0.0085class^2 - 0.0033\Delta W_{opt}{}^2 - 0.0012\sigma_3{}^2 + 0.0001PL^2 \qquad \text{(Eq. 8)}$$
$$+ 0.0278LI^2 - 0.0017S^2 - 38.44\log S - 0.2222\log\sigma_d$$

$$M_R(psi, C, \phi) = 2860.94 + 275C + 128\sigma_1\tan\phi + 118\theta \qquad \text{(Eq. 9)}$$

$$M_R(psi, \sigma_3, \theta) = (18.28 + 0.4917\sigma_3)0.4098 + 150.7\theta \qquad \text{(Eq. 10)}$$

TABLE 1 - Soil index properties and resilient modulus test data used for regression analysis conducted by Drumm et al. (1993).

Soil Index Property and Resilient Modulus Test Data	Symbol
Liquid limit	LL
Plastic limit	PL
Plasticity index	PI
Liquidity index	LI
Percent passing the No. 200 sieve by washing (e.g. 10.2%=10.2)	P_{200}
Percent clay (e.g. 20.4%=20.4)	P_{clay}
AASHTO classification (e.g. A-7-6=7.6 or A-1-a=1.1)	$class$
Specific gravity	G
CBR at 2.54 mm penetration	$CBR_{2.54}$
CBR at 5.08 mm penetration	$CBR_{5.08}$
Optimum water content	W_{opt}
Maximum dry density	γ_{dmax}
Resilient modulus (ksi)	M_r
Confining pressure (psi)	σ_3
Deviator stress (psi)	σ_d
Dry density of the specimen (pcf)	γ_d
Water content of the specimen	W_s
Deviation from maximum dry density	$\Delta\gamma_{dmax}$
Deviation from optimum water content	ΔW_{opt}
Percent saturation (e.g. 30.6%=30.6)	S
Initial tangent modulus from unconfined compression tests	$1/a$
Parameter corresponding to unconfined compressive strength	$1/b$

Although the California bearing ratio has been used extensively to estimate the strength characteristics of a soil, CBR does not correlate well to resilient modulus. The static loading in the CBR test does not represent the repetitive dynamic loading characteristics of the resilient modulus test. However, a commonly referenced correlation between CBR and resilient modulus of fine grained soils (Equation 11) is suggested by the AASHTO Guide (AASHTO 1993b). Comparisons of the moduli from Equation 11 to laboratory test moduli (Drumm et al, 1993) show a wide range of variation including some moduli outside the range of 3 to 0.75 times CBR (Figure 1).

$$M_R(CBR) = 1.5CBR \text{ ksi} \qquad \text{(Eq. 11)}$$

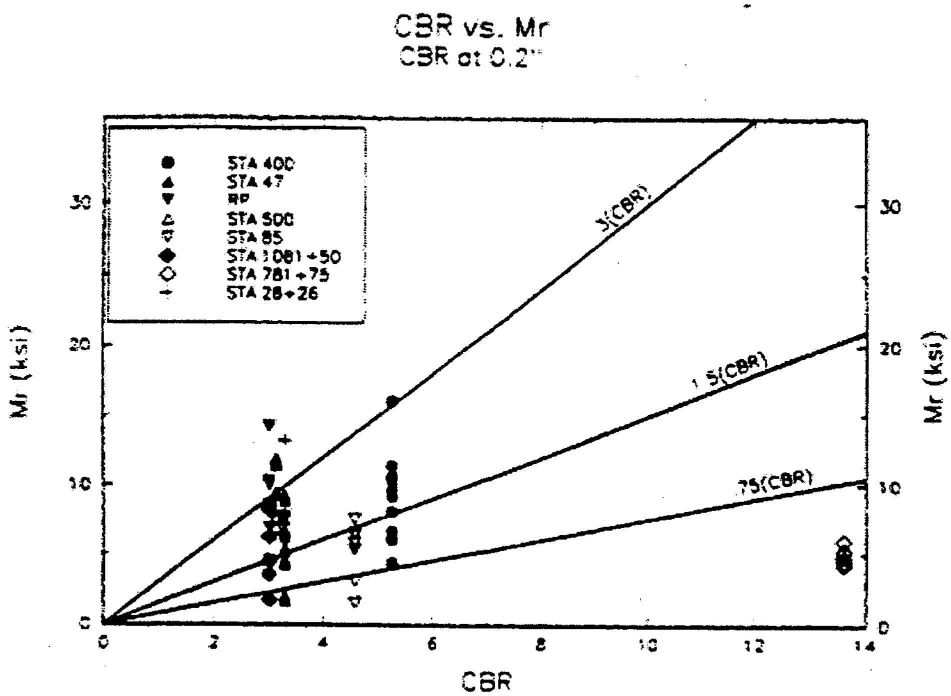


FIGURE 1 - Resilient moduli of eleven type 2 soils from Tennessee compared to correlations with CBR (Drumm et al., 1993).

The AASHTO Guide also recommends factors for Equation 11 for changes in bulk stress. Actual factors backcalculated from laboratory resilient modulus tests have been shown to be an average 75% smaller than those given by AASHTO (Laguros et al., 1993). The results of the comparison are shown in Table 2.

TABLE 2 - Correlation between resilient modulus and CBR as a function of bulk stress as recommended by the AASHTO Guide and as determined by Laguros et al. (AASHTO, 1993b; Laguros et al., 1993).

Bulk Stress, θ (psi)	M_R(*CBR*, in ksi) AASHTO Guide	M_R(*CBR*, in ksi) Laguros et al.	Laguros et al. vs. AASHTO Guide
100	0.74 *CBR*	0.193 *CBR*	74% lower
30	0.44 *CBR*	0.096 *CBR*	78% lower
20	0.34 *CBR*	0.082 *CBR*	76% lower
14	0.288 *CBR* *	0.074 *CBR*	74% lower
10	0.25 *CBR*	n/a	n/a

* Value computed assuming a linear relationship between bulk stresses of 30 and 10-psi.

In resilient modulus testing soils are divided into two types. Type 1 soils are generally granular bases and type 2 soils are cohesive subbases and subgrade soils. The laboratory resilient moduli of six type 1 and eleven type 2 soils from Maine were measured by Law Engineering (1992) using the Strategic Highway Research Program's (SHRP) Protocol 46 resilient modulus testing procedure. This data was used in conjunction with soil index and falling weight deflectometer (FWD) data obtained from the SHRP Long Term Pavement Performance (LTPP) database to examine three methods for resilient modulus determination.

The first method used FWD data from road sections in Brunswick, Damariscotta, North Freeport, and South Freeport. Two backcalculation software packages, MODCOMP 4 (Irwin, 1997) and MODULUS 5.1 (Michalak and Scullion, 1995) were used for the analysis. The LTPP database contained a large amount of FWD loading and deflection data. On the average, 16 FWD tests were done at every 7.62-m (25-ft) station for 152.4 m (500 ft). The exact location of the laboratory specimens were unknown therefore the average deflection basin profile and load pressure for every 30.5 m (100 ft) were used in the analysis. The five backcalculated moduli were averaged to obtain a single representative value to compare against the laboratory resilient modulus.

The backcalculated resilient moduli were compared to the laboratory resilient moduli. The backcalculated resilient moduli from MODCOMP 4 (Figure 3) and MODULUS 5.1 did not correlate well with the laboratory moduli when the program automatically estimated the depth to underlying hard layer. However, after removing two unreasonably high moduli from the eight data points (M_R > 2000 MPa) in the initial MODULUS 5.1 analysis a reasonably favorable correlation was acheived (R^2=0.76) (Figure 4). Furthermore the remaining six moduli were within the same magnitude as those predicted by the initial MODCOMP 4 analysis.

The LTPP database did not provide depths to hard layer for the four sites. However, the depths to refusal of a standard split spoon test were available for two of the sites. These depths of refusal did not compare well with those estimated by the programs. Using these refusal depths for depths to hard layer did not produce good correlation between backcalculated and laboratory resilient moduli. Therefore refusal depth should generally not be used for the depth to hard layer in resilient modulus backcalculation.

The effect of Poisson's ratio on backcalculated resilient modulus was also examined. The backcalculated resilient modulus of base and subgrade soils from MODULUS 5.1 are less sensitive to changes in Poisson's ratio of surface and subbase layers than the backcalculated resilient moduli from MODCOMP 4.

The second method used the constants and equations from the Law Engineering (1992) test data to develop equivalent constants for seven commonly used constitutive resilient modulus equations. Although the database is very small, a range of constants for use with Maine soils for the seven equations was developed. The average and standard deviation of each coefficient was also reported. The results of this analysis are shown in Tables 2 and 3.

The final approach was done by performing a linear regression of the soil index and resilient modulus test data to develop correlations to resilient modulus. Soil index properties and testing data from the Maine soils were used to develop a database for a statistical analysis software program. The program performed a forward and backward stepwise regression adding and removing independent variables based on their initial correlation with the dependent variable, laboratory resilient modulus. The number of independent variables was limited to one less that the number of soil specimens. For type 1 soils the best correlation was found using Equation 12. The correlation coefficient was very high (R^2=0.991) and the standard error was moderate (2003 psi). The laboratory test data fit within the range of minimum and maximum predicted resilient moduli. The same approach was used for type 2 soils. The best correlation was found using Equation 13. The correlation coefficient for the best fit line was very high (R^2=0.996) and the standard error was 950 psi. The results of the regression are shown in Table 4.

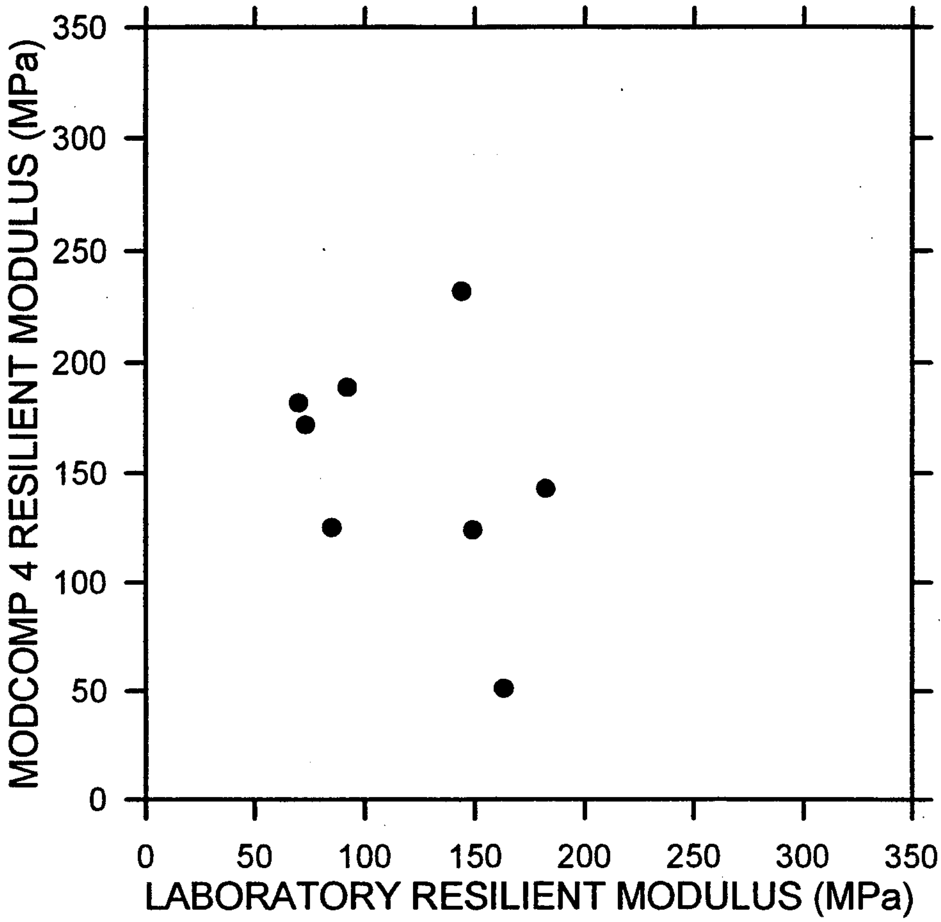


FIGURE 3 - Laboratory resilient moduli versus MODCOMP 4 resilient moduli with depth to hard layer as estimated by MODCOMP 4.

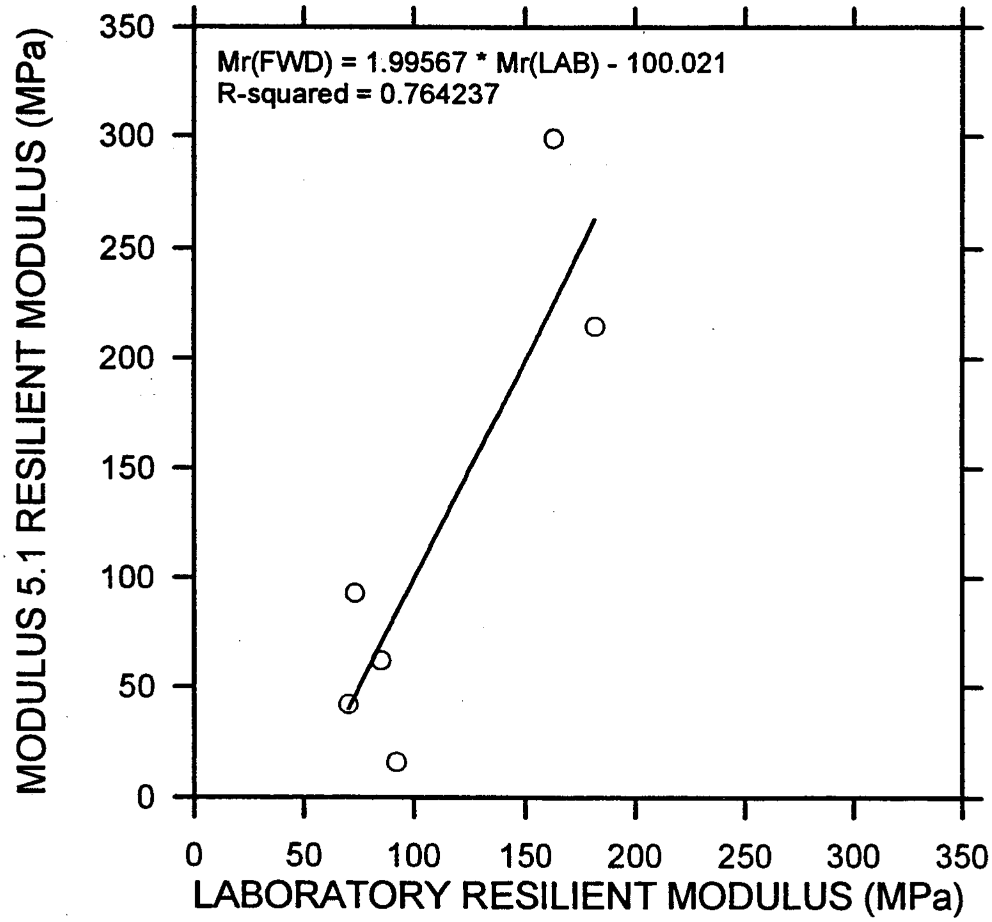


FIGURE 4 - Laboratory resilient moduli versus MODULUS 5.1 resilient moduli less than 2000 MPa with depth to hard layer as estimated by MODULUS 5.1.

TABLE 3 - Average, range, and standard deviation of the K_n constants for the equations in Chapter 4 from six type 1 Maine soils.

Number	Equation	M_R units	K_n	Range of Values	AverageValue	Standard Deviation
Eq. 4-1	$M_R = K_1(\theta)^{K_2}$	psi	K_1	675 to 2001	1356	485
		psi	K_2	0.465 to 0.862	0.707	0.136
Eq. 4-2	$M_R = K_3 \mathrm{P}_a \left(\frac{\theta}{\mathrm{P}_a}\right)^{K_4}$	psi or Pa	K_3	466 to 798	642	138
		psi or Pa	K_4	0.465 to 0.862	0.707	0.136
Eq. 4-3	$M_R = K_5 \mathrm{P}_a \left(\frac{\theta}{\mathrm{P}_a}\right)^{K_6} \left(\frac{\sigma_d}{\mathrm{P}_a}\right)^{K_7}$	psi or Pa	K_5	449 to 1101	679	245
		psi or Pa	K_6	0.337 to 0.931	0.729	0.204
		psi or Pa	K_7	-0.001 to 0.185	-0.019	0.121

TABLE 3 (continued) - Average, range, and standard deviation of the K_n constants for the equations in Chapter 4 from six type 1 Maine soils.

Eq. 4-4	$M_R = K_8(\sigma_{cyclic})^{K_9} K_{10}(1+\sigma_3)^{K_{11}}$	psi	K_8	1914 to 4240	3058	1006
		psi	K_9	0.091 to 0.240	0.156	0.058
		n/a	K_{10}	assumed = 1.0	n/a	n/a
		psi	K_{11}	0.562 to 0.755	0.636	0.070

TABLE 4 - Average, range, and standard deviation of the K_n constants for the equations in Chapter 4 from nine type 2 Maine soils.

Number	Equation	M_R units	K_n	Range of Values	Average Value	Standard Deviation
Eq. 4-5	$M_R = K_{12}(\sigma_d)^{K_{13}}$	psi	K_{12}	8076 to 10520	8897	755
		psi	K_{13}	-0.041 to 0.184	0.083	0.070
Eq. 4-6	$M_R = K_{14} P_a \left(\frac{\sigma_d}{P_a}\right)^{K_{15}}$	psi or Pa	K_{14}	561 to 928	755	100
		psi or Pa	K_{15}	-0.041 to 0.184	0.083	0.070
Eq. 4-7	$M_R = K_{16}(\sigma_d)^{K_{17}} K_{18}(1+\sigma_3)^{K_{19}}$	psi or Pa	K_{16}	3289 to 4619	3863	431
		psi or Pa	K_{17}	-0.042 to 0.187	0.084	0.071
		n/a	K_{18}	assumed = 1.0	n/a	n/a
		psi or Pa	K_{19}	0.463 to 0.642	0.546	0.049

Unfortunately, the database was not large enough to test the validity of Equations 12 and 13 on soil specimens outside the database. Each soil in the database was at a single set of soil index properties. Therefore the changes in resilient modulus due to differential water contents, dry densities, and other properties could not be accurately accounted for. However, removing the stress terms from the equations produced excessively high standard error estimates for both soil types (Type 1 standard error of estimate > 8300 psi, Type 2 standard error of estimate > 2200 psi). The resilient modulus of type 1 soils in the database is highly dependent on bulk stress. Values for the bulk stress term in Equation 12 should be between 12 and 100 psi. The resilient modulus of type 2 soils in the database is highly dependent upon confining pressure. Values for the deviator stress term in Equation 13 should be between 2 and 10 psi. Values for the confining pressure term in Equation 13 should be between 2 and 6 psi.

TABLE 5 - Linear regression equations correlating soil index properties and resilient modulus test data of six type 1 soils and eight type 2 soils from Maine.

Number	Equation	Standard Error of Estimate
12	$M_R(LRA\,Type1) = -6350\Delta\gamma_{d\,\max} + 170S - 280\%pass25mm + 730\%pass2mm + 330\theta$	2003 psi
13	$M_R(LRA\,Type2) = 263\Delta\gamma_{d\,\max} - 234W_{opt} + 31S + 165\%pass76mm - 34\%pass0.08mm + 190\sigma_d - 1215\sigma_3$	950 psi

NOTE: Equations 6-8 and 6-11 should not be used to estimate the resilient modulus of soils from Aroostook County.

where: $\Delta\gamma_{dmax}$ = difference between maximum dry density and dry density at time of testing in pcf
W_{opt} = optimum water content
S = percent saturation (percent, e.g. 98.1% = 98.1)
%pass76mm = percent passing 76 mm (3-in.) sieve (percent)
%pass25mm = percent passing 25 mm (1-in.) sieve (percent)
%pass2mm = percent passing 2 mm (0.08-in.,#10) sieve (percent)
%pass0.08mm = percent passing 0.08 mm (0.003-in., #200) sieve (percent)
θ = bulk stress in psi
σ_d = deviator stress in psi
σ_3 = confining pressure in psi

To summarize, there are several practical methods to determine resilient modulus for use in pavement design. Computer software can be used to backcalculate the in-situ resilient modulus of pavement and soil layers using FWD data. Backcalculation requires knowledge of depth to hard layer, Poisson's ratio, surface and soil layer configuration, and reasonable initial values of resilient modulus. MODCOMP 4 and MODULUS 5.1 do not backcalculate resilient moduli that correlate well with moduli obtained from laboratory tests. Several constitutive relationships (Equations 1 through 7) based on stress state are available and have been shown to accurately predict resilient modulus. The regression constants (K_n) for six type 1 soils and nine type 2 soils from Maine have been determined. The range, average, and standard deviation for the regression constants of these soils are available and can be applied to similar soil types having corresponding classification, gradation, water content, and dry density. Resilient modulus can also be determined by correlating resilient modulus to soil index property data and stress state. A correlation equation for each soil type has been developed for Maine soils. The type 1 equation (Equation 12) was developed using resilient modulus and soil index data from six type 1 Maine soils whereas the type 2 equation (Equation 13) was developed from nine type 2 Maine soils. To use the equations, the stress state (bulk stress, deviator stress, and confining pressure), deviation from dry density, optimum water content, and grain size distribution must be known. Equations 12 and 13 have not been tested using soils outside the database. Furthermore, some soil index properties such as liquidity index, plasticity index, and plastic limit were not available for use in the regression analysis. Therefore equations 12 and 13 should be used with caution.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 Background

In the past two decades, much emphasis has been placed on rehabilitation and improvement of the nation's transportation system. This prompted national organizations such as the Transportation Research Board (TRB) and American Association of State Highway and Transportation Officials (AASHTO) to sponsor research leading to better methods of predicting pavement failures such as cracking and rutting. Past design methods have relied heavily on properties that measure strength and penetration resistance, such as the California bearing ratio (CBR). Recognizing that the present CBR testing procedure does not accurately represent the conditions leading to typical modes of pavement failure, much research has been directed at developing the concept of resilient modulus which is a measure of the elasticity of a soil layer.

The Maine Department of Transportation (MDOT) commissioned this study to determine a reasonable procedure for measuring resilient modulus of subbase and subgrade soils. Resilient modulus is now recommended by AASHTO as the basis for pavement design (AASHTO, 1993a). Previously, MDOT used design methods based on CBR and structural number (SN). The CBR test has several disadvantages. The testing procedures are relatively time consuming and relatively large samples are required. Moreover, the pavement design models based on CBR do not accurately represent field conditions. Although MDOT owns laboratory equipment to determine resilient modulus, it is outdated and no longer adequate. The results of this project provide MDOT with guidance for determination of resilient modulus and related parameters for typical Maine subbase and subgrade soils.

1.2 Definition of Resilient Modulus

Resilient modulus (M_R) is defined as the slope of the applied stress vs. deformation relationship in response to a cyclic load as shown in Figure 1.1.

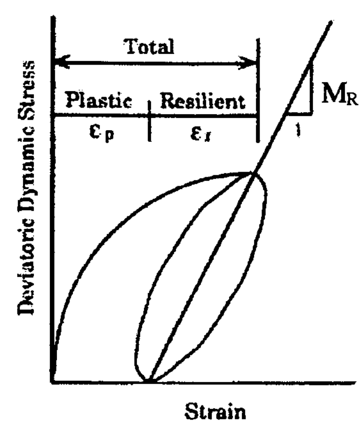


FIGURE 1.1 Sketch of the stress / strain relationship and resilient modulus (Laguros et al., 1993).

Resilient modulus is highly dependent on stress level, soil type, environmental conditions, and construction processes. Its accuracy and variability are influenced by sampling, type of testing equipment, testing procedures, technician skill levels, and use of proper mathematical models. A high resilient modulus subgrade is "stiffer" than a subgrade with a lower resilient modulus and will deform less under the same applied stress. Resilient modulus can be used to gage the elastic behavior of in situ asphalt, base, subbase, and subgrade layers. Use of resilient modulus in pavement design requires sound engineering judgment (AASHTO, 1993a).

1.3 Reasons for Changing the AASHTO Design Procedure

As a result of the research efforts of universities, states, and private agencies nationwide, the AASHTO 1993 Guide for Design of Pavement Structures (AASHTO Guide) changed from a CBR based approach to using resilient modulus for the reasons given below.

1. The CBR is a measure of penetration resistance whereas the resilient modulus is a measure of elastic behavior and therefore is more representative of actual traffic loading.
2. The resilient modulus is a basic material property unique to soil type that can be used in a mechanistic analysis of multilayered systems for predicting pavement roughness, cracking, rutting, and faulting (AASHTO, 1993a).
3. The resilient modulus of many in situ materials can be estimated by nondestructive tests such as falling weight deflectometer (AASHTO, 1993a).
4. The resilient modulus has been recognized internationally as a method for characterizing materials for use in pavement design and evaluation (AASHTO 1993a).

AASHTO specification T294-92, "Resilient Modulus of Unbound Granular Base/Subbase Materials and Subgrade Soils - Strategic Highway Research Program (SHRP) Protocol P46" (AASHTO T294-92), gives the current laboratory procedure for resilient modulus determination (AASHTO, 1993b). Several AASHTO Guide supplemental pamphlets have been published by the Federal Highway Administration (FHWA) to aid agencies in the development of testing facilities and design procedures (Alavi et al., 1997, Killingsworth and Von Quintus, 1997; Von Quintus and Killingsworth, 1997a,b; Ostram et al., 1997). The AASHTO Guide recommends that its user agencies purchase modernized equipment if extensive and routine testing is to take place (AASHTO, 1993a). Recognizing that the equipment is expensive, the AASHTO Guide also provides alternative methods for computing resilient modulus from non-destructive testing (NDT) and CBR conversion.

1.4 Resilient Modulus Specifications

The current specification for determining the resilient modulus of soils and aggregate materials is AASHTO TP46-94 (AASHTO, 1995). Because it is a relatively new specification, its procedures were not followed by the cited literature. The reviewed literature uses several preceding versions of AASHTO TP46-94. They are AASHTO T274-82, SHRP Protocol P46, and AASHTO T294-92. The predominant specification used by the reviewed literature is AASHTO T294-92 therefore its methodology will be discussed in Chapter 2.

1.5 Overview

This report begins with a literature review of the laboratory methods associated with resilient modulus determination. Wherever possible the data and results from the cited literature are reported in SI units. Environmental factors affecting resilient modulus are discussed. This report also examines several mathematical models used to compute resilient modulus from laboratory testing. Alternative methods for determining resilient modulus are examined including correlations to soil index properties and backcalculation techniques employing computer software. Software packages were used to develop empirical relationships for subgrade soils native to Maine with data stored in the SHRP Long Term Pavement Performance (LTPP) database. The report discusses the results of a comparison between laboratory methods and backcalculation techniques, and concludes with recommendations for a reasonable approach for determining resilient modulus and associated parameters of subbase and subgrade soils for roadway design in Maine.

CHAPTER 2

LABORATORY DETERMINATION OF RESILIENT MODULUS

Specimen preparation and testing procedures have been the subject of much scrutiny in recent years. The original specification has been modified twice since originally written in 1982. AASHTO expects additional modifications in the near future as the result of ongoing research. Since the magnitude of resilient modulus is highly dependent on proper specimen preparation and laboratory testing criteria, these factors must accurately represent the in-place conditions of the specimen in question. This chapter gives an overview of the current testing procedure and apparatus. It also discusses possible sources of error due to specimen identification, preparation, conditioning, deviator stress, confining pressure, and strain measurement.

2.1 1993 AASHTO Specification T294-92 and SHRP Protocol P46 Test Procedure

The test procedure for determining the resilient modulus of unbound granular base and subbase materials and subgrade soil is given in AASHTO T294-92 (AASHTO, 1993b). This specification has been revised to include portions of Protocol P46 developed by the SHRP (SHRP Protocol P46) for resilient modulus testing. Using representative specimens, the procedure attempts to simulate the repeated loading a pavement, base, subbase, or subgrade will experience after construction. The testing methodology can be summarized as follows:

1. A bulk soil sample is prepared in a fashion similar to standard triaxial testing using techniques appropriate to its AASHTO classification.
2. The sample is placed in a triaxial cell. A load cell is attached between the loading frame and piston to monitor the applied deviator stress. The minor principle stress is provided by the confining pressure inside the cell. Measuring devices are attached to the specimen to record axial deformation. The sample is subjected to a conditioning phase prior to further repetitive loading.
3. The specimen is systematically loaded at different levels of confining pressure and deviator stress for a given number of repetitions at each level. Each loading repetition is called a "sequence".
4. The change in deformation (strain) for each sequence is used to determine resilient modulus for the corresponding stress state.
5. The resilient modulus for all sequences are graphed as functions of increasing bulk or deviator stresses.

Modern equipment and electronics are used to automate the testing procedure. Figures 2.1 and 2.2 show typical configurations of resilient modulus testing equipment and triaxial cell setup. The initial start up cost of the equipment can range from $60,000 to $80,000 (Pezo et al., 1992).

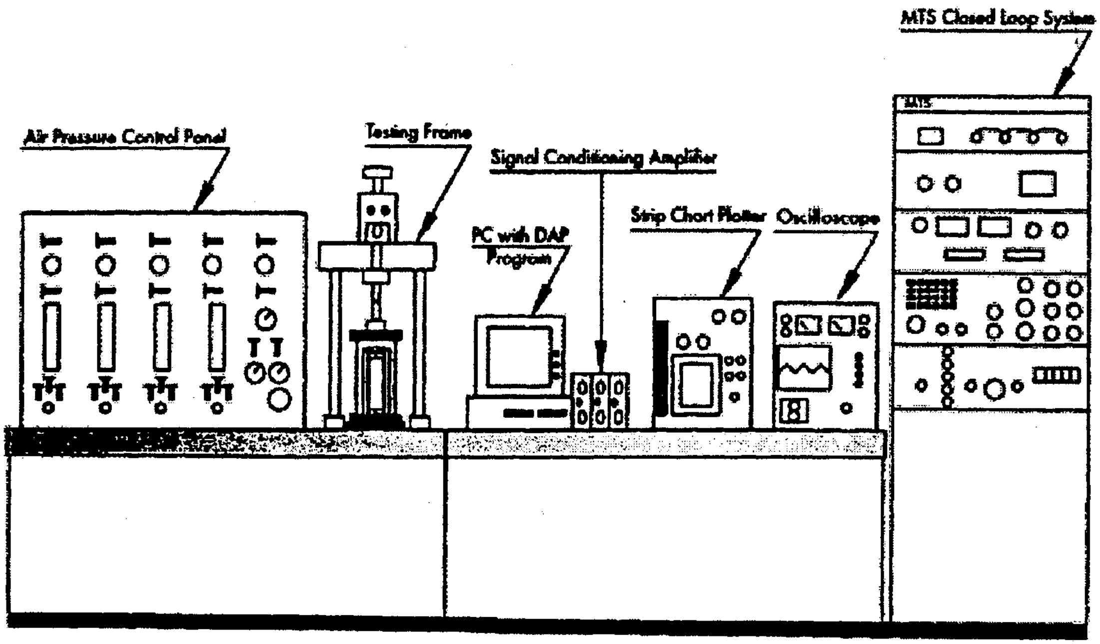


FIGURE 2.1 - Sketch of the resilient modulus testing equipment developed at the University of Texas at Austin (Pezo et al., 1992).

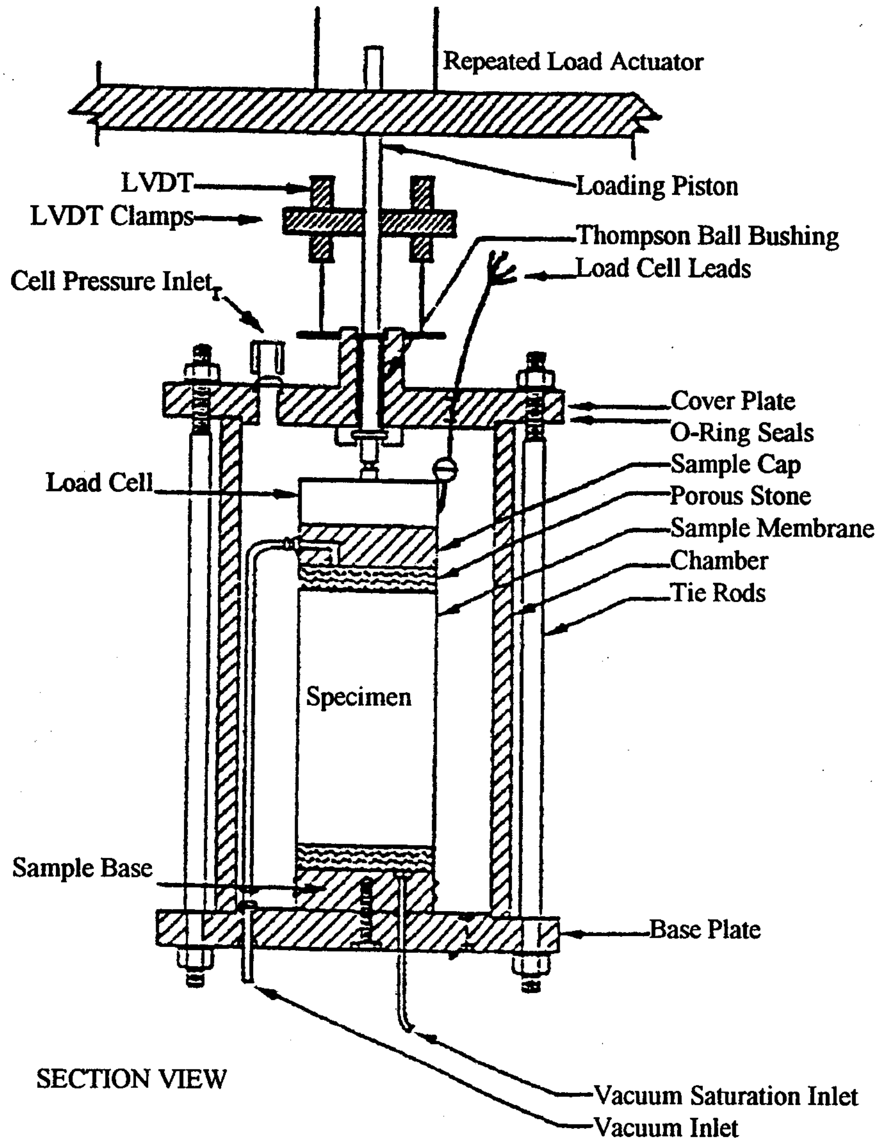


FIGURE 2.2 - Resilient modulus triaxial cell apparatus with external LVDTs (AASHTO, 1993b).

Even with an extensive configuration, uncertainties are introduced into the system by imperfections in the apparatus, apparatus configuration, and human error known collectively as system compliance. Technicians need to be highly skilled and familiar with soil testing procedure, electronics, and computer operation.

In 1994, the SHRP studied testing procedures and equipment (Steel et al., 1994). The purposes of this study were to verify calibration and results of the participating agencies, and to use the results to develop a practical quality assurance and control program (QA/QC). The study involved 13 laboratories completing a round-robin resilient modulus testing schedule of three synthetic reference samples under AASHTO T294-92. AASHTO T294-92 outlines two procedures for preparation and resilient modulus determination depending upon classification. Details of the classification are discussed in the next section. For this examination the synthetic specimens were classified as type 1 materials. Individually, the laboratories could produce resilient moduli within 6% of the average (Steel et al., 1994). The collective accuracy of the laboratories was within 22% for all of the specimens except the Teflon specimen which was 40% (Steel et al., 1994). From this information, the SHRP concluded that the test results within an individual laboratory are highly repeatable but varied considerably when compared between laboratories.

Alavi et al. (1997) published a standard QA/QC protocol for resilient modulus testing of unbound materials. The protocol was developed to improve the accuracy and reliability of the raw data collected using a closed-loop servo-hydraulic system. This protocol includes equipment requirements, performance and verification procedures for electronic systems and overall system calibration, technician proficiency, and testing protocol.

2.2 Specimen Identification

Resilient modulus specimens are identified according to AASHTO M145-91, "The Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes" (AASHTO, 1993b). Table 2.1 presents the criteria used for this classification system.

TABLE 2.1 - AASHTO classification of soils and soil-aggregate mixtures (AASHTO, 1993b).

GENERAL	GRANULAR MATERIALS Less than 35% passing 0.075 mm							SILT-CLAY MATERIALS Greater than or equal to 35% passing 0.075 mm			
GROUP	A-1			A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
SEIVE ANALYSIS % PASSING											
2.00 mm	50 max	–		–	–	–	–	–	–	–	–
0.452 mm	30 max	50 max	51 min	–	–	–	–	–	–	–	–
0.075 mm	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
CHARACTERISTICS OF FRACTION PASSING 0.452 MM											
Liquid Limit	–		–	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		NP	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min
OTHER											
Constituents	stone fragments, gravel and sand		fine sand	silty or clayey gravel and sand				silty soils		clayey soils	
Subgrade Rating	excellent to good							fair to poor			

AASHTO T294-92 separates soils into two different types based on AASHTO M145-91 classification. In general, type 1 materials are granular soils with little or no fines and type 2 materials are soils with greater amounts of silts and clays. More specifically, material type 1 includes all unbound granular base and subbase material and all untreated subgrade soils which meet the criteria of less than 70 percent passing the 2 mm sieve and maximum of 20 percent passing the 0.075 mm sieve. Type 1 materials include all A-1-a soils and may include A-1-b, A-2, and A-3 soils (AASHTO, 1993b). Material type 2 soils are all soils not meeting the type 1 criteria. A-4, A-5, A-6, and A-7 soils are always type 2 materials in addition to some A-1-b, A-2, and A-3 soils (AASHTO, 1993b).

TABLE 2.2 - AASHTO T294-92 material type determination.

Material Type AASHTO T294-92	Type 1	Type 1 or Type 2			Type 2			
Classification AASHTO M145-91	A-1-a	A-1-b	A-2	A-3	A-4	A-5	A-6	A-7
Condition	Always	Type 1 if unbound base or subbase and untreated subgrade. Type 2 if bound or treated.			Always			

Identifying the soil type can be confusing. Under AASHTO TP46-94 untreated granular base and subbase materials and all untreated subgrade soils which meet the T294-92 sieve criteria and have a plasticity index of ten or less are type 1 soils. Type 2 soils are all untreated bases and subbases as well as untreated subgrade soils that do not meet the T294-92 type 1 criteria.

2.3 Specimen Preparation

Soil type governs the method of specimen compaction, dry density, and water content. Undisturbed field specimens are preferred for laboratory resilient modulus testing, however, this is often not possible. AASHTO T294-92 provides guidelines for the reconstitution of disturbed type 1 and type 2 soils. The reconstitution of disturbed samples is a possible source of discrepancy because compaction method, relative density, and water content affect the value of resilient modulus.

2.3.1 Type 1 Specimen Preparation

Reconstituted type 1 specimens are laboratory compacted using a vibratory compactor in a minimum of three equal lifts. The specimen should be prepared within one percentage point of the in- situ moisture content and with three percentage points of the in-situ dry density if they are known. If the in-situ moisture content and dry density are unknown, the optimum moisture content and 95 percent of the maximum dry density as determined by AASHTO T180 should be used. Specimens are prepared in a metal split mold measuring 102 mm (4 in.) in diameter by 204 mm (8 in.) high or 152 mm (6 in.) in diameter by 304 mm (12 in.) high. The smaller mold is used for nominal particle sizes not exceeding 19 mm (3/4 in.) whereas the larger mold accommodates maximum particle sizes between 19 mm (3/4 in.) and 32 mm (1¼ in.) (AASHTO, 1993b).

Reproducing the in-place dry density is important since increased relative density is accompanied by an increase in resilient modulus. Hicks and Monismith (1971) compared the effect of variations in relative density due to vibratory compaction of crushed and partially crushed aggregates. Different relative densities were accomplished by increasing the number of layers in a 102-mm diameter by 204-mm high (4-in. by 8-in.) specimen. The vibration period was 15 seconds for each layer. As seen in Figure 2.3, the increase in resilient modulus due to density change is more pronounced in partially crushed aggregates than wholly crushed aggregates. Similar trends were observed by Ishibashi et al. (1984) when examining a well graded gravelly silty sand base (A-1-b).

Reproducing the in-place water content is also important since increased water content can decrease the resilient modulus of type 1 soils. Ishibashi et al. (1984) examined the effect of changing water content on resilient modulus of the same A-1-b sample mentioned in the previous paragraph. Figure 2.4 shows the reduction of resilient modulus due to small changes of water content.

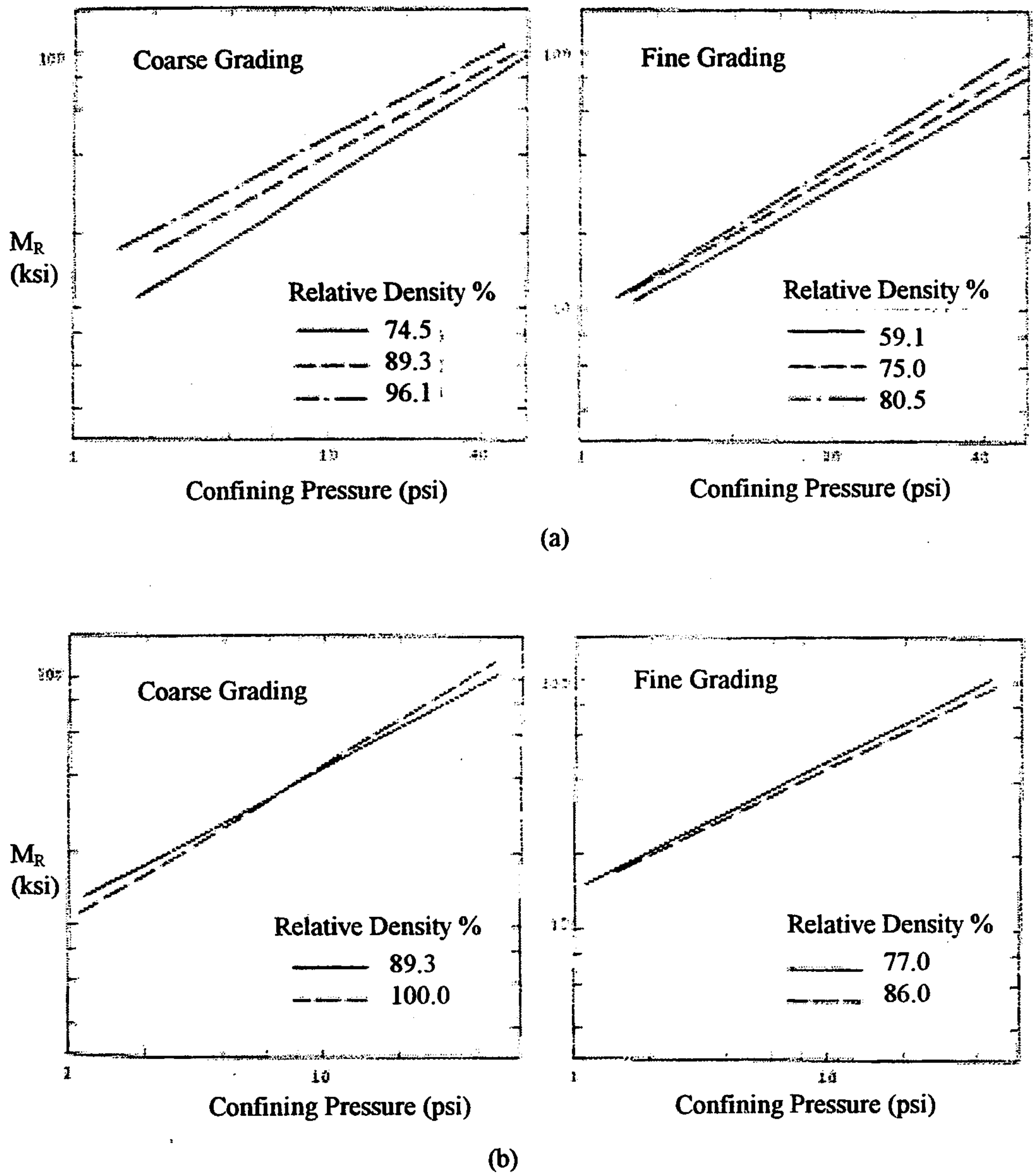


FIGURE 2.3 - Effect of relative density and confining pressure on resilient modulus of (a) partially crushed aggregate and (b) wholly crushed aggregate. (Hicks and Monismith, 1971).

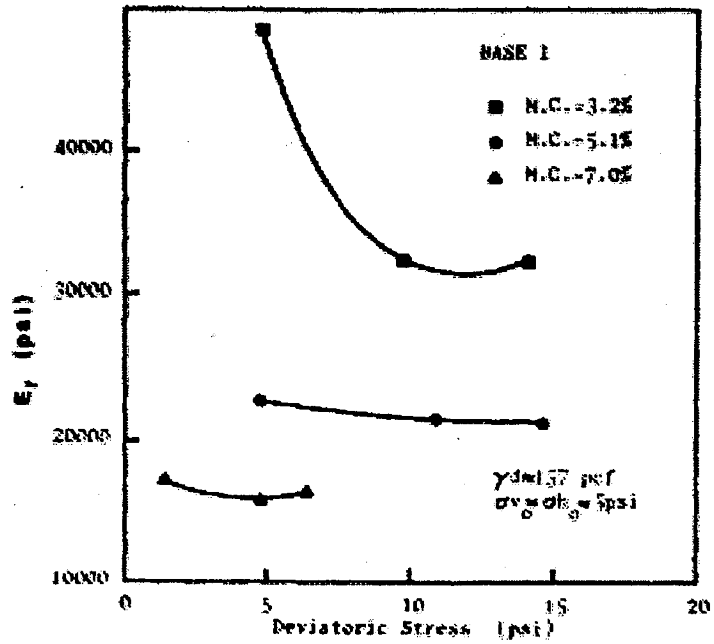


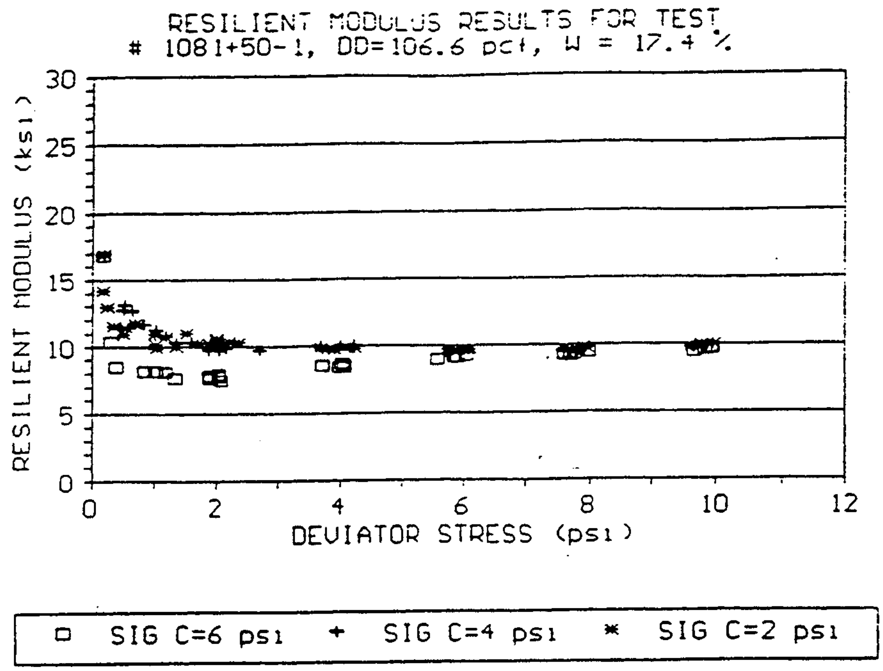
FIGURE 2.4 - Resilient modulus as a function of deviator stress and moisture content (Ishibashi et al., 1984).

2.3.2 Type 2 Specimen Preparation

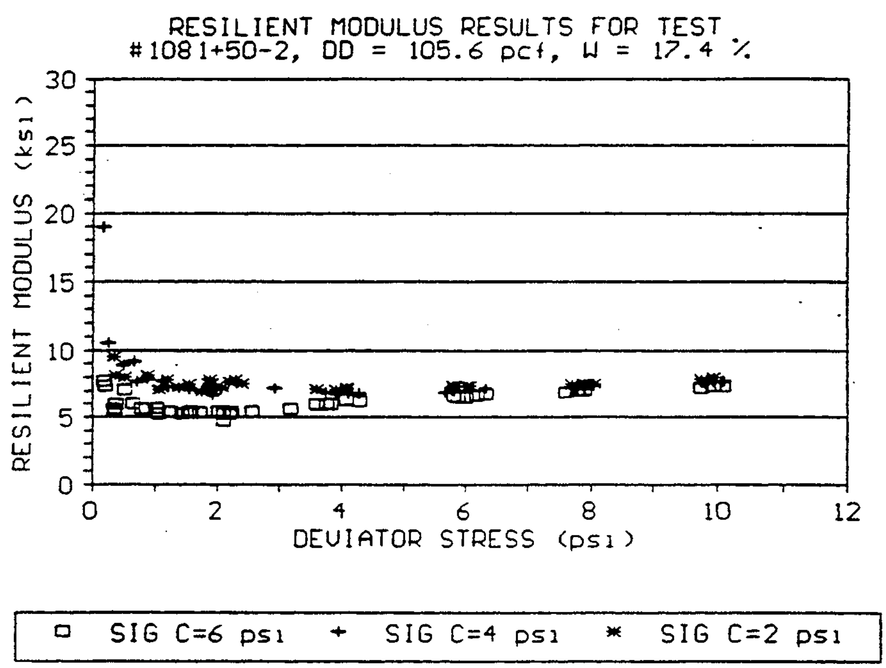
Type 2 soils are preferably taken directly from thin-walled sample tubes to minimize sample disturbance due to handling. If thin-walled samples are not available, type 2 soils are recompacted in a 71-mm (2.8-in.) diameter by 142-mm (5.6-in.) high specimen mold. If in-situ dry density and water contents are unknown, the specimen should be compacted at the optimum water content to within five percentage points of the maximum dry density. Vibratory compaction methods are not used for type 2 specimens. Type 2 soils are recompacted in five layers of equal mass by the static double plunger method. The double plunger method applies pressure from the top and bottom of the specimen to preserve uniform density throughout the specimen (AASHTO, 1993b).

Unfortunately the soil fabric of field specimens cannot always be reliably reproduced by laboratory compaction methods (Prapaharan et al., 1991). A medium plastic clay was compacted in the field by two methods: a 28,600-kg (63,000-lb) static segmented-pad roller and a 11,500-kg (25,000-lb) vibratory segmented-pad roller. Three different energy levels were applied. In addition, remolded specimens were compacted in the laboratory by kneading and impact methods. Kneading compaction was done using a pneumatic compactor and standard Proctor mold. Both methods were done using low-level, standard, and modified Proctor compaction energies. Pore size distribution curves were determined using the mercury intrusion method. For dry of optimum conditions, the soil fabric produced in the laboratory was not the same as the fabric produced in the field, regardless of method. For specimens prepared wet of optimum, no significant differences in soil fabric were noted (Prapaharan et al., 1991).

Small changes in dry density also affects the value of resilient modulus. For A-4 and A-6 soils studied by Drumm et al. (1993), an increase in resilient modulus accompanied an increase in dry density. Figure 2.5 shows the laboratory resilient modulus of an A-6 soil at high and low densities. Specimens were compacted in 71-mm (2.8-in.) diameter by 142-mm (5.6-in.) high molds using kneading techniques.



(a)

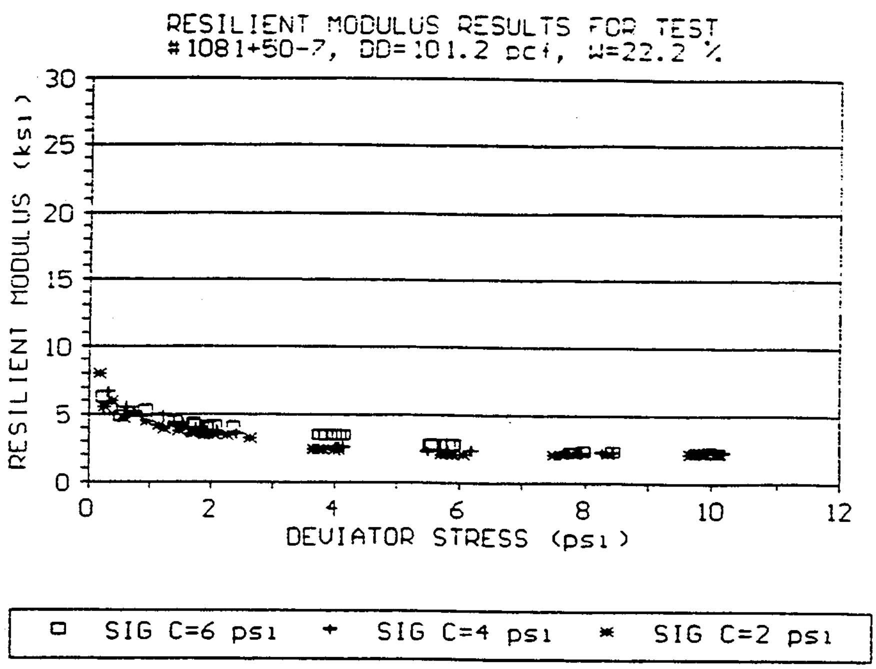


(b)

FIGURE 2.5 - Resilient modulus as a function of deviator stress at dry densities of (a) 16.7 kN/m^3 (106.6 pcf) and (b) 16.6 kN/m^3 (105.6 pcf) (Drumm et al., 1993).

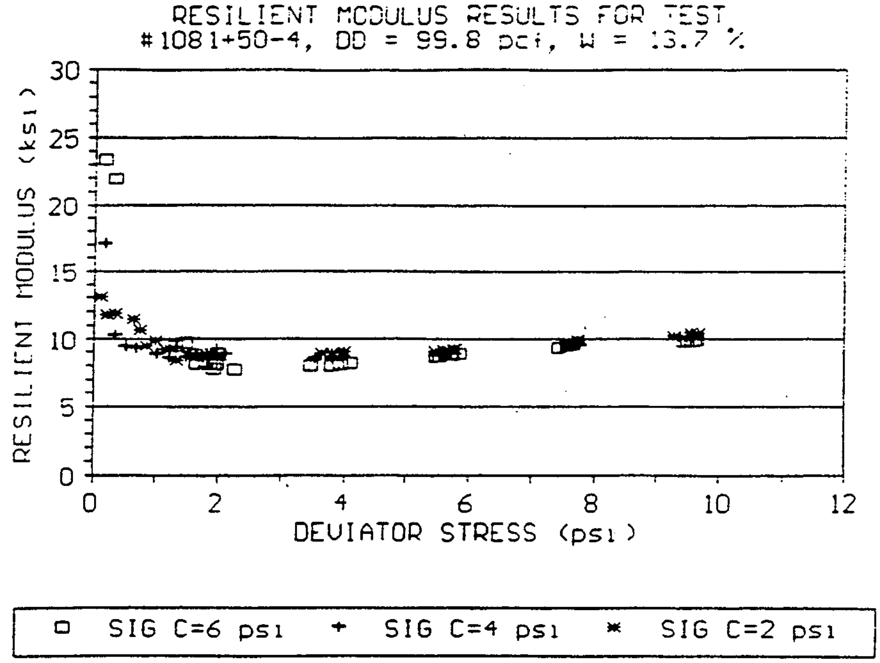
For type 2 soils increasing the moisture content decreases the resilient modulus. Figure 2.6 shows the laboratory resilient modulus of three A-6 specimens at moisture contents of 13.7%, 18.3% (optimum), and 22.2% (Drumm et al., 1993).

Increased time between specimen preparation and testing causes increased resilient modulus of type 2 soils due to thixotropic effects. Seed et al. (1962) studied several laboraotory compacted specimens of the same clay used as a subbase soil for the AASHO Road Test. The results of specimens with ages between fifteen minutes and fifty days are shown in Figure 2.7. Pezo et al. (1992) observed the same effect to varying degrees for fifteen type 2 soils including AASHTO classifications A-4, A-6, A-7, and A-7-6.

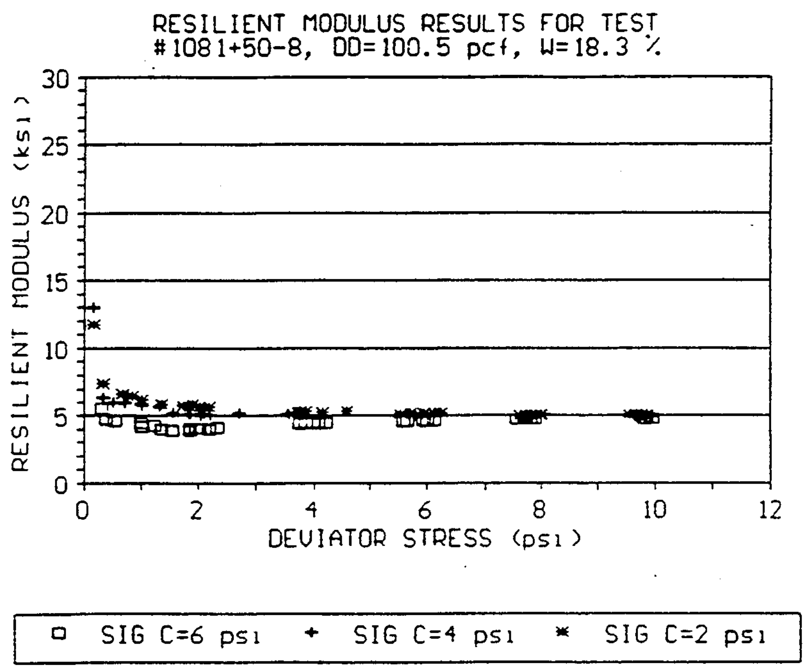


(a)

FIGURE 2.6 – Resilient modulus as a function of deviator stress with varying water content (a) 22.2% (Drumm et al., 1993).



(b)



(c)

FIGURE 2.6 - Resilient modulus as a function of deviator stress with varying water content (b) 13.7%, (c) 18.3% (optimum) (Drumm et al., 1993).

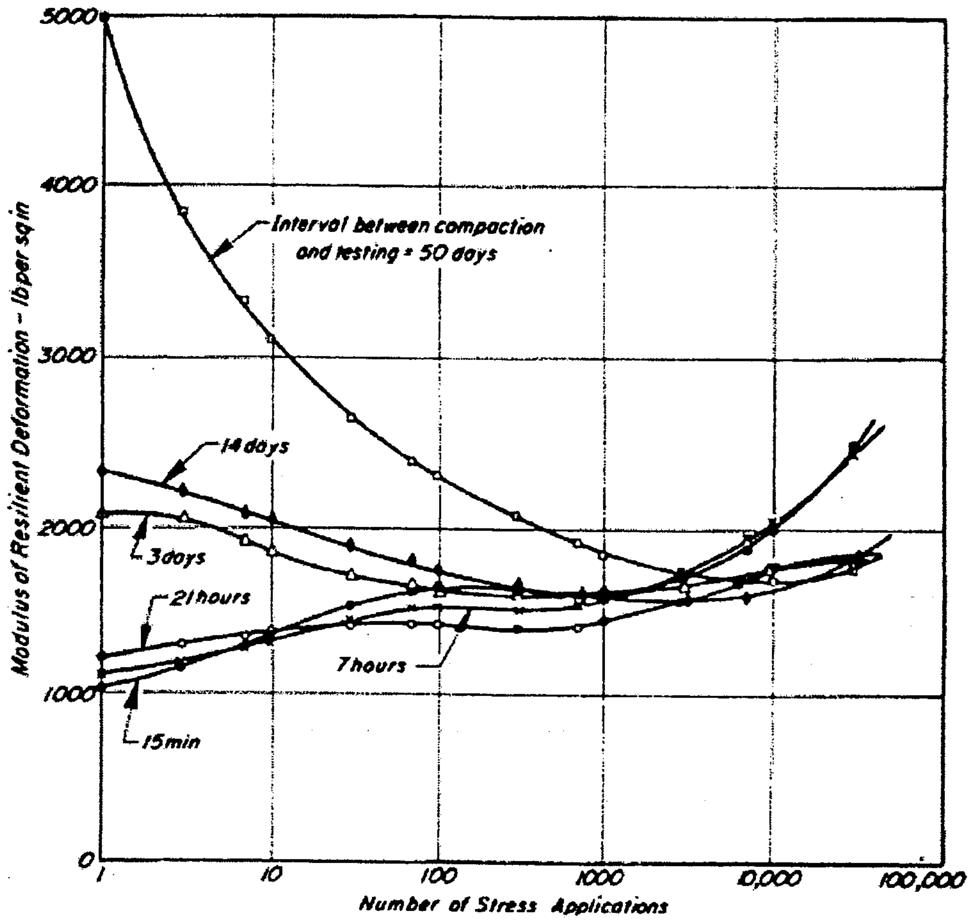


FIGURE 2.7 - Resilient modulus of a type 2 soil as a function of different specimen ages (Seed et al., 1962).

2.4 Specimen Loading

In laboratory resilient modulus testing, different stress conditions are applied cyclically to simulate the stress of a travelling vehicle above a given soil element. As shown in Figure 2.8, the major principle stress is simulated by the piston and end plate arrangement. The minor principle stress is simulated by the confining pressure exerted on the specimen by the fluid in the triaxial chamber. The deviator stress is the difference between the major and minor principle stresses. It is also referred to as the devatoric stress.

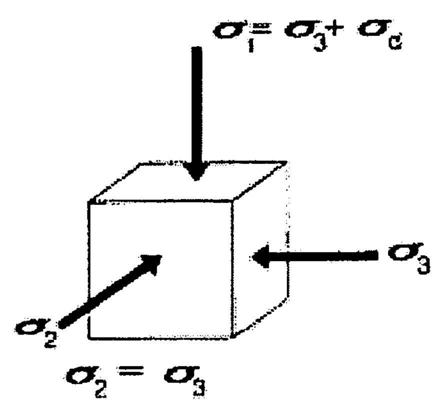


FIGURE 2.8 - Sketch of the major and minor principle stresses acting upon a soil element (Santha, 1994). σ_1 = major principle stress, σ_2 = intermediate principle stress, σ_3 = minor principle stress, and σ_d = is the deviator stress.

Different stress levels are applied to type 1 and 2 soils. Although each material type is preconditioned with 1,000 repetitions and subsequently tested through fifteen sequences each having 100 repetitions, the stress levels are lower for type 2 materials. The duration of the preconditioning sequence, variations in the magnitude of the deviator stress, number of principle stress repetitions and loading pulse shape, and variations in confining pressure levels affect the resulting magnitude of resilient modulus.

2.4.1 Preconditioning Sequence

The specimen preconditioning sequence is prescribed to reduce discrepancies caused by thixotropic effects between specimen preparation and the initial loading sequence. Preconditioning is also needed to improve intimate contact between the top and bottom of the specimen and the end plates of the triaxial apparatus. AASHTO T294-92 requires 1000 repetitions of prescribed deviator stress and confining pressure for this conditioning sequence. A "repetition" consists of a 0.1-second load pulse duration followed by a 0.9-second resting period. For conditioning type 1 soil, the deviator stress is 41 kPa (6 psi) and the confining pressure is 28 kPa (4 psi). For conditioning type 2 soil, the deviator stress and confining pressure are each 103 kPa (15 psi).

Evidence of decreased resilient modulus with increasing loading repetitions can be seen in Figure 2.7. Seed et al. (1962) observed that despite large differences in initial resilient moduli the resilient moduli converged to a similar value after 1,000 repetitions.

To create intimate contact between the specimen and the triaxial end plates, the concept of capping concrete cylinders with molten sulfur has been applied to resilient modulus testing. Pezo et al. (1992) compared grouted and non-grouted A-4 and A-7 soil specimens subjected to 2,000 load applications at a bulk stress level of 110 kPa (16 psi).

Figure 2.9 shows that for these soils, the resilient modulus of grouted specimens remain relatively unchanged with increasing load repetitions whereas the resilient modulus of the same ungrouted specimen is lower and increases slightly with increasing load repetitions. In sharp contrast to the grouted specimens, permanent deformation of ungrouted specimens increase drastically with increasing load repetitions.

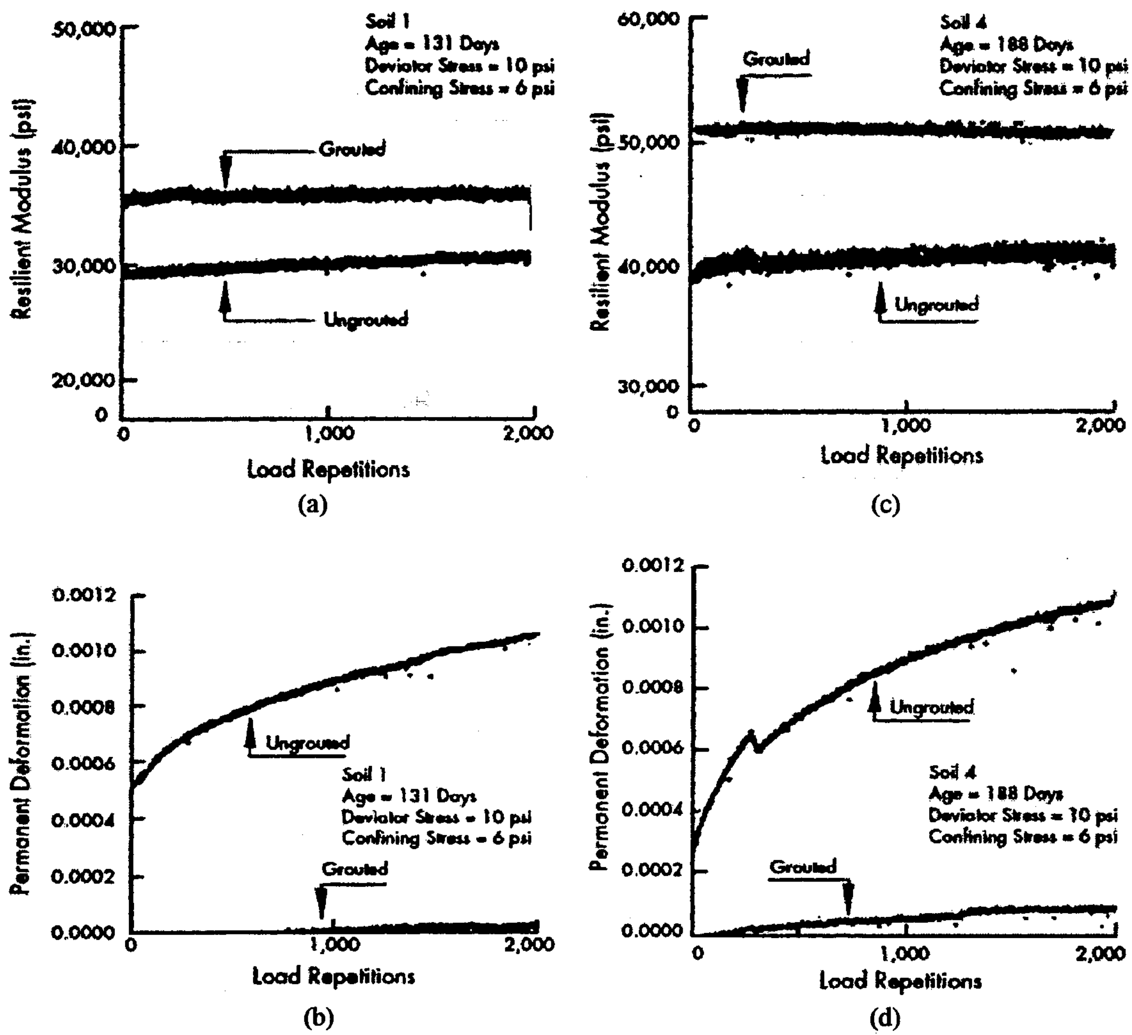


FIGURE 2.9 - Resilient modulus and permanent deformation of (a,b) an A-7 and (c,d) an A-4 soil (Pezo et al., 1992).

Grouting may reduce the number of repetitions needed to condition the specimen. Pezo et al. (1992) measured the resilient modulus of two A-4 soils, two A-6 soils, and four A-7-6 soils to 200 stress repetitions. The specimens were prepared and grouted in the same fashion as described above. For these grouted specimens the resilient modulus did not change significantly for the duration of the test. Representative results are shown in Figures 2.10 through 2.12.

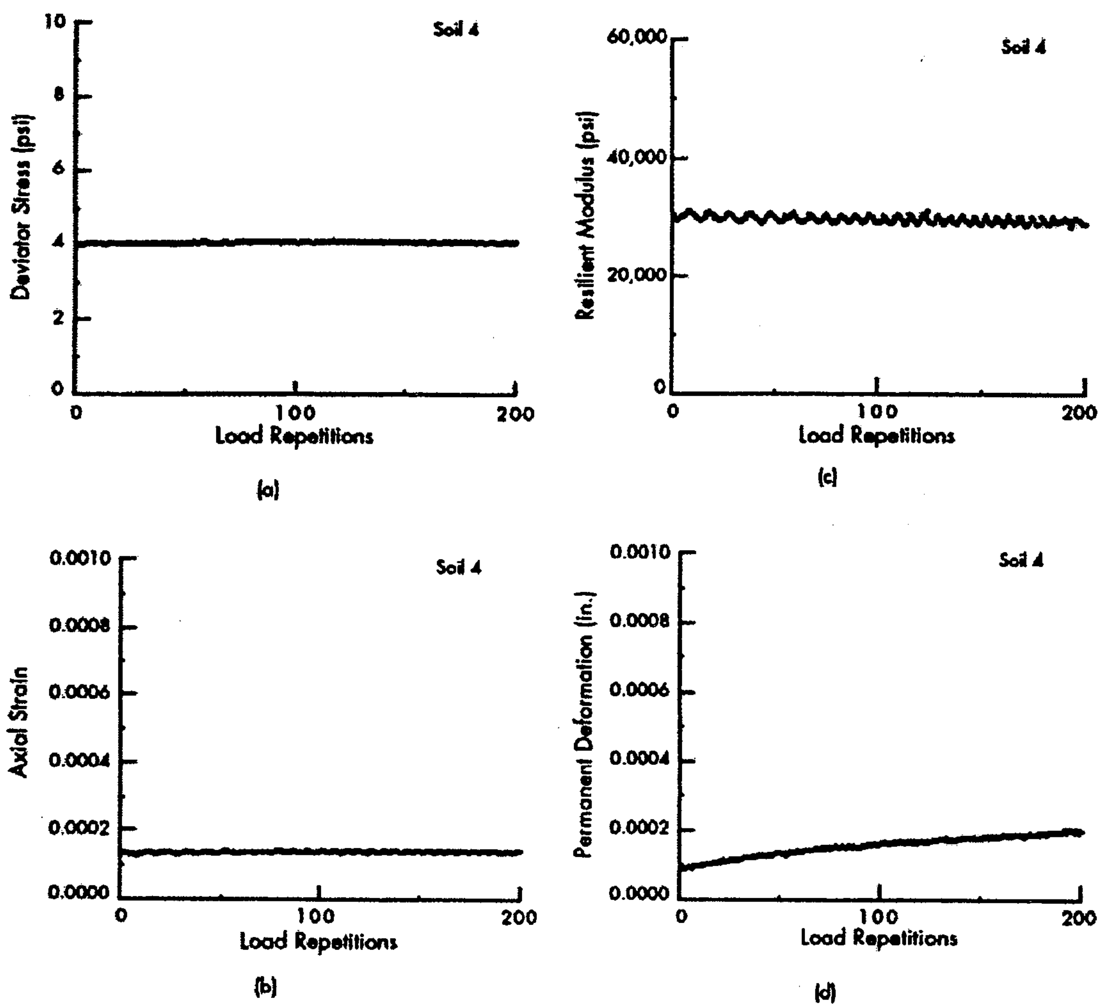


FIGURE 2.10 - (a) Deviator stress, (b) axial strain, (c) resilient modulus, and (d) permanent deformation as functions of number of loading repetitions of a grouted A-4 soil (Pezo et al., 1992).

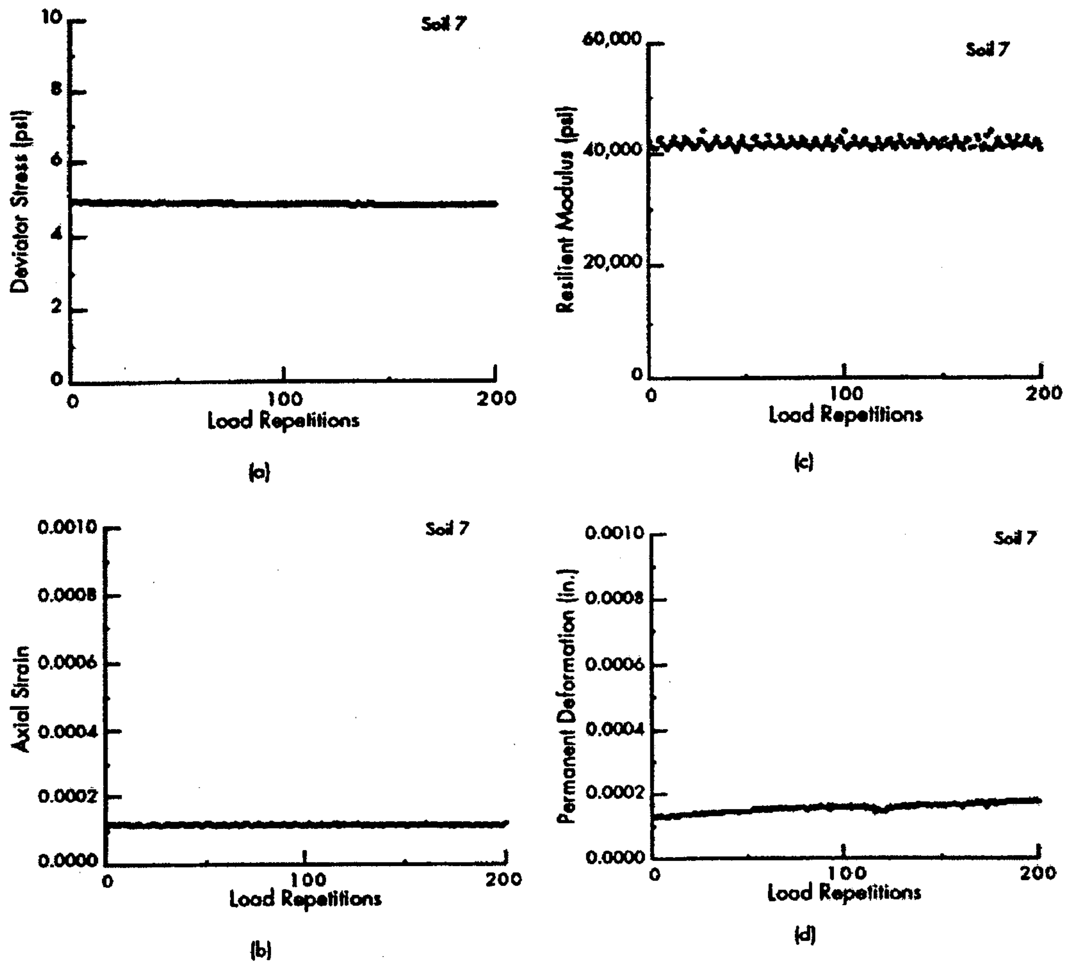


FIGURE 2.11 - (a) Deviator stress, (b) axial strain, (c) resilient modulus, (d) and permanent deformation as functions of number of loading repetitions of a grouted A-6 soil (Pezo et al., 1992).

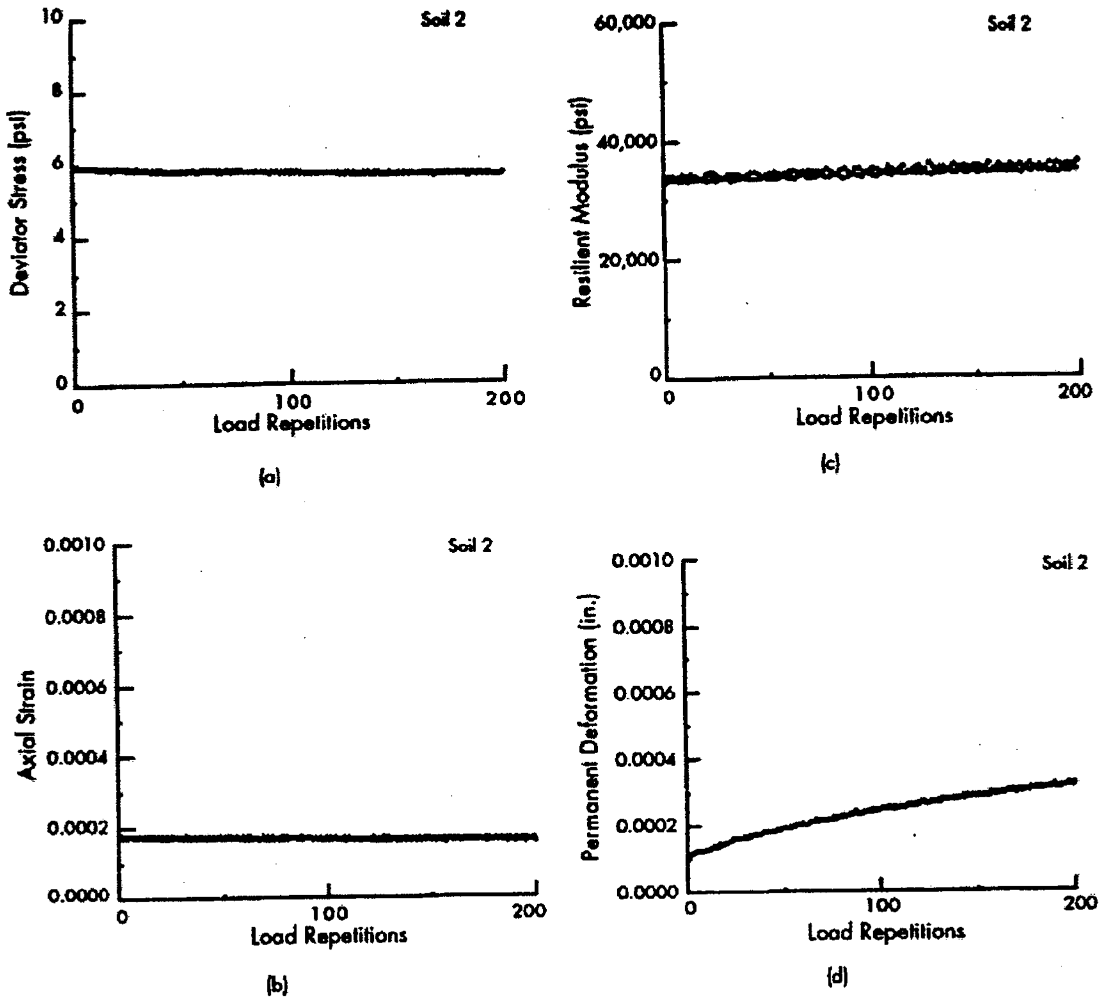
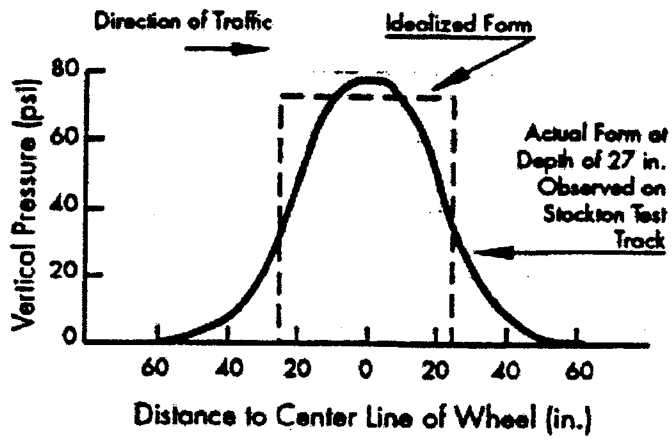


FIGURE 2.12 - (a) Deviator stress, (b) axial strain, (c) resilient modulus, and (d) permanent deformation as functions of number of loading repetitions of a grouted A-7-6 soil (Pezo et al., 1992).

2.4.2 Deviator Stress

In laboratory resilient modulus testing, deviator stress is cyclically applied at each sequence by the piston and end plate of the triaxial apparatus. Some studies have examined the effect of the shape of the loading pulse. Changes in resilient modulus have been observed as a result of different levels of deviator stress and number of applied repetitions.

To simulate in-situ conditions of a load moving over a given soil element a haversine shaped loading pulse is prescribed by AASHTO T294-92. Figure 2.13 compares the downward force exerted on a soil element by a moving wheel load as a function of time to a haversine loading pulse. Figure 2.14 shows the resilient modulus load pulse configuration. The same pulse configuration is used for the preconditioning sequence and all other sequences and repetitions.



NOTE: The dashed line is the work equivalent idealized form of the haversine shape.

FIGURE 2.13 – Changes in stress caused by a moving load (Seed and McNeill, 1958 as reproduced in Laguros et al., 1993).

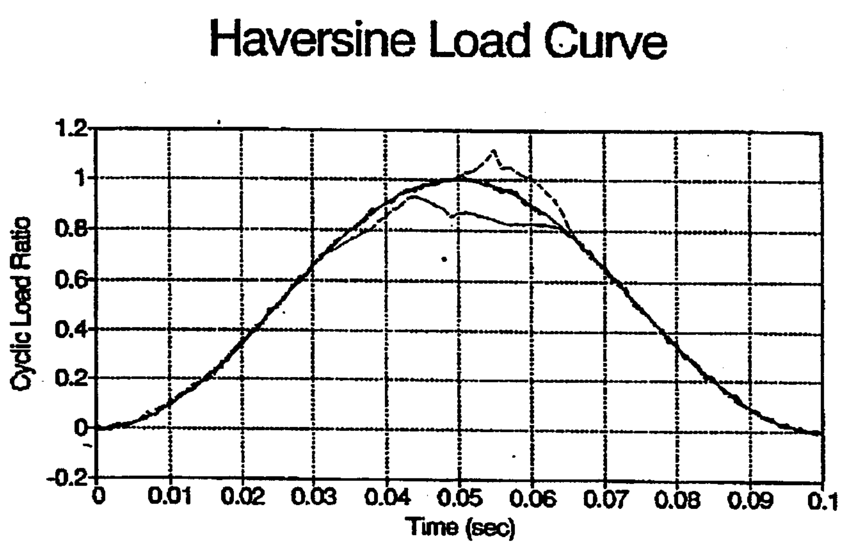


FIGURE 2.14 – Haversine loading pulse configuration versus time (Baus and Ray, 1992).

Although the square shaped pulse and the haversine pulse do not apply the load in the same manner, Laguros et al. (1993) assumed a square shaped vertical pulse of equal work could be used as a reasonable approximation of actual conditions. Barksdale et al. (1997) recommended using the haversine load pulse, and stated that if a pneumatic system is used, modifications to the air supply and exhaust port should be made to allow for greater control of the pulse shape.

The magnitude of deviator stress affects the magnitude of resilient modulus. The range of deviator stress for type 1 soils is 21 kPa (3 psi) to 276 kPa (40 psi) whereas the range for type 2 soils is 14 kPa (2 psi) to 69 kPa (10 psi). Ishibashi et al. (1984) tested multiple specimens of two A-6 subgrade soils and two A-1-b base soils at various dry densities and water contents. Compaction was achieved using an automatic kneading device. The deviator stress was applied using a haversine pulse shape such that the maximum level was attained after 0.2 seconds. Therefore the pulse duration was 0.4 seconds. A 1.0-second resting period was allowed between pulses. The duration of each repetition was 1.4 seconds. For each of the seven loading sequences, 210 repetitions were completed with data taken at selected repetitions throughout the sequence. The deviator stress and confining pressure levels were changed after each sequence was completed. For the A-6 soils, the resilient modulus decreased very rapidly as the deviator stress increased. For the A-1-b soils, a less rapid decrease followed by an eventual increase in resilient modulus was observed. Figures 2.15 and 2.16 show some of these test results. Figure 2.17 shows similar evidence as compiled by Seed et al. (1962) for kneading compacted clays.

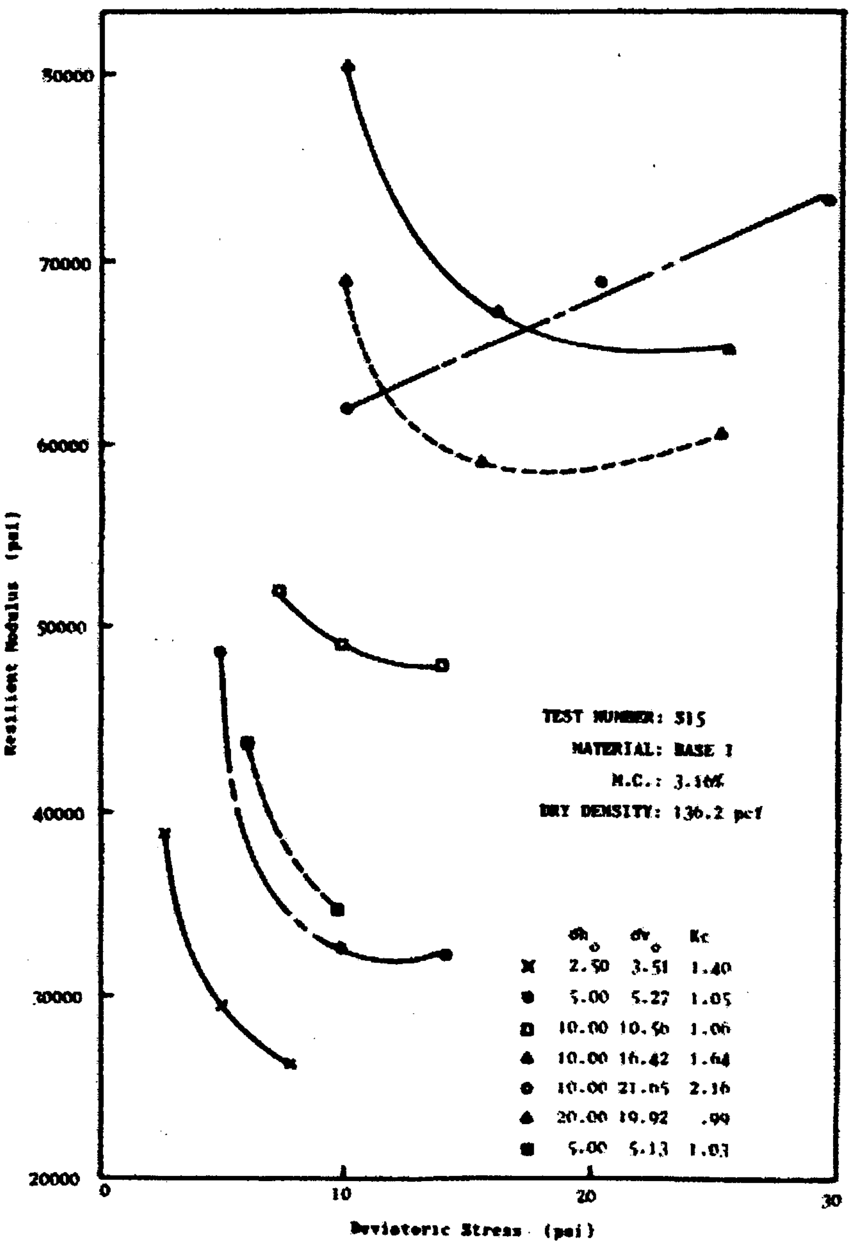


FIGURE 2.15 - Change in resilient modulus with respect to increasing deviator stress of an A-1-b soil (Ishibashi et al., 1984).

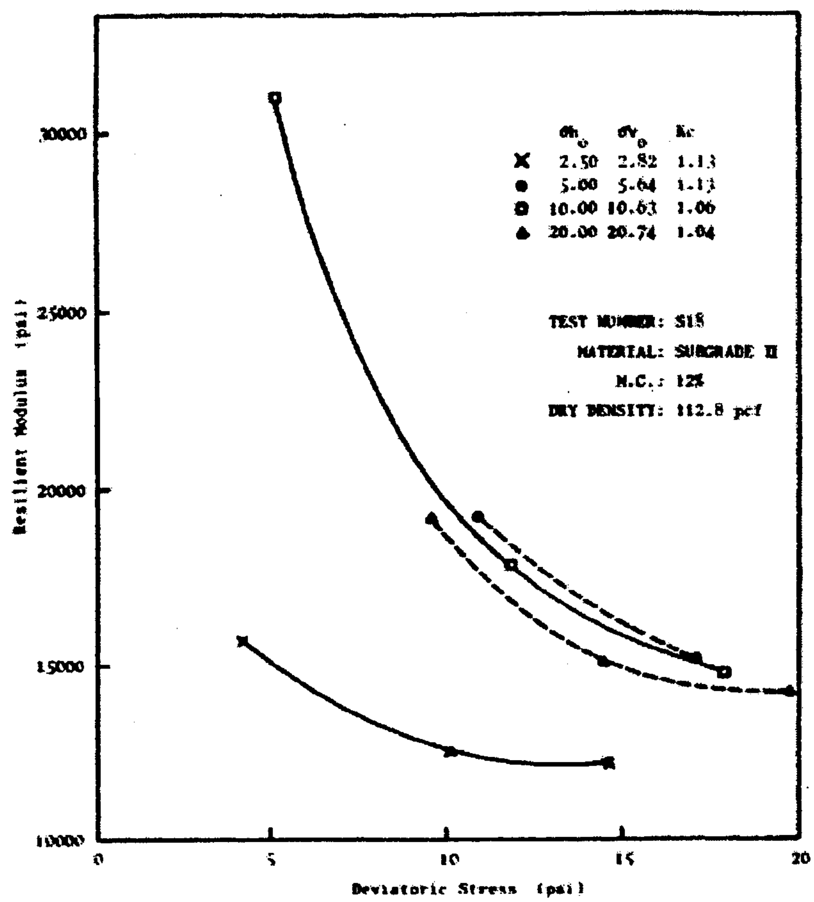


FIGURE 2.16 - Change in resilient modulus with respect to increasing deviator stress of an A-6 soil (Ishibashi et al., 1984).

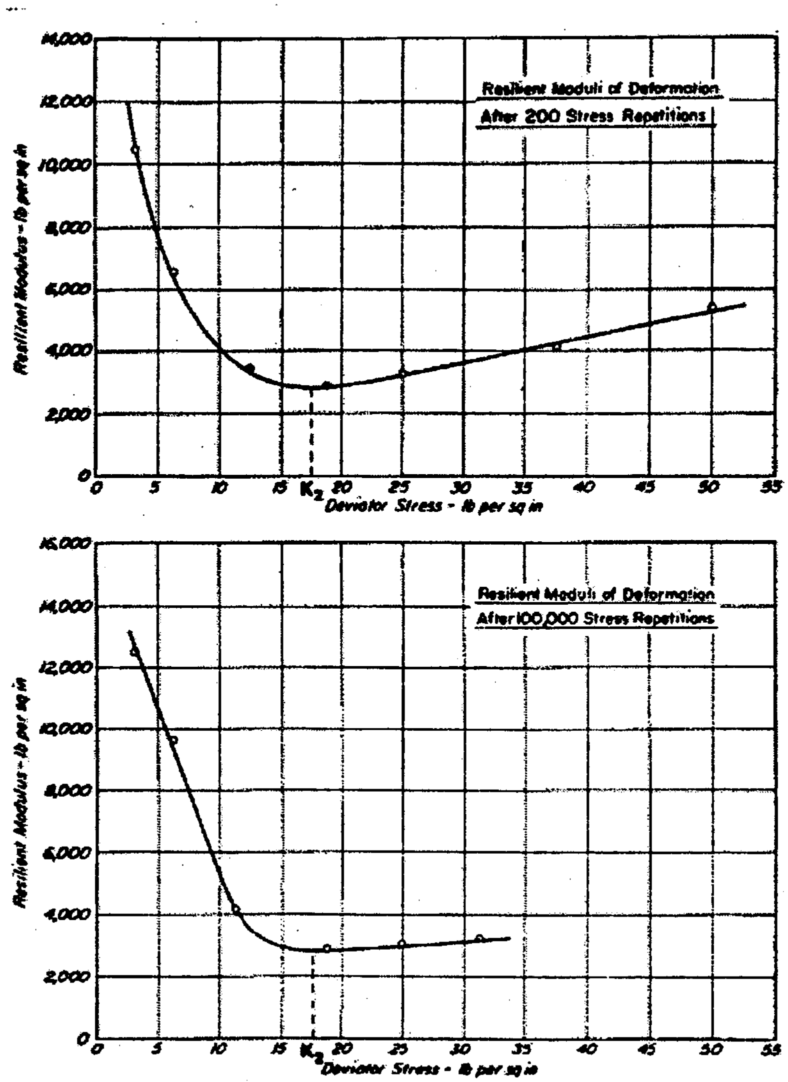


FIGURE 2.17 - The effect of deviator stress on resilient modulus after 200 and 100,000 repetitions (Seed et al., 1962).

A "breakpoint" in the resilient modulus behavior occurred between principle stresses of 103 kPa (15 psi) and 138 kPa (20 psi) after which the change in resilient modulus was considerably less than the change observed at lower deviator stresses.

2.4.3 Confining Pressure

Confining pressure is the pressure exerted on the specimen by the fluid inside the triaxial chamber. The magnitude of the confining pressure required by AASHTO T294-92 depends on soil type and loading sequence.

Drumm et al. (1993) observed an increase in resilient modulus as confining pressure levels were increased from 14 kPa (2 psi) to 41 kPa (6 psi) for two A-4 soils with a water content ½ percent above optimum. In contrast, for the same A-4 soils with water contents below optimum, the resilient modulus was approximately the same for the range of confining pressures tested. These results are shown in Figure 2.18.

In the same study, Drumm et al. (1993) observed an increase in resilient modulus with increased confining pressure for other A-6, A-7-5, and A-7-6 soils. Further examination led Drumm et al. (1993) to conclude that this correlation was attributed to increased density and stiffness after the first stress sequence was completed. It was recommended that using the maximum deviator stress prescribed in the testing sequences for the preconditioning sequence stress would minimize this effect.

Ishibashi et al. (1984) observed an increase in resilient modulus with increasing confining pressure for the same A-1-b and A-6 soils discussed previously in Section 2.3.2 (see Figures 2.15 and 2.16). The increase in resilient modulus is more apparent for the type 1 A-1-b soil in Figure 2.15 than for the type 2 A-6 soil in Figure 2.16.

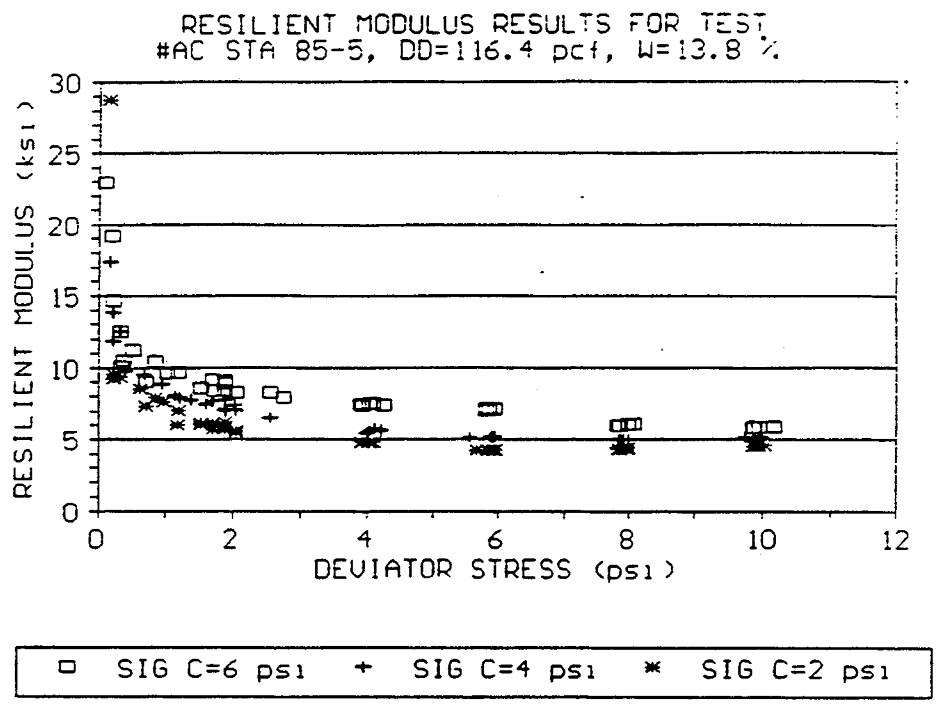


FIGURE 2.18 - Reduction in resilient modulus of an A-4 soil at ½ percent above optimum water content due to decreasing confining pressure (Drumm et al., 1993).

2.5 Strain Measurement

A final aspect of resilient modulus testing is strain measurement. Strain measurement is a crucial element to resilient modulus testing. AASHTO T294-92 specifies two linear variable differential transformers (LVDT) to be installed on the loading piston outside of the triaxial chamber. As the load is applied, the LVDTs measure the displacement of the piston with respect to the top of the triaxial cell. Since the piston rod, load cell, and piston head are included in the range of measurement, possible deformation in the apparatus will be included with the specimen strain itself. Several researchers have examined the LVDTs mounted externally, internally, and other possible methods for measuring specimen strain as discussed in the following paragraphs.

Mohammad et al. (1995) proposed that an internal LVDT system was better because it measured specimen strain without including deformation of apparatus components. Pezo et al. (1992) concluded that two internally mounted LVDTs proved to be the most effective method for monitoring accurate and reliable axial deformations.

Burczyk et al. (1994) examined the differences between internally and externally mounted LVDTs. Internal LVDTs were mounted on a ring clamped directly to the outside of the sample membrane. When compared to externally mounted LVDTs, the internal LVDTs consistently gave higher resilient modulus values. Although Burczyk et al. (1994) also commented that internal LVDTs are more complicated to set up and may cause sample disturbance due to the attachment configuration.

A comprehensive review of methods for internal strain measurement was made by Barksdale et al. (1997). Clamp and stud mounted LVDTs, non-contact proximity sensors, and optical measurement systems were examined as possible methods of strain measurement. Barksdale et al. (1997) recommends that an optical extensometer be used for most accurate results.

2.6 Summary

Due to the complexity of laboratory resilient modulus testing as prescribed by AASHTO T294-92, the procedure is prone to several sources of error. Repeatability of resilient moduli measurements has been shown to be excellent within individual laboratories but poor when compared between laboratories. Soils are classified into two types each with their own specimen preparation and loading procedures. In general type 1 soils are granular with limited fines and type 2 soils have higher amounts of silt and clay. A-1-a soils are always type 1 soils. A-4, A-5, A-6, and all A-7 soils are always type 2 soils. A-1-b, A-2, and A-3 can sometimes be type 1 or type 2 soils depending upon their fines contents and plasticities.

Reconstitution of type 2 samples for laboratory resilient modulus testing may not accurately represent in-situ conditions. The soil fabric of laboratory compacted clays has been shown to be different than in-situ samples of the same soil. Increased relative density is accompanied by increased resilient modulus. A decrease in resilient modulus is observed with increasing water content. Thus, it is important that the density and water content of the lab sample be the same as the field.

Preconditioning the specimen for 1000 repetitions of the maximum stress level prescribed in the loading sequence has been shown to reduce thixotropic effects on the value of resilient modulus. Intimate contact between the specimen and resilient modulus test apparatus is assured if the specimen ends are grouted to the end platens. The resilient modulus of specimens grouted to the end platens is not significantly affected after 2000 maximum stress level applications are applied.

Therefore grouting may reduce resilient modulus test duration.

The effect on resilient modulus of changing stress magnitudes, number of loading repetitions, and loading pulse shape has been examined. For type 1 soils, increasing the deviator stress causes an initial reduction in resilient modulus followed by a gradual increase in resilient modulus with further increase in deviator stress. For type 2 soils, a drastic decrease in resilient modulus is observed as the deviator stress increases. Increased confining pressure causes an increase in resilient modulus for both soils but is more pronounced in granular soils than fine grained soils. A haversine 0.1-second loading followed by a 0.9-second resting period is recommended by AASHTO T294-92 for the shape of the deviator stress loading pulse.

Accurate strain measurement is crucial to resilient modulus testing. Internal LVDTs have been shown to give higher resilient modulus values than externally mounted LVDTs. For optimum results, optical extensometers are recommended. Barksdale et al. (1997) states amongst his significant general findings that "for production resilient modulus testing, a completely automated, modern electro-hydraulic loading and data acquisition system is a necessity to maximize the number of tests performed and to minimize the potential for testing and data reduction errors".

CHAPTER 3

ALTERNATIVE METHODS FOR RESILIENT MODULUS DETERMINATION

The current procedure for resilient modulus determination is complex, time consuming, and requires specialized expensive equipment. Research has been conducted to find alternative methods for resilient modulus determination. Torsional resonant column testing, torsional shear testing, and gyratory testing have been examined. Another method that has been proposed employs an instrumented hammer and oscilloscope to measure the resilient modulus as well as principle stress difference. Computer software has been developed to backcalculate the resilient modulus of in situ soils using deflection data obtained from falling weight deflectometer tests. This chapter will introduce these alternative methods and report their correlations with standard laboratory resilient modulus tests.

3.1 Torsional Resonant Column and Shear Devices

Torsional resonant column (RC) and torsional shear devices (TS) have been used to determine the resilient modulus of soil specimens. Because of equipment limitations most laboratory resilient modulus test equipment can only accurately measure to about 0.01 percent strain. Kim and Stokoe (1992) used the RC and TS devices to measure between 0.0001 and 0.01 percent strain. This allows the complete stress-strain behavior to be observed. In RC tests, the specimen is vibrated into first mode torsional motion. After first mode vibration is achieved, measurements of the amplitude and frequency are made.

Combining these measurements with equipment properties and specimen dimensions allows the computation of shear wave velocity, shear modulus, and shear strain amplitude. Figure 3.1 shows the RC device and a typical RC frequency response curve.

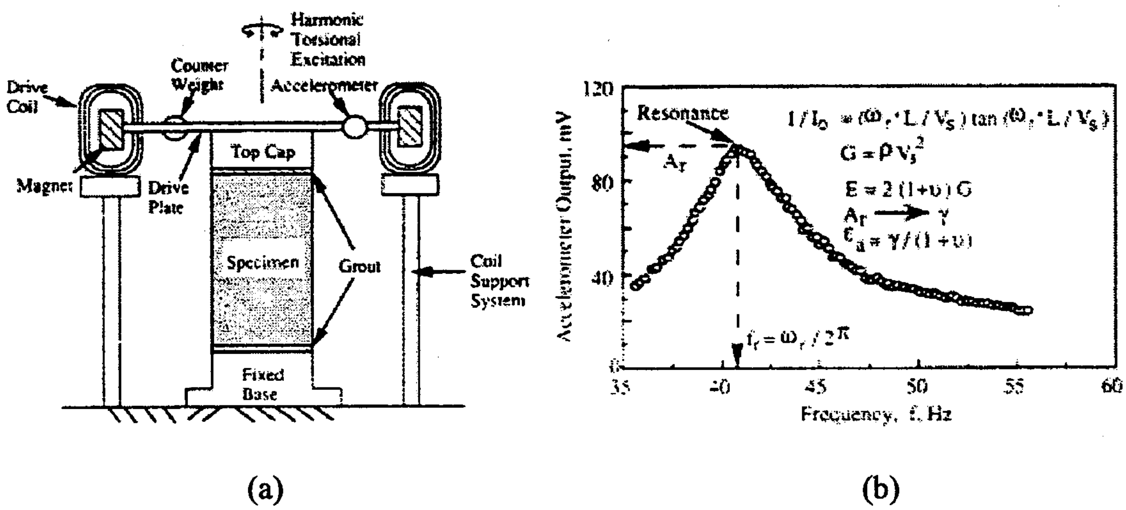
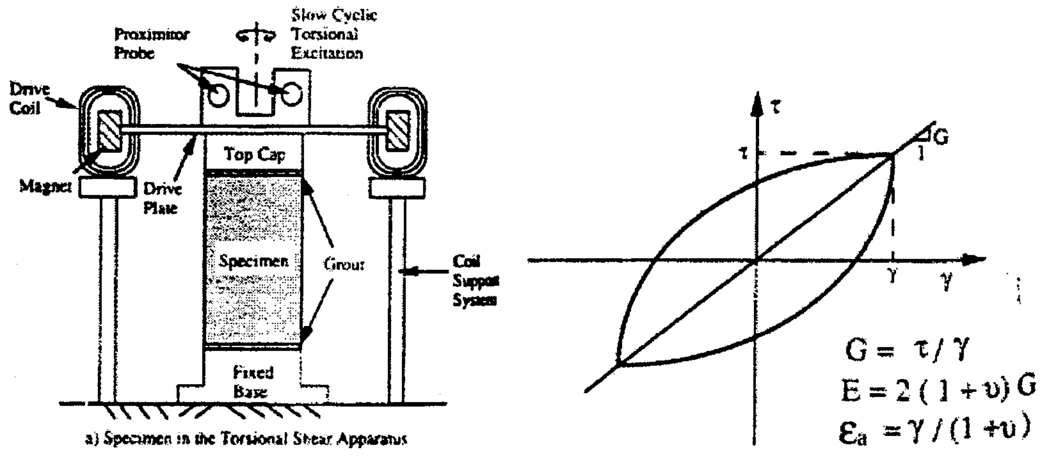


FIGURE 3.1 - (a) Resonant column test apparatus and (b) frequency response curve (Kim and Stokoe, 1992).

TS testing is accomplished with the same equipment but the applied torsional force is cyclic. Resonant frequency is not determined in TS tests, rather, the stress-strain relationship is measured using the torque-twist response of the specimen. Figure 3.2 shows the TS device and typical hysteresis response curve.



(a) (b)

FIGURE 3.2 - (a) Torsional shear testing apparatus and (b) hysteresis response curve (Kim and Stokoe, 1992).

The shear modulus (*G*) from RC or TS tests is determined from the stress-strain relationship as shown in the the hysteresis response curve in Figure 3.2b. The shear modulus is proportional to Young's modulus (*E*). The shear modulus is used to determine resilient modulus by assuming Young's modulus equal to resilient modulus. The equations used for resilient modulus and strain determination (Eqs. 3-1 and 3-2) also assume that the specimen is homogeneous and isotropic. Values for Poisson's ratio (ν) are also required (Kim and Stokoe, 1992).

$$E = 2G(1+\nu) \qquad \text{(Eq. 3-1)}$$

$$\varepsilon_a = \frac{\gamma}{(1+\nu)} \qquad \text{(Eq. 3-2)}$$

where: $\varepsilon_a =$ axial strain

$\gamma =$ shearing strain

Kim and Stokoe (1992) tested multiple specimens of ten type 2 soils using RC, TS, and standard laboratory resilient modulus testing. The specimens were recompacted and grouted to the end plates to reduce slippage. The moduli obtained from RC and TS testing were converted into equivalent resilient moduli using Equations 3-1 and 3-2. The results from the three testing methods compared very well as shown in Figure 3.3

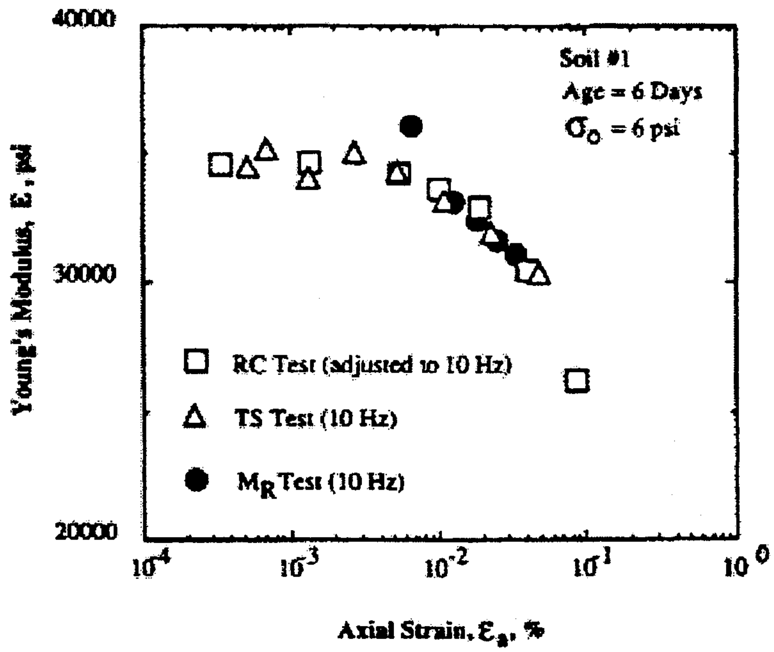


FIGURE 3.3 - Comparison of resilient modulus values for RC, TS, and resilient modulus testing (Kim and Stokoe, 1992).

3.2 Gyratory Testing Machine

George and Uddin (1993) explored the use of the U.S. Army Corps of Engineers gyratory testing machine (GTM). The GTM is used for the quality control of bituminous mixtures as well as base, subbase, and subgrade soils. The GTM may be a good alternative to resilient modulus testing because it is a combination kneading, dynamic consolidation, and shear testing machine. Another advantage of the GTM is that it is able to simulate shear stress reversals inherent to moving loads. Figure 3.4 shows a diagram of the GTM apparatus.

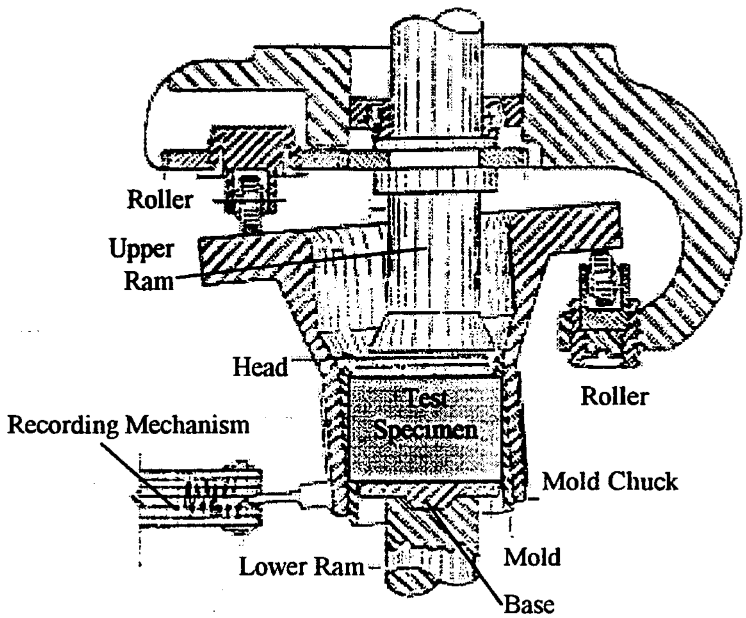


FIGURE 3.4 - Schematic illustration of a GTM (George and Uddin, 1993).

George and Uddin (1993) compared the results of the resilient modulus obtained through GTM to several values obtained using the original AASHTO resilient modulus specification T274-82. Five type 1 and four type 2 soils were used in the study. Three specimens from each soil were tested. Results showed that the resilient moduli from the GTM were 8 to 47 percent lower than those obtained using AASHTO T274-82. George and Uddin (1993) also studied the effect of gyration angle on resilient modulus. Increasing the gyration angle from 0.0 to 0.1 degrees reduced the average resilient modulus by 45 percent for type 1 soils and 37 percent reduction for type 2 soils. With these results George and Uddin (1993) suggest that with some apparatus modifications and further study, the GTM may be an alternative method for resilient modulus prediction.

3.3 Falling Weight Impact Alternative Laboratory Test Method

Drumm et al. (1996) developed a simplified alternative laboratory test method that closely resembles a common nondestructive field testing method known as the falling weight deflectometer (FWD). The falling weight impact alternative testing method (ATM) apparatus consists of a standard Proctor mold, an instrumented drop hammer, and an oscilloscope. To prevent the specimen from extruding through the rigid base-Proctor mold interface a thin metal disk is inserted into the bottom of the mold prior to compacting. The mold is lubricated with a thin coating of vegetable oil. The specimen is then compacted as prescribed by AASHTO T-99. The system is modeled as a falling block mass impacting an equivalent mass of soil supported by a linearly compressible spring as shown in Figure 3.5. The acceleration of the drop hammer is measured and plotted as a function of time as shown in Figure 3.6. The resilient modulus ($M_R(ATM)$) and deviator stress are determined using Equations 3-3, 3-4, and 3-5. A full discussion of the derivation of these equations with consideration to stress paths is given by Drumm et al. (1996).

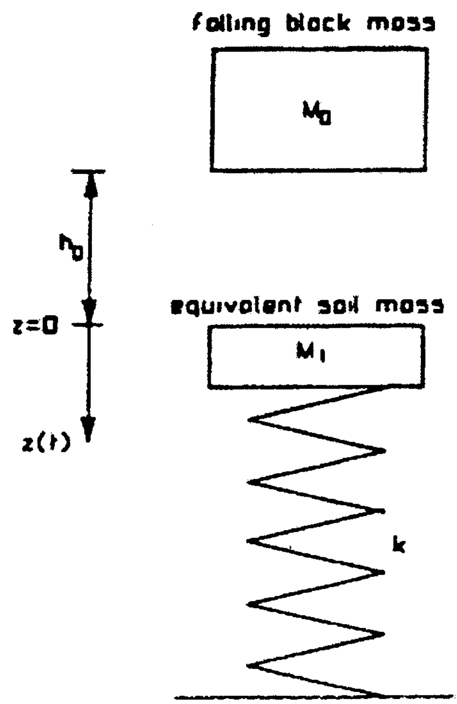


FIGURE 3.5 - Idealized single degree of freedom mass / spring model for ATM (Drumm et al., 1996).

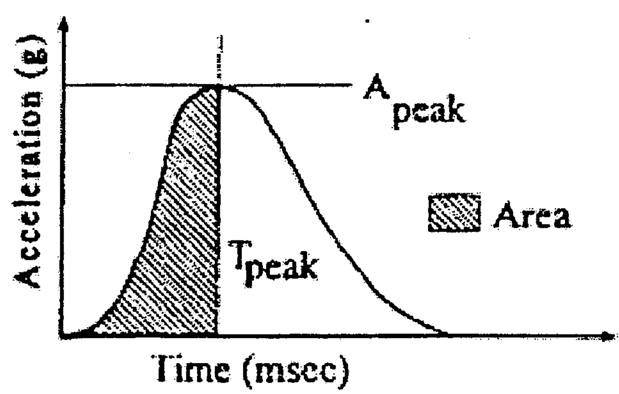


FIGURE 3.6 - Acceleration as a function of time for ATM (Drumm et al., 1996).

Peak Acceleration Method

$$M_R(ATM) = \frac{(1+\nu)(1-2\nu)}{(1-\nu)} \cdot \frac{(M_0+M_1)^2 L}{M_0 2h_0 g A_{sec}} \cdot A_p^2 \qquad \text{(Eq. 3-3)}$$

Impulse to the Peak Acceleration

$$M_R(ATM) = \frac{(1+\nu)(1-2\nu)}{(1-\nu)} \cdot \frac{(M_0+M_1)^2 L}{Area^2 M_0 g A_{sec}} \cdot A_p^2 \qquad \text{(Eq. 3-4)}$$

Deviator Stress

$$\sigma_d(ATM) = \frac{(1-2\nu)}{1-\nu} \cdot \frac{A_p M_0}{A_{sec}} \qquad \text{(Eq. 3-5)}$$

where: M_0 = mass of the hammer

M_1 = mass of the soil specimen

L = length of the soil specimen

h_0 = drop height

g = measured acceleration

A_{sec} = cross sectional area of the specimen

A_p = measured peak acceleration

$Area$ = area under the acceleration curve

Using the ATM requires assumed or known values of Poisson's ratio. Multiple deviator stresses are obtained by varying the mass and height of the drop hammer. Acceptable acceleration curves are haversine shaped and can be monitored by the oscilloscope. A sketch of the ATM apparatus is shown in Figure 3.7.

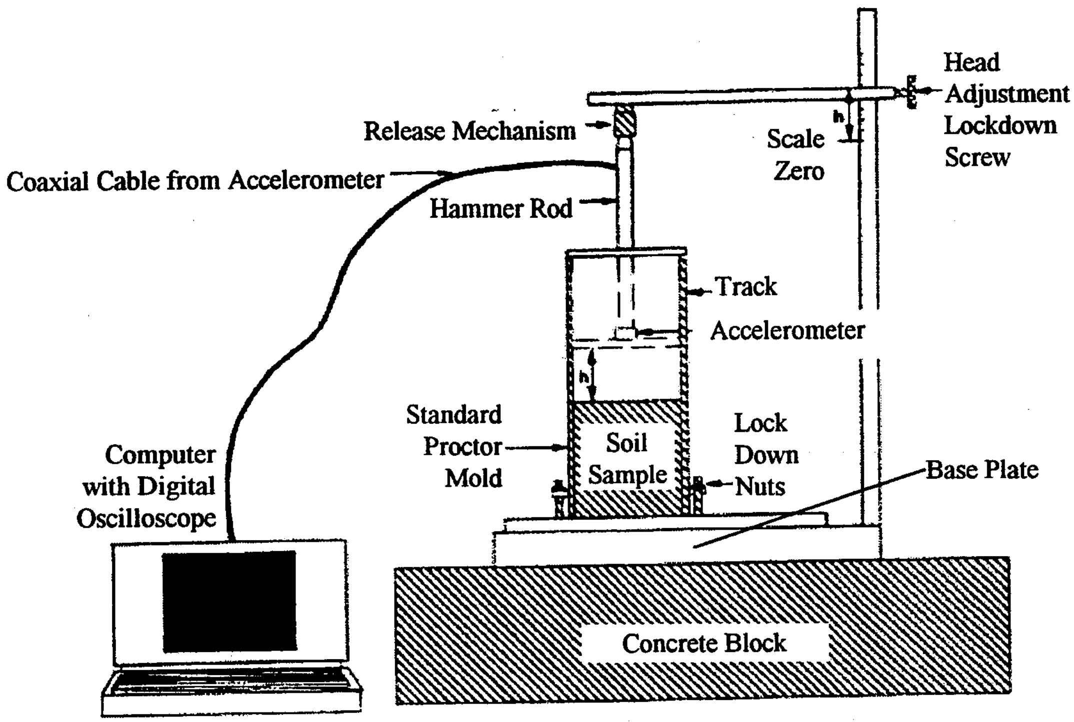


FIGURE 3.7 - ATM apparatus (Drumm et al., 1996).

Drumm et al. (1996) used the ATM to determine the resilient moduli of fourteen type 2 soils from Tennessee. The resilient moduli obtained from the ATM compared favorably to the resilient moduli found using SHRP Protocol P46. Furthermore, the ATM was able to distinguish between soils with high and low resilient moduli. Figure 3.8 shows these results.

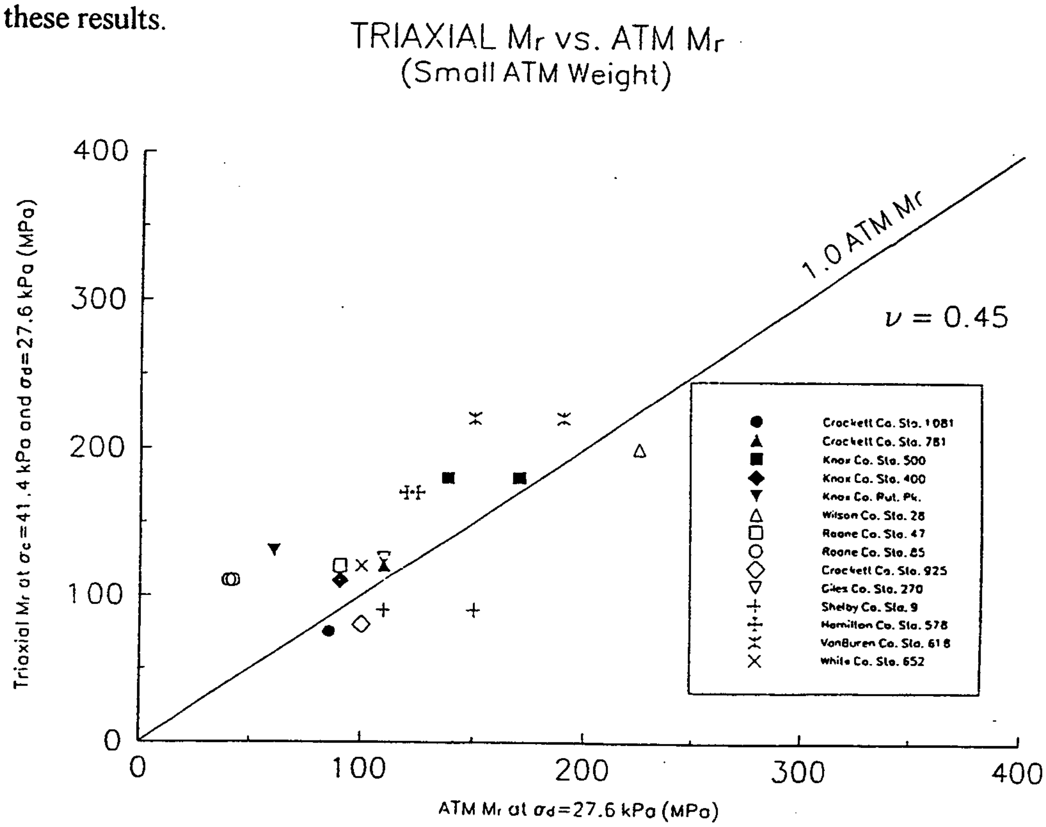


FIGURE 3.8 - Comparison of SHRP Protocol P46 and ATM resilient moduli values (Drumm et al., 1996).

3.4 Backcalculation of Resilient Modulus

Nondestructive testing methods (NDT) have been useful tools to evaluate existing pavement conditions. Falling weight deflectometer (FWD) testing is a common NDT method. The FWD has been explored as a potential method for determining the in-situ resilient modulus of roadway pavements, base, subbase, and subgrade soils using back calculation computer programs. The most common of these programs is MODULUS (Baus and Ray, 1992). Others mentioned in the literature review include MODCOMP 4 and WESDEF. The resilient moduli computed from the deflection basins measured by the FWD are referred to as "backcalculated" moduli. The deflection basin profile is iteratively compared to a database of known profiles and properties, and a backcalculated resilient modulus is generated after the comparison error is minimized. The concept relies heavily on the elastic layer response of soils. It also assumes a decrease in stress with increasing distance from the loading point. Several authors have used versions of MODULUS to compare the backcalculated resilient moduli with different methods of resilient modulus determination. Some authors have found errors inherent to the backcalculation process. These errors as well as correlations between backcalculated resilient moduli and laboratory resilient moduli will be discussed in this section.

3.4.1 Problems Associated with Resilient Modulus Backcalculation

Using FWD data from eighteen general pavement study sections stored in the LTPP database, Killingsworth and Von Quintus (1997) evaluated six backcalculation software packages. These packages were MODULUS 4.2 (Michalak and Scullion, 1993), MODCOMP 3 Version 3.6 (Irwin, 1994), WESDEF, WESNET, MICHBACK 1.0 (Harichandran, 1995), and FWD-DYN (Foinquinos, 1993). MODULUS 4.2, MODCOMP 3.6, MICHBACK 1.0, and WESDEF produced similar error terms at the 40-kN (9,000-lb) load level.

An initial backcalculation procedure was used to evaluate the programs. Results from this procedure were compared to results from MODULUS 4.0 (Rhode and Scullion, 1990). Based on this comparison five programs were eliminated from further examination. The results from MODULUS 4.2 were not significantly different from those of MODULUS 4.0 therefore MODULUS 4.2 was not selected for continued evaluation. Although FWD-DYN provided consistently low error terms it estimated negative soil layer thicknesses. MICHBACK 1.0 and MODCOMP 3.6 computed higher than actual subgrade moduli. WESNET is restricted to conventional three layer systems of conventional asphalt over granular base over subgrade therefore it was also neglected from the examination.

The remaining programs MODULUS 4.0 and WESDEF were chosen to continue the backcalculation procedure. Although the backcalculation procedure is very subjective the results from these two programs were very similar.

In a related publication Von Quintus and Killingsworth (1997b) reported several possible errors inherent in the backcalculation process. These errors are:

1. Some software has a limit to the allowable number of soil layers, therefore multiple soil layers may need to be combined to fit the software analysis.
2. Sensor measurement inaccuracy or inherent noise.
3. Discontinuities in the pavement, such as cracks between the applied load and sensor.
4. Improper assumption of the location of an apparent stiff soil or rock layer.
5. Discrepancies between assumed and actual layer thicknesses.
6. Non-uniform loading distribution under the loading device.
7. Non-linear, heterogeneous, or anisotropic materials in the pavement or subgrade.

To reduce the effect of these errors the Von Quintus and Killingsworth (1997b) developed a comprehensive procedure around the MODULUS and WESDEF programs. The procedure addresses the definition of resilient moduli ranges for different layers, pavement structure modeling, and analysis of results. Von Quintus and Killingsworth (1997b) also advised that there is no unique solution for a specific deflection basin using elastic layer theory. The layer moduli determined from the backcalculation process represent equivalent moduli and should be reviewed for reasonableness. The procedure is as follows:

1. "Normalize and review the measured deflection basins to ensure that the deflections decrease consistently with those sensors farther from the applied load. Identify unique deflection basins that are inconsistent with elastic layer theory."
2. "Review the materials and soils recovered from the pavement cores and borings. Separate significantly different pavement materials and subgrade soils or subsurface conditions into different layers and identify the depth to a stiff or rigid layer."
3. "Identify potential problem layers included in the structure. For example, weak soils above stiffer soils, sandwich sections, and thin and thick layers relative to adjacent layers."
4. "Determine the pavement cross section to be used in the backcalculation process."
5. "Backcalculate the modulus of each layer and calculate the error term for each measured basin or the sum of the total percentage difference between the measured and calculated basins."
6. "For large errors, review the pavement structure used in the backcalculation process with cores and borings. Recombine or separate layers, if necessary, to decrease the error term."

7. "Review the moduli ratios between unbound layers to identify unrealistic or improbable conditions."
8. "For those basins that consistently hit the upper limit set for the modulus of a particular material, the structure should be reviewed in an attempt to reduce the error term while maintaining reasonable modulus values. For basins that hit the lower limit for a particular material, the lower limit can be further reduced. Low modulus values may be reasonable because of contamination of underlying materials, the presence of cracks or internal damage, or the weakening of some unbound materials with an increase in moisture or a decrease in density."

Von Quintus and Killingsworth (1998) made further recommendations to aid in error reduction. First, software not compatible with the elastic layer theory should not be used. Second, it was found that delineating the soil profile into four layers considerably reduced error when compared to three, five, and six layer profiles. Third, temperature affects the flexibility of asphalt concrete pavements therefore a temperature at mid-depth of the pavement should be used. And finally, representative stress states used in backcalculating resilient modulus should be determined at a depth of 45-cm (18-in.) into the subgrade and at depths of one quarter the thicknesses of the base and subbase layers. For a given layer, the following equation is used to relate laboratory resilient modulus ($M_R(LAB)$) with resilient moduli values backcalculated from deflection basins measured by FWD ($M_R(FWD)$):

$$M_R(LAB) = C \times M_R(FWD) \qquad \text{(Eq. 3-6)}$$

where: C = adjustment coefficient dependent on material type

Tables 3.1, 3.2, and 3.3 list the mean value of the adjustment coefficient for dense graded asphalt concrete mixtures, unbound granular base and subbase materials, and materials for embankments.

TABLE 3.1 - Adjustment coefficients for mid-depth temperature of dense graded asphalt concrete mixtures (Von Quintus and Killingsworth, 1997b).

Mid-Depth Temperature, °C	Mean C-Value
5	1.0
25	0.36
40	0.25

TABLE 3.2 - Adjustment coefficients for unbound granular base and subbase materials (Von Quintus and Killingsworth, 1997b).

Layer Type and Location	Mean C-Value	Coefficient of Variation, %
Granular base or subbase under a Portland cement concrete surface	1.32	74
Granular base or subbase under an asphalt concrete surface or base mixture	0.62	44
Granular base or subbase between a stabilized material and asphalt concrete surface or base mixture	1.43	80

TABLE 3.3 - Adjustment coefficients for embankment materials (Von Quintus and Killingsworth, 1997b).

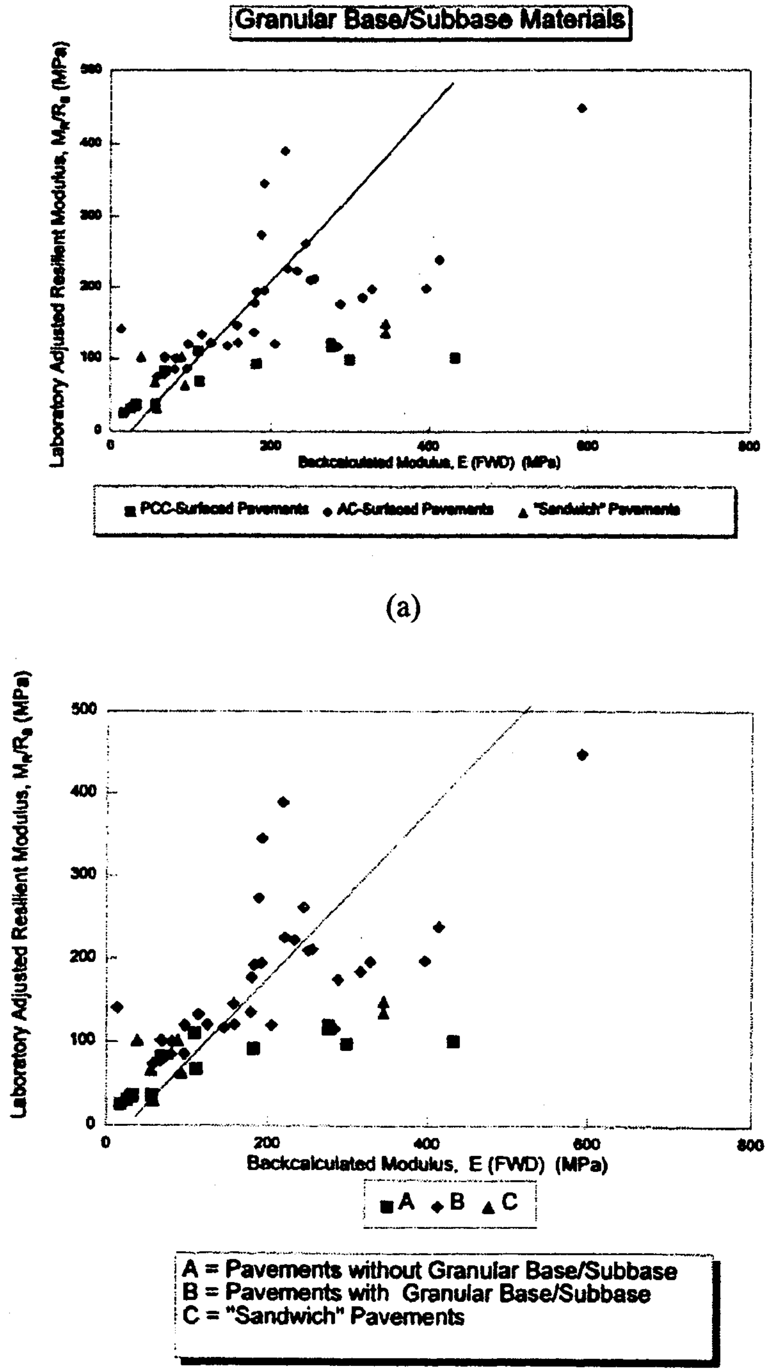
Pavement and Material Type	Mean C-Value	Coefficient of Variation, %
Embankment materials below a stabilized subbase	1.32	80
Embankment materials below a pavement without an unbound granular base and or subbase layer and no stabilized subgrade	0.52	37
Embankment materials below a pavement with an unbound granular base and or subbase layer but not stabilized subgrade	0.35	49

3.4.2 Correlations Between Backcalculated Resilient Moduli and Other Methods of Resilient Modulus Determination

Correlations between backcalculated resilient modulus and resilient modulus computed from laboratory tests, AASHTO recommendations, and in-situ instrumentation have been examined. The LTPP database has been the most widely used source of data.

Von Quintus and Killingsworth (1998) used the LTPP database to compare the resilient moduli determined by backcalculation with those from laboratory testing. The backcalculated resilient moduli were generally higher than the laboratory resilient moduli at equivalent stress states especially for higher moduli. These results are shown in Figure 3.9.

Daleiden et al. (1994) used an extensive set of LTPP data to find correlations between backcalculated and laboratory resilient modulus. The estimated resilient modulus was determined by assuming that the modulus of elasticity of the subgrade ($E_{subgrade}$) was equal to the resilient modulus. For this study Equation 3-7 (AASHTO, 1993a) was used to estimate resilient modulus. Equation 3-7 is based on the deflection basin profile in response to a given load. A fundamental assumption incorporated into Equation 3-7 is that the resilient modulus increases as the stress within the soil layer is spread over a greater area (Daleiden et al., 1994). Therefore the results from Equation 3-7 are semi-empirical. Equation 3-7 yielded considerably higher results when compared to laboratory and FWD resilient moduli. Direct comparisons of the three prediction methods showed a wide range of ratios as shown in Table 3.4.



(b)

FIGURE 3.9 - Comparison of backcalcluated and laboratory for (a) resilient moduli base and subbase and (b) subgrade soil layers (Von Quintus and Killingsworth, 1998).

$$E_{subgrade} = \frac{0.2792P}{\delta r \times r} \quad \text{(Eq. 3-7)}$$

where: $P =$ plate load of the NDT device (lbs).

$\delta r =$ measured deflection at radial distance r from the center of the loading plate (mils).

$r =$ radial distance from the plate load center to the point of deflection measurement (in.).

TABLE 3.4 - Direct comparison of backcalculated, laboratory, and resilient moduli using Equation 3-7 (Daleiden et al., 1994).

Ratio	Mean	Standard Deviation	Maximum	Minimum
$M_R(LAB) / M_R(FWD)$	0.57	0.67	10.34	0.01
$E_{subgrade} / M_R(LAB)$	4.65	3.81	58.09	1.10
$E_{subgrade} / M_R(FWD)$	2.34	2.94	36.56	0.20

Daleiden et al. (1994) also performed statistical regression analyses and developed good relationships for prediction of resilient modulus of clays, silts, and sands from FWD and soil property data. Equations 3-8, 3-9, and 3-10 are the results from the regression analysis. Daleiden et al. (1994) also conducted a sensitivity analysis to gage the accuracy of these equations. The variables and the ranges of values used in the sensitivity analysis are shown in Table 3.6. Using these values, Equation 3-8 (for sands) appeared to produce reasonable values for the backcalculated resilient moduli. For silts Equation 3-9 produced negative moduli above specific gravities (G_s) of 2.70. However for specific gravities between 2.30 and 2.60, Equation 3-9 produced reasonable moduli values. For clays Equation 3-10 yielded reasonable resilient moduli values as long as the percent saturation (S) was above 30%.

TABLE 3.5 - Data used for regression and sensitivity analyses for Equations 3-8, 3-9, and 3-10 (Daleiden et al., 1994).

Property	Symbol	Range used in Sensitivity Analysis
Seventh sensor reading from FWD	*seventh*	0.25 - 2 mils
Load from FWD	P	8,000 - 10,000 lbs
Asphalt or concrete + treated base thickness	t	4 - 12 in.
Untreated granular base thickness (in.)	b	4 - 20 in.
Specific gravity	G	2.00 - 3.00
Percent Saturation (integer)	S	10 - 100 %
Dry density of the subgrade	$\gamma_{dsubgrade}$	85 - 115 pcf

For Clays:

$$M_R(clay) = 0.88\frac{P}{seventh} + 90.13\frac{b^2}{seventh} - 0.00488b^2P + 0.000147S^2P - 0.08b^2t^2 + 116{,}774\frac{b^2}{S} + 94{,}749\frac{t^2}{S^2} \quad \text{(Eq. 3-8)}$$

$R^2 = 0.8886$, Standard error of estimate = 6997 psi

For Silts:

$$M_R(silt) = 30.17\frac{b^2}{seventh^2} + 0.000384P^2 + \frac{611{,}120}{G} + 630.12\frac{b^2}{t^2} - 23.54b^2G + 2{,}439.62\frac{t}{G} - 258{,}797 \quad \text{(Eq. 3-9)}$$

$R^2 = 0.7809$, Standard error of estimate = 11,419 psi

For Sands:

$$M_R(sand) = \frac{-2{,}834{,}967}{\gamma_{dsubgrade}} + 0.000131P^2 seventh + 15.04b^2 seventh + 371.33\frac{\gamma_{dsubgrade}}{seventh} - 0.00000301P^2\gamma_{dsubgrade} - 2{,}751.43\frac{t^2}{\gamma_{dsubgrade}} + 22{,}372 \quad \text{(Eq. 3-10)}$$

$R^2 = 0.8371$, Standard error of estimate = 15033 psi

3.5 Summary

Investigations have been made to find alternative methods of determining resilient modulus without the use of traditional laboratory methods and equipment. Torsional shear and resonant column tests allow the measurement of resilient modulus at very small values of strain. These values are comparable to the strain seen by subbase soils. Values for these tests have correlated well with standard laboratory test methods. With significant additional study the gyratory testing machine, most often used for testing asphalt concrete, could be used to determine the resilient modulus of soils. Drumm et al. (1996) developed an alternative laboratory test apparatus and method for estimating resilient modulus. The apparatus measures the peak acceleration of a drop hammer as it impacts a soil specimen prepared in a standard Proctor mold. The concept models the system as a falling block mass impacting an equivalent mass of soil supported by a linearly compressible spring. With further development this method may offer a portable and inexpensive alternative method to standard laboratory resilient modulus determination.

Computer software packages for backcalculation of resilient modulus using data from nondestructive testing methods is available. The most common program is MODULUS however other programs exist and have been used with varying degrees of accuracy. Other programs include MODCOMP 4, WESDEF, WESNET, MICHBACK 1.0, and FWD-DYN.

There are several possible errors that influence the results of resilient modulus backcalculation. These errors can be caused by limitations in the number of soil layers by some backcalculation programs, sensor measurement inaccuracy and inherent noise, pavement discontinuities, improper estimation of the depth of an apparent stiff layer, non-uniform load distribution, and soils that are not homogeneous and do not exhibit isotropic behavior.

The approach to resilient modulus backcalculation is highly subjective, however, an approach has been recommended to minimize errors in the backcalculation procedure. This approach conducts a thorough examination of deflection data, layer types and thicknesses, and soil properties. The approach also reviews the backcalculation results to insure that they are consistent with reasonable assumptions or typical values of resilient modulus.

A large database of soil index properties, resilient modulus test results, and FWD data is available in the LTPP database. Problems inherent with the database include assumptions incorporated into the software used to compute resilient modulus. This data has been used to develop correlations with basic soil index properties and laboratory measured, backcalculated, and empirically estimated resilient moduli. When comparing $M_R(LAB)$ to $M_R(FWD)$, the $M_R(FWD)$ values need to be adjusted for pavement and soil layer temperature, layer type and location, and for roadway construction on top of embankments.

CHAPTER 4

CONSTITUTIVE RELATIONSHIPS FOR RESILIENT MODULUS

The nonlinear elastic behavior of resilient modulus with respect to changes in stress conditions can be represented by one of several constitutive relationships. These relationships have varying degrees of complexity. As discussed in Chapter 2, the resilient moduli of type 1 and type 2 soils are a function of stress conditions. The magnitude of the resilient modulus of type 1 soils increases with increased confining pressure. Alternately, some relationships for type 1 soils use bulk stress rather than confining pressure. In contrast, confining pressure has a negligible effect on the resilient modulus of type 2 soils. Type 2 soil relationships do not consider confining pressure and are largely based on the effect of deviator stress. Some universal relationships, applicable to both soil types, are used and incorporate the effects of confining pressure and deviator stress. All relationships have constant terms (K_n) that are specific to the soil tested. A common method of determining these coefficients is to perform a linear regression analysis on the results of laboratory resilient modulus tests. A linear regression analysis obtains the best fit between the laboratory resilient moduli and a linearized form of the appropriate constitutive relationship. The accuracy of the correlation is reported by the square of the correlation coefficient (R^2). If all laboratory resilient moduli fall exactly on the linearized relationship, the correlation is perfect and $R^2 = 1.0$. As the scatter of the testing results increases, R^2 will be proportionally less than 1.0. The results of linear regression analyses for several commonly used relationships are presented in this chapter.

4.1 Bulk Stress Relationship for Type 1 Soils

AASHTO T294-92 (AASHTO, 1993b) specifies a simple relationship for the resilient modulus of type 1 soils based on bulk stress (θ).

$$M_R = K_1(\theta)^{K_2} \tag{Eq. 4-1}$$

The constants in Equation 4-1 are unit specific and it is inconvenient to convert between SI and English units. To solve this problem, atmospheric pressure (P_a) is introduced into the equation to make the K_n constants unitless. The form of this equation is shown below:

$$M_R = K_3 P_a \left(\frac{\theta}{P_a}\right)^{K_4} \tag{Eq. 4-2}$$

AASHTO T294-92 (1993b) specifies plotting the results of resilient modulus tests on a Log M_R versus Log θ. K_1 is then the modulus at θ = 6.9 kPa (1psi) of the best fit line and K_2 is the slope of the line.

To characterize the resilient modulus behavior of type 1 soils from New England Lee et al. (1997) used the results of laboratory resilient modulus tests to determine K_1 and K_2 in Equation 4-1 for eight representative subbases. Six samples were of AASHTO classification A-1-a and two were A-1-b soil. Two of the A-1-a soils were supplied by

MDOT as representative subbases used for roadway construction in Maine. Each sample was prepared at optimum moisture content and dry density as prescribed by AASHTO T180-90. Laboratory resilient modulus tests were done at room temperature (20°C ; 68°F) using AASHTO T292-91, the predecessor to the current specification. Specimen deformation was measured using four internal LVDTs. Eighteen data points were collected for each specimen for the resilient modulus at different levels of bulk stress. Soil properties, K_n constants, and correlation coefficients for each soil sample are listed individually in Appendix A. Resilient modulus has units of ksi and bulk stress has units of psi. The range of correlation coefficients for all eight soils was 0.98 to 0.79 indicating that the model provided a good fit to the actual data. The correlation coefficient for the Maine type 1 A-1-a Frenchville soil was 0.80 whereas the correlation coefficient for the type 1 A-1-a Sabattus soil was 0.93. Figure 4.1 shows that K_2 tends to decrease as K_1 increases. Relationships such as this can be used to ensure that the K_1 and K_2 values used in the analysis are compatible. The correlation coefficient for the K_1-K_2 relationship is 0.828. Resilient modulus as a function of bulk stress for the Maine soils are shown in Figures 4.2 and 4.3.

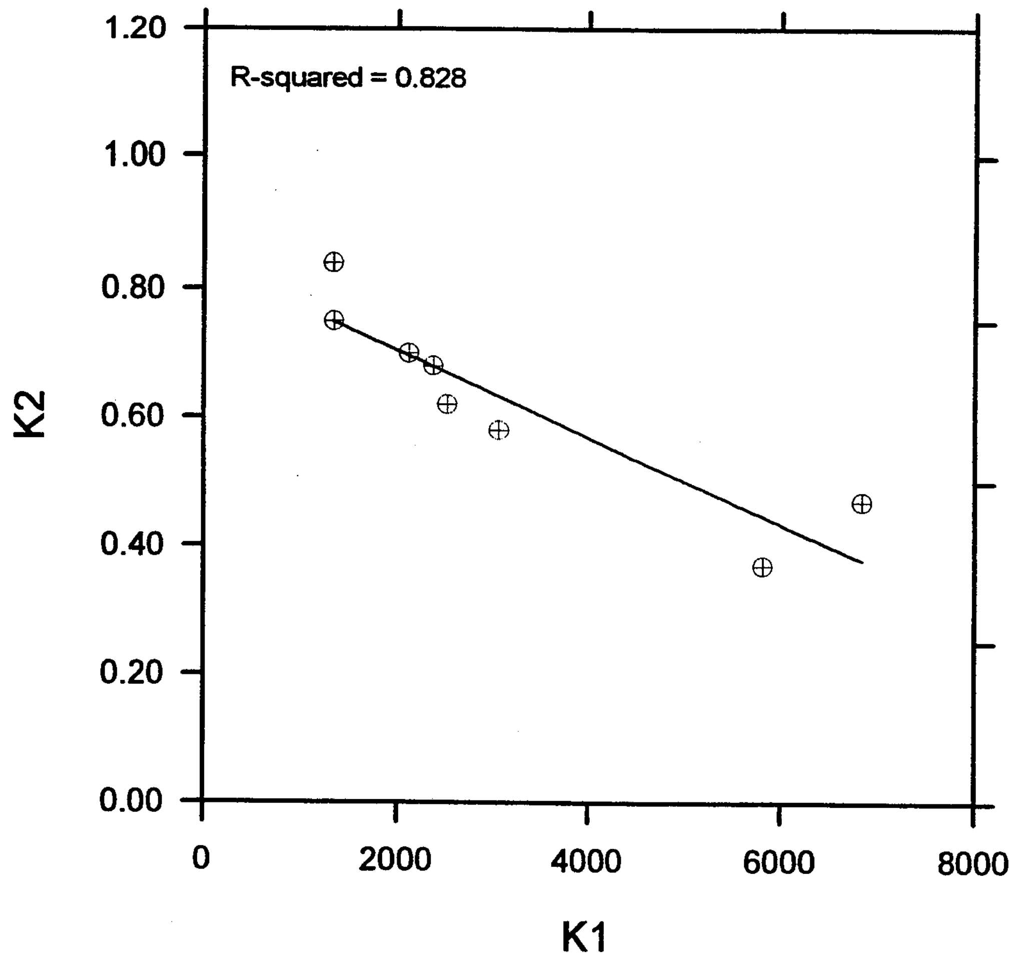


FIGURE 4.1 - The K_1 and K_2 relationship for eight type 1 soils from New England using English units and Equation 4-1 (based on the data from Lee et al., 1997).

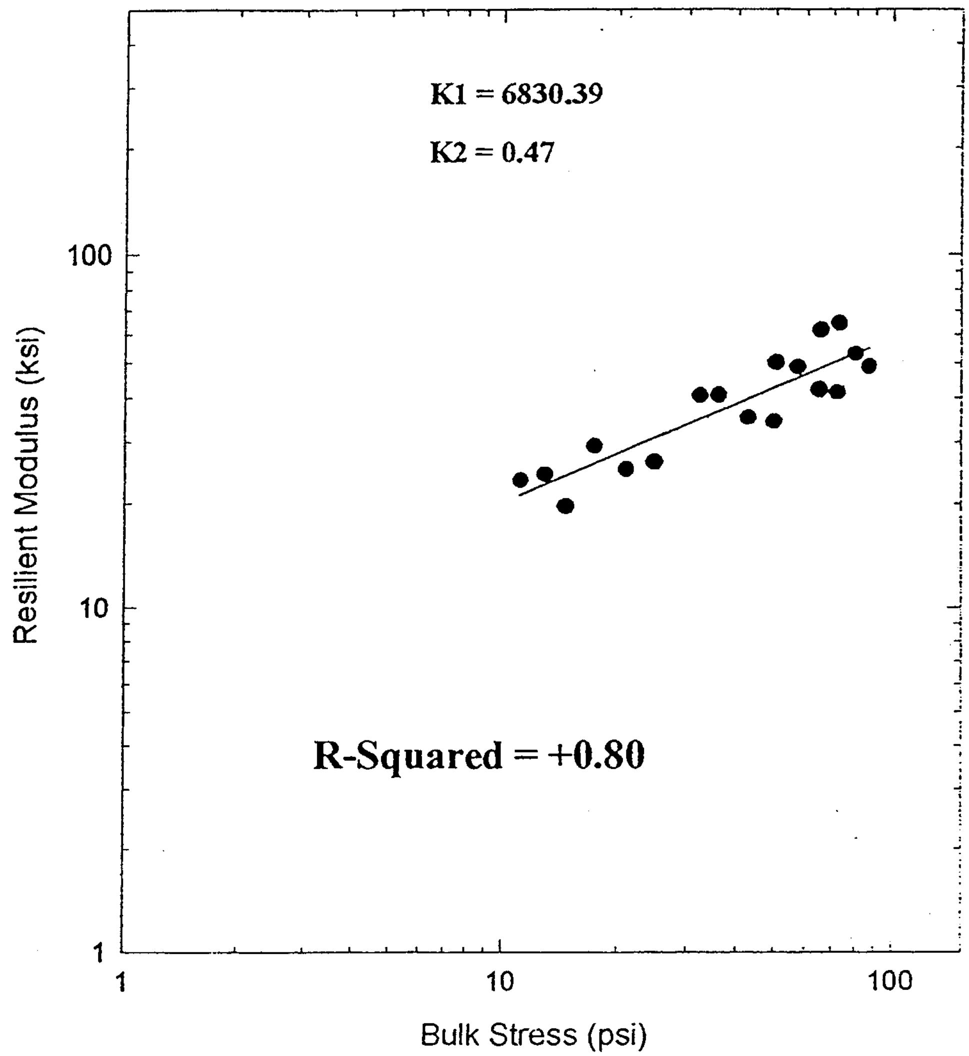


FIGURE 4.2 - AASHTO T292-91 resilient modulus results for processed A-1-a type 1 Frenchville subbase from Maine (Lee et al., 1997).

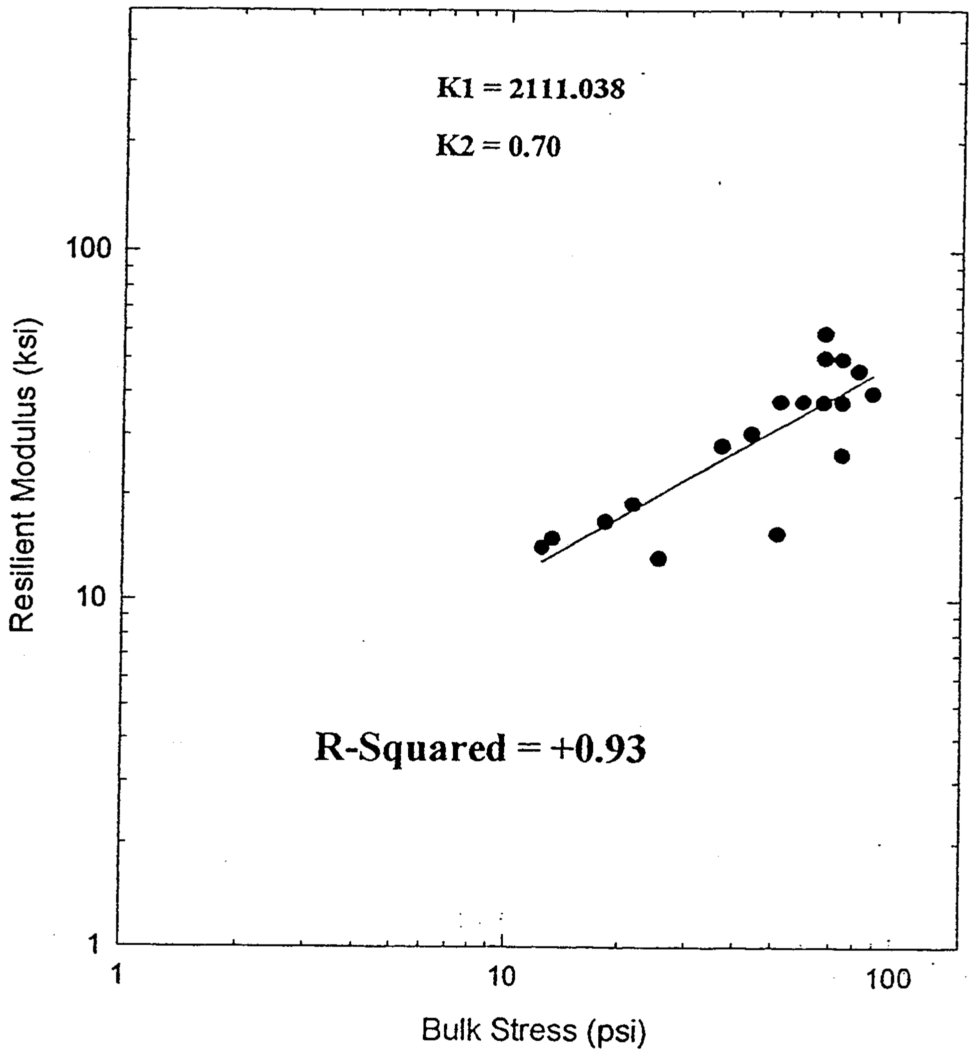


FIGURE 4.3 - AASHTO T292-91 resilient modulus results for bank run A-1-a type 1 Sabattus gravel (Lee et al., 1997).

Santha (1994) developed K_3 and K_4 in Equation 4-2 from a linear regression of the resilient moduli from multiple specimens of fifteen type 1 soils from Georgia. Three specimens were tested from each sample. Soil index properties were obtained prior specimen preparation. The specimens were statically compacted in three equal layers to dry densities corresponding to 1.5 percentage points lower than optimum water content, optimum water content, and 1.5 percentage points higher than optimum water content. Laboratory resilient modulus testing was done using an early resilient modulus specification, AASHTO T274-82. Santha compared the observed laboratory resilient moduli with the resilient moduli predicted using Equation 4-2. These results are shown below in Figure 4.4.

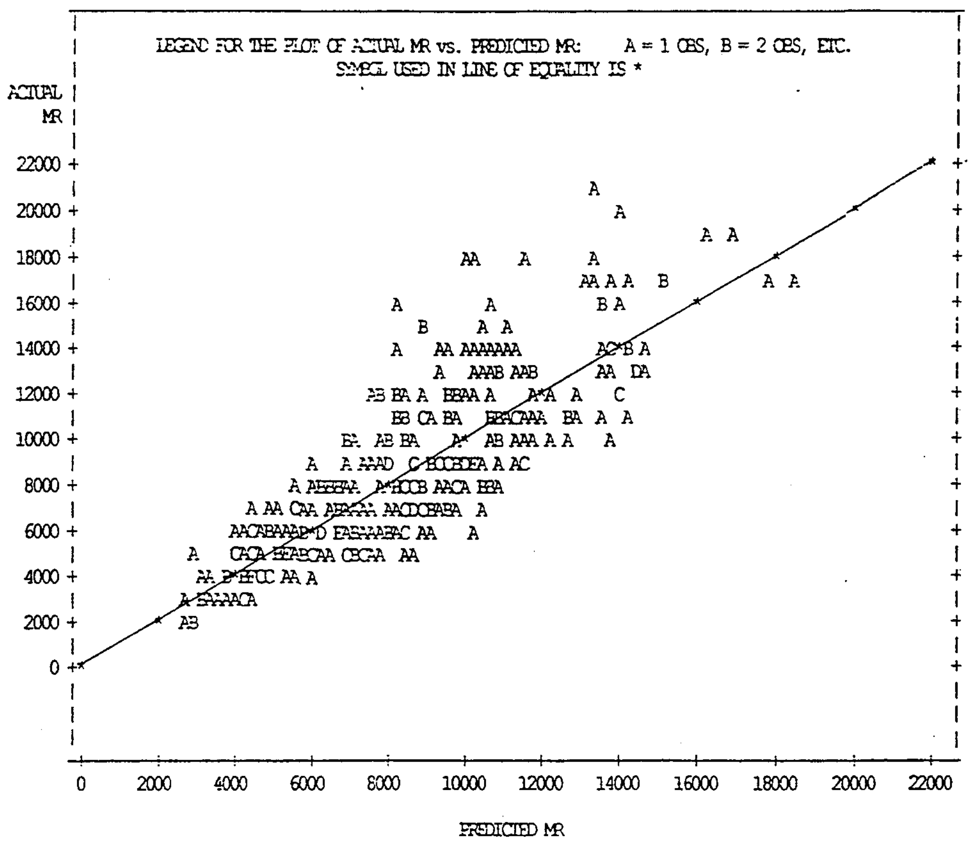


FIGURE 4.4 - Resilient moduli obtained from the AASHTO T274-82 resilient modulus test compared to predicted resilient moduli using the bulk stress relationship described by Equation 4-2 (Santha, 1994).

4.2 Universal Relationship for Type 1 Soils

Another relationship for modeling resilient modulus behavior is the universal relationship shown below. This equation incorporates the effects of both bulk stress and deviator stress (σ_d).

$$M_R = K_5 P_a \left(\frac{\theta}{P_a}\right)^{K_6} \left(\frac{\sigma_d}{P_a}\right)^{K_7} \qquad \text{(Eq. 4-3)}$$

For a linearly elastic material $K_6 = 0$ and $K_7 = 0$. By setting $K_7 = 0$ the bulk stress relationship in Equation 4-2 is obtained.

The universal relationship has been used to represent the resilient modulus of type 1 soils. Santha (1994) used the same resilient modulus results discussed in Section 4.1 to conduct a linear regression analysis using Equation 4-3. Soil properties, values of K_5, K_6, and K_7, and correlation coefficients for the individual soil specimens are shown in Appendix B. The range of correlation coefficients for the 45 soil specimens was between 0.98 and 0.75. The average correlation coefficient for these soils was 0.92. Therefore, the universal relationship provided a good fit to the resilient moduli for these soils. The relationships between K_5, K_6, and K_7 are shown in Figures 4.5, 4.6, and 4.7. Figure 4.5 shows that K_6 tends to decrease as K_5 increases and has a correlation coefficient of 0.207. Figure 4.6 shows that K_7 increases with increasing K_5 having a correlation coefficient of 0.254. Figure 4.7 shows that K_7 tends to decrease as K_6 increases however, the correlation coefficient was 0.062, which is extremely low. Santha (1994) found that Equation 4-3 predicted the observed laboratory resilient moduli quite well. The comparison is shown in Figure 4.8.

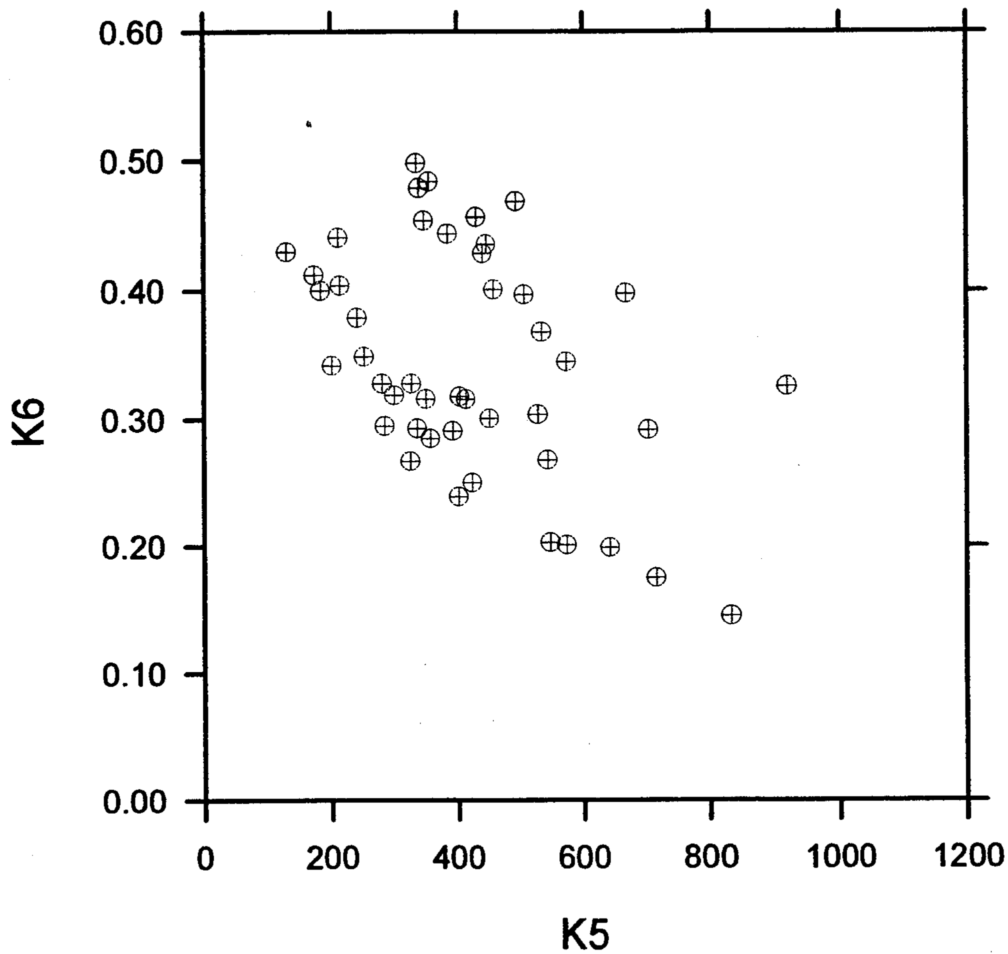


FIGURE 4.5 - The relationship between K_5 and K_6 for 45 specimens of type 1 soils from Georgia using Equation 4-3 (based on the data from Santha, 1994).

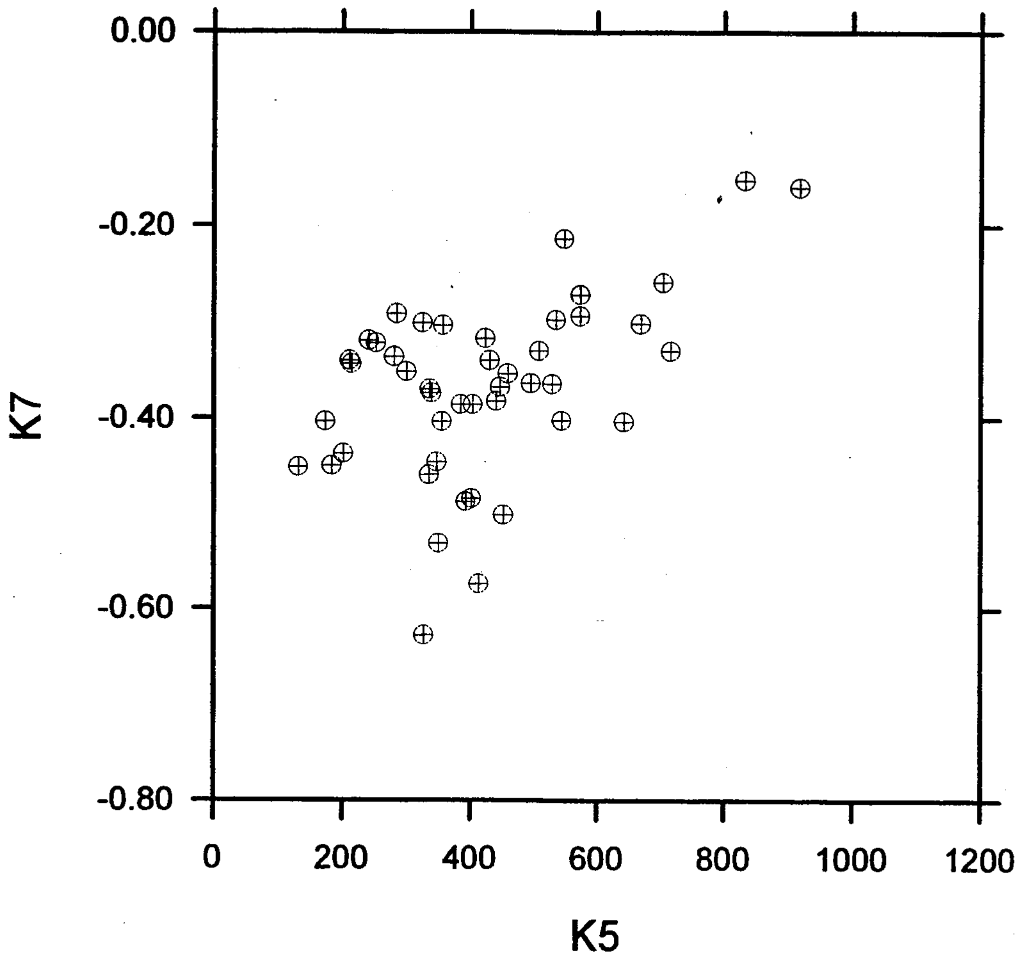


FIGURE 4.6 - The relationship between K_5 and K_7 for 45 specimens of type 1 soils from Georgia using Equation 4-3 (based on the data from Santha, 1994).

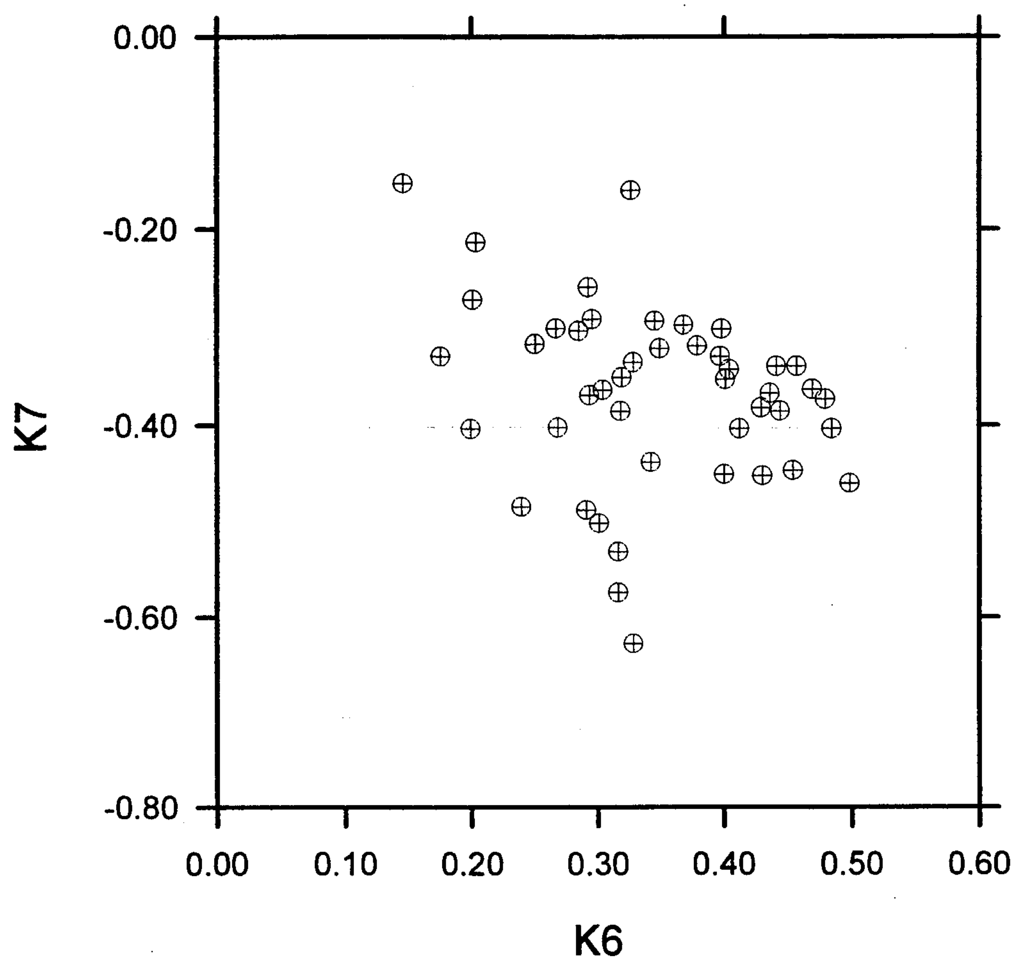


FIGURE 4.7 - The relationship between K_6 and K_7 for 45 specimens of type 1 soils from Georgia using Equation 4-3 (based on the data from Santha, 1994).

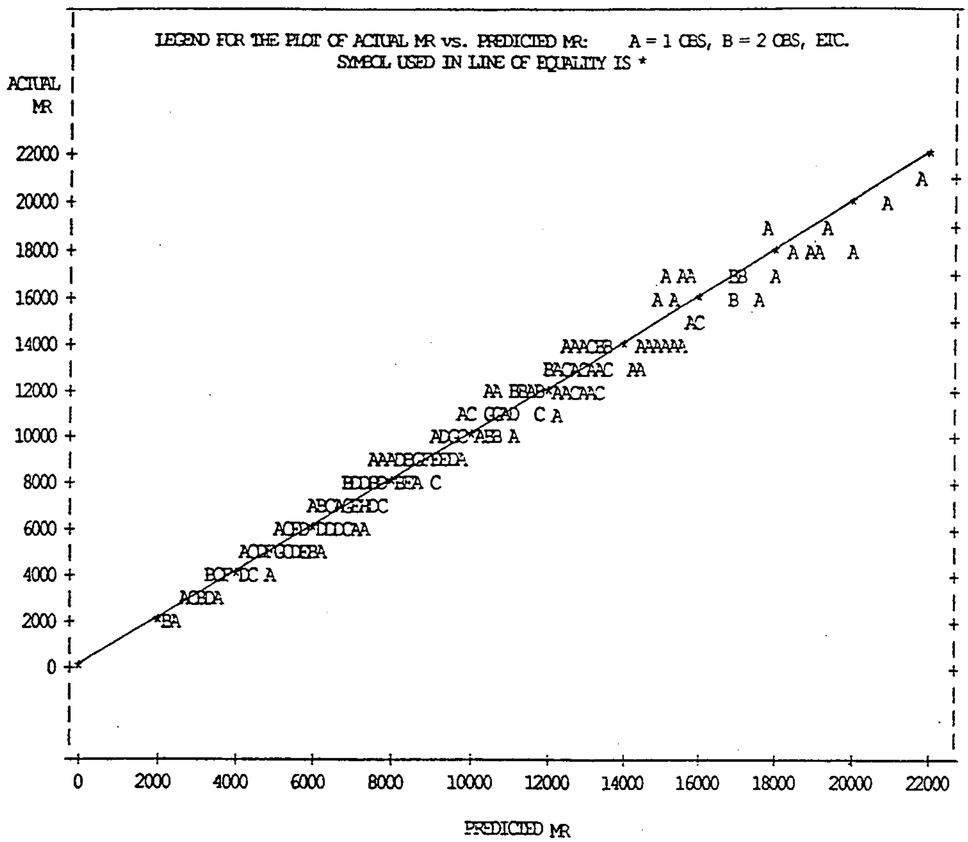


FIGURE 4.8 - Resilient moduli obtained from AASHTO T274-82 resilient modulus testing compared to predicted resilient moduli using Equation 4-3 (Santha, 1994).

Santha (1994) compared the resilient moduli predicted by the universal and bulk stress relationships. The experimental versus predicted resilient moduli using Equation 4-3 (Figure 4.8) provided better correlations when compared to Equation 4-2 (Figure 4.4).

Using the universal relationship Von Quintus and Killingsworth (1998) conducted a regression analysis on soil index property and laboratory resilient modulus data stored in the FHWA LTPP database to obtain values of K_5, K_6, and K_7. Von Quintus and Killingsworth (1998) used SI units. The data is compiled from 372 low volume road projects throughout the United States. The individual data and relationship correlations were not given, however, the average correlation coefficient exceeded 0.85 (Von Quintus and Killingsworth, 1998). The results of the analysis are summarized in Table 4.1.

TABLE 4.1 - Results of linear regression analysis from repeated-load triaxial compression test results from the LTPP database of samples of unbound pavement materials and subgrade soils from 372 low volume road projects using Equation 4-3 and SI units(Von Quintus and Killingsworth, 1998).

Soil	K_5	K_6	K_7
Sands	598	0.44	-0.12
Gravels	836	0.23	-0.08
Bases	869	0.65	-0.04

4.3 Cyclic Stress and Confining Pressure Relationship for Type 1 Soils

Another relationship that is gaining popularity is based on cyclic stress and confining pressure. The cyclic load is defined as the difference between the maximum load applied during a test sequence and the holding load used to maintain contact between the end platens and the test specimen. The holding load is typically 10 percent of the maximum load. Therefore, the cyclic stress is 90 percent of the deviator stress. Some authors assume cyclic stress equal to deviator stress (Von Quintus and Killingsworth, 1998). The relationship is described by Equation 4-4.

$$M_R = K_8(\sigma_{cyclic})^{K_9} K_{10}(1+\sigma_3)^{K_{11}} \quad \text{(Eq. 4-4)}$$

Six A-1-a type 1 soil samples from Maine have undergone resilient modulus testing using SHRP Protocol P46. Using the cyclic stress relationship described by Equation 4-4, values for K_8, K_9, and K_{11} were generated by conducting a linear regression analysis of the test results. K_{10} was omitted from the analysis and can be assumed equal to 1 to preserve the form of Equation 4-4 presented above. Stresses and moduli had units of psi. Soil properties, values for K_8, K_9, and K_{11}, and correlation coefficients for the individual soil samples are listed in Appendix C. The range of the correlation coefficients was 0.997 to 0.984 showing that the equation provides excellent fits to the actual resilient moduli for these soils. The relationships between K_8, K_9, and K_{11} are shown in Figures 4.9, 4.10, and 4.11. Figures 4.9 and 4.10 show that K_9 and K_{11} decrease as K_8 increases. The K_8-K_9 relationship has a low correlation coefficient of 0.222 whereas the K_8-K_{11} relationship has a higher correlation coefficient of 0.658. Figure 4.11 shows that there is no significant correlation between K_{11} and K_9 as evidenced by an extremely low correlation coefficient of 0.009. A sample log-log plot of resilient modulus versus cyclic stress is given in Figure 4.12.

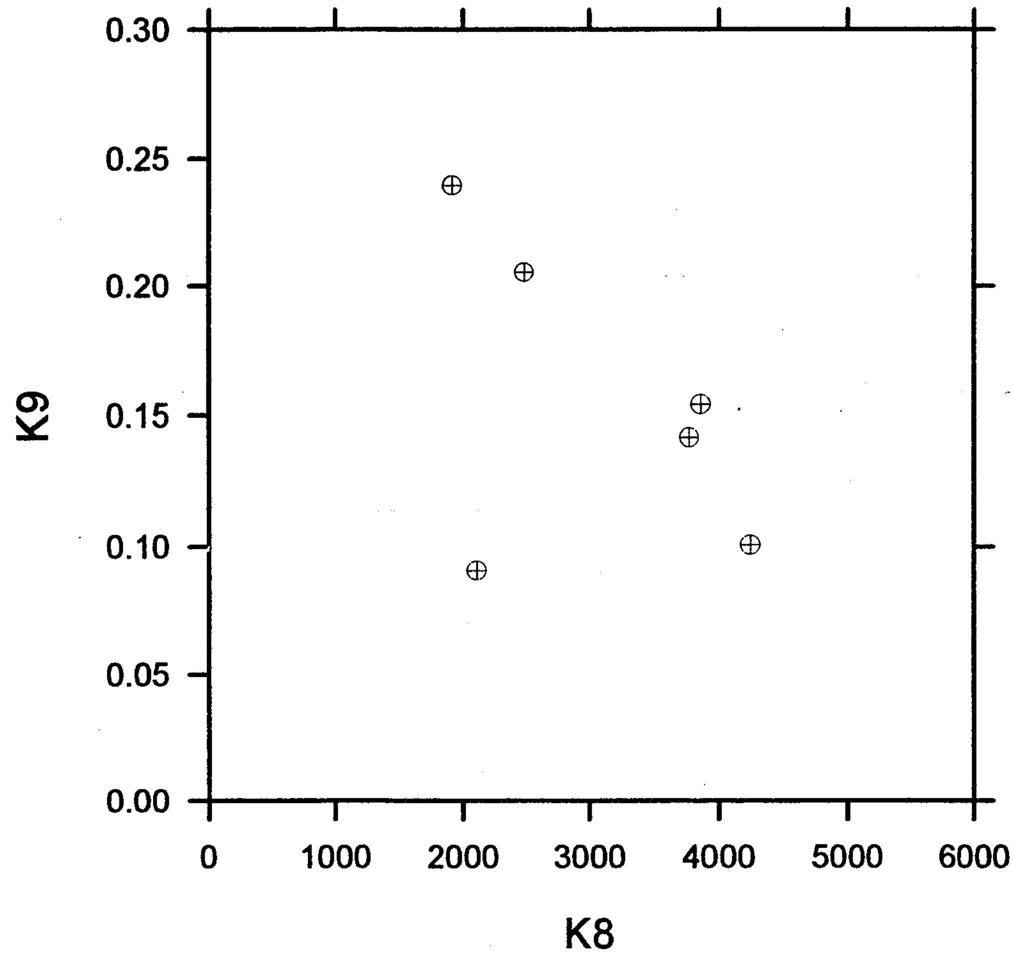


FIGURE 4.9 - The relationship between K_8 and K_9 for six type 1 soils from Maine using Equation 4-4 and English units (based on the data from Law Engineering, 1992).

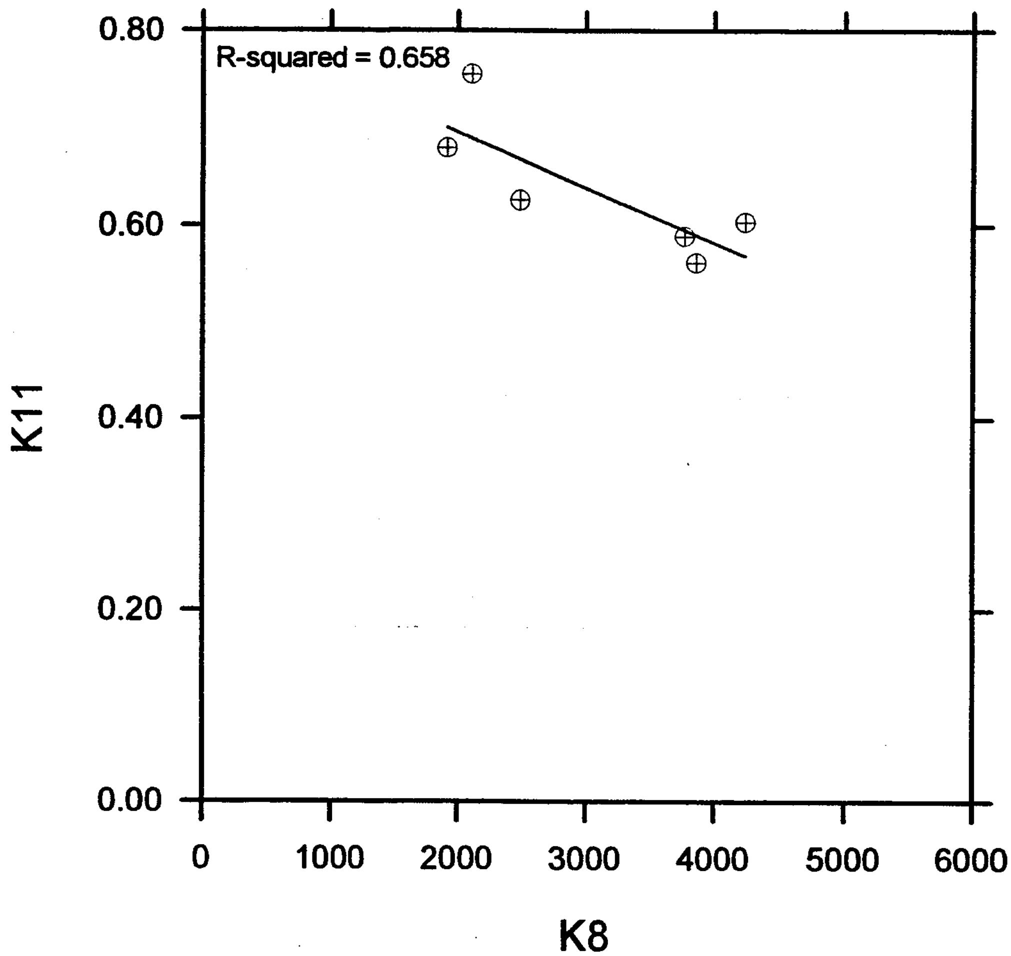


FIGURE 4.10 - The relationship between K_8 and K_{11} for six type 1 soils from Maine using Equation 4-4 and English units (based on the data from Law Engineering, 1992).

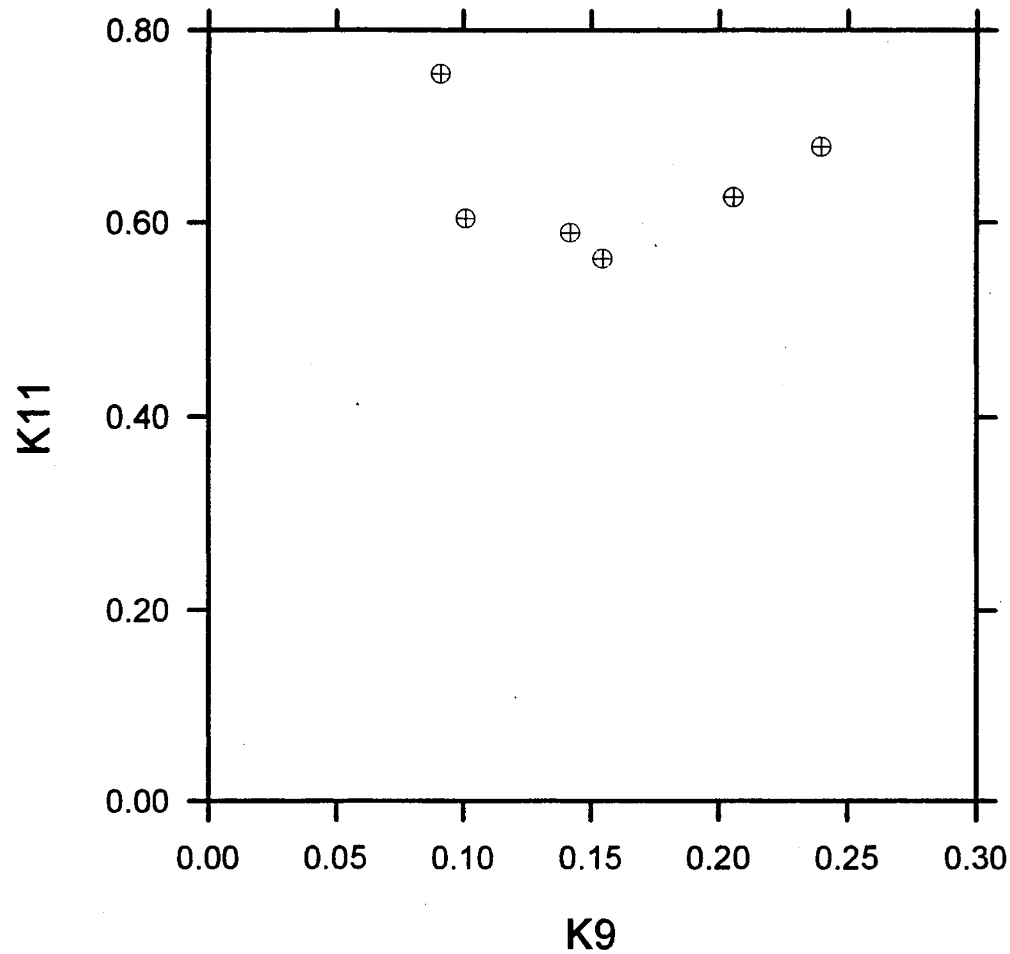


FIGURE 4.11 - The relationship between K_9 and K_{11} for six type 1 soils from Maine using Equation 4-4 and English units (based on the data from Law Engineering, 1992).

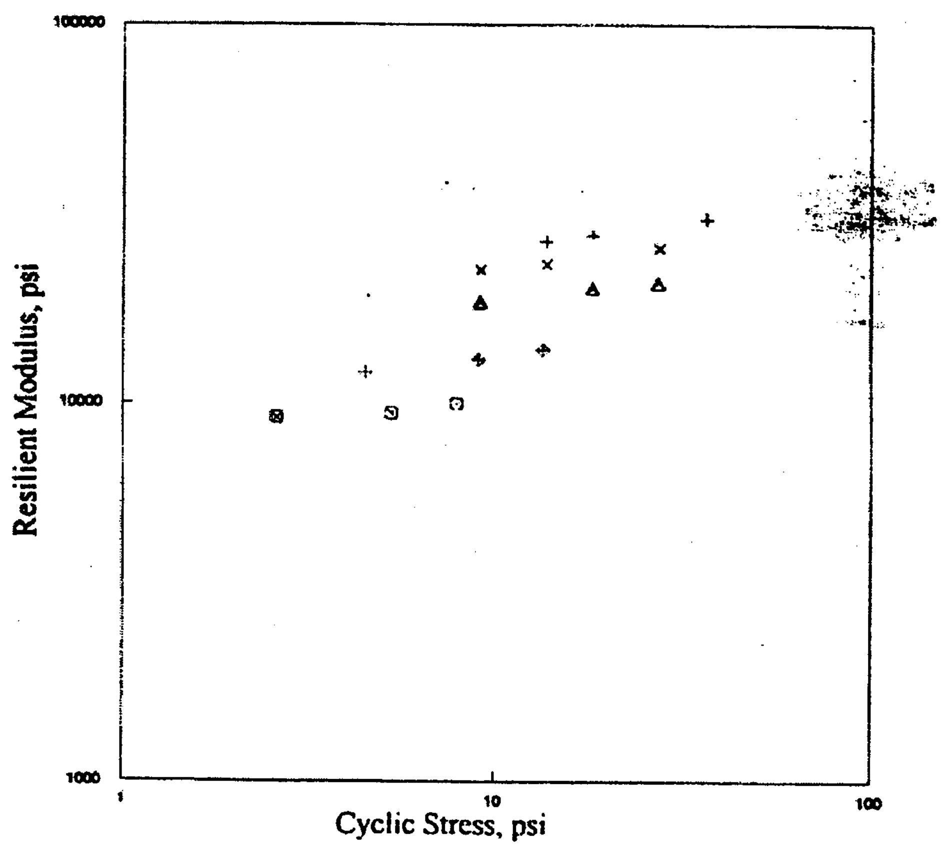


FIGURE 4.12 - Sample log-log plot of resilient modulus versus cyclic stress of an A-1-a soil specimen from Brunswick, Maine (Law Engineering, 1992).

Von Quintus and Killingsworth (1998) conducted a linear regression for type 1 soils using Equation 4-4 and assuming cyclic stress equal to deviator stress. Von Quintus and Killingsworth (1998) used SI units (resilient modulus and stresses expressed in kPa). The results are part of the same study using the SHRP LTPP database discussed in Section 4.1. Assuming that bases, gravel, and sand are type 1 soils, the resulting values of K_8, K_9, and K_{11} are listed in Table 4.2. K_{10} was omitted from the analysis and can be assumed equal to 1.0 in Equation 4-4.

TABLE 4.2 - Summary of linear regression analysis statistics from repeated-load triaxial compression test results from the LTPP database of unbound pavement materials and subgrade soils using Equation 4-4 and SI units (Von Quintus and Killingsworth, 1998).

Soil Type	K_8	K_9	K_{11}
Sands	5400	0.14	0.45
Gravels	8100	-0.02	0.46
Bases	5500	0.21	0.59

4.4 Deviator Stress Relationship for Type 2 Soils

AASHTO T294-92 (AASHTO, 1993b) specifies a simple relationship for the resilient modulus of type 2 soils based on deviator stress.

$$M_R = K_{12}(\sigma_d)^{K_{13}} \qquad \text{(Eq. 4-5)}$$

The same unit conversion problem encountered with Equation 4-1 is present in Equation 4-5. Thus, atmospheric pressure (P_a) is introduced into the equation to make the K_n constants unitless. This form of the equation is shown below:

$$M_R = K_{14}P_a\left(\frac{\sigma_d}{P_a}\right)^{K_{15}} \qquad \text{(Eq. 4-6)}$$

The universal relationship described by Equation 4-3 can be transformed into the deviator stress relationship by setting $K_6 = 0$.

Using Equation 4-5 Drumm et al. (1993) conducted a linear regression analysis on laboratory resilient moduli of eight type 2 soils from Tennessee to determine K_{12} and K_{13}. Two soils were AASHTO classification A-4, two were A-6, one was A-7-5, and the remaining three were classification A-7-6. Optimum moisture contents and dry densities were determined using AASHTO T-99. A range of optimum moisture contents was developed by computing the 100 and 95 percent densities. Specimens were compacted using kneading techniques. The specimens were stored for six to eight days in a high humidity environment. Laboratory resilient modulus tests were done using SHRP Protocol P46 and pneumatic testing apparatus. Two external LVDTs were attached to the top platen to measure deformation. Soil properties and values for K_{12} and K_{13} for the individual soil specimens are listed in Appendix D. The relationship between K_{12} and K_{13} is shown in Figure 4.13. K_{13} tends to decrease with increasing K_{12}. The K_{12}-K_{13} relationship had an extremely low correlation coefficient of 0.072. Although the correlation coefficients for each specimen were not given Drumm et al. (1993) reported that Equation 4-5 predicted the observed resilient moduli very well. Figure 4.14 is a sample plot of the measured resilient moduli and the values predicted by Equation 4-5.

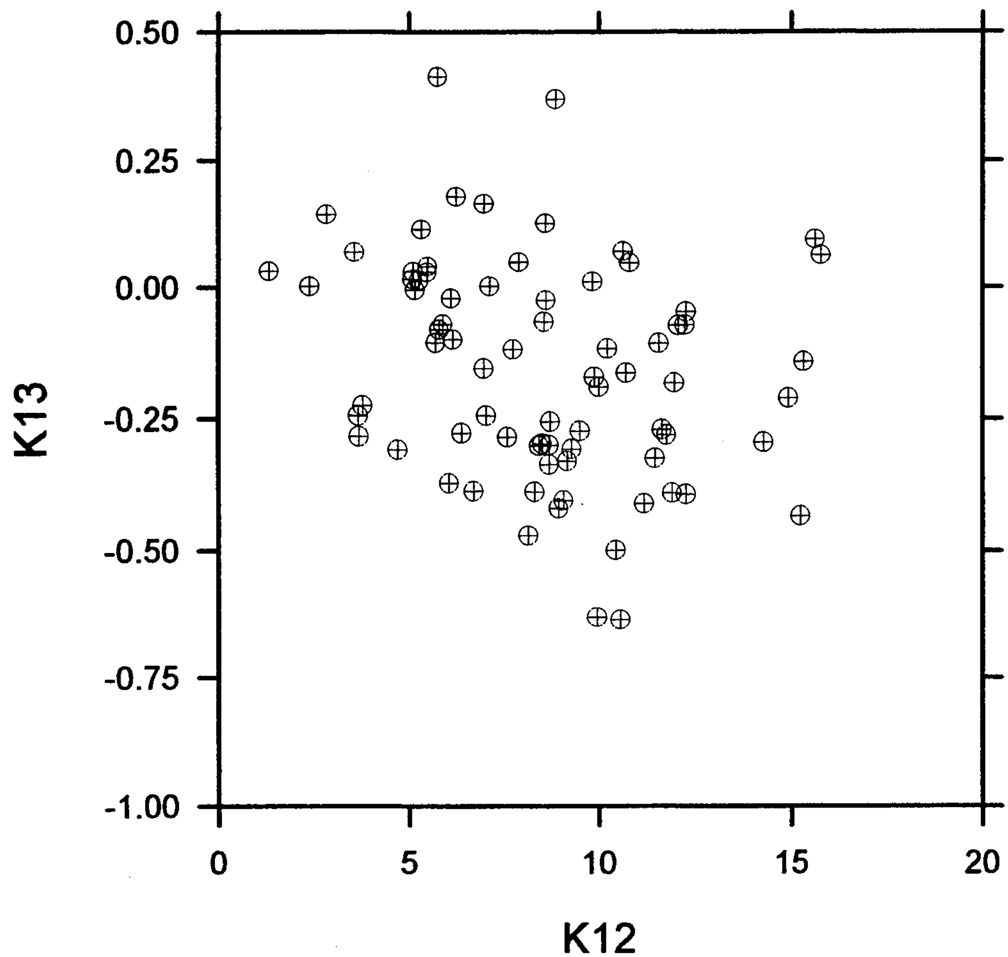


FIGURE 4.13 - The relationship between K_{12} and K_{13} using Equation 4-5 from 75 specimens of eight type 2 soils from Tennessee (based on the data from Drumm et al., 1993).

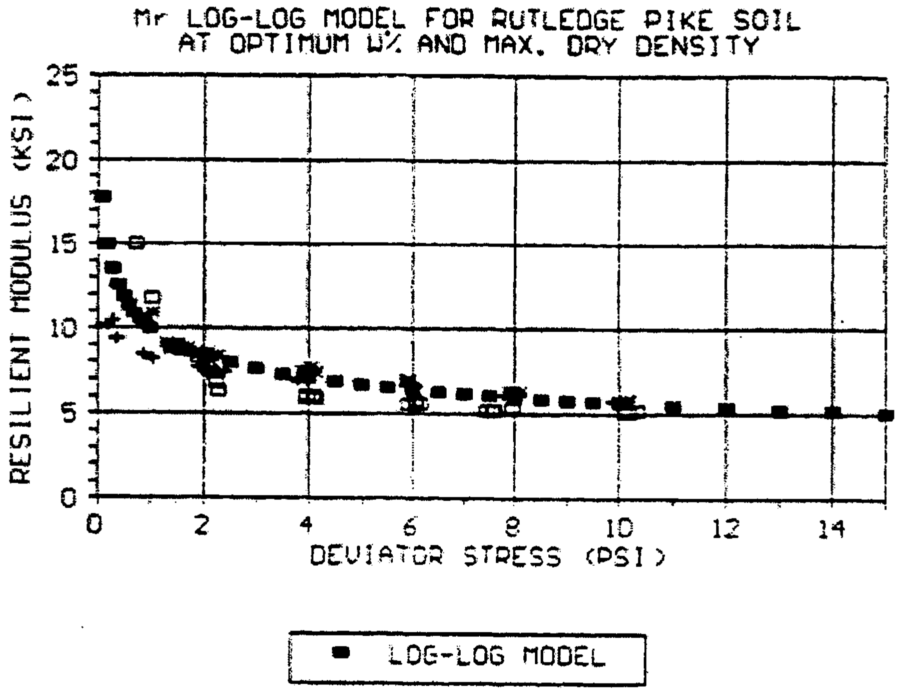


FIGURE 4.14 - Resilient modulus test results using SHRP P46 Protocol and predicted resilient modulus using the deviator stress relationship described by Equation 4-5 of an A-7-6 soil from Tennessee. $K_{12} = 9.09$ and $K_{13} = 0.40$ (Drumm et al., 1993).

Values for K_{14} and K_{15} were generated from a linear regression analysis using Equation 4-6 and resilient moduli tests from multiple specimens of 14 type 2 soil samples from Georgia (Santha, 1994). Soil index properties were obtained prior to resilient modulus testing. Three specimens were used from each sample. The specimens were statically compacted in three equal layers to dry densities corresponding to 1.5 percent lower than optimum, optimum, and 1.5 percent higher than optimum water contents. Laboratory resilient modulus testing was done using AASHTO T274-82. Soil properties, values for K_{14} and K_{15}, and correlation coefficients for the individual soil specimens are listed in Appendix B. The range of correlation coefficients for the 42 specimens was 0.51

and 0.99 with an average of 0.90. There was a general trend of increasing K_{15} with increasing K_{14} as shown in Figure 4.15. The K_{14}-K_{15} relationship has a correlation coefficient of 0.720. Santha (1994) compared the observed laboratory resilient moduli with the resilient moduli predicted using Equation 4-6. These results are shown in Figure 4.16.

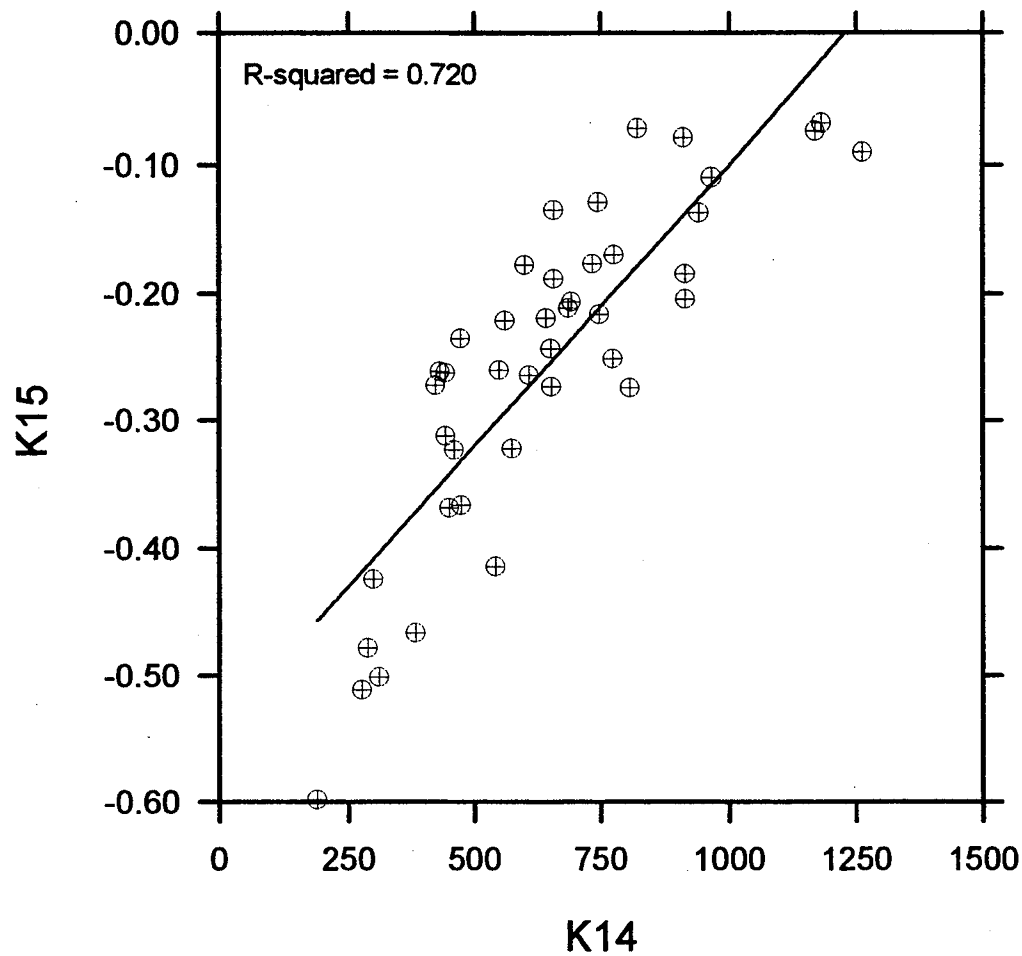


FIGURE 4.15 - The relationship between K_{14} and K_{15} using Equation 4-6 from 42 specimens of 14 type 2 soils from Georgia (based on the data from Santha, 1994).

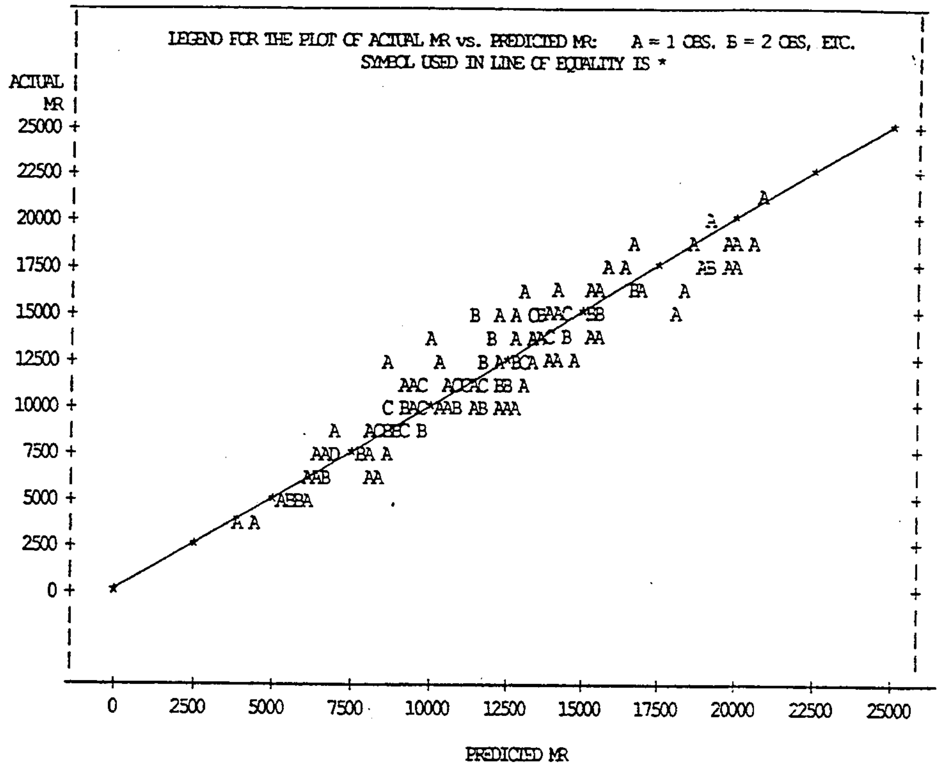


FIGURE 4.16 - Resilient moduli obtained from AASHTO T274-82 resilient modulus tests compared to resilient moduli predicted using Equation 4-6 (Santha, 1994).

4.5 Universal Relationship for Type 2 Soils

Von Quintus and Killingsworth (1998) conducted a linear regression analysis for type 2 soils using Equation 4-3. The results are part of the same study using the SHRP LTPP database discussed in Section 4.2. The data was compiled from 372 low volume road projects throughout the United States. The individual data and relationships were not given, however, the average correlation coefficient exceeded 0.85 (Von Quintus and Killingsworth, 1998). The resulting values of K_5, K_6, and K_7 are listed in Table 4.3.

TABLE 4.3 - Results of linear regression analysis from repeated-load triaxial compression test results from the LTPP database of samples of unbound pavement materials and subgrade soils from 372 low volume road projects using Equation 4-3 and SI units (Von Quintus and Killingsworth, 1998).

Soil Type	K_5	K_6	K_7
Clays	594	0.44	-0.19
Silts	426	0.42	-0.23

4.6 Deviator Stress and Confining Pressure Stress Relationship for Type 2 Soils

Another deviator stress relationship used to describe the resilient modulus of type 2 soils is similar to the cyclic stress relationship for type 1 soils (Equation 4-4). The distinction between this equation and Equation 4-4 is that the cyclic stress term is changed to deviator stress. The equation is as follows:

$$M_R = K_{16}(\sigma_d)^{K_{17}} K_{18}(1+\sigma_3)^{K_{19}} \qquad \text{(Eq. 4-7)}$$

Von Quintus and Killingsworth (1998) conducted a linear regression analysis for type 2 soils using this deviator stress and confining pressure relationship. The results are part of the same experiment using the LTPP database discussed in Section 4.1. The resulting values of K_{16}, K_{17}, and K_{19} are listed in Table 4.4. K_{18} was omitted from the analysis and should be taken as 1.0 in Equation 4-7.

TABLE 4.4 - Summary of linear regression analysis statistics from repeated-load triaxial compression test results from the LTPP database from unbound pavement materials and subgrade soils from 372 low volume road projects using Equation 4-7 and SI units (Von Quintus and Killingsworth, 1998).

Soil Type	K_{16}	K_{17}	K_{19}
Clays	8300	-0.08	0.26
Silts	5800	0.08	0.48

Von Quintus and Killingsworth (1998) found no significant difference between the resilient modulus predicted from Equations 4-5 and 4-7. This indicates that the simpler Equation 4-5 can be used under most circumstances.

Eleven type 2 soil samples from Maine have undergone resilient modulus testing using SHRP Protocol P46. Using the deviator stress relationship described by Equation 4-7 values for K_{16}, K_{17}, and K_{19} were generated by conducting a linear regression analysis of the test results. K_{18} was omitted from the analysis. Soil properties, values for K_{16}, K_{17}, and K_{19}, and correlation coefficients for the individual soil samples are listed in Appendix D. The range of correlation coefficients for the eleven soil specimens was 0.930 to 0.998. Figures 4.17 and 4.18 show a general trend of decreasing K_{17} and K_{19} with increasing K_{16}. The K_{16}-K_{17} relationship has an extremely low correlation coefficient of 0.097. The K_{16}-K_{19} relationship has a correlation coefficient of 0.238. Figure 4.19 shows decreasing K_{19}

with increasing K_{17} however the correlation coefficient is 0.160. A sample log-log plot of resilient modulus as a function of deviator stress is given in Figure 4.20.

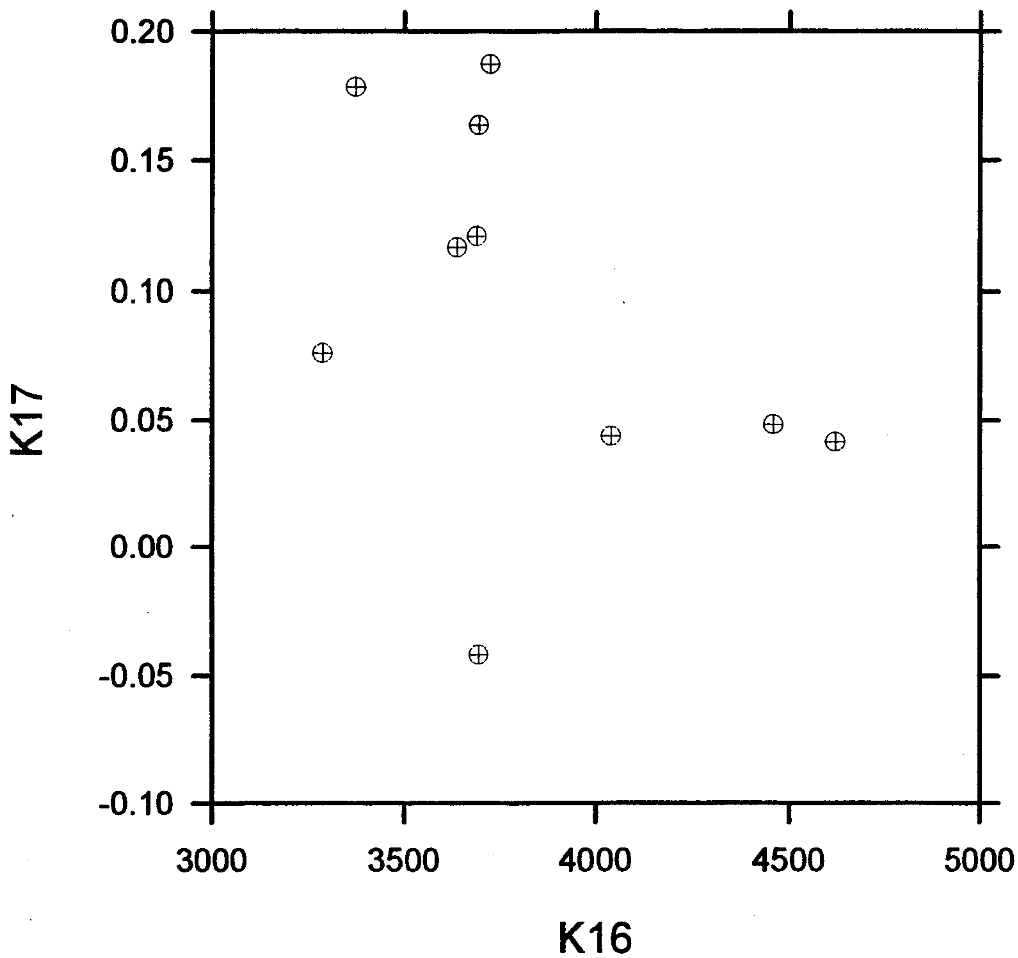


FIGURE 4.17 - The relationship between K_{16} and K_{17} using Equation 4-7 and English units from eleven type 2 soils from Maine (based on data from Law Engineering, 1992).

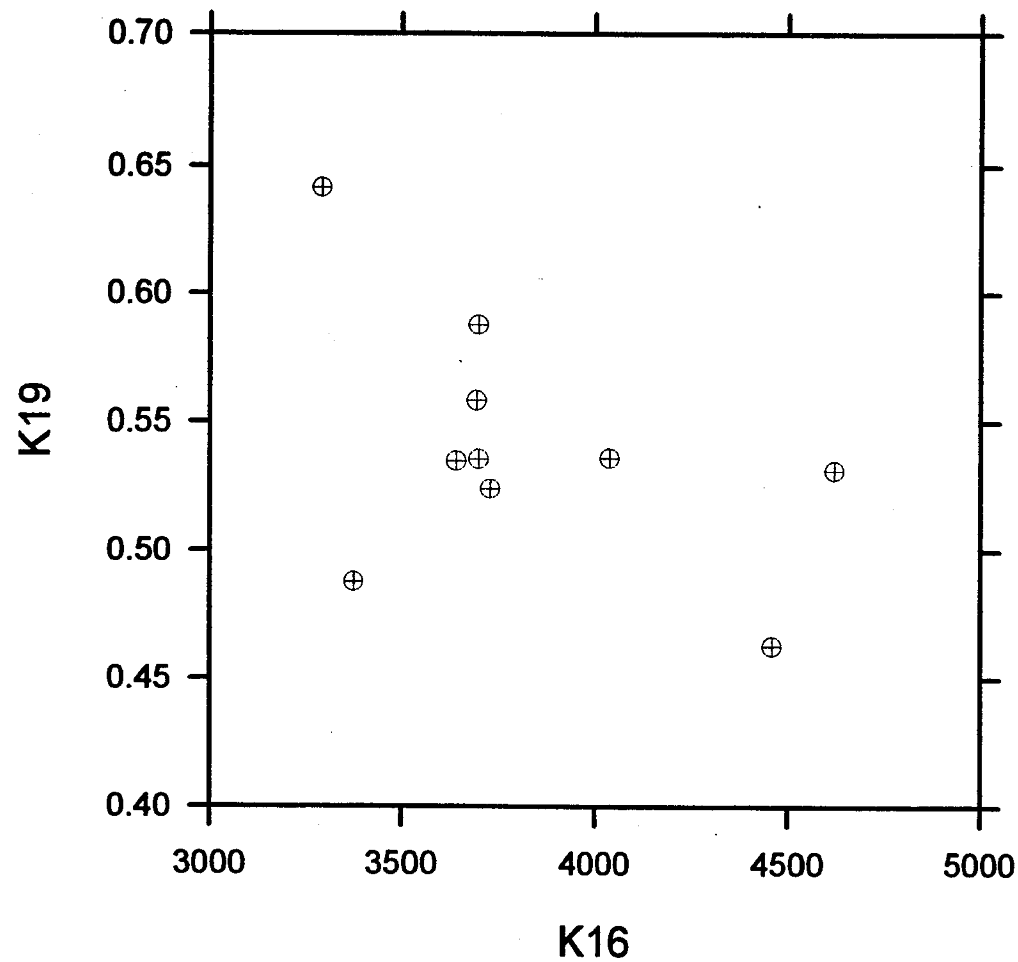


FIGURE 4.18 - The relationship between K_{16} and K_{19} using Equation 4-7 and English units from 11 type 2 soils from Maine (based on data from Law Engineering, 1992).

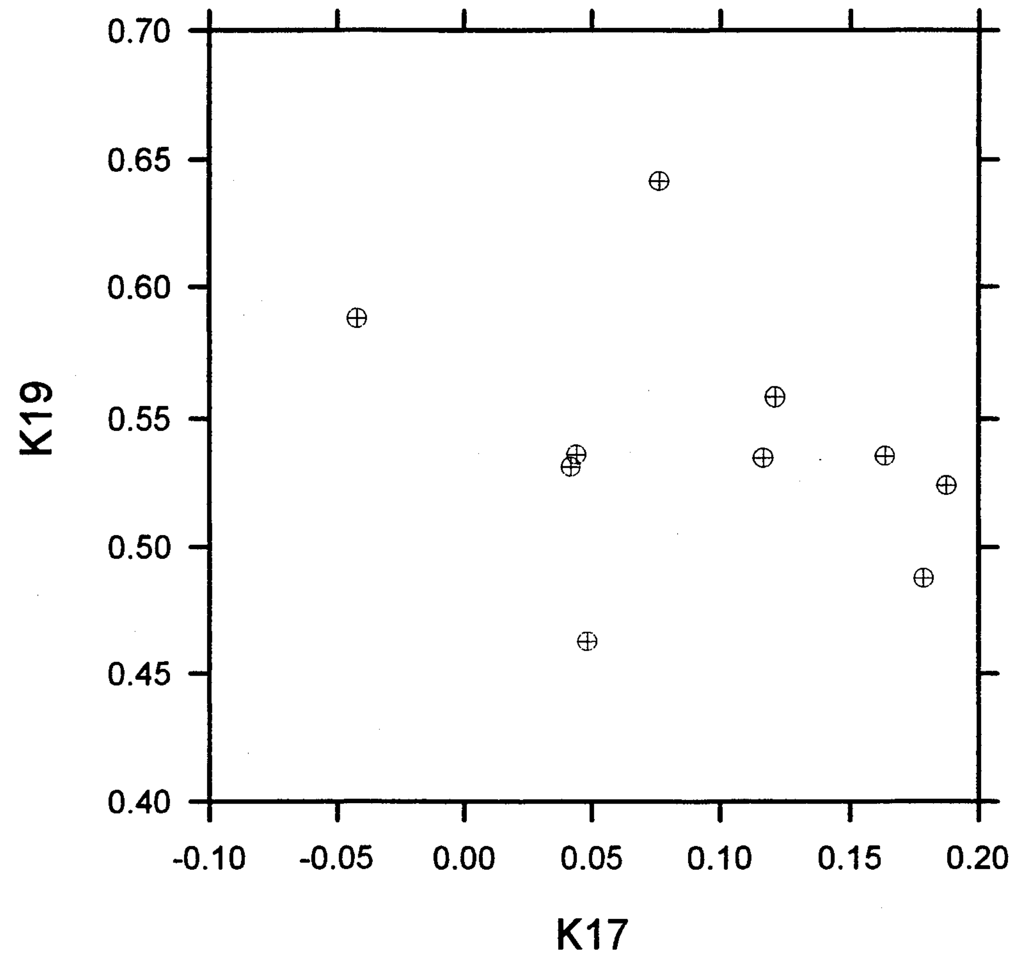


FIGURE 4.19 - The relationship between K_{17} and K_{19} using Equation 4-7 and English units from 11 type 2 soils from Maine (based on data from Law Engineering, 1992).

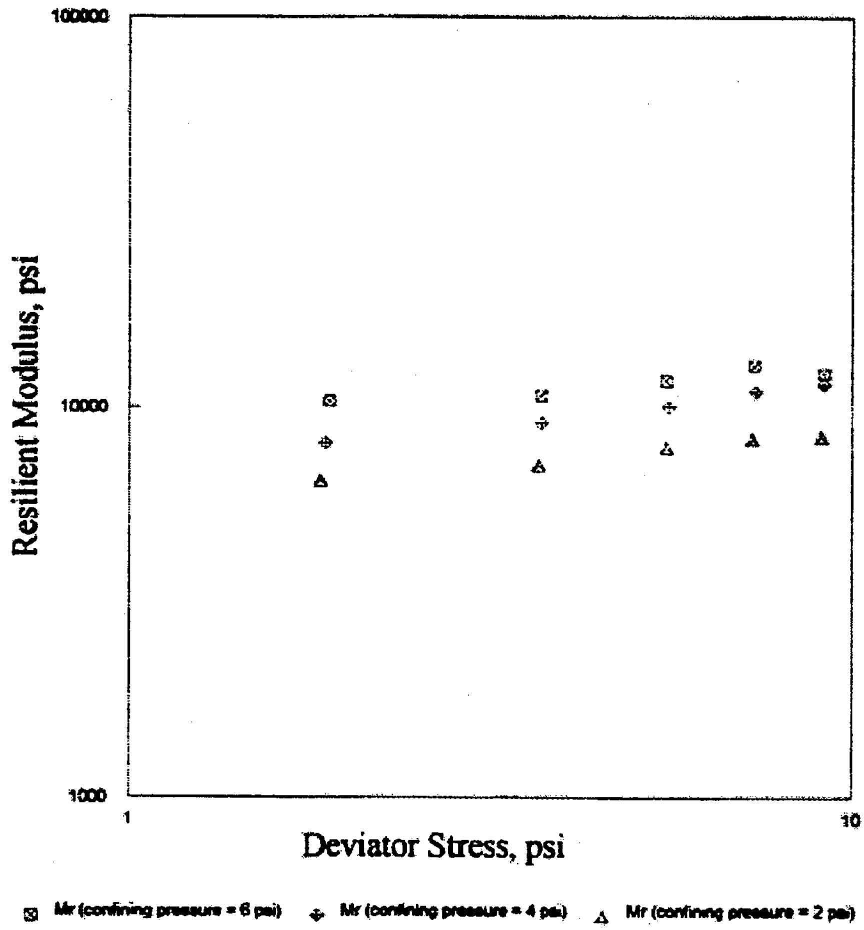


FIGURE 4.20 - Sample log-log plot of resilient modulus versus deviator stress of an A-7-6 soil specimen from South Freeport, Maine (Law Engineering, 1992).

4.7 Summary

Several constitutive relationships have been used to represent the nonlinear behavior of resilient modulus. These equations relate resilient modulus to stress conditions for use in numerical models of pavement performance.

$$M_R = K_1(\theta)^{K_2} \qquad \text{(Type 1 soil)} \qquad \text{(Eq. 4-1)}$$

$$M_R = K_3 \mathrm{P}_a \left(\frac{\theta}{\mathrm{P}_a}\right)^{K_4} \qquad \text{(Type 1 soil)} \qquad \text{(Eq. 4-2)}$$

$$M_R = K_5 \mathrm{P}_a \left(\frac{\theta}{\mathrm{P}_a}\right)^{K_6} \left(\frac{\sigma_d}{\mathrm{P}_a}\right)^{K_7} \qquad \text{(Type 1 or 2 soil)} \qquad \text{(Eq. 4-3)}$$

$$M_R = K_8(\sigma_{cyclic})^{K_9} K_{10}(1+\sigma_3)^{K_{11}} \qquad \text{(Type 1 soil)} \qquad \text{(Eq. 4-4)}$$

$$M_R = K_{12}(\sigma_d)^{K_{13}} \qquad \text{(Type 2 soil)} \qquad \text{(Eq. 4-5)}$$

$$M_R = K_{14} P_a \left(\frac{\sigma_d}{\mathrm{P}_a}\right)^{K_{15}} \qquad \text{(Type 2 soil)} \qquad \text{(Eq. 4-6)}$$

$$M_R = K_{16}(\sigma_d)^{K_{17}} K_{18}(1+\sigma_3)^{K_{19}} \qquad \text{(Type 2 soil)} \qquad \text{(Eq. 4-7)}$$

Because stress state affects type 1 soils differently than type 2 soils these relationships are often soil type dependent. However, a universal relationship that is applicable to both soil types has also been proposed.

The K_n constants have been determined for some Maine soils. Available values are summarized in this chapter. Correlations between K_n constants for a given equation were examined. Good correlations were found between K_1 and K_2 (Eq. 4-1), K_8 and K_{11} (Eq. 4-

4), and K_{14} and K_{15} (Eq. 4-6). By inspection, poor correlations were found between the K_n constants of the remaining equations (Eqs. 4-2, 4-3, 4-5, and 4-7). These correlations are helpful when choosing a consistent set of K_n constants for a given soil type.

Lee et al. (1997) determined the resilient modulus of eight type 1 soils from New England using AASHTO T292-91. Two soils were from Maine: a processed Frenchville A-1-a subbase and a bank run Sabattus A-1-a gravel. Using Equation 4-1 Lee et al. (1997) performed a linear regression on the test results to determine the material specific K_n constants. The best fit line for the Frenchville subbase had a correlation coefficient of 0.80 and the Sabattus subbase correlation coefficient was 0.93. The high values of the correlation coefficient show that Equation 4-1 predicted the actual test data for the individual soil samples extremely well. Furthermore there was a good correlation between K_1 and K_2 for all eight soils in the data base. This correlation can be helpful in choosing compatible pairs of K_1 and K_2.

Santha (1994) compared the results from actual resilient modulus test data to predicted resilient moduli from Equations 4-2 and 4-3. Santha (1994) used 45 specimens from 15 type 1 soils from Georgia. The resilient modulus specification used for testing was AASHTO T274-82. Predicted resilient moduli from Equation 4-3 showed better correlation to laboratory resilient moduli than Equation 4-2. The correlation coefficient for the relationships between K_5, K_6, and K_7 did not rise above 0.254 indicating that there is little relationship between these coefficients.

Six type 1 soils from Maine have been tested using SHRP Protocol P46 (Law Engineering, 1994). This specification uses Equation 4-4, which is based on cyclic stress. Cyclic stress is 90 percent of the maximum deviator stress applied during the loading sequence. Some authors assume cyclic stress equal to deviator stress (Von Quintus and Killingsworth, 1998). For the six Maine soils, Equation 4-4 fit the test resilient moduli extremely well. The correlation coefficients for these soils were in excess of 0.987. There was little correlation between K_8 and K_9 or between K_9 and K_{11}. However there was a general trend that K_{11} decreased as K_8 increased. K_{10} was omitted from the analysis and was taken to be 1.0.

Drumm et al. (1993) used the resilient modulus test results of 75 specimens of eight type 2 soils from Tennessee to develop representative values for the K_n constants in Equation 4-5. A linear regression analysis was used to determine these values. The correlations for the best fit lines were not given however Drumm et al. (1994) reported that Equation 4-5 fit the data very well.

Santha (1994) used resilient modulus test data from 42 specimens of 14 type 2 soils from Georgia to develop representative values for the K_n constants in Equation 4-6. AASHTO T274-82 was the specification used for resilient modulus testing. Equation 4-6 predicted the actual resilient moduli very well. The range of the correlation coefficients was 0.51 to 0.99 with an average of 0.90. A linear regression was determined between the two the K_n constants. The relationship between K_{14} and K_{15} is very strong with a correlation coefficient of 0.720.

Equation 4-7 is a deviator stress version of Equation 4-4. Eleven type 2 soils from Maine have undergone resilient modulus testing using SHRP Protocol P46 (Law Engineering, 1994). Equation 4-7 fit the resilient modulus test data extremely well. The individual best fit lines for the Maine soils had a range of correlation coefficients between 0.930 to 0.998. However, the correlation coefficients for the relationships between K_{16}, K_{17}, and K_{19} did not rise above 0.238. Therefore there is no strong relationship between the K_n constants of Equation 4-7. K_{18} was omitted from the analysis and was taken to be 1.0.

Universal equations for both soil types have been examined. Von Quintus and Killingsworth (1998) used resilient modulus test data of type 1 and type 2 unbound pavement materials and subgrade soils from 372 low volume roadway project stored in the LTPP database to develop representative values for the K_n constants of Equations 4-3, 4-4, and 4-7. The results are given in Appendix C. The average correlation coefficient for this analysis exceeded 0.85.

CHAPTER 5

CORRELATIONS OF RESILIENT MODULUS WITH SOIL INDEX PROPERTIES

To reduce the need for extensive laboratory testing, researchers have directed their efforts to develop correlations between resilient modulus and soil index properties. Baus and Ray (1992) conducted a nationwide poll of state agencies to determine, among other things, which relationship the agency used when designing by the AASHTO Guide. At the time of the survey, MDOT did not use the AASHTO Guide and had not developed any correlation between CBR or soil index properties and resilient modulus. The MDOT did indicate that it had established a method of determining resilient modulus for pavement overlay work from nondestructive test methods.

In the absence of laboratory resilient modulus testing equipment, many agencies have tried to develop correlations between resilient modulus and soil index properties. An important factor affecting the reliability of the correlations is the accuracy of the laboratory resilient moduli that form the basis of the correlation. As discussed in Chapter 2 the study conducted by Steel et al. (1994) showed that the resilient moduli of identical samples determined by several labs varied over a wide range.

In this chapter, correlations between resilient modulus and soil index properties, resilient modulus test data, strength parameters, CBR, and correlation with K_n constants are examined. Drumm et al. (1993) and Laguros et al. (1993) used small sets of moduli and soil index properties from in-house laboratory resilient modulus testing of locally available soils. Von Quintus and Killingsworth (1998) used moduli and soil index properties from 372 nationwide low volume road projects stored in the LTPP database.

5.1 Correlation with Soil Index Properties and Resilient Modulus Test Data

Drumm et al. (1993) was able to develop a good constitutive relationship combining soil index properties and resilient modulus test data. Soil index properties were obtained from eight type 2 soils from Tennessee. Several soil index properties were used for the study and are listed in Table 5.1. Resilient modulus testing was done according to SHRP Protocol P46.

Several models were developed and compared to the laboratory resilient modulus test results. Drumm et al. (1993) found the most significant factors affecting resilient modulus were classification and deviation from optimum water content. The final product did not include all the properties listed in Table 5.1 and is listed below in Equation 5-1. The soil index properties for the eight Tennessee type 2 soils examined are listed in Appendix D.

TABLE 5.1 - Soil index properties and resilient modulus test data used for regression analysis conducted by Drumm et al. (1993).

Soil Index Property and Resilient Modulus Test Data	Symbol
Liquid limit	LL
Plastic limit	PL
Plasticity index	PI
Liquidity index	LI
Percent passing the No. 200 sieve by washing (e.g. 10.2%=10.2)	P_{200}
Percent clay (e.g. 20.4%=20.4)	P_{clay}
AASHTO classification (e.g. A-7-6=7.6 or A-1-a=1.1)	$class$
Specific gravity	G
CBR at 2.54 mm penetration	$CBR_{2.54}$
CBR at 5.08 mm penetration	$CBR_{5.08}$
Optimum water content	W_{opt}
Maximum dry density	γ_{dmax}
Resilient modulus (ksi)	M_r
Confining pressure (psi)	σ_3
Deviator stress (psi)	σ_d
Dry density of the specimen (pcf)	γ_{ds}
Water content of the specimen	W_s
Deviation from maximum dry density	$\Delta\gamma_{dmax}$
Deviation from optimum water content	ΔW_{opt}
Percent saturation (e.g. 30.6%=30.6)	S
Initial tangent modulus from unconfined compression tests	$1/a$
Parameter corresponding to unconfined compressive strength	$1/b$

$$\begin{aligned}\log M_R(psi) = {} & 46.93 + 0.0188\sigma_d + 0.0333\Delta\gamma_{d\max} - 0.1143LI + 0.4680S \\ & + 0.0085class^2 - 0.0033\Delta W_{opt}{}^2 - 0.0012\sigma_3^2 + 0.0001PL^2 \qquad \text{(Eq. 5-1)} \\ & + 0.0278LI^2 - 0.0017S^2 - 38.44\log S - 0.2222\log\sigma_d\end{aligned}$$

$R^2 = 0.70$

Drumm et al. (1993) tested the accuracy of Equation 5-1 with four of the eight soils used for creating the database. Three samples of the first soil, an A-7-5, were prepared and tested at varying degrees of density. The second soil, an A-6, had a higher than optimum water content and low density. The third and fourth soils, an A-4 and an A-7-6, were of low density and lower than optimum water content. The specimens were tested at three levels of confining pressure (41 kPa, 28 kPa, 14 kPa ; 6 psi, 4 psi, 2 psi) between deviator stresses of 0 and 69 kPa (10 psi). Equation 5-1 accurately predicted the resilient moduli of the specimens within the database with the exception of the high density / low water content A-7-5 specimen. However, this is not an independent check of the the validity of the equation because it was tested using the properties of the soils within the database. The results are shown in Figures 5.1 through 5.6.

5.2 Correlation with Soil Index Properties and K_n Constants

Linear regression analyses have been done between soil index properties and the K_n constants from the stress dependent equations in Chapter 4. Von Quintus and Killingsworth (1998) used soil index properties of the soils in the LTPP database to generate equations for the values of the K_n constants from Equation 4-4. Correlations were developed for clays, silts, and sands. Inadequate soil index data existed for gravels and bases at the time of the study therefore no correlation could be generated for these soils. Several index properties were used for the linear regression analysis. These properties are listed in Table 5.2.

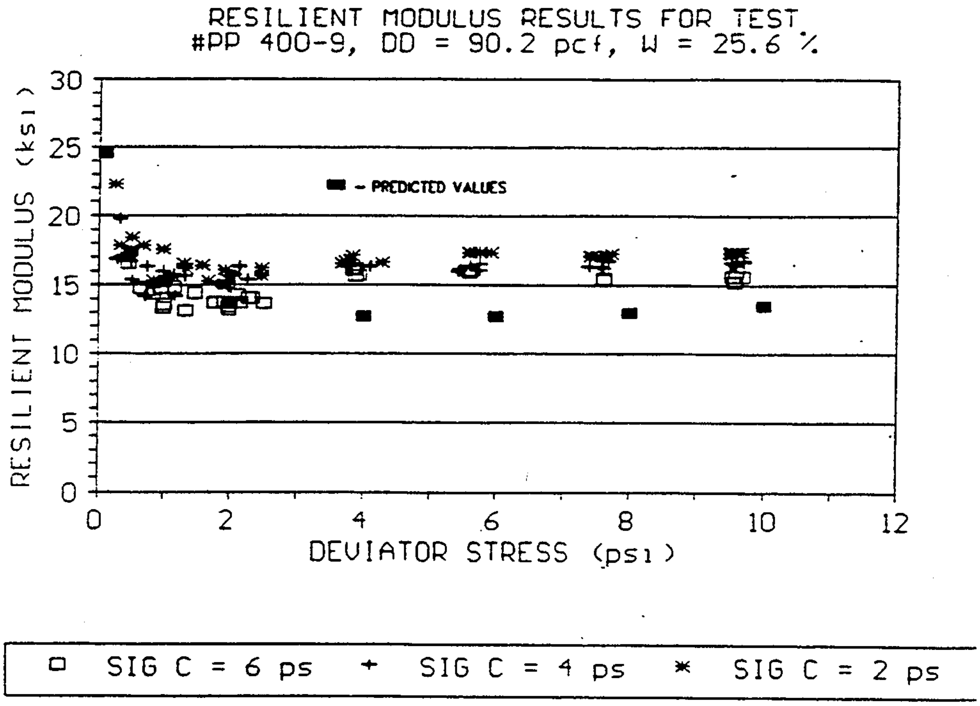


FIGURE 5.1 - Laboratory test and predicted values of resilient modulus using Equation 5-1 of an A-7-5 specimen at low water content (W=25.6%) and high density (1.44 Mg/m^3; 90.2 pcf)(Drumm et al., 1993)

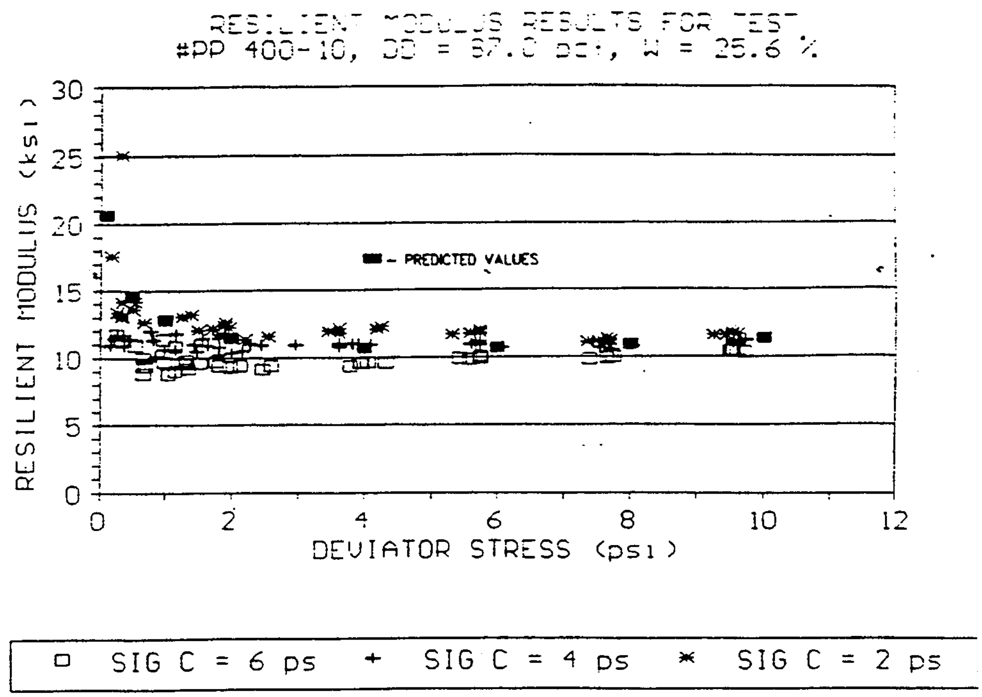


FIGURE 5.2 - Laboratory test and predicted values of resilient modulus using Equation 5-1 of an A-7-5 specimen at low water content (W=25.6%) and low density (1.39 Mg/m^3; 87 pcf) (Drumm et al., 1993).

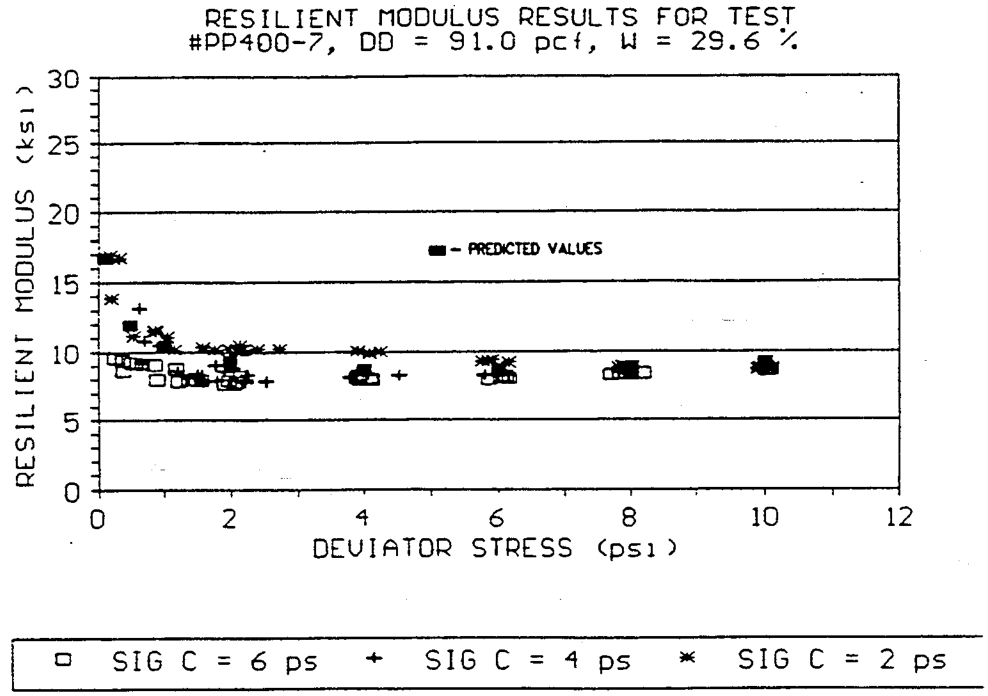


FIGURE 5.3 - Laboratory test and predicted values of resilient modulus using Equation 5-1 of an A-7-5 specimen at optimum water content (W=29.6%) and high density (14.6 Mg/m^3; 91 pcf) (Drumm et al., 1993).

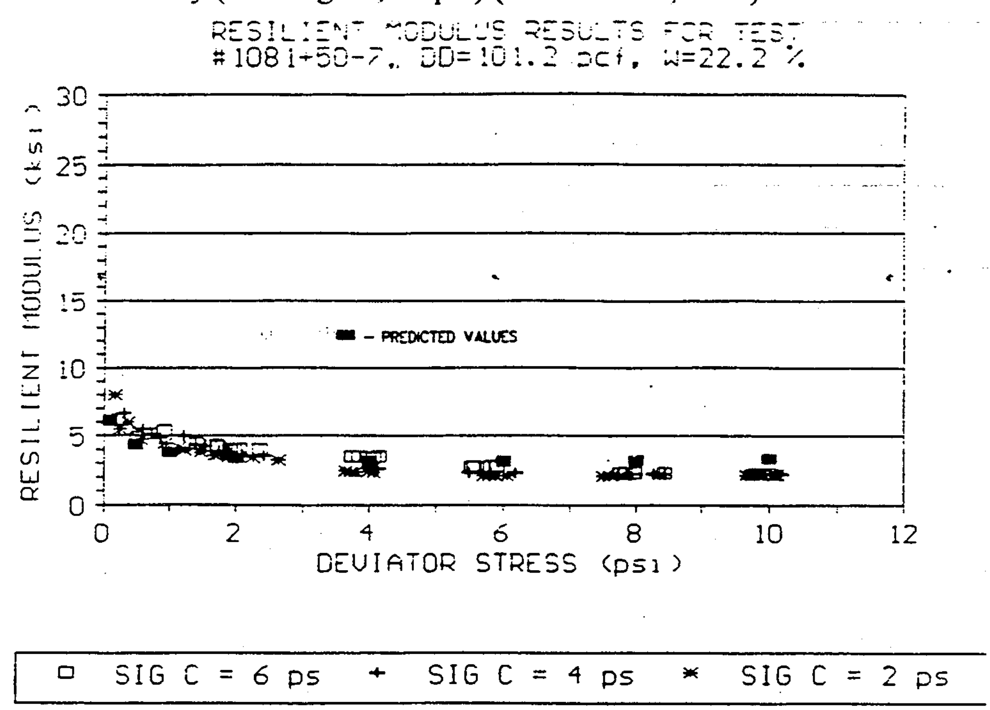


FIGURE 5.4 - Laboratory test and predicted values of resilient modulus using Equation 5-1 of an A-6 specimen at high water content (W=22.2%) and low density (1.62 Mg/m^3; 101.2 pcf) (Drumm et al., 1993).

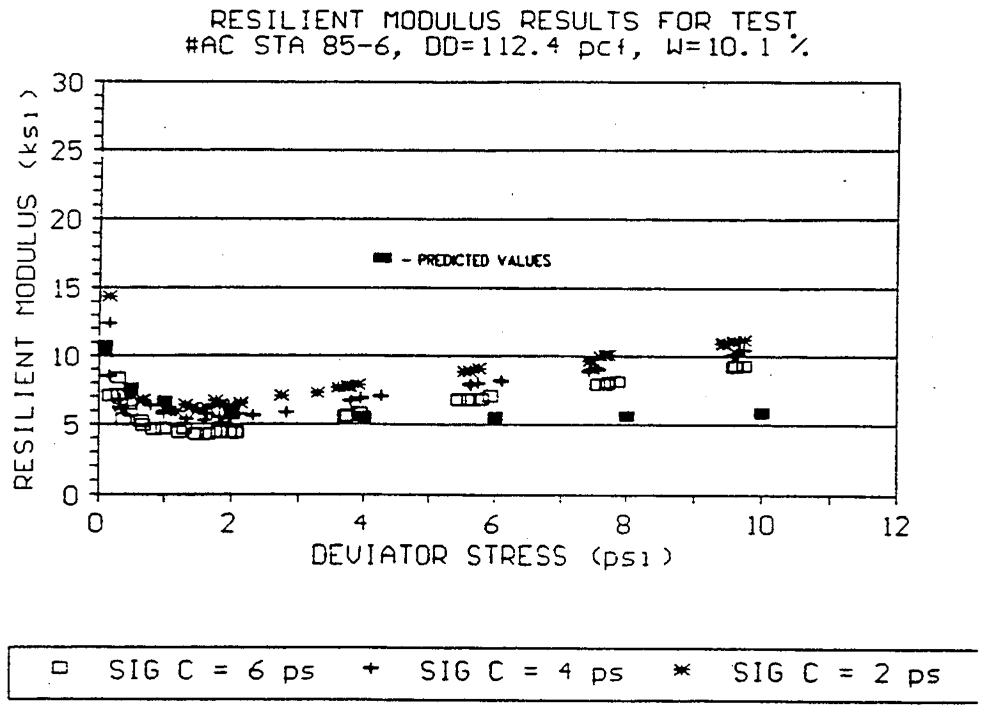


FIGURE 5.5 - Laboratory test and predicted values of resilient modulus using Equation 5-1 of an A-4 specimen at low water content (W=10.1%) and low density (1.80 Mg/m^3; 112.4 pcf)(Drumm et al., 1993).

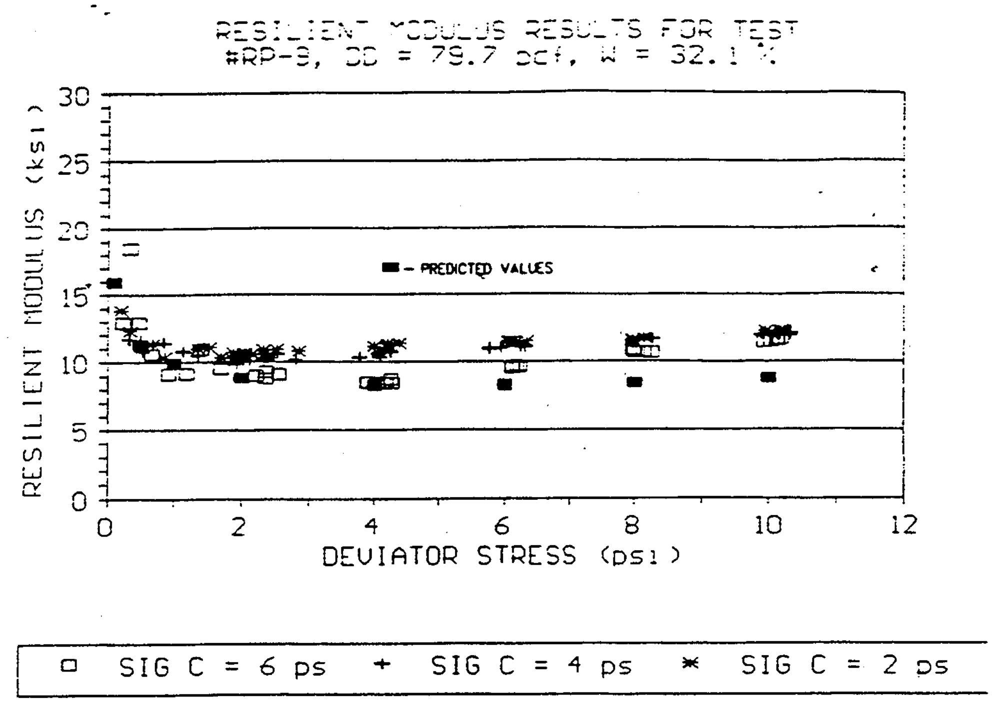


FIGURE 5.6 - Laboratory test and predicted values of resilient modulus using Equation 5-1 of an A-7-6 specimen at low water content (W=32.1%) and low density (1.28 Mg/m^3; 79.7 pcf) (Drumm et al., 1993).

TABLE 5.2 - Soil index properties used for regression analysis conducted by Von Quintus and Killingsworth (1998).

Soil Index Property	Symbol
Optimum water content	W_{opt}
Water content of the specimen	W
Dry density of the specimen	γ_d
Maximum dry density	γ_{dmax}
Percentage of silt (percent)	$\%silt$
Liquid limit	LL
Plasticity index	PI
Percent passing the No. 40 sieve	P_{40}
Saturation Degree (percent)	S

The correlation coefficients for the K_n constants of the three soil types are very different. The equations for clays (Equations 5-2 and 5-3) have high correlation coefficients (R^2=1.0, R^2=0.81). The equations for silts (Equations 5-4, 5-5, and 5-6) have intermediate correlation coefficients (R^2=0.81, R^2=0.688, R^2=0.568). The equations for sands (Equations 5-7, 5-8, and 5-9) have the lowest correlation coefficients (R^2=0.160, R^2=0.226, R^2=0.304).

For Clays:

$$\log K_8 = 17.662 - 0.2647W_{opt} - 0.4439W + 2.6732\left(\frac{\gamma_d}{\gamma_{d\max}}\right) + 0.1320\%silt + 0.6422LL - 0.3742PI - 0.1963\gamma_{d\max} - 0.00087P_{40}S \qquad R^2 = 1.0 \qquad \text{(Eq. 5-2)}$$

$$K_9 = 0, K_{10} = 1$$

$$K_{11} = 3.3673 - 0.01464W_{opt} - 1.7371\left(\frac{W}{W_{opt}}\right) - 0.1264\left(\frac{\gamma_d}{\gamma_{d\max}}\right) \qquad R^2 = 0.81 \qquad \text{(Eq. 5-3)}$$

For Silts:

$$\log K_8 = 1.9823 + 0.01394W_{opt} - 0.5934\left(\frac{W}{W_{opt}}\right) + 0.1500\left(\frac{\gamma_d}{\gamma_{d\max}}\right) + 0.00831\gamma_{d\max} + 0.00034\left(\frac{\gamma_{d\max}^{2}}{P_{40}}\right) \qquad R^2 = 0.810 \qquad \text{(Eq. 5-4)}$$

$$K_9 = 6.4676 - 0.0861W_{opt} - 0.5458\left(\frac{\gamma_d}{\gamma_{d\max}}\right) + 0.00800S - 0.04226\gamma_{d\max} \qquad R^2 = 0.688 \qquad \text{(Eq. 5-5)}$$

$$K_{10} = 1$$

$$K_{11} = 5.7391 + 0.07929W - 1.1778\left(\frac{W}{W_{opt}}\right) + 0.008037\%silt + 0.04549\gamma_{d\max} \qquad R^2 = 0.568 \qquad \text{(Eq. 5-6)}$$

For Sands:

$$\log K_8 = 2.7602 - 0.00702W - 0.0706\left(\frac{W}{W_{opt}}\right) + 0.05750\left(\frac{\gamma_d}{\gamma_{d\max}}\right) + 0.000279\gamma_{d\max} \qquad R^2 = 0.160 \qquad \text{(Eq. 5-7)}$$

$$K_9 = 0.7386 - 0.0149W_{opt} + 0.3916\left(\frac{\gamma_d}{\gamma_{d\max}}\right) - 0.00604S - 0.00157\gamma_{d\max} \qquad R^2 = 0.226 \qquad \text{(Eq. 5-8)}$$

$$K_{10} = 1$$

$$K_{11} = -0.04978 - 0.0092W + 0.008377\left(\frac{W}{W_{opt}}\right) - 0.0052\%silt + 0.000487\gamma_{d\max} \qquad R^2 = 0.304 \qquad \text{(Eq. 5-9)}$$

5.3 Correlation with Cohesion, Internal Friction Angle, and Elasticity

Correlating resilient modulus with soil specimen cohesion (*C*), internal friction angle (ϕ), and elasticity have been examined. Laguros et al. (1993) used data from six type 1 soils collected from Oklahoma to develop correlations for predicting resilient modulus. Laboratory resilient modulus tests were done using AASHTO T294-92. Equation 5-10 incorporates cohesion, internal friction angle, and the effect of changes in bulk stress.

$$M_R(psi, C, \phi) = 2860.94 + 275C + 128\sigma_1 \tan\phi + 118\theta \qquad \text{(Eq. 5-10)}$$

Equation 5-10 predicted the resilient moduli of the six type 1 soils with moderate accuracy. The range of the correlation coefficient for the six specimens was 0.5374 to 0.8345 with an average of 0.7336. The predicted values of resilient modulus using Equation 5-10 were compared with actual test data. For three of the soils Equation 5-10 predicted values in the upper range of the test results. For the remaining three soils Equation 5-10 predicted values in the lower range of the test results. Figures 5.7 and 5.8 show examples of these comparisons.

Laguros et al. (1993) found that by increasing the confining pressure the initial tangent modulus of elasticity increased (*E*). From this correlation an attempt was made to find a correlation between resilient modulus and modulus of elasticity. Equation 5-11 incorporates the effects of confining pressure and bulk stress.

$$M_R(psi,\sigma_3,\theta) = (18.28 + 0.4917\sigma_3)0.4098 + 150.7\theta \qquad \text{(Eq. 5-11)}$$

No correlation coefficient was given for the fitting of Equation 5-11. Two soils had predicted resilient moduli in the upper range, two soils in the lower range, and two soils average range of the test data. Equation 5-11 predicted resilient moduli lower than Equation 5-10 for five of the six soils. Therefore Laguros et al. (1993) found that a better estimate of resilient modulus for design may be determined using correlations to cohesion and internal angle of friction when compared to correlations to initial tangent elastic modulus. Figures 5.9, 5.10, and 5.11 show examples of the comparisons.

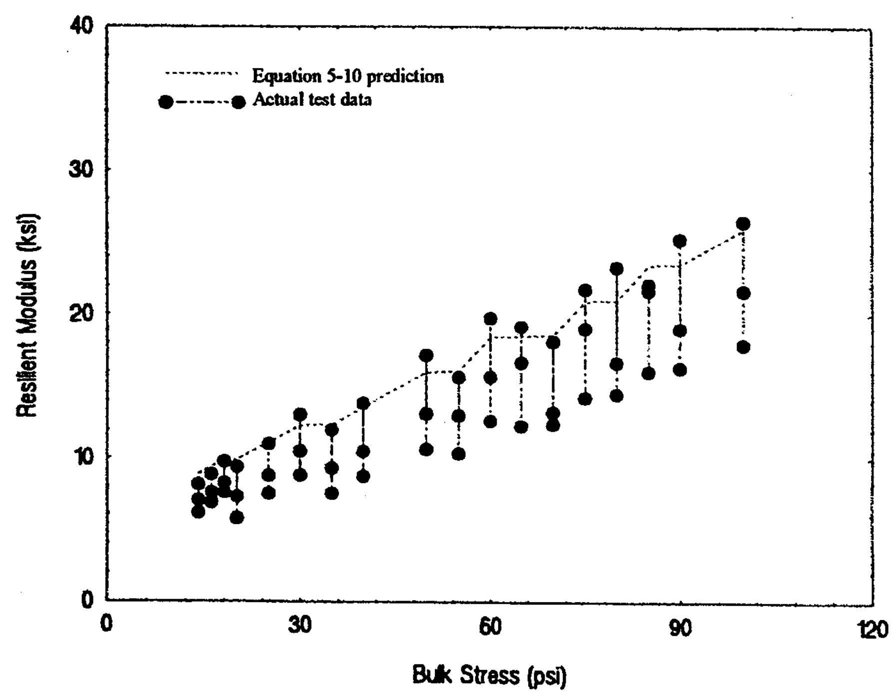


FIGURE 5.7 - Comparison of upper prediction range of Equation 5-10 and actual test data (Laguros et al., 1993).

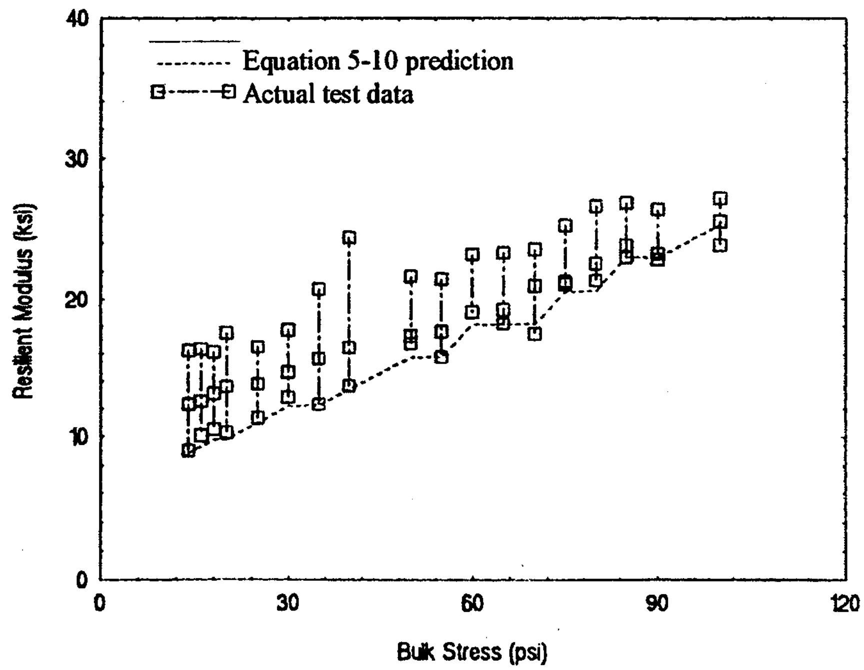


FIGURE 5.8 - Comparison of lower prediction range of Equation 5-10 and actual test data (Laguros et al., 1993).

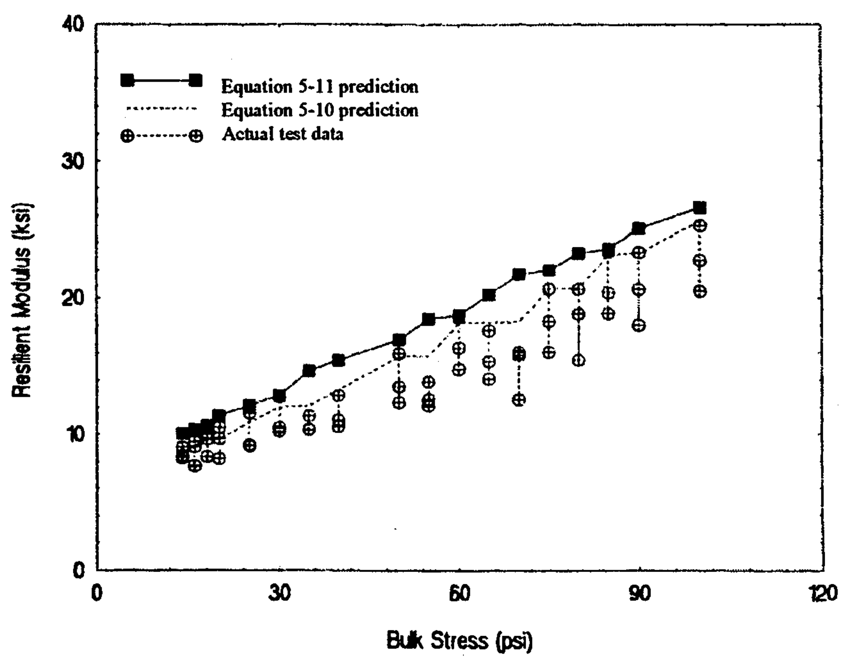


FIGURE 5.9 - Upper range predictions of resilient modulus using Equations 5-11, 5-10 and actual test data of an A-1-b specimen (Laguros et al., 1993).

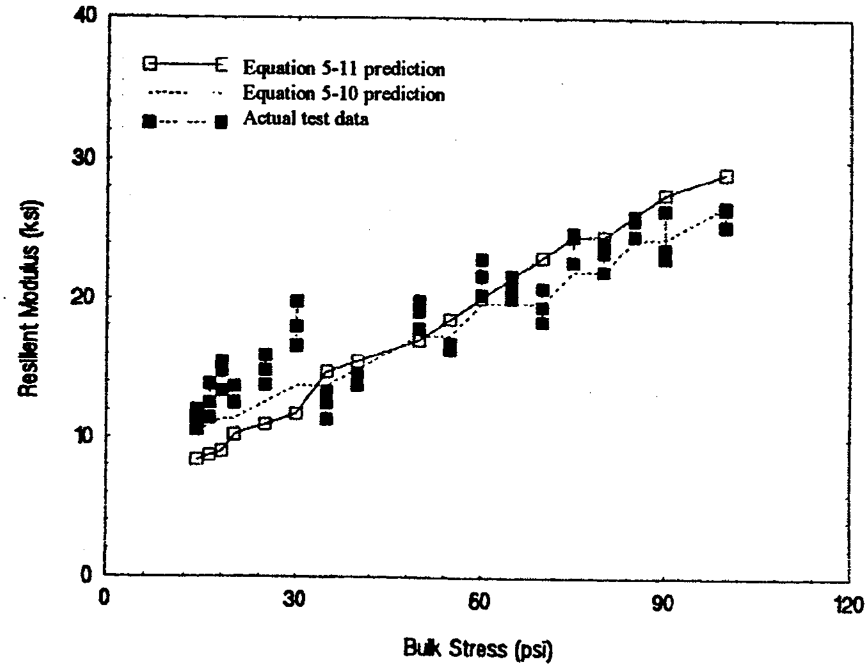


FIGURE 5.10 - Average range predictions of resilient modulus using Equations 5-11, 5-10 and actual test data of an A-1-b specimen (Laguros et al., 1993).

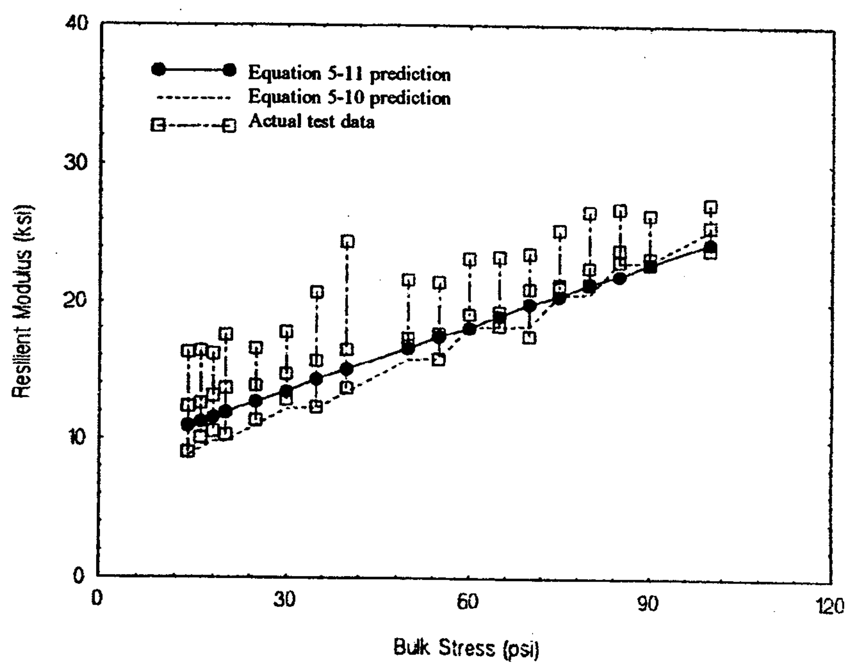


FIGURE 5.11 - Lower range predictions of resilient modulus using Equations 5-11, 5-10 and actual test data of an A-1-b specimen (Laguros et al., 1993).

5.4 Correlation with California Bearing Ratio

CBR is a common test for determining the strength characteristics of a soil layer. The CBR test prescribes static loading of a confined specimen which limits the swelling and bending to a single direction. In contrast the resilient modulus specimen is dynamically loaded for multiple repetitions and is allowed to bend and swell axially as well as radially. Furthermore the CBR loading piston does not cover the entire specimen and could come in contact with a large aggregate particle. Since resilient modulus is a measure of stiffness, it is generally believed that correlations between CBR and resilient moduli are inaccurate. As discussed in Chapter 2, resilient moduli of soils are highly stress dependent. Yet in CBR tests the stress conditions are highly variable at different locations in the specimen and they do not match field conditions.

A commonly referenced correlation as suggested by the AASHTO Guide (AASHTO, 1993b) is shown below in Equation 5-12. The equation is recommended for fine grained soils with a soaked CBR of ten or less.

$$M_R(CBR) = 1.5CBR \text{ ksi} \qquad \text{(Eq. 5-12)}$$

Drumm et al. (1993) compared the resilient moduli at a deviator stress of 28 kPa (4 psi) and confining pressure of 41kPa (6 psi) to the value determined using Equation 5-12 and the values of CBR at 2.54-mm (0.10-in.) and 5.08-mm (0.20-in.) penetrations for eleven type 2 soils from Tennessee. Figure 5.12 shows a wide range of variations including some values outside the range of 3 to 0.75 times CBR.

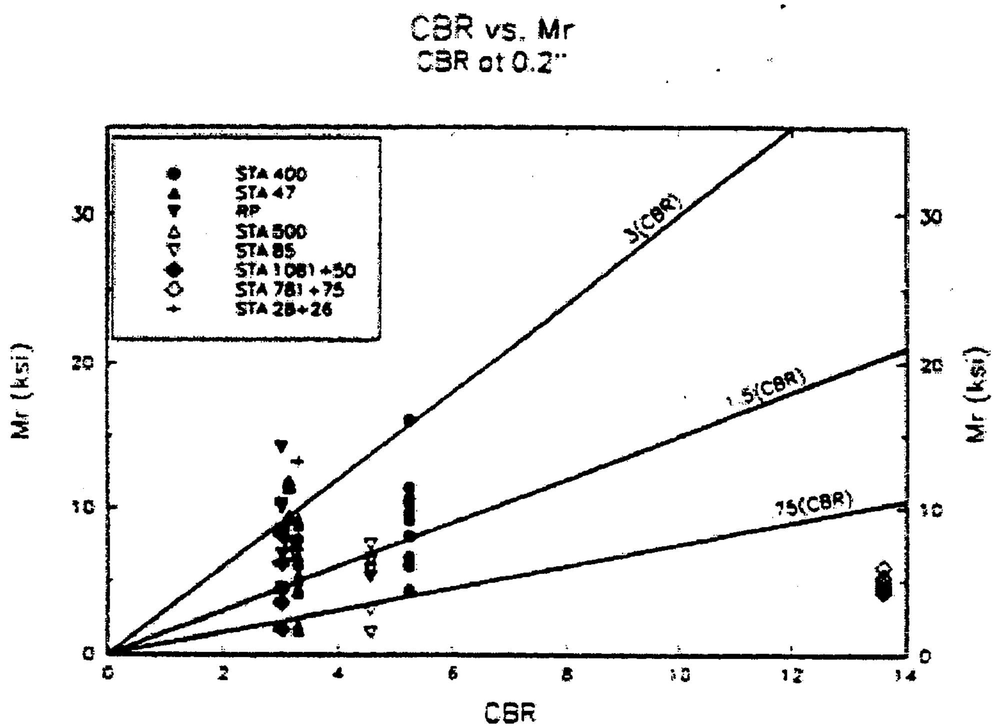


FIGURE 5.12 - Resilient moduli of eleven type 2 soils from Tennessee compared to correlations with CBR (Drumm et al., 1993).

The AASHTO Guide also gives a correlation between resilient modulus and CBR as a function of bulk stress. Laguros et al. (1993) compared the correlation between CBR and resilient modulus based on six type 1 soils to those given by the AASHTO Guide. Table 5.3 shows the results of this comparison. The factor relating CBR to resilient modulus determined by Laguros et al. (1993) was much smaller than given by AASHTO.

TABLE 5.3 - Correlation between resilient modulus and CBR as a function of bulk stress as recommended by the AASHTO Guide and as determined by Laguros et al. (AASHTO, 1993b; Laguros et al., 1993).

Bulk Stress, θ (psi)	M_R(*CBR*, in ksi) AASHTO Guide	M_R(*CBR*, in ksi) Laguros et al.	Laguros et al. vs. AASHTO Guide
100	0.74 *CBR*	0.193 *CBR*	74% lower
30	0.44 *CBR*	0.096 *CBR*	78% lower
20	0.34 *CBR*	0.082 *CBR*	76% lower
14	0.288 *CBR* *	0.074 *CBR*	74% lower
10	0.25 *CBR*	n/a	n/a

* Value computed assuming a linear relationship between bulk stresses of 30 and 10-psi.

5.5 Summary

Correlation between resilient modulus and soil index properties, strength parameters, and CBR has been examined. If enough soil index data is available, fairly accurate equations for predicting resilient modulus can be developed using statistical regression analysis.

Drumm et al. (1993) performed a linear regression using the resilient moduli and soil index properties of multiple specimens of eight type 2 soils. The multiple specimens were prepared at different levels of dry density and water contents. The resulting regression is given as Equation 5-1. The variables are defined in Table 5-1. The database was developed from soil index properties and resilient moduli from eight soils. Equation 5-1 has eight independent variables therefore the database should be comprised of at least nine different soils. This brings the accuracy of the correlation into question.

$$\begin{aligned}\log M_R(psi) = {} & 46.93 + 0.0188\sigma_d + 0.0333\Delta\gamma_{d\max} - 0.1143LI + 0.4680S \\ & + 0.0085class^2 - 0.0033\Delta W_{opt}^{\ 2} - 0.0012\sigma_3^{\ 2} + 0.0001PL^2 \qquad \text{(Eq. 5-1)} \\ & + 0.0278LI^2 - 0.0017S^2 - 38.44\log S - 0.2222\log\sigma_d\end{aligned}$$

Correlations between soil index data in the Long Term Pavement Performance database and the K_n constants from the stress dependent models in Chapter 4 were developed. Von Quintus and Killingsworth (1998) used the index properties from clays, silts, and sands to develop Equations 5-2 through 5-9 to predict the values of the K_n constants of Equation 4-4. Good correlations were developed for clays however the correlations for silts and sands were moderate to poor. Correlations for gravels and bases were not examined due to insufficient amounts of data.

For type 1 soils, correlations based on cohesion (C), internal angle of friction (ϕ), bulk stress (θ), and confining pressure (σ_3) gave better estimates of resilient modulus than correlations based on initial tangent modulus. Laguros et al. (1993) developed Equations 5-10 and 5-11 from six type 1 soils from Oklahoma. The equations predicted the actual test data moderately well. The average correlation coefficient for the six soils using Equation 5-10 was 0.7336. However this average was from a wide range of values (0.5374 to 0.8345) for the six soils. Equation 5-11 is based only on stress conditions. No correlation coefficient was given for the Equation 5-11 predictions.

$$M_R(C,\phi) = 2860.94 + 275C + 128\sigma_1 \tan\phi + 118\theta \quad \text{psi} \qquad \text{(Eq. 5-10)}$$

$$M_R(\sigma_3,\theta) = (18.28 + 0.4917\sigma_3)0.4098 + 150.7\theta \quad \text{psi} \qquad \text{(Eq. 5-11)}$$

Loading conditions in the California Bearing Ratio (CBR) test are considerably different than the resilient modulus test. The CBR test uses a small diameter piston to statically load a larger diameter specimen. The specimen is confined by the mold therefore it is not allowed to deform radially. The resilient modulus test dynamically and repetitiously loads the specimen over its entire cross section and allows deformation in the axial and radial directions. Furthermore the haversine shaped loading pulse prescribed for resilient modulus testing better represents the stresses caused by moving vehicles. Since CBR is a measure of strength rather than stiffness correlations to resilient moduli are generally inaccurate.

Drumm et al. (1993) compared the resilient moduli of eleven type 2 soils from Tennessee to a commonly referenced CBR-resilient modulus relationship (Equation 5-12). The range of factors for the resilient moduli predicted by Equation 5-12 was generally between 0.75 and 3.0 although some results fell outside this range.

$$M_R(CBR) = 1.5CBR \text{ ksi} \qquad \text{(Eq. 5-12)}$$

The AASHTO Guide gives relationships between CBR and resilient moduli as functions of bulk stress (Table 5.3). Laguros et al. (1993) used these relationships to compare their predictions to actual resilient moduli of six type 1 soils from Oklahoma. For bulk stresses between 689 kPa (100 psi) and 69 kPa (10 psi) Laguros et al. (1993) recommended factors between 0.193 and 0.074 times the CBR. These factors were 74% to 78% lower than those recommended by AASHTO.

CHAPTER 6

DETERMINATION OF RESILIENT MODULUS FOR MAINE SOILS

Available data on the resilient modulus of Maine subgrade and subbase soils was gathered. The LTPP database provides only a limited amount of soil index data however, it does provide a considerable amount of FWD data that can be used to determine resilient modulus. Moreover, laboratory resilient modulus tests were conducted by Law Engineering in 1992 using SHRP Protocol P46. Six type 1 soils from Bethel, Damariscotta, North Freeport, and South Freeport were tested. Notably absent from this list are weaker subbase aggregates from Northern Maine. Eleven type 2 soils from Brunswick, Damariscotta, North Freeport, South Freeport, Topsham, and Wilton were also tested. Two of the type 2 soils were tested using the cyclic stress sequencing normally associated with type 1 soils because they were subbase soils. Cyclic stress sequencing is more representative of the higher stress states seen in subbase soil layers than in subgrade soil layers. Therefore SHRP Protocol P46 permits the testing of type 2 soils used for the subbase layer. A possible approach to determining the resilient modulus of a soil sample is to pair its index properties such as grain size distribution, water content, and dry density with a soil whose resilient modulus is already known. For this reason, available data in presented in Appendix F.

Despite the limited data, three methods of estimating the resilient modulus of Maine soils without further laboratory tests were examined. The first method was to compute resilient modulus from FWD test data ($M_R(FWD)$) using MODCOMP 4 version

H and MODULUS 5.1. The relationship between backcalculated and laboratory resilient modulus was investigated. The second method employed the stress dependent constitutive equations presented in Chapter 4. The K_n coefficients from the resilient modulus test data have been transformed to equivalent coefficients for several equations in Chapter 4. The final approach correlates soil index data and estimated stress state to resilient modulus using linear regression analysis.

6.1 Resilient Modulus using FWD Data

The relationship between laboratory resilient modulus and backcalculated resilient modulus has been examined. The LTPP database provided FWD test data done in June 1989, three years before the laboratory tests were performed. FWD test data is available for four of the seven Maine sites. These road test sites are located near Brunswick, Damariscotta, North Freeport, and South Freeport. The length of each project was approximately 152.4 m (500 ft). Test data was taken at 7.62-m (25-ft) station intervals.

The FWD device was configured per SHRP specifications. Deflection sensors were placed 0, 203, 305, 457, 610, 914, and 1524 mm (0, 8, 12, 18, 24, 36, and 60 in.) from the 150-mm (5.91-in.) radius loading plate. For the Brunswick, Damariscotta, and North Freeport sites, average loads of 390, 570, 760, and 1010 kPa (8145, 11900, 15900, 21100 lb) were used. Each load was dropped four times for a total of sixteen drops at each location. Average loads of 570, 760, and 1010 kPa (1190, 15900, and 21100 lb) were used for the South Freeport site. Each load for the South Freeport site was dropped four times for a total of twelve drops at each location. MODCOMP 4 allowed English and SI units for the load and deflection data. MODCOMP 4 also provided the option to input the load as the plate pressure or the weight of the hammer. The load input for MODULUS 5.1 was limited to the weight of the hammer in pounds and the deflection in mils. The load and deflection data is listed in Appendix E.

The exact locations of the soil samples are unknown therefore the average deflection bowl for every 30.5-m (100-ft) at each of the load levels was used for resilient modulus backcalculation. Twenty load/deflection bowls were used for the Damariscotta, North Freeport, and South Freeport projects. Fifteen load/deflection bowls were used for the Brunswick project.

The backcalculation software required layer thicknesses and values for Poisson's ratio. Values for Poisson's ratios for MODCOMP 4 and MODULUS 5.1 were taken from the MODCOMP 4 help menus (0.35 for asphalt, 0.15 for concrete, 0.35 for base soils, and 0.40 for subgrade and subbase soils). Layer thicknesses for the surface, base, subbase soil layers were taken from the LTPP database. The maximum number of layers for MODCOMP 4 was seven whereas MODULUS 5.1 was limited to four. Therefore some similar soil layers needed to be combined for MODULUS 5.1. The subgrade soil layer thickness and the depth to the nearest hard layer was not reported in the LTPP database. MODCOMP 4 and MODULUS 5.1 automatically estimated the thickness of the subgrade soil layer based on the deflection basin profile. The backcalculated moduli were compared to the laboratory resilient moduli provided by Law Engineering (1992).

The laboratory values are the representative resilient moduli for the soil as called out on the laboratory data sheet however they may not represent the actual in-situ stress state. Representative resilient moduli for type 1 soils are based on a cyclic stress of 2.2 kPa (15 psi) and confining pressure of 2.2 kPa (15 psi). The representative resilient moduli for type 2 soils is based on a deviator stress of 0.87 kPa (6 psi) and confining pressure of 0.58 kPa (4 psi). In resilient modulus tests, the at rest earth pressure coefficient (K_O) can be defined as:

$$K_O = \frac{\sigma_H}{\sigma_V} = \frac{\sigma_3}{\sigma_3 + \sigma_d} \qquad \text{(Eq. 6-1)}$$

where: σ_H = horizontal stress

σ_V = vertical stress

There is no deviator stress on the specimen when it is at rest inside the triaxial chamber prior to preconditioning. Therefore the vertical and horizontal stresses are equal and K_O for the representative resilient moduli for both soil types is 1.0. In contrast, the in-situ values for K_O are approximately 0.50.

Initially the backcalculated resilient moduli from MODCOMP 4 and MODULUS 5.1 did not correlated well with the laboratory resilient moduli (Figures 6.1 and 6.2). However removing the backcalculated resilient moduli in excess of 2000 MPa from the MODULUS 5.1 analysis significantly improved the correlation (R^2=0.76) (Figure 6.3). Furthermore the MODULUS 5.1 resilient moduli were within the same order of magnitude as the initial moduli from MODCOMP 4. There were no obvious outliers from the initial MODCOMP 4 analysis to remove therfore this step was not done with the initial MODCOMP 4 data. The inverse slope ($M_R(LAB)$ / $M_R(FWD)$) of the MODULUS 5.1 best fit line (0.0501) was close to the mean ratio (Mean = 0.57, Standard Deviation = 0.67) as reported in Table 3.4 (Dalieden et al., 1994).

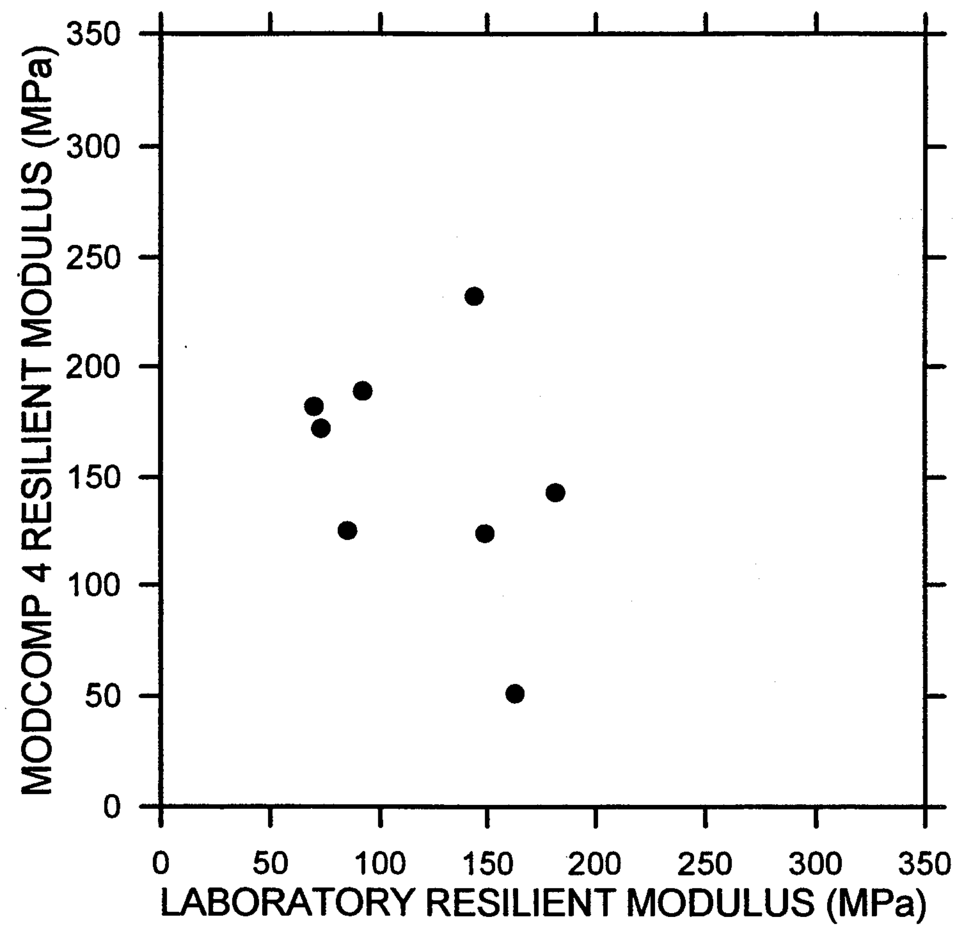


FIGURE 6.1 - Laboratory resilient moduli versus MODCOMP 4 resilient moduli with depth to hard layer as estimated by MODCOMP 4.

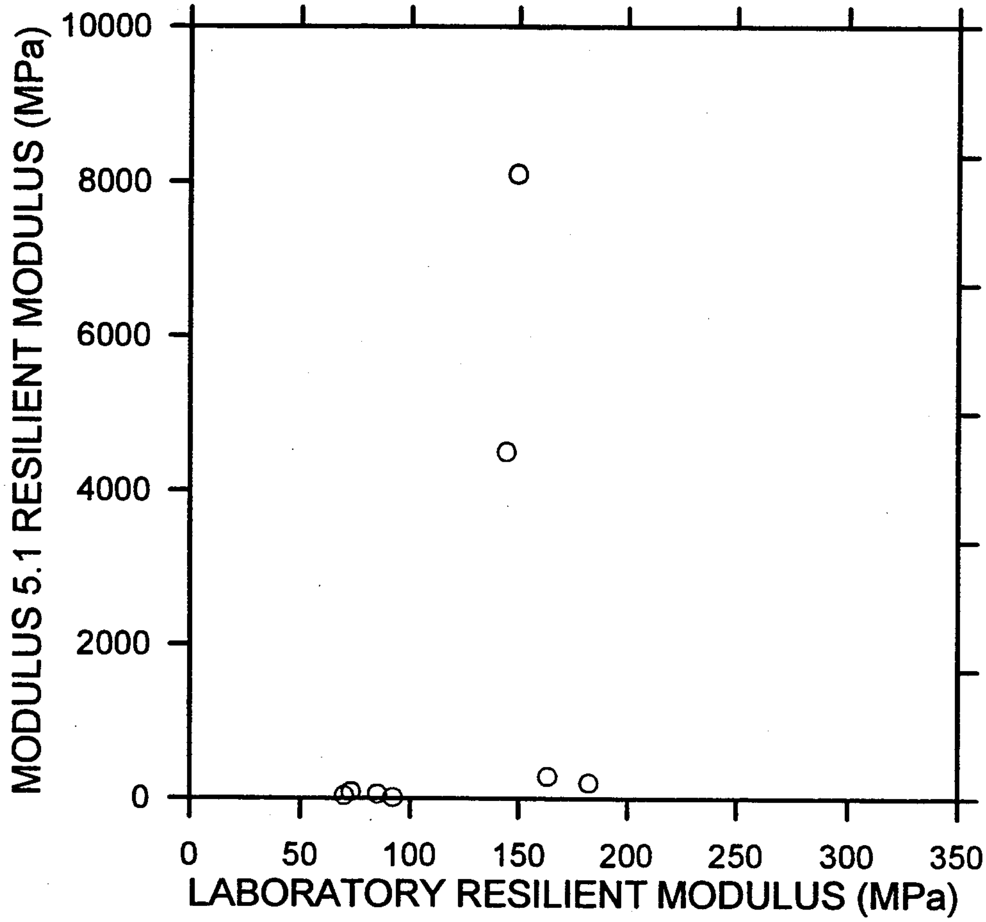


FIGURE 6.2 - Laboratory resilient moduli versus MODULUS 5.1 resilient moduli with depth to hard layer as estimated by MODULUS 5.1.

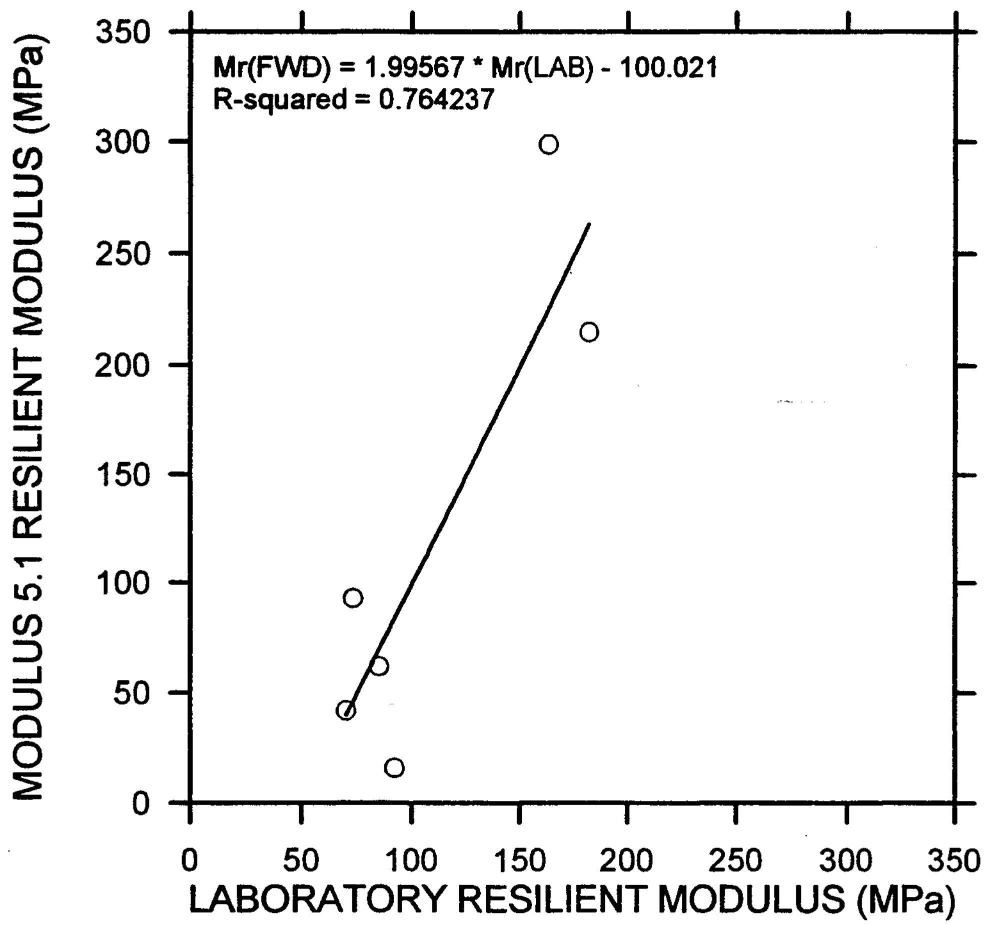


FIGURE 6.3 - Laboratory resilient moduli versus MODULUS 5.1 resilient moduli less than 2000 MPa with depth to hard layer as estimated by MODULUS 5.1.

Backcalculation depends on the values chosen for the depth to hard layer and Poisson's ratio. To examine the effect of hard layer depth, the depths to refusal were used as depths to hard layers for the South Freeport and Brunswick sites. Two soils were provided from each site. The soils from the South Freeport site were type 2 soils (a subbase soil and a subgrade soil). The soils from the Brunswick site were also type 2 (a base soil and a subbase soil). The refusal depth was determined by driving a standard split spoon sampler to refusal. The LTPP database provided the depth to refusal for these sites but did not provide them for the Damariscotta or North Freeport sites. These depths were subtracted from the sum of the layer thicknesses to obtain the subgrade soil layer thickness and the depth to hard layer. The LTPP refusal depths and the computer program estimated depths to hard layers are shown in Table 6.1. The estimated depths to hard layers do not compare well with the refusal depths. The estimate by MODCOMP 4 for the South Freeport site is listed as "infinite" because the hard layer is estimated at a depth deep enough such that its effect on resilient modulus is negligible.

TABLE 6.1 - Refusal depths from the LTPP database and estimated depths to hard layer from MODCOMP 4 and MODULUS 5.1.

Depth	Refusal (m)	Average Estimated to Hard Layer (m)	
Source	LTPP	MODCOMP 4	MODULUS 5.1
Brunswick	1.52	7.8	3.0
South Freeport	1.37	infinite	6.9

Changes in the depth to hard layer caused wide resilient modulus variations in the four soils. The resilient moduli were backcalculated using the refusal depths in Table 6.1 and compared to the laboratory resilient moduli. The results are shown in Table 6.2. The refusal depth moduli did not compare favorably to the average estimated depth to hard layer moduli. This suggests that the refusal depth should not be used for the depth to hard layer in resilient modulus backcalculation.

TABLE 6.2 - Laboratory, MODCOMP 4, and MODULUS 5.1 resilient moduli refusal and estimated depths to hard layer (Laboratory resilient moduli from Law Engineering, 1992)

Source		MODCOMP 4 $M_R(FWD)$ (MPa)		MODULUS 5.1 $M_R(FWD)$ (MPa)	
Site	Layer	Based on Refusal Depth	Based on Ave. Est. Depth	Based on Refusal Depth	Bsaed on Ave. Est. Depth
Brunswick	Base	2402	232	11422	4505
Brunswick	Subbase	26	182	23	42
South Freeport	Subbase	380	51	68	299
South Freeport	Subgrade	12	172	20322	93

Changes in the value of Poisson's ratio of an asphalt or soil layer may or may not have an effect on backcalculated resilient modulus. The resilient moduli of a base soil and a subgrade soil from the Damariscotta site were backcalulated for various Poisson's ratio of the asphalt layer and the subbase layer. The depth to hard layer was estimated by the programs. Poisson's ratios for the asphalt layer were 0.15, 0.35, and 0.45. Poisson's ratio values for the subbase layer were 0.25, 0.35, and 0.45. Increasing the asphalt layer Poisson's ratio caused a decreasing trend in backcalculated resilient moduli of the base and subgrade soils. MODULUS 5.1 backcalculated a wider range of resilient moduli than MODCOMP 4. Increasing the subbase layer Poisson's ratio caused an increase in backcalculated resilient modulus of the base soil. MODULUS 5.1 backcalculated a wider range of resilient moduli for the base soil than MODCOMP 4. However with increasing subbase Poisson's ratio, MODCOMP 4 backcalculated resilient moduli decreasing in value. MODULUS 5.1 backcalculated subgrade resilient moduli increasing in value. A narrower range of resilient moduli was backcalculated by MODULUS 5.1 than MODCOMP 4. All backcalculated resilient moduli compared poorly with the laboratory resilient moduli provided by Law Engineering (1992). Therefore accurate Poisson's ratios should be used for the backcalulation process. The results for this analysis are shown in Table 6.3.

TABLE 6.3 - Variations in backcalculated resilient modulus due to changes in Poisson's ratio for a base soil and a subbase soil from Damariscotta.

Program		MODCOMP 4 $M_R(FWD)$ (MPa)		MODULUS 5.1 $M_R(FWD)$ (MPa)	
Layer	Poisson's Ratio	Base	Subgrade	Base	Subgrade
Asphalt	0.15	300	71	204	58
Asphalt	0.35	215	62	215	62
Asphalt	0.45	185	56	243	62
Subbase	0.25	146	126	139	127
Subbase	0.35	143	125	143	125
Subbase	0.45	136	124	145	122

NOTE: M_R base soil = 182 MPa, M_R subgrade soil = 85 MPa (Law Engineering, 1992)

6.2 K_n Constants for Stress Dependent Resilient Moduli Equations

The laboratory resilient modulus test data from Law Engineering has been used to determine K_n constants for many of the constitutive equations listed in Chapter 4. This will allow the pavement designer to chose the constitutive equation best suited to a particular site. A set of K_n constants was determined for several individual Maine soil samples.

6.2.1 K_n Constants for Type 1 Soils

The stress dependent equations for type 1 soils are Equations 4-1, 4-2, 4-3, and 4-4. Six type 1 soils from southern and western Maine were use in the analysis. Law Engineering (1992) performed resilient modulus tests and determined K_8, K_9, and K_{11} (assuming K_{10} equal to 1.0 for Equation 4-4). The test data was provided in tabular and graphical form. Because Equation 4-4 is based on cyclic stress (σ_{cyclic}) and Equations 4-1, 4-2, and 4-3 are based on bulk stress (θ) the cyclic stress needed to be transformed to an equivalent bulk stress. Assuming that cyclic stress is equal to deviator stress (σ_d)* the relationship between cyclic stress and bulk stress is described below:

* This assumption is reasonable since σ_{cyclic} is taken by various researchers as either $0.9\sigma_d$ or equal to σ_d.

$$\sigma_{cyclic} = \sigma_d = \sigma_1 - \sigma_3 \quad \text{(Eq. 6-2)}$$

$$\sigma_1 = \sigma_{cyclic} + \sigma_3 \quad \text{(Eq. 6-3)}$$

$$\theta = \sigma_1 + 2\sigma_3 \quad \text{(Eq. 6-4)}$$

$$\theta = \sigma_{cyclic} + 3\sigma_3 \quad \text{(Eq. 6-5)}$$

The following procedure was used to obtain values for K_1 and K_2 of Equation 4-1 for each soil using Equation 4-4:

1. Compute the bulk stresses corresponding to the confining pressures and deviator stresses (assuming deviator stress equal to cyclic stress) given in the laboratory resilient modulus database (Law Engineering 1992) using Equation 6-5.

2. Plot the resilient moduli from the laboratory data (Equation 4-4) as a function of the computed bulk stress.

3. Determine the best fit straight line on a log-log plot of resilient modulus vs. computed bulk stress.

An example of the results is shown in Figure 6.4. The best fit straight line correlation coefficients and values for K_1 and K_2 are listed in Table 6.4. The correlations between the best fit straight line and the resilient moduli from Equation 4-4 (step 2) were very good (R^2 ranging between 0.95 and 0.99).

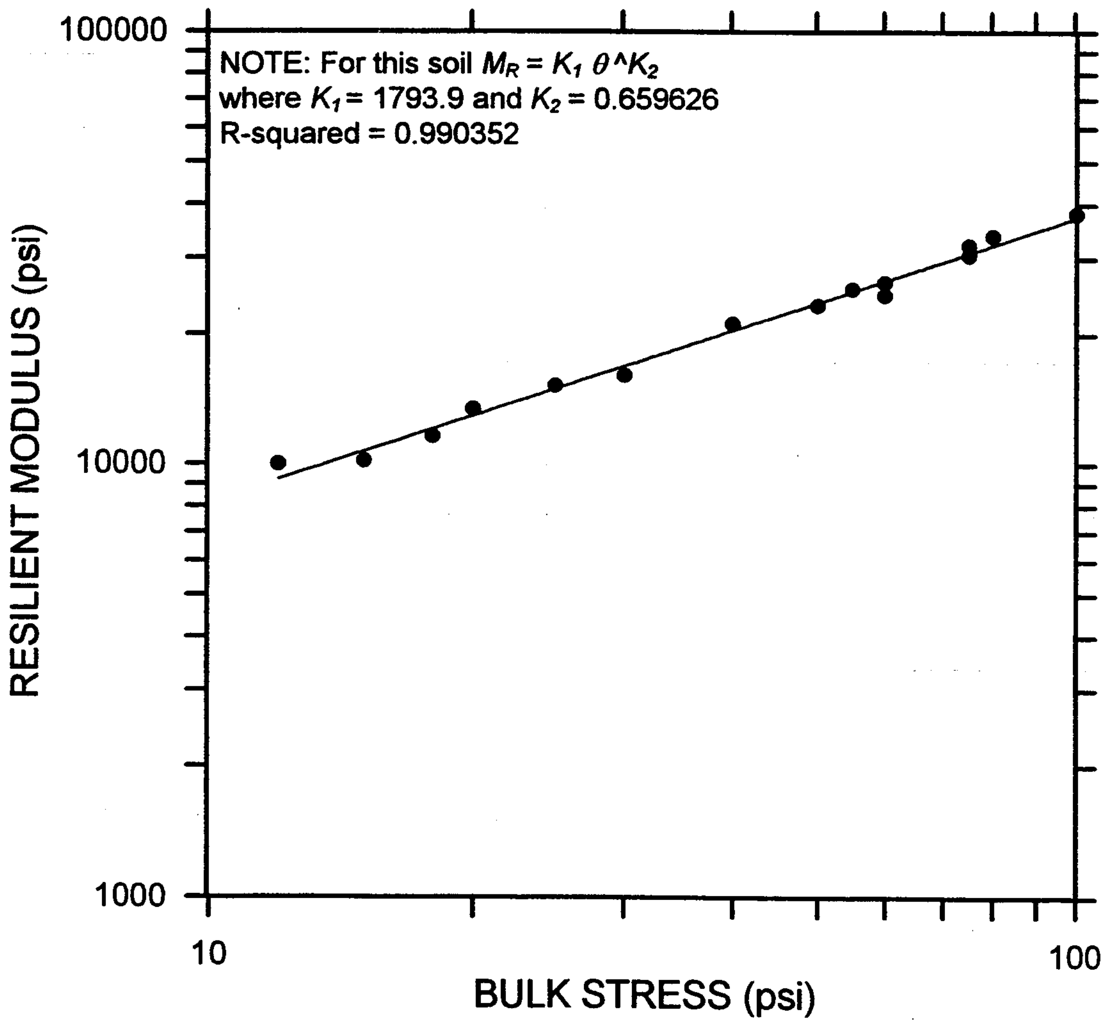


FIGURE 6.4 - Resilient modulus as a function of computed bulk stress for a type 1 A-1-a soil from Damariscotta used to determine the K_n constants for Equation 4-1.

TABLE 6.4 - Values of K_1 and K_2 for use with Equation 4-1 of six type 1 soils from Maine.

SHRP ID	Location	Classification	K_1	K_2	R^2
231009 BABG	Damariscotta	A-1-a	1794	0.660	0.99
231009 TPBG55	Damariscotta	A-1-a	675	0.862	0.99
231012 TPBG56	South Freeport	A-1-a	1151	0.724	0.95
231028 BA1BS01	Bethel	A-1-a	1405	0.790	0.95
237023 BABG-1	North Freeport	A-1-a	1112	0.738	0.97
237023 BABG-2	North Freeport	A-1-a	2001	0.465	0.98

NOTE: $M_R = K_1(\theta)^{K_2}$ (Eq. 4-1), K_n constants are dependent on units of psi; θ and M_R must be in units of psi.

Equation 4-2 is similar to Equation 4-1 except that the constants are rendered unitless due to the addition of the atmospheric pressure term (P_a). K_3 and K_4 for Equation 4-2 were obtained from K_1 and K_2 the following procedure:

1. Compute bulk stresses using Equation 6-4 and confining pressures of 1 and 5 psi and deviator stresses of 2, 5, 7.5, 10, and 15 psi (as recommended by Santha, 1994).

2. Use Equation 4-1 and its corresponding values of K_1 and K_2 to compute resilient moduli for the range of bulk stresses.

3. Divide the bulk stresses and the resilient moduli by atmospheric pressure (P_a) (14.7 psi).

4. On log-log scales, plot resilient modulus divided by P_a (M_R/P_a) as a function of bulk stress divided by P_a (θ / P_a).

5. Determine the best fit straight line on the log-log plot of M_R/P_a vs. θ / P_a.

An example of the results is shown in Figure 6.5. Fit 2 is the best fit straight line in log-log form. The coefficients in the equation are K_3 and K_4. The best fit straight line correlation coefficients and values for K_3 and K_4 are listed in Table 6.5. The correlations between the best fit straight line and the resilient moduli from Equation 4-1 (step 2) were exact (R^2=1.0). This was expected since Equation 4-1 is linear on a log-log plot of resilient modulus versus bulk stress.

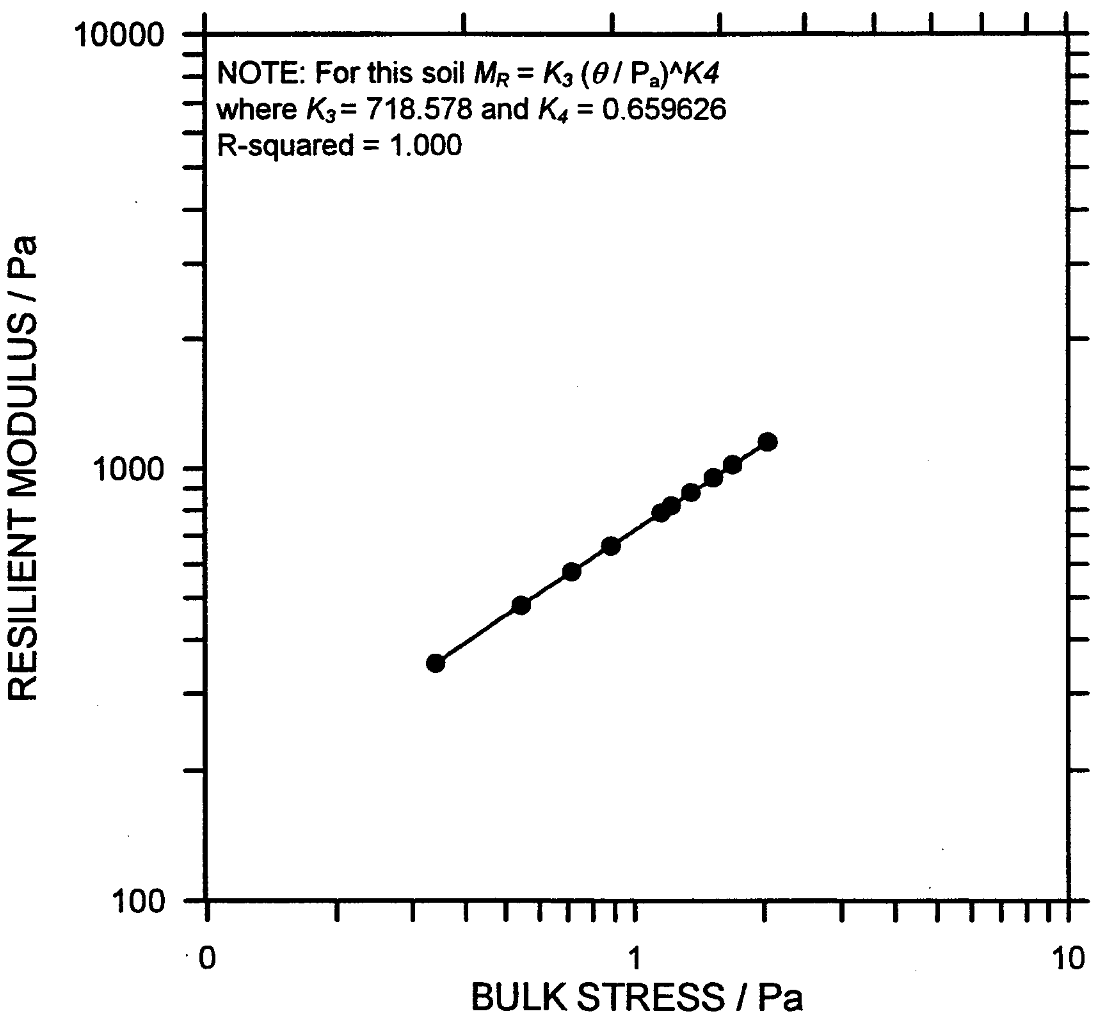


FIGURE 6.5 - Resilient modulus divided by atmospheric pressure as a function of changing equivalent bulk stress divided by atmospheric pressure for a type 1 A-1-a soil from Damariscotta used to determine the K_n constants for Equation 4-2 .

TABLE 6.5 - Values of K_3 and K_4 for use with Equations 4-2 of six type 1 soils from Maine.

SHRP ID	Location	Classification	K_3	K_4	R^2
231009 BABG	Damariscotta	A-1-a	719	0.660	1.0
231009 TPBG56	Damariscotta	A-1-a	466	0.862	1.0
231012 TPBG56	South Freeport	A-1-a	548	0.724	1.0
231028 BA1BS01	Bethel	A-1-a	798	0.790	1.0
237023 BABG-1	North Freeport	A-1-a	550	0.738	1.0
237023 BABG-2	North Freeport	A-1-a	771	0.465	1.0

NOTE: $M_R = K_3 P_a \left(\frac{\theta}{P_a}\right)^{K_4}$ (Eq. 4-2), K_n constants are independent of units

Unlike Equations 4-1 and 4-2, Equation 4-3 has two independent variables (bulk stress and deviator stress). To obtain the K_n constants of the soils for Equation 4-3, Equation 4-3 was first transformed into its logarithmic form (Equation 6-6). A linear regression analysis using SYSTAT 7.01 was done with log M_R as the dependent variable and log θ / P_a and log σ_d / P_a as the independent variables. The results of the regression are two coefficients and a constant. The coefficient of log (θ / P_a) is K_6. The coefficient of log (σ_d / P_a) is K_7. The constant term (N) is equal to log $(K_5 P_a)$. K_5 is obtained by substituting N into Equation 6-7. The coefficients for the six type 1 soils from Maine are listed in Table 6.6. The correlations for the regressions were very good ($0.772 < R^2 < 0.997$).

$$\log M_R = N + K_6 \log\left(\frac{\theta}{P_a}\right) + K_7 \log\left(\frac{\sigma_d}{P_a}\right) \qquad \text{(Eq. 6-6)}$$

where: $N = \quad \log(K_5 P_a)$

$$K_5 = 10^{(N - \log P_a)} \qquad \text{(Eq. 6-7)}$$

For Equation 4-4, the actual resilient modulus test data from Law Engineering provided the coefficients K_8, K_9, and K_{11} for each soil. The best fit line correlation coefficients are given with the K_n coefficients in Table 6.7.

TABLE 6.6 - Values of K_5, K_6, and K_7 for use with Equations 4-3 of six type 1 soils from Maine.

SHRP ID	Location	Classification	K_5	K_6	K_7	R^2
231009 BABG	Damariscotta	A-1-a	1101	0.337	0.185	0.772
231009 TPBG56	Damariscotta	A-1-a	449	0.931	-0.074	0.997
231012 TPBG56	South Freeport	A-1-a	548	0.724	-0.001	0.974
231028 BA1BS01	Bethel	A-1-a	824	0.761	0.029	0.975
237023 BABG-1	North Freeport	A-1-a	505	0.818	-0.08	0.99
237023 BABG-2	North Freeport	A-1-a	644	0.803	-0.172	0.997

$M_R = K_5 P_a \left(\frac{\theta}{P_a}\right)^{K_6} \left(\frac{\sigma_d}{P_a}\right)^{K_7}$ (Eq. 4-3), K_n constants are independent of units

TABLE 6.7 - Values of K_8, K_9, and K_{11} for use with Equation 4-4 of six type 1 soils from Maine assuming K_{10} = 1.0 (Law Engineering, 1994).

SHRP ID	Location	Classification	K_8	K_9	K_{11}	R^2
231009 BABG	Damariscotta	A-1-a	3836	0.154	0.562	0.997
231009 TPBG56	Damariscotta	A-1-a	1914	0.240	0.680	0.992
231012 TPBG56	South Freeport	A-1-a	2105	0.091	0.755	0.985
231028 BA1BS01	Bethel	A-1-a	3771	0.142	0.590	0.991
237023 BABG-1	North Freeport	A-1-a	2483	0.206	0.626	0.984
237023 BABG-2	North Freeport	A-1-a	4240	0.101	0.604	0.997

NOTE: $M_R = K_8(\sigma_{cyclic})^{K_9} K_{10}(1+\sigma_3)^{K_{11}}$ (Eq. 4-4), K_n constants are dependent on units of psi; θ and M_R must be in units of psi.

6.2.2 K_n Constants for Type 2 Soils

The stress dependent equations for type 2 soils are Equations 4-5, 4-6, and 4-7. Nine of eleven type 2 soils from southern and western Maine were use in the analysis. Two of the eleven type 2 soils were tested using the bulk stress sequencing normally used for type 1 soils (AASHTO T294-92) and could not be used in the analysis. Law Engineering (1992) performed resilient modulus testing and determined K_{16}, K_{17}, and K_{19} assuming K_{18} equal to 1.0 for Equation 4-7. The test data was provided in tabular and graphical forms. With the exception of computing bulk stress from cyclic stress, the same procedure to obtain the K_n constants for the resilient modulus equations of type 1 soils (Section 6.2.1) was used for type 2 soils.

Figure 6.6 shows a typical plot for obtaining K_{12} and K_{13} for Equation 4-5 from the resilient modulus test data. Three different trends of resilient moduli are shown because three different levels of confining pressure were used for each level of deviator stress. The high and low trends were included in the analysis therefore the correlation coefficient for the best fit line is extremely low. However, the best fit line still represents the resilient modulus - deviator stress relationship.

Values of confining pressure and deviator stress recommended by Santha (1994) (6.9 kPa $< \sigma_3 <$ 34.5 kPa ; 13.8 kPa $< \sigma_d <$ 103.4 kPa) were used to generate the resilient moduli from Equation 4-7. Figure 6.7 shows a typical plot for obtaining K_{14} and K_{15} for Equation 4-6. Tables 6.8,and 6.9 show the resulting values of the K_n constants and the correlation coefficients for Equations 4-5 and 4-6 using this procedure. Table 6.10 shows the K_n constants for Equation 4-7 (Law Engineering, 1992).

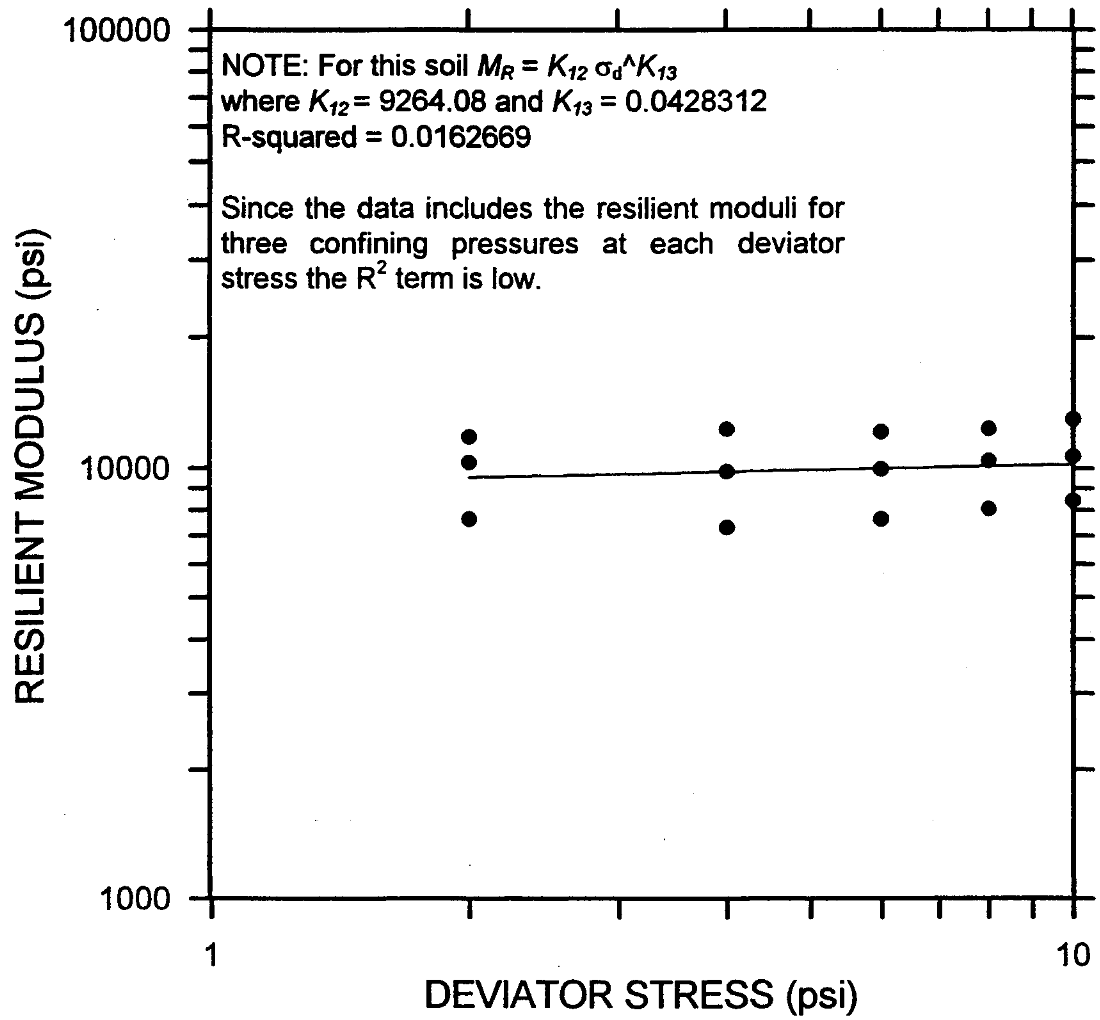


FIGURE 6.6 - Resilient modulus as a function of deviator stress for a type 2 A-4 soil from Damariscotta used to determine the K_n constants for Equation 4-5.

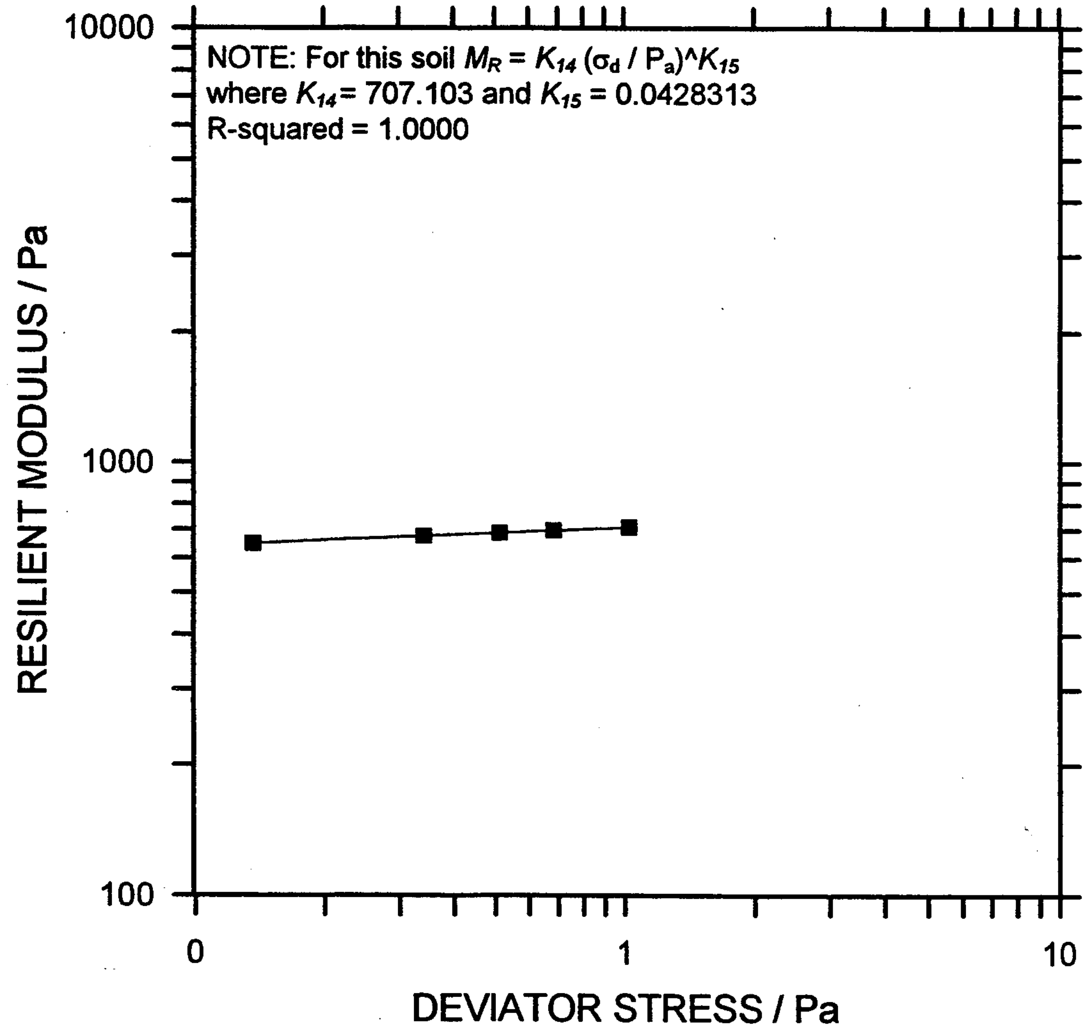


FIGURE 6.7 - Resilient modulus divided by atmospheric pressure as a function of deviator stress divided by atmospheric pressure for a type 2 A-4 soil from Damariscotta used to determine the K_n constants for Equation 4-6.

TABLE 6.8 - Values of K_{12} and K_{13} for Equation 4-5 of nine type 2 soils from Maine.

SHRP ID	Location	Classification	K_{12}	K_{13}	R^2
231009 TPBS55	Damariscotta	A-4	9264	0.043	0.016**
231012 TPBS55	South Freeport	A-7-6	9223	-0.041	0.013**
231026 TPBS55	Wilton	A-4	10520	0.041	0.015**
233013 BABS-1	Brunswick	A-2-4	9126	0.047	0.025**
233103 BABS-2	Brunswick	A-2-4	8295	0.114	0.105**
233014 BA5BS06	Topsham	A-1-b	8076	0.120	0.111**
233014 BABS	Topsham	A-1-b	8856	0.076	0.035**
237023 BABS-1	North Freeport	A-2-?*	8390	0.162	0.192**
237023 BABS-2	North Freeport	A-2-?*	8320	0.184	0.238**

NOTE: $M_R = K_{12}(\sigma_d)^{K_{13}}$ (Eq. 4-5), K_n constants are dependent on units of psi.,
* Complete classification not reported. ** The data includes the resilient moduli for three confining pressures at each deviator stress therefore the R^2 term is low.

TABLE 6.9 - Values of K_{14} and K_{15} for Equation 4-6 of nine type 2 soils from Maine.

SHRP ID	Location	Classification	K_{14}	K_{15}	R^2
231009 TPBS55	Damariscotta	A-4	707	0.043	1.0
231012 TPBS55	South Freeport	A-7-6	561	-0.041	1.0
231026 TPBS55	Wilton	A-4	798	0.041	1.0
233013 BABS-1	Brunswick	A-2-4	705	0.047	1.0
233103 BABS-2	Brunswick	A-2-4	768	0.114	1.0
233014 BA5BS06	Topsham	A-1-b	758	0.120	1.0
233014 BABS	Topsham	A-1-b	738	0.076	1.0
237023 BABS-1	North Freeport	A-2-?*	833	0.162	1.0
237023 BABS-2	North Freeport	A-2-?*	928	0.184	1.0

NOTE: $M_R = K_{14} P_a \left(\frac{\sigma_d}{P_a} \right)^{K_{15}}$ (Eq. 4-6), K_n constants are independent of units,

* Complete classification not reported.

TABLE 6.10 - Values of K_{16}, K_{17}, and K_{19} for Equation 4-7 of nine type 2 soils from Maine assuming $K_{18} = 1.0$.

SHRP ID	Location	Classification	K_{16}	K_{17}	K_{19}	R^2
231009 TPBS55	Damariscotta	A-4	4039	0.044	0.536	0.978
231012 TPBS55	South Freeport	A-7-6	3696	-0.042	0.588	0.959
231026 TPBS55	Wilton	A-4	4619	0.042	0.532	0.976
233013 BABS-1	Brunswick	A-2-4	4458	0.048	0.463	0.930
233103 BABS-2	Brunswick	A-2-4	3690	0.121	0.558	0.998
233014 BA5BS06	Topsham	A-1-b	3558	0.120	0.535	0.955
233014 BABS	Topsham	A-1-b	3289	0.076	0.642	0.994
237023 BABS-1	North Freeport	A-2-?*	3726	0.187	0.524	0.965
237023 BABS-2	North Freeport	A-2-?*	3695	0.164	0.536	0.978

NOTE: $M_R = K_{16}(\sigma_d)^{K_{17}} K_{18}(1+\sigma_3)^{K_{19}}$ $K_{19} = 1.0$ (Eq. 4-7), K_n constants are dependent on units of psi., * Complete classification not reported.

6.3 Linear Regression Analysis Using Soil Index and Resilient Modulus Test Data

The final approach used to determine resilient modulus of Maine soils was to perform a linear regression analysis to correlate laboratory resilient modulus data to soil index properties. The objective of this section was to develop a simple linear equation to estimate resilient modulus based on easily obtainable soil index data and stress state. The soil index data for the six type 1 and eight type 2 soils from Maine was obtained from the LTPP database. Two type 2 soils were tested using the type 1 stress sequence and one type 2 soil had incomplete soil index data. The laboratory resilient modulus test data was obtained from Law Engineering (1992). The software used for the linear regression analysis was SYSTAT, version 7.0.1.

Type 1 soils are more dependent on confining pressure than type 2 soils (Chapter 2). Constitutive relationships for type 1 soil incorporate confining pressures into bulk stress terms whereas relationships for type 2 soils are deviator stress dependent (Chapter 4). As discussed in Chapter 5, the best correlations to the value of resilient modulus exist with classification, water content, and stress conditions.

This section presents the results of linear regression analyses for type 1 and type 2 soils. The index properties for the soils used in the analysis are listed in Appendix F.

6.3.1 Linear Regression Analysis of Type 1 Soils

The soil index properties and the resilient modulus test data for six type 1 soils was used to create a database for the type 1 linear regression analysis. The soils used for this analysis were soil BABG and TPBG55 from Damariscotta, TPBG56 from South Freeport, BA1BS01 from Bethel, and BABG-1 and BABG-2 from North Freeport.

An automatic stepwise regression was chosen as the method of analysis. Choosing an automatic stepwise regression allowed SYSTAT to automatically add or remove each variable based on the magnitude of a computed "remove" or "enter" value. The "remove" and "enter" values are a measure of the variable's correlation to the variables previously entered into the regression equation. Forward stepwise regression begins with an equation with no variables and sequentially adds a variable at each step based on the lowest "enter" value. Backward stepwise regression begins with all variables in the equation and systematically removes one variable at each step based on the largest "remove" value. Equation 6-8 (five independent variables) is the resulting equation for stepwise regression in both directions for type 1 soils. The minimum and maximum values of resilient moduli were determined using Equation 6-8 and actual laboratory test data for the six soils. As expected the actual resilient moduli from the database generally fall within the minimum and maximum values of Equation 6-8 (Figure 6.8).

$$\begin{aligned} M_R(LRA1) = &-6350\Delta\gamma_{d\,\max} + 170S - 280\% pass25mm \\ &+ 730\% pass2mm + 330\theta \end{aligned} \qquad \text{(Eq. 6-8)}$$

$R^2 = 0.991$, Standard error of estimate = 2003 psi

where: $M_R(LRA1)$ = resilient modulus of type 1 soils in psi

$\Delta\gamma_{dmax}$ = difference between maximum dry density and dry density at time of testing in pcf

S = percent saturation (percent, e.g. 20.3% = 20.3)

%pass25mm = percent passing 25 mm (1-in.) sieve (percent)

%pass2mm = percent passing 2 mm (0.08-in.,#10) sieve (percent)

θ = bulk stress in psi

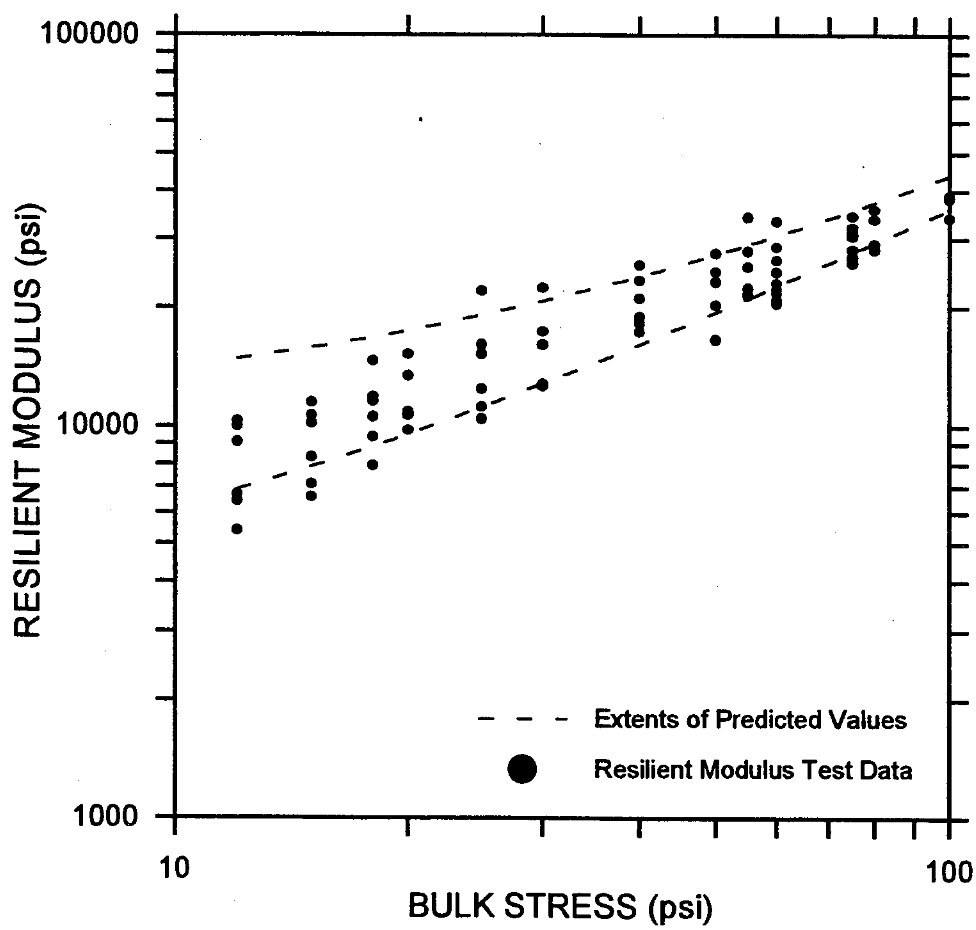


FIGURE 6.8 - Minimum and maximum resilient moduli from Equation 6-8 and the actual laboratory test data for six type 1 soils from Maine.

More linear regressions were done to examine the resilient modulus dependency on stress and its direct correlation to soil index properties. In the second regression (Equation 6-9) the bulk stress term from Equation 6-8 was removed. The correlation coefficient was 0.843 and the standard error of estimate increased to 8380 psi. The third regression (Equation 6-10) determined the correlation between resilient modulus and maximum dry density, optimum moisture content, and percents passing the number 10, 80, and 200 sieves. The correlation coefficient for this regression was 0.845 and the standard error of estimate was 8326 psi. The standard errors of estimate are excessively high. Furthermore Equations 6-9 and 6-10 do not predict resilient modulus as a function of stress condition.

$$\begin{aligned} M_R(LRA1) = &-7716\Delta\gamma_{d\max} + 443S - 316\%pass25mm \\ &+ 816\%pass2mm \end{aligned} \quad \text{(Eq. 6-9)}$$

$R^2 = 0.845$, Standard error of estimate = 8380 psi

$$\begin{aligned} M_R(LRA1) = &\ 329\gamma_{d\max} - 11786W_{opt} + 771\%pass2mm \\ &+ 2034\%pass0.34mm + 649\%pass0.08mm \end{aligned} \quad \text{(Eq. 6-10)}$$

$R^2 = 0.843$, Standard error of estimate = 8326 psi

where: $M_R(LRA1)$ = resilient modulus of type 1 soils in psi

γ_{dmax} = maximum dry density in pcf

%pass0.34mm = percent passing 0.34 mm (0.013-in.,#80) sieve (percent)

%pass0.08mm = percent passing 0.08 mm (0.003-in.,#200) sieve (percent)

6.3.2 Linear Regression Analysis of Type 2 Soils

For the type 2 soils the same approach discussed in the previous section was applied. Two of the eleven type 2 soils were omitted from the analysis because they were tested using the type 1 stress sequencing. One of the eleven type 2 soils was excluded from the analysis because it had an incomplete data set. The remaining index properties and resilient moduli of the eight soils in Appendix F comprised the database for the regression analysis. The soils used in this regression analysis are TPBS55 from Damariscotta, TPBS55 from South Freeport, TPBS55 from Wilton, BABS-1 and BABS-2 from Brunswick, BA5BS06 and BABS from Topsham, and BABS-1 and BABS-2 from North Freeport. Some soil index properties that Drumm et al. (1993) determined were important for the regression analysis (liquid and plastic limits) were not in the LTPP database.

The linear regression analysis generated Equation 6-11 (seven independent variables) for the type 2 soils. It is a combination of the different variables chosen by the forward and backward stepwise regression analysis. The range of resilient modulus was determined using Equation 6-11 and the minimum and maximum values of the soil index properties in the database. The range and the actual laboratory test data are plotted in Figure 6.9. The actual test data included resilient moduli at three different confining pressures. The best fit line through the minimum and maximum predicted resilient moduli fall in the middle third of the data.

$$M_R(LRA2) = 263\Delta\gamma_{d\max} - 234W_{opt} + 31S + 165\%pass76mm - 34\%pass0.08mm + 190\sigma_d - 1215\sigma_3 \qquad \text{(Eq. 6-11)}$$

$R^2 = 0.996$, Standard error of estimate = 950 psi

where: $M_R(LRA2)$ = resilient modulus of type 2 soils in psi

$\Delta\gamma_{dmax}$ = difference between maximum dry density and dry density at time of testing in pcf

W_{opt} = optimum water content

S = percent saturation (percent, e.g. 36.4% = 36.4)

%pass76mm = percent passing 76 mm (3-in.) sieve (percent)

%pass0.08mm = percent passing 0.08 mm (0.003-in., #200) sieve (percent)

σ_d = deviator stress in psi

σ_3 = confining pressure in psi

Further regression analyses were done to determine the effect of confining pressure and deviator stress. Equation 6-12 is Equation 6-11 without the confining pressure term. The resulting correlation coefficient was 0.979 and the standard error of estimate was 2245 psi. Removing the deviator stress and the confining pressure terms from Equation 6-11 (Equation 6-13) resulting in a standard error of estimate of 2302 psi. These are significantly higher than for Equation 6-11, therefore confining pressure has a significant effect on the resilient modulus of the Maine soils in the database.

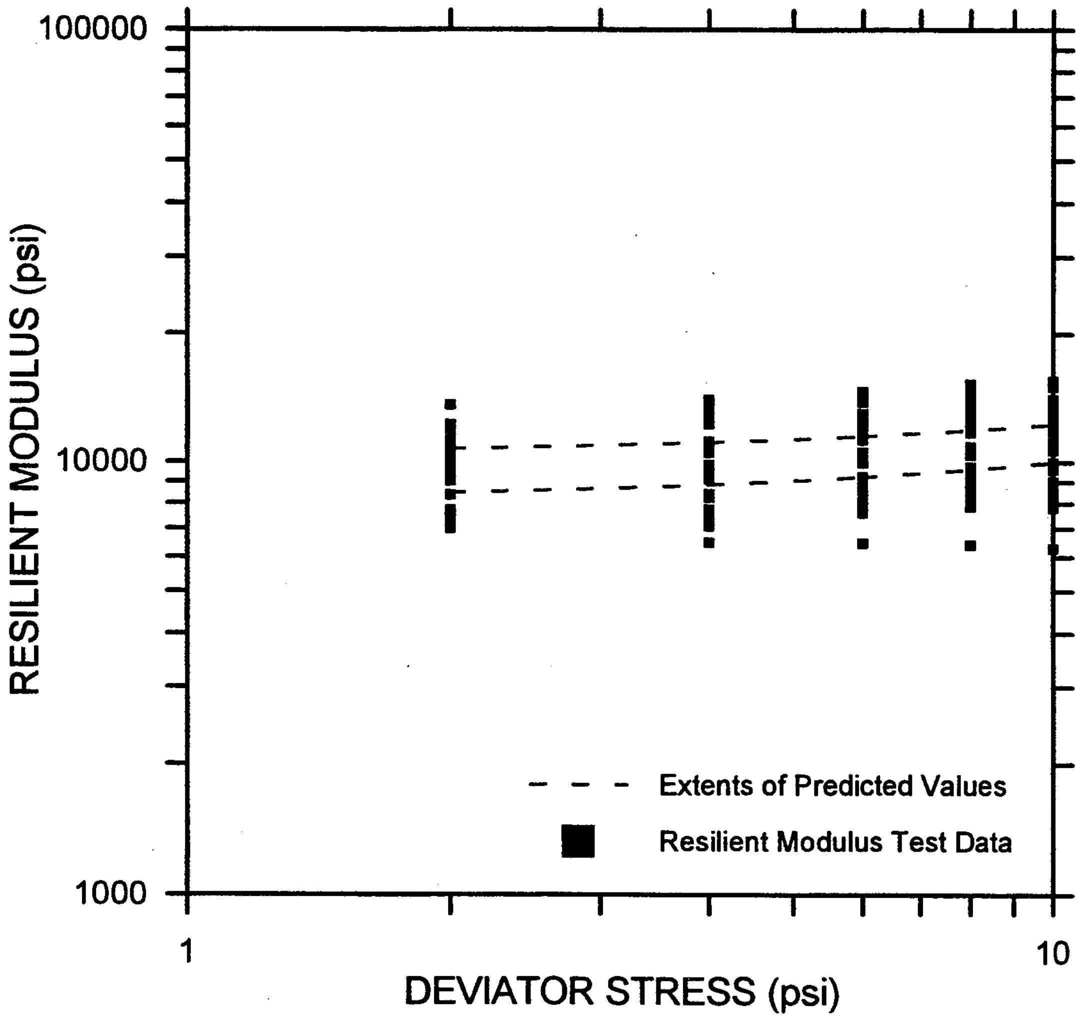


FIGURE 6.9 - Minimum and maximum resilient moduli from Equation 6-11 and the actual laboratory test data for eight type 2 soils from Maine

$$M_R(LRA2) = 263\Delta\gamma_{d\max} - 234W_{opt} + 31S + 116\%pass76mm - 34\%pass0.08mm + 190\sigma_d \quad \text{(Eq. 6-12)}$$

$R^2 = 0.979$, Standard error of estimate = 2245 psi

$$M_R(LRA2) = 263\Delta\gamma_{d\max} - 234W_{opt} + 31S + 128\%pass76mm - 34\%pass0.08mm \quad \text{(Eq. 6-13)}$$

$R^2 = 0.958$, Standard error of estimate = 2302 psi

6.4 Summary

Three methods for estimating the resilient modulus of Maine soils were examined. (1.) The resilient modulus was backcalculated from FWD data using MODCOMP 4 version H and MODULUS 5.1. (2.) K_n constants for all the constitutive relationships in Chapter 4 were developed from Equations 4-4 and 4-7. (3.) A linear regression analysis was done to correlate the laboratory resilient modulus to soil index properties.

The available data for Maine soils was limited to six type 1 and eleven type 2 soils. Laboratory resilient modulus test data, done by Law Engineering (1992) using AASHTO TP46-94 was provided by MDOT. Soil index property data was obtained from the Long Term Pavement Performance Program (LTPP) database. The LTPP database also provided falling weight deflectormeter (FWD) results for four Maine test sites.

MODCOMP 4 and MODULUS 5.1 were used to backcalculate the resilient moduli of eight soils from four FWD test sites in Maine (Brunswick, Damariscotta, North Freeport, and South Freeport).

Initially, backcalculated resilient moduli from MODCOMP 4 and MODULUS 5.1 did not correlate well with the laboratory resilient moduli. However removing two unreasonably high resilient moduli from the MODULUS 5.1 results significantly improved the MODULUS 5.1 / laboratory resilient modulus correlation (R^2=0.76). There were no obvious outliers to remove from the initial MODCOMP 4 analysis therefore attempts to improve the correlation were not done. The backcalculated / laboratory resilient modulus ratio for MODULUS 5.1 was 1.99 with a Y-intercept of -100 MPa (14.5 ksi). The inverse ratio (0.501) correlated well with the mean ratio (Mean =0.57) reported in Table 3.4 (Dalieden et al., 1994).

For two of the sites, standard split spoon refusal depths did not compare well with the software estimated depth to hard layer. These refusal depths were used to backcalculate resilient moduli of the base and subgrade layers. The refusal depth moduli did not compare favorably to the average estimated depth to hard layer moduli. Therefore the split spoon refusal depths should generally not be used for depths to hard layer.

The backcalculated resilient moduli of base and subgrade soils from MODULUS 5.1 are less sensitive to changes in Poisson's ratio of surface and subbase layers than the backcalculated resilient moduli from MODCOMP 4. Therefore accurate valuse of Poisson's ratios should be used when backcalculating resilient modulus.

The K_n constants in the constitutive equations in Chapter 4 were determined for several Maine soils. This gives designers a choice of constitutive equations when analyzing pavement performance. The K_n constants for Equation 4-4 had already been determined for six type 1 soils from Maine. Likewise, the constants for Equation 4-7 had already been determined for nine type 2 soils from Maine (Law Engineering, 1992). These equations and constants were used as the basis for determining the constants for other equations.

Equations 4-4 and 4-7 give resilient modulus as a function of cyclic stress (σ_{cyclic}) and confining or minor principle stress (σ_3). These equations were used to calculate resilient moduli for a range of σ_{cyclic} and σ_3 representative of field conditions. Equations 4-1 and 4-2 for type 1 soils give resilient modulus as a function of bulk stress (θ), while Equation 4-3 gives resilient modulus as a function of θ and deviator stress (σ_d). To determine the constants for these equations, it was necessary to assume that σ_{cyclic} was equal to σ_d. This is reasonable since σ_{cyclic} is either equal to or 90% of σ_d. This assumption allowed θ to be calculated using Equation 6-5. For each soil type a linear regression was performed with the resilient moduli calculated from Equation 4-4 to determine the K_n constants for Equations 4-1m 4-2, and 4-3. The results are summarized in Table 6.11. Constants for each soil type are given in Tables 6.4, 6.5, 6.6, and 6.7.

A similar approach was taken to determine the constants for constitutive equations applicable to type 2 soils (Equations 4-5 and 4-6). These equations are based on deviator stress, so it was again necessary to assume that σ_{cyclic}, used in Equation 4-7, was equal to σ_d. For each soil type, a linear regression was performed with the resilient moduli calculated from Equation 4-7 to determine the K_n constants for Equations 4-5 and 4-6. The results are summarized in Table 6.12. Constants for each soil are given in Tables 6.8, 6.9, and 6.10.

A linear regression analysis was done to develop correlations between resilient modulus and soil index properties. Equations for each soil type were generated. The maximum number of independent variables for type 1 equations was limited to five because there were only six soils used to build the database. The best type 1 regression equation (Equation 6-8) included the difference in dry density from maximum dry density, percent saturation, percents passing the 25 mm and 2 mm sieves, and bulk stress. The test data fit within the minimum and maximum predictions of Equation 6-8 very well however it was not tested on soils independent of the database. Notably absent from the database are the weaker subbase aggregates from Northern Maine. Therefore Equation 6-9 should not be used for estimating the resilient modulus of soils in Aroostook County. The correlation coefficient for Equation 6-8 was 0.991 and the standard error of estimate was 2003 psi. The effect of bulk stress was checked by performing a regression analysis without bulk stress. The resulting equation (Equation 6-9) had a significantly high standard error of estimate and is therefore not recommended for design.

TABLE 6.11 - Average, range, and standard deviation of the K_n constants for the equations in Chapter 4 from six type 1 Maine soils.

Number	Equation	M_R units	K_n	Range of Values	AverageValue	Standard Deviation
Eq. 4-1	$M_R = K_1(\theta)^{K_2}$	psi	K_1	675 to 2001	1356	485
		psi	K_2	0.465 to 0.862	0.707	0.136
Eq. 4-2	$M_R = K_3 \mathrm{P}_a \left(\frac{\theta}{\mathrm{P}_a}\right)^{K_4}$	psi or Pa	K_3	466 to 798	642	138
		psi or Pa	K_4	0.465 to 0.862	0.707	0.136
Eq. 4-3	$M_R = K_5 \mathrm{P}_a \left(\frac{\theta}{\mathrm{P}_a}\right)^{K_6} \left(\frac{\sigma_d}{\mathrm{P}_a}\right)^{K_7}$	psi or Pa	K_5	449 to 1101	679	245
		psi or Pa	K_6	0.337 to 0.931	0.729	0.204
		psi or Pa	K_7	-0.001 to 0.185	-0.019	0.121

TABLE 6.11 (continued) - Average, range, and standard deviation of the K_n constants for the equations in Chapter 4 from six type 1 Maine soils.

Eq. 4-4	$M_R = K_8(\sigma_{cyclic})^{K_9} K_{10}(1+\sigma_3)^{K_{11}}$	psi	K_8	1914 to 4240	3058	1006
		psi	K_9	0.091 to 0.240	0.156	0.058
		n/a	K_{10}	assumed = 1.0	n/a	n/a
		psi	K_{11}	0.562 to 0.755	0.636	0.070

TABLE 6.12 - Average, range, and standard deviation of the K_n constants for the equations in Chapter 4 from nine type 2 Maine soils.

Number	Equation	M_R units	K_n	Range of Values	AverageValue	Standard Deviation
Eq. 4-5	$M_R = K_{12}(\sigma_d)^{K_{13}}$	psi	K_{12}	8076 to 10520	8897	755
		psi	K_{13}	-0.041 to 0.184	0.083	0.070
Eq. 4-6	$M_R = K_{14}P_a\left(\frac{\sigma_d}{\mathrm{P}_a}\right)^{K_{15}}$	psi or Pa	K_{14}	561 to 928	755	100
		psi or Pa	K_{15}	-0.041 to 0.184	0.083	0.070
Eq. 4-7	$M_R = K_{16}(\sigma_d)^{K_{17}} K_{18}(1+\sigma_3)^{K_{19}}$	psi or Pa	K_{16}	3289 to 4619	3863	431
		psi or Pa	K_{17}	-0.042 to 0.187	0.084	0.071
		n/a	K_{18}	assumed = 1.0	n/a	n/a
		psi or Pa	K_{19}	0.463 to 0.642	0.546	0.049

$$M_R(LRA1) = -6350\Delta\gamma_{d\max} + 170S - 280\% pass25mm + 730\% pass2mm + 330\theta \qquad \text{(Eq. 6-8)}$$

Regression equations were also developed for type 2 soils. The database contained a more complete set soil index properties and resilient modulus test data for eight of the eleven Maine type 2 soils. The best correlation (Equation 6-11) was achieved with seven variables. A shortcoming of the available data was that the liquid and plastic limits were unavailable. Furthermore properties from the weaker subgrade soils in Northern Maine were not included in the database. Therefore Equation 6-11 should not be used for estimating the resilient modulus of soils in Aroostook County. Equation 6-11 fit the test data quite well (R^2=0.996) however it was not tested using soils independent of the database. The prediction range of Equation 6-11 was in the middle third of the measured laboratory resilient moduli. The standard error of estimate was 950 psi.

$$M_R(LRA2) = 263\Delta\gamma_{d\max} - 234W_{opt} + 31S + 165\% pass76mm - 34\% pass0.08mm + 190\sigma_d - 1215\sigma_3 \qquad \text{(Eq. 6-11)}$$

The samples were tested at three different confining pressures, therefore the dependence upon confining pressure and deviator stress was examined. Removing the confining pressure term from Equation 6-11 produced a standard error estimate of 2245 psi (Equation 6-12). By removing both the deviator stress and the confining pressure terms (Equation 6-13) the standard error of estimate was 2302 psi. Therefore the resilient modulus of the type 2 soils from Maine in the database is highly correlated with stress conditions. Use of equations 6-12 and 6-13 is discouraged.

CHAPTER 7

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

7.1 Summary

The Maine Department of Transportation (MDOT) uses the 1993 AASHTO Guide for Design of Pavement Structures (AASHTO Guide) as a design aid to determining the thickness of asphalt cement concrete paving, base, and subbase layers. Prior to 1986 the design of roadway construction was primarily based on the California Bearing Ratio (CBR). The AASHTO Guide now bases its design methods on resilient modulus rather than CBR. In response, the MDOT purchased laboratory resilient modulus test equipment consistent with the testing methods at the time. The equipment is pneumatically actuated and may now be obsolete. MDOT commissioned this study because it has no practical method to determine resilient modulus. The following tasks were accomplished:

1. A literature review was conducted of laboratory tests to examine the factors most affecting the magnitude of resilient modulus. Procedures for estimating resilient modulus parameters from soil index data, laboratory testing, and nondestructive test methods were also examined.

2. Actual resilient modulus test data for seventeen Maine soils was obtained from published results (Law Engineering, 1992). Soil index data for these soils was obtained from the Strategic Highway Research Program's (SHRP) Long Term

Pavement Performance (LTPP) database. Falling weight deflectometer data from four Maine sites was also obtained from the LTPP database.

3. The data was used to examine three methods for resilient modulus determination for Maine soils. The three methods were resilient modulus backcalculation using MODCOMP 4 version H and MODULUS 5.1, determination of the soil specific K_n constants for seven constitutive relationships, and linear regression to correlate resilient modulus to soil index and resilient modulus test data.

The literature review included a study to determine if current laboratory resilient modulus test methods yielded repeatable results (Steel et al., 1994). Thirteen agencies tested three synthetic material specimens using similar laboratory equipment and the specification current at the time (AASHTO T294-92). An individual laboratory could produce repeatable results with an average variation in resilient modulus of only 6%. However, when the results from all the laboratories were viewed collectively the average variation was 22% for two of the specimens and 40% for the third. Thus the measured resilient modulus is highly dependent on the lab that does the test. More recently, quality control and assurance standards have been published to reduce the margin of laboratory error (Alavi et al., 1997).

The literature review examined factors that affect resilient modulus measured in the laboratory. Changes in dry density, moisture content, soil fabric, and stress state affect the resilient modulus. Therefore reproducing the in-situ conditions in the laboratory is extremely important to obtaining accurate resilient moduli. For laboratory resilient modulus tests, soils are divided into two types. Most granular bases and subbases are type 1 soils. Finer grained soils, including some subbases and most subgrades, are type 2 soils. For type 1 soils, increasing the deviator stress causes an initial reduction in resilient modulus followed by a gradual increase in resilient modulus with further increases in deviator stress. Increased confining pressure causes an increase in resilient moduli for type 1 soils and to a lesser degree for type 2 soils.

System compliance of the laboratory equipment affects the value of resilient modulus. Linear variable differential transformers (LVDT) mounted on clamps attached directly to the middle third of the specimen give higher resilient moduli than those obtained using externally mounted LVDTs. Furthermore, a haversine shaped loading pulse most accurately represents the actual field loading conditions. Ideally, resilient modulus testing equipment should have electro-hydraulic loading and a data acquisition system that is completely computer automated (Barksdale et al., 1997).

Four alternative laboratory test methods were found in the literature. The torsional shear (TS) and resonant column (RC) have been used to measure the shear strain of soil specimens. Correlations between shear strain and resilient modulus have been proposed. A modified gyratory test machine (GTM), normally used to test asphalt concrete specimens, has been used for resilient modulus measurement. A fourth alternative test method (ATM) apparatus developed by Drumm et al. (1996) is similar to a small-scale falling weight deflectometer. With further investigation the GTM and the ATM are good candidates for alternative methods for resilient modulus estimation.

The falling weight deflectometer (FWD) is a common nondestructive field test method used to evaluate pavement performance. Several software programs exist to backcalculate resilient modulus from FWD deflection basin data. The programs most suited for resilient modulus backcalculation are based on elastic layer theory. FWD equipment characteristics, deflection basin data, initial seed resilient moduli, and roadway cross section information are required to perform the backcalculation analysis. The input data preparation and output evaluation portions of the backcalculation procedure are tedious. Good engineering judgement is needed to choose appropriate values for input parameters and evaluate the results.

Initially, the backcalculated resilient moduli from MODCOMP 4 and MODULUS 5.1 and the laboratory resilient moduli did not correlate well. After removing unrealistic moduli (M_R > 2000 MPa) from the MODULUS 5.1 analysis, the backcalculated resilient modulus correlation from MODULUS 5.1 improved significantly (R^2=0.76). Furthermore, the modified MODULUS 5.1 and MODCOMP 4 moduli were within the same order of magnitude. The backcalculated / laboratory resilient modulus ratio using the six of eight moduli from MODULUS 5.1 was 1.99 with a Y-intercept of -100 MPa. Estimates for the depth to hard layer were done automatically by the programs based on deflection basin profiles.

The LTPP database provided refusal depths of a standard split spoon test for two of the four sites. The refusal depths and the moduli computed with the refusal depths did not compare well with the estimated depths to hard layer and the moduli from the estimated depths to hard layer. Therefore the depths to refusal should generally not be used for depths to hard layer.

The backcalculated resilient moduli of base and subgrade soils from MODULUS 5.1 are less sensitive to changes in Poisson's ratio of surface and subbase layers than the backcalculated resilient moduli from MODCOMP 4. Therefore accurate Poisson's ratios should be used when backcalculating resilient modulus.

Stress dependent constitutive relationships for resilient modulus have been proposed. These equations relate resilient modulus to stress conditions. The experimental constants for these relationships are determined by a best fit line through a set of laboratory resilient modulus test results. The in-situ stress conditions affect type 1 soils differently than type 2 soils. Therefore constitutive relationships are most often unique to soil type. However a universal relationship for types 1 and 2 soils has been proposed (Tables 7.1 and 7.2). Law Engineering (1992) determined the laboratory resilient moduli of six type 1 and eleven type 2 soils from southern and western Maine and then found K_n constants for Equations 4-4 and 4-7. Weaker subbase aggregates from Northern Maine were notably absent from the soils tested. The test results provided the basis for determining equivalent K_n constants for three type 1 and two type 2 equations. The range, average, and standard deviation of the K_n constants for each equation are given in Tables 7.1 and 7.2. These equations give designers a choice of constitutive relationships when analyzing pavement performance.

Correlations between resilient modulus and soil index properties were examined. Drumm et al. (1993) identified several key properties and resilient modulus test data which influence resilient modulus. They are listed in Table 7.3. Drumm et al. (1993) found the most significant factors affecting resilient modulus were classification and deviation from optimum water content. Drumm et al. (1993) perfomed a linear regression analysis using properties and test data from eight type 2 soils from Tennessee. The final product did not include all the soil properties in Table 7.3 and is listed below in Equation 5-1. The equation has eight independent variables. Equation 5-1 fit the test data with reasonable accuracy (R^2=0.70) however it was tested on four soils within the database. Testing the accuracy of a regression equation should be done on soils independent of the database.

$$\begin{aligned}\log M_R(psi) &= 46.93 + 0.0188\sigma_d + 0.0333\Delta\gamma_{d\max} - 0.1143LI + 0.4680S \\ &+ 0.0085class^2 - 0.0033\Delta W_{opt}{}^2 - 0.0012\sigma_3{}^2 + 0.0001PL^2 \\ &+ 0.0278LI^2 - 0.0017S^2 - 38.44\log S - 0.2222\log\sigma_d\end{aligned} \qquad \text{(Eq. 5-1)}$$

TABLE 7.3 - Average, range, and standard deviation of the K_n constants for the equations in Chapter 4 from six type 1 Maine soils.

Number	Equation	M_R units	K_n	Range of Values	AverageValue	Standard Deviation
Eq. 4-1	$M_R = K_1(\theta)^{K_2}$	psi	K_1	675 to 2001	1356	485
		psi	K_2	0.465 to 0.862	0.707	0.136
Eq. 4-2	$M_R = K_3 \mathrm{P}_a \left(\frac{\theta}{\mathrm{P}_a}\right)^{K_4}$	psi or Pa	K_3	466 to 798	642	138
		psi or Pa	K_4	0.465 to 0.862	0.707	0.136
Eq. 4-3	$M_R = K_5 \mathrm{P}_a \left(\frac{\theta}{\mathrm{P}_a}\right)^{K_6} \left(\frac{\sigma_d}{\mathrm{P}_a}\right)^{K_7}$	psi or Pa	K_5	449 to 1101	679	245
		psi or Pa	K_6	0.337 to 0.931	0.729	0.204
		psi or Pa	K_7	-0.001 to 0.185	-0.019	0.121

TABLE 7.3 (continued) - Average, range, and standard deviation of the K_n constants for the equations in Chapter 4 from six type 1 Maine soils.

Eq. 4-4	$M_R = K_8(\sigma_{cyclic})^{K_9} K_{10}(1+\sigma_3)^{K_{11}}$	psi	K_8	1914 to 4240	3058	1006
		psi	K_9	0.091 to 0.240	0.156	0.058
		n/a	K_{10}	assumed = 1.0	n/a	n/a
		psi	K_{11}	0.562 to 0.755	0.636	0.070

TABLE 7.4 - Average, range, and standard deviation of the K_n constants for the equations in Chapter 4 from nine type 2 Maine soils.

Number	Equation	M_R units	K_n	Range of Values	Average Value	Standard Deviation
Eq. 4-5	$M_R = K_{12}(\sigma_d)^{K_{13}}$	psi	K_{12}	8076 to 10520	8897	755
		psi	K_{13}	-0.041 to 0.184	0.083	0.070
Eq. 4-6	$M_R = K_{14}P_a\left(\frac{\sigma_d}{P_a}\right)^{K_{15}}$	psi or Pa	K_{14}	561 to 928	755	100
		psi or Pa	K_{15}	-0.041 to 0.184	0.083	0.070
Eq. 4-7	$M_R = K_{16}(\sigma_d)^{K_{17}} K_{18}(1+\sigma_3)^{K_{19}}$	psi or Pa	K_{16}	3289 to 4619	3863	431
		psi or Pa	K_{17}	-0.042 to 0.187	0.084	0.071
		n/a	K_{18}	assumed = 1.0	n/a	n/a
		psi or Pa	K_{19}	0.463 to 0.642	0.546	0.049

TABLE 7.3 - **Soil index properties and resilient modulus test data used for regression analysis conducted by Drumm et al. (1993).**

Soil Index Property and Resilient Modulus Test Data	Symbol
Liquid limit	LL
Plastic limit	PL
Plasticity index	PI
Liquidity index	LI
Percent passing the No. 200 sieve by washing (e.g. 10.2%=10.2)	P_{200}
Percent clay (e.g. 20.4%=20.4)	P_{clay}
AASHTO classification (e.g. A-7-6=7.6 or A-1-a=1.1)	$class$
Specific gravity	G
CBR at 2.54 mm penetration	$CBR_{2.54}$
CBR at 5.08 mm penetration	$CBR_{5.08}$
Optimum water content	W_{opt}
Maximum dry density	γ_{dmax}
Resilient modulus (ksi)	M_r
Confining pressure (psi)	σ_3
Deviator stress (psi)	σ_d
Dry density of the specimen (pcf)	γ_d
Water content of the specimen	W_s
Deviation from maximum dry density	$\Delta\gamma_{dmax}$
Deviation from optimum water content	ΔW_{opt}
Percent saturation (e.g. 30.6%=30.6)	S
Initial tangent modulus from unconfined compression tests	$1/a$
Parameter corresponding to unconfined compressive strength	$1/b$

Correlations between resilient modulus with cohesion (C), internal friction angle (ϕ), bulk stress (θ), and confining pressure (σ_3), and elasticity (initial tangent modulus) have been examined. Laguros et al. (1993) used these properties from six type 1 soils from Oklahoma to perform a regression analysis. The resulting equations (Equations 5-10 and 5-11) predicted the actual resilient modulus test data moderately well. The average correlation coefficient for Equation 5-10 was 0.7336. However this average was from a wide range of values (0.5374 to 0.8345) for the six soils. No correlation coefficient was given for Equation 5-11.

$$M_R(psi, C, \phi) = 2860.94 + 275C + 128\sigma_1 \tan\phi + 118\theta \qquad \text{(Eq. 5-10)}$$

$$M_R(psi, \sigma_3, \theta) = (18.28 + 0.4917\sigma_3)0.4098 + 150.7\theta \qquad \text{(Eq. 5-11)}$$

The LTPP database provided a small amount of index data for the six type 1 and nine type 2 soils from Maine. Laboratory resilient modulus tests were done by Law Engineering (1992). For type 1 soils, Equation 6-8 (Table 7.3) fit the test data very well (R^2=0.991, Standard error of estimate = 2003 psi). The actual test data was within the minimum and maximum predictions (Figure 6.10). The standard estimate of error was considerably higher (8380 psi) without the bulk stress term (θ) and nearly as high (8326 psi) when correlated to dry density, water content, and some sieves defining AASHTO soil classification. The true accuracy of Equation 6-8 is unknown because it was not used to predict resilient moduli of soils outside those in the database. Furthermore, index properties from the weaker subgrade soils from Northern Maine were absent from the database. Therefore Equation 6-8 should not be used to estimate the resilient modulus of soils from Aroostook County. The resilient modulus of the six type 1 Maine soils in the database is highly dependent on the bulk stress.

Eight type 2 soils from Maine were used to develop correlations between soil index properties and resilient modulus test data. The best correlation equation was achieved with seven independent variables. For the type 2 soils, Equation 6-11 (Table

7.4) fit the test data very well (R^2=0.996, Standard error of estimate = 950 psi). The prediction range of Equation 6-11 was in the middle third of the laboratory resilient moduli. Without the confining pressure term the standard error of estimate was 2245 psi. The correlation to dry density, optimum water content, percent saturation, and percents passing the 76-mm (3-in.) and 0.08-mm (0.003-in., #200) sieves produced a similar error term of 2302 psi. The true accuracy of Equation 6-11 is unknown because it was not used to predict resilient moduli of soils outside those in the database. . Furthermore, index properties from the weaker subgrade soils from Northern Maine were absent from the database. Therefore Equation 6-11 should not be used to estimate the resilient modulus of soils from Aroostook County. The resilient modulus of the eight type 2 Maine soils in the database is dependent on confining pressure.

TABLE 7.4 - Linear regression equations correlating soil index properties and resilient modulus test data of six type 1 soils and eight type 2 soils from Maine.

Equation Number	Equation	Standard Error of Estimate
6-8	$M_R(LRAType1) = -6350\Delta\gamma_{d\max} + 170S - 280\%pass25mm + 730\%pass2mm + 330\theta$	2003 psi
6-11	$M_R(LRAType2) = 263\Delta\gamma_{d\max} - 234W_{opt} + 31S + 165\%pass76mm - 34\%pass0.08mm + 190\sigma_d - 1215\sigma_3$	950 psi

NOTE: Equations 6-8 and 6-11 should not be used to estimate the resilient modulus of soils from Aroostook County.

where: $\Delta\gamma_{dmax}$ = difference between maximum dry density and dry density at time of testing in pcf

W_{opt} = optimum water content

S = percent saturation (percent, e.g. 98.1% = 98.1)

%pass76mm = percent passing 76 mm (3-in.) sieve (percent)

%pass25mm = percent passing 25 mm (1-in.) sieve (percent)

%pass2mm = percent passing 2 mm (0.08-in.,#10) sieve (percent)

%pass0.08mm = percent passing 0.08 mm (0.003-in., #200) sieve (percent)

θ = bulk stress in psi

σ_d = deviator stress in psi

σ_3 = confining pressure in psi

The California bearing ratio (CBR) has been used extensively as an estimation of a soil's strength characteristics. Unfortunately CBR correlates poorly to resilient modulus. The static loading method used in CBR testing does not represent the repetitious dynamic loading done in resilient modulus tests. Furthermore CBR allows specimen deformation in only the axial direction. Resilient modulus testing allows deformation in the axial and radial directions. CBR test data was not available for the Maine soils in the LTPP database, therefore its correlation to resilient modulus was not examined. However, the AASHTO Guide correlates the resilient modulus of fine grained soils to CBR (Equation 5-12). Comparison to actual test data (Drumm et al., 1993) has shown a wide range of variations including some values outside the range of 3 to 0.75 times CBR (Figure 5.11).

$$M_R = 1.5CBR \text{ ksi} \qquad \text{(Eq. 5-12)}$$

The AASHTO Guide also gives a correlation between resilient modulus and CBR as a function of bulk stress. Laguros et al. (1993) compared the correlation between resilient modulus and CBR basedd on six type 1 soils from Oklahoma. The factors relating CBR to resilient modulus were an average 76% lower than those given by AASHTO.

7.2 Conclusions

The conclusions to this study are listed below:

1. Determining the resilient modulus of a soil in the laboratory by repeated load triaxial testing is a complicated procedure and is affected by many factors including: specimen identification, compaction method, soil fabric, density, water content, specimen storage time, loading pulse configuration, preconditioning, stress state, strain measurement, and system compliance. Therefore the results of the laboratory resilient modulus test are in many respects an index test.

2. A single laboratory can obtain consistent results from resilient modulus tests, however, when compared to other laboratories the range of resilient moduli of similar specimens is rather wide. Therefore, the accuracy of resilient moduli are a function of the details of the test equipment and the skill of the technician who performed the test.

3. With addtional research the resonant column, torsional shear, gyratory testing machine, and the ATM could offer a practical alternative laboratory method to determine the resilient modulus of Maine soils.

4. Backcalculating resilient modulus from FWD data requires initial reasonable estimates of values for resilient modulus and accurate values of depth to hard layer, layer thicknesses, and Poisson's ratio. MODCOMP 4 is more sensitive to changes in Poisson's ratio than MODULUS 5.1.

5. Split spoon depths to refusal should generally not be used for depth to hard layer in MODCOMP 4 and MODULUS 5.1.

6. The backcalculated resilient moduli from MODCOMP 4 and MODULUS 5.1 do not correlate well with moduli from laboratory tests when the programs automatically estimate the depth to hard layer. The backcalculated resilient moduli from MODULUS 5.1 correlate well with laboratory moduli if moduli in excess of 2000 MPa are neglected from the correlation.

7. There are several constitutive relationships that can accurately relate resilient modulus to stress state. These rely on K_n specific to a given soil. The constants are presented for fourteen Maine soils. They can be used with soils that have similar classification, dry density, and water content.

8. Useable correlations of resilient modulus with soil properties can be developed. The best correlations to resilient modulus include soil property data and stress state. The correlations developed in this study should be used with caution since they were based on a limited data set

9. California bearing ratio does not correlate well with laboratory resilient modulus. The use of correlations involving CBR is discouraged.

7.3 Recommendations for Further Research

To develop a standard procedure for estimating the resilient modulus of Maine base, subbase, and subgrade soil layers a larger database of soil index properties and resilient modulus test data needs to be created. The data base should include multiple specimens of several samples of soils typical to those used for Maine roadway construction and from all AASHTO classifications. Wherever possible, the soil specimens should be undisturbed. The specimens should reflect estimated changes in moisture content and dry density so that the corresponding change in resilient modulus can be studied.

A thorough series of index testing should be done on Maine soils representative of those used in roadway construction. There should be a minimum of ten soils from Aroostook County and a total of 20 soils from other Maine locations. Soil index properties should include specific gravity, dry density, maximum dry density, in-situ density, water content, optimum water content, in-situ water content, percent saturation, plastic limit, liquid limit, plasticity index, and liquidity index. Grain size distribution should also be done. The samples should be identified according to AASHTO classification. Resilient modulus testing should be done by a single state-of-the-art laboratory with adequately skilled technicians and current resilient modulus specifications. Resilient modulus testing should be done on multiple specimens of a soil sample with varying densities and water contents.

Investigation into backcalculation software for use with FWD data should be continued. In addition to the specimen's resilient modulus and soil index properties, the specimen's location, depth to hard layer, and Poisson's ratio should be obtained to compare the FWD backcalculation results to laboratory resilient modulus tests. The programs should be able to directly access the FWD data from the field to expedite the backcalculation process.

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APPENDIX A

SOIL INDEX PROPERTIES AND K_n CONSTANTS FOR EIGHT NEW ENGLAND SOILS EXAMINED BY LEE et al.. (1997).

TABLE A.1 - New England Type 1 Soils (Lee et al.,1997)

Soil	Source	Classification	W_{opt} (%)	γ_{dopt} (kg/m^3)	K_1	K_2	R^2
Vermont Crushed Stone	Processed	A-1-a	8.0	2162.4	1333	0.75	0.98
Massachussettes Gravel	Processed	A-1-a	8.5	2003.2	3058	0.58	0.95
Maine Sabattus Subbase	Bank Run	A-1-a	8.1	2181.1	2111	0.70	0.93
Connecticut Gravel	Bank Run	A-1-a	8.6	2020.8	2518	0.62	0.86
Maine Frenchville Gravel	Processed	A-1-a	6.1	2329.0	6830	0.47	0.80
New Hampshire Subbase	Bank Run	A-1-a	8.6	1888.0	2365	0.68	0.82
Rhode Island Subbase	Bank Run	A-1-b	6.0	2078.5	5809	0.37	0.84
Massachussettes Crushed Stone	Processed	A-1-b	7.1	2235.5	1326	0.84	0.79

APPENDIX B

SOIL INDEX PROPERTIES AND K_N CONSTANTS FOR FIFTEEN TYPE 1 AND FOURTEEN TYPE 2 SOILS FROM GEORGIA (SANTHA, 1994).

TABLE B.1 - Georgia Type 1 Soils (Santha, 1994)

Soil ID	W_{opt} (%)	W (%)	γ_d (pcf)	K_5	K_6	K_7	R^2
1-I	12	11.2	121	392	0.291	-0.487	0.97
1-II	12	12.5	121	349	0.316	-0.531	0.89
1-III	12	9.7	121	543	0.268	-0.402	0.98
2-I	18	17.0	108	401	0.239	-0.484	0.98
2-II	18	19.5	108	326	0.328	-0.627	0.93
2-III	18	14.9	108	715	0.175	-0.330	0.94
3-I	16	16.0	113	451	0.301	-0.501	0.97
3-II	16	17.4	113	413	0.316	-0.574	0.94
3-III	16	14.8	113	642	0.199	-0.403	0.97
4-I	25	24.5	92	528	0.304	-0.364	0.94
4-II	25	25.9	92	403	0.318	-0.385	0.84
4-III	25	22.9	92	703	0.292	-0.259	0.93

TABLE B.1 (continued) - Georgia Type 1 Soils (Santha, 1994)

Soil ID	W_{opt} (%)	W (%)	γ_d (pcf)	K_5	K_6	K_7	R^2
5-I	16	15.2	108	356	0.285	-0.304	0.93
5-II	16	17.0	108	335	0.293	-0.369	0.90
5-III	16	13.8	108	547	0.203	-0.213	0.75
6-I	17	15.7	108	573	0.201	-0.272	0.82
6-II	17	17.2	108	423	0.250	-0.317	0.83
6-III	17	14.0	108	832	0.145	-0.152	0.68
7-I	19	19.3	103	214	0.404	-0.343	0.91
7-II	19	20.6	103	173	0.412	-0.403	0.98
7-III	19	17.1	103	299	0.319	-0.351	0.95
8-I	16	15.7	110	241	0.379	-0.319	0.95
8-II	16	17.0	110	211	0.441	-0.340	0.92
8-III	16	14.0	110	284	0.295	-0.292	0.91

TABLE B.1 (continued) - Georgia Type 1 Soils (Santha, 1994)

Soil ID	W_{opt} (%)	W (%)	γ_d (pcf)	K_5	K_6	K_7	R^2
9-I	11	13.5	114	280	0.328	-0.336	0.90
9-II	11	14.3	114	252	0.349	-0.322	0.94
9-III	11	11.1	114	324	0.267	-0.301	0.91
10-I	16	15.7	106	430	0.457	-0.340	0.95
10-II	16	16.6	106	338	0.479	-0.373	0.90
10-III	16	13.4	106	534	0.368	-0.298	0.91
11-I	14	13.1	111	458	0.401	-0.353	0.96
11-II	14	14.4	111	384	0.444	-0.385	0.89
11-III	14	11.6	111	573	0.345	-0.294	0.94
12-I	14	13.6	112	668	0.398	-0.302	0.94
12-II	14	15.2	112	494	0.469	-0.363	0.94
12-III	14	12.1	112	918	0.326	-0.159	0.93

TABLE B.1 (continued) - Georgia Type 1 Soils (Santha, 1994)

Soil ID	W_{opt} (%)	W (%)	γ_d (pcf)	K_5	K_6	K_7	R^2
13-I	12	11.5	114	354	0.484	-0.403	0.95
13-II	12	12.7	114	334	0.498	-0.459	0.90
13-III	12	9.8	114	446	0.436	-0.367	0.94
14-I	16	14.7	107	440	0.429	-0.382	0.95
14-II	16	16.3	107	346	0.454	-0.446	0.90
14-III	16	13.4	107	507	0.397	-0.330	0.96
15-I	20	19.8	123	183	0.400	-0.450	0.89
15-II	20	21.4	123	130	0.430	-0.451	0.90
15-III	20	18.2	123	201	0.342	-0.437	0.93

TABLE B.1 (continued) - Georgia Type 1 Soils (Santha, 1994)

Soil ID	W_{opt} (%)	W (%)	γ_d (pcf)	*LL,PI*	K_{14}	K_{15}	R^2
1-I	20	19.2	105	40.3,20.9	382	-0.466	0.94
1-II	20	21.7	105	40.3,20.9	287	-0.478	0.97
1-III	20	18.4	105	40.3,20.9	574	-0.322	0.89
2-I	20	20.2	107	35,10.8	276	-0.511	0.98
2-II	20	20.8	107	35,10.8	188	-0.598	0.98
2-III	20	17.5	107	35,10.8	450	-0.368	0.96
3-I	20	19.9	106	38.9,19.2	657	-0.188	0.80
3-II	20	21.1	106	38.9,19.2	431	-0.261	0.80
3-III	20	18.1	106	38.9,19.2	745	-0.128	0.51

TABLE B.2 - Georgia Type 2 Soils (Santha, 1994)

Soil ID	W_{opt} (%)	W (%)	γ_d (pcf)	*LL,PI*	K_{14}	K_{15}	R^2
4-I	19	18.1	108	36,17.5	608	-0.264	0.95
4-II	19	19.9	108	36,17.5	423	-0.272	0.97
4-III	19	16.4	108	36,17.5	774	-0.251	0.88
5-I	20	19.2	106	40.5,17.8	641	-0.219	0.99
5-II	20	20.6	106	40.5,17.8	442	-0.312	0.90
5-III	20	17.7	106	40.5,17.8	657	-0.134	0.78
6-I	17	16.6	109	46.5,30.4	777	-0.169	0.71
6-II	17	18.2	109	46.5,30.4	473	-0.235	0.74
6-III	17	14.6	109	46.5,30.4	913	-0.079	0.70
7-I	19	18.4	104	43,18	651	-0.273	0.94
7-II	19	19.1	104	43,18	549	-0.260	0.90
7-III	19	16.6	104	43,18	943	-0.136	0.98

TABLE B.2 (continued) - Georgia Type 2 Soils (Santha, 1994)

Soil ID	W_{opt} (%)	W (%)	γ_d (pcf)	*LL,PI*	K_{14}	K_{15}	R^2
8-I	18	17.7	107	40,13.1	460	-0.323	0.90
8-II	18	18.6	107	40,13.1	299	-0.424	0.97
8-III	18	15.9	107	40,13.1	599	-0.177	0.78
9-I	15	14.2	112	33,11.3	650	-0.243	0.93
9-II	15	15.5	112	33,11.3	474	-0.366	0.97
9-III	15	12.2	112	33,11.3	823	-0.072	0.97
10-I	16	15.6	111	49,32	917	-0.204	0.98
10-II	16	16.3	111	49,32	685	-0.211	0.90
10-III	16	13.6	111	49,32	1169	-0.074	0.97
11-I	21	20.2	103	59,18	916	-0.184	0.95
11-II	21	21.5	103	59,18	748	-0.216	0.84
11-III	21	18.5	103	59,18	1263	-0.090	0.99

TABLE B.2 (continued) - Georgia Type 2 Soils (Santha, 1994)

Soil ID	W_{opt} (%)	W (%)	γ_d (pcf)	*LL,PI*	K_{14}	K_{15}	R^2
12-I	16	15.5	110	30,12	541	-0.414	0.89
12-II	16	17.1	110	30,12	310	-0.501	0.98
12-III	16	13.6	110	30,12	808	-0.274	0.86
13-I	18	19.1	111	34,14	967	-0.109	0.89
13-II	18	20.5	111	34,14	734	-0.176	0.82
13-III	18	17.7	111	34,14	1181	-0.068	0.98
14-I	22	19.9	101	39,14	560	-0.221	0.91
14-II	22	21.7	101	39,14	442	-0.262	0.93
14-III	22	18.5	101	39,14	691	-0.206	0.95

APPENDIX C

SOIL INDEX PROPERTIES AND K_n CONSTANTS FOR SIX TYPE 1 SOILS AND ELEVEN TYPE 2 FROM MAINE (LAW ENGINEERING, 1992).

TABLE C.1 - Maine Type 1 Soils (Law Engineering, 1992)

Source	Classification	W (%)	γ_d (pcf)	K_8	K_9	K_{11}	R^2
South Freeport	A-1-a	13.2	111.3	2105	0.09079	0.75476	0.985
Damariscotta	A-1-a	5.9	133.1	3863	0.15436	0.56229	0.997
Damariscotta	A-1-a	6.7	127.4	1914	0.23975	0.67971	0.992
Bethel	A-1-a	6.3	118.9	3771	0.14196	0.58948	0.991
North Freeport	A-1-a	8.3	123.4	2483	0.20555	0.62645	0.984
North Freeport	A-1-a	9.0	121.7	4240	0.10076	0.60387	0.997

TABLE C.2 - Maine Type 2 Soils (Law Engineering, 1992)

Source	Classification	W (%)	γ_d (pcf)	K_{16}	K_{17}	K_{19}	R^2
South Freeport	A-7-6	8.0	120.6	3374	0.17833	0.48778	0.977
South Freeport	A-7-6	14.7	105.9	3696	-0.04203	0.58782	0.959
Topsham	A-1-b	11.3	108.4	3558	0.12004	0.53467	0.955
Topsham	A-1-b	12.4	107.6	3289	0.07636	0.64159	0.994
Brunswick	A-2-4	9.8	115.5	4458	0.04826	0.46286	0.930
Brunswick	A-1-b	6.6	113.3	3690	0.12091	0.55820	0.998
Brunswick	A-2-4	6.2	114.9	3638	0.11653	0.53500	0.976
Wilton	A-4	6.2	118.9	4619	0.04164	0.53164	0.976
North Freeport	A-2-*	2.8	115.9	3695	0.16353	0.53579	0.978
North Freeport	A-2-*	6.0	122.0	3726	0.18735	0.52423	0.965
Damariscotta	A-4	5.6	111.6	4039	0.04382	0.53631	0.978

*Plasticity data to further classify these speciens was not available. The classifications shown are reflective of grain size distribution only.

APPENDIX D

SOIL INDEX PROPERTIES AND K_n CONSTANTS FOR EIGHT TYPE 2 SOILS FROM TENNESSEE (DRUMM et al., 1993).

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TABLE D.1 - Tennessee Type 2 Soils (Drumm et al., 1993)

Classification and Source	Specimen	W (%)	γ_d (pcf)	K_{12}	K_{13}
A-7-6	1	39.4	82.5	6.0673	-0.3692
	2	39.4	79.6	6.7198	-0.3839
	3	32.4	82.5	8.6169	-0.0251
	4	32.4	82.4	12.2786	-0.3909
	5	35.7	82.2	9.0876	-0.4016
	6	35.7	79.4	8.3214	-0.3859
	7	34.1	77.2	7.7577	-0.1178
	8	34.1	77.0	11.9046	-0.3883
	9	32.1	79.7	10.6158	0.0713
	10	32.1	85.0	12.2780	-0.0464

TABLE D.1 (continued) - Tennessee Type 2 Soils (Drumm et al., 1993)

Classification	Specimen	W (%)	γ_d (pcf)	K_{12}	K_{13}
A-7-5	1	29.8	87.1	11.6387	-0.2692
	2	29.8	86.4	8.7345	-0.2545
	3	26.3	88.5	14.2715	-0.2937
	4	26.3	84.9	12.2416	-0.0718
	5	33.6	86.0	5.8074	-0.0804
	6	33.6	87.7	8.5143	-0.2962
	7	29.6	91.0	9.8835	-0.1710
	8	29.6	91.9	12.0642	-0.0728
	9	25.6	90.2	15.6214	0.0948
	10	25.6	87.0	11.5623	-0.1066

TABLE D.1 (continued) - Tennessee Type 2 Soils (Drumm et al., 1993)

Classification	Specimen	W (%)	γ_d (pcf)	K_{12}	K_{13}
A-7-6	1	105.9	17.7	14.9126	-0.2107
	2	101.4	17.7	11.9616	-0.1824
	3	100.5	14.1	9.5198	-0.2714
	4	99.6	14.1	11.7531	-0.279
	5	101.3	18.6	15.2328	-0.4319
	7	106.8	18.6	15.3235	-0.1423
	8	102.7	22.3	10.5641	-0.6355
	9	101.6	22.3	10.4411	-0.4992
	10	99.7	14.2	7.9052	0.0503
	11	107.5	14.2	6.9751	0.1664

TABLE D.1 (continued) - Tennessee Type 2 Soils (Drumm et al., 1993)

Classification	Specimen	W (%)	γ_d (pcf)	K_{12}	K_{13}
A-6	3	19.5	104.8	6.1696	-0.0992
	6	14.2	102.6	5.4862	0.0421
	7	17.3	108.6	5.9035	-0.0706
	8	17.3	111.2	9.2982	-0.3055
	9	17.9	103.1	6.4020	-0.2759
	10	17.9	110.3	10.0081	-0.1895
	11	21.1	105.3	3.6634	-0.2440
	12	21.1	107.8	3.7818	-0.2242
	13	13.8	97.1	7.0443	-0.2431
	14	18.8	103.9	11.4633	-0.3230
	15	13.1	101.5	8.4349	-0.2997
	16	13.1	104.7	7.6012	-0.2828

TABLE D.1 (continued) - Tennessee Type 2 Soils (Drumm et al., 1993)

Classification	Specimen	W (%)	γ_d (pcf)	K_{12}	K_{13}
A-6 (cont.)	17	13.3	104.3	9.1826	-0.3281
	18	13.3	110.2	8.5724	-0.0653
A-4	1	13.5	115.5	8.7099	-0.3346
	2	13.5	115.5	8.9439	-0.4173
	3	10.1	113.8	6.2427	0.1805
	4	10.1	114.1	8.1582	-0.47
	5	13.8	116.4	9.9482	-0.6313
	6	10.1	112.4	5.7433	0.4131
	7	15.0	117.6	1.3262	0.0328
	8	15.0	113.3	2.4095	0.0025
	9	13.5	113.0	6.9803	-0.1547

TABLE D.1 (continued) - Tennessee Type 2 Soils (Drumm et al., 1993)

Classification	Specimen	W (%)	γ_d (pcf)	K_{12}	K_{13}
A-4	1	20.8	101.4	3.5627	0.0706
	2	20.8	101.2	2.8263	0.1499
	3	12.8	97.9	5.1024	0.0322
	4	12.8	101.8	5.3168	0.1151
	5	18.1	101.6	5.4660	0.0298
	6	18.1	101.9	6.1178	-0.0202
	7	20.7	100.9	5.2426	0.0141
	8	20.7	102.4	5.0755	0.0164
	9	17.5	97.4	5.1424	-0.0045
A-6	1	17.4	106.6	10.6955	-0.1640
	2	17.4	105.6	7.1393	0.0028
	3	18.1	105.1	11.1668	-0.409

TABLE D.1 (continued) - Tennessee Type 2 Soils (Drumm et al., 1993)

Classification	Specimen	W (%)	γ_d (pcf)	K_{12}	K_{13}
A-6 (cont.)	4	13.7	99.8	10.2252	-0.1166
	5	14.0	105.2	8.8657	0.3690
	6	22.2	102.5	3.6812	-0.2818
	7	22.2	101.2	4.7011	-0.3077
	8	18.3	100.5	5.6919	-0.1058
A-7-6	1	16.2	98.1	10.780	0.049
	2	16.2	94.1	8.610	0.128
	3	21	100	8.695	-0.298
	4	21	93.5	15.770	0.064
	5	28.2	92.0	9.835	0.011

APPENDIX E

MAINE FALLING WEIGHT DEFLECTOMETER DATA FROM DAMARISCOTTA, SOUTH FREEPORT, BRUNSWICK, AND NORTH FREEPORT.

TABLE E.1 - 231009 Damariscotta Falling Weight Deflectometer Data

Section begins 1.1 miles North of intersection of Route 1 with Business Route 1 North of Damariscotta.

TABLE E.1.1 - Average of Stations 0+00 to 1+00

Load (kPa)	392	577	771	1017
#1 (microns)	179	256	330	411
#2	149	214	275	342
#3	127	183	235	292
#4	98	143	185	231
#5	75	110	144	182
#6	45	66	88	113
#7	20	29	39	51

TABLE E.1.2 - Average of Stations 1+25 to 2+00

Load (kPa)	390	569	765	1011
#1 (microns)	188	273	357	447
#2	162	235	306	383
#3	141	207	269	338
#4	114	168	221	279
#5	91	136	180	228
#6	59	89	119	153
#7	28	41	56	73

TABLE E.1.3 - Average of Stations 2+25 to 3+00

Load (kPa)	384	566	760	1005
#1 (microns)	189	277	362	454
#2	161	237	309	387
#3	140	207	270	339
#4	113	167	220	278
#5	89	134	177	225
#6	57	86	115	148
#7	26	39	53	69

TABLE E.1 (continued) - 231009 Damariscotta Falling Weight Deflectometer Data

TABLE E.1.4 - Average of Stations 3+25 to 4+00

Load (kPa)	387	564	760	1008
#1 (microns)	178	261	343	432
#2	151	223	292	368
#3	132	194	255	322
#4	105	157	208	263
#5	83	125	167	213
#6	53	80	108	140
#7	25	36	50	66

TABLE E.1.5 - Average of Stations 4+25 to 5+00

Load (kPa)	387	565	759	1007
#1 (microns)	174	253	329	414
#2	148	216	280	352
#3	129	189	245	308
#4	104	152	199	252
#5	82	122	161	204
#6	52	78	104	134
#7	24	35	48	63

TABLE E.1.6 - Layer Details

Layer	Material	Mean Thickness (mm)
original surface	dense graded, hot mixed, hot laid AC	76.2
AC layer binder course	dense graded, hot laid central plant mix	76.2
base	crushed stone, gravel, or slag	101.6
subbase	sand	508
subgrade soil	n/a	n/a

TABLE E.2 - 231012 South Freeport Falling Weight Deflectometer Data

Section begins 0.62 miles South of "Desert of Maine" road overpass and proceeds south for 500 feet.

TABLE E.2.1 - Average of Stations 0+00 to 1+00

Load (kPa)	385	578	781	1028
#1 (microns)	140	212	282	355
#2	117	178	235	295
#3	103	158	210	263
#4	86	133	178	223
#5	71	111	149	188
#6	48	76	104	132
#7	25	40	56	72

TABLE E.2.2 - Average of Stations 1+25 to 2+00

Load (kPa)	436	542	744	985
#1 (microns)	160	202	272	345
#2	133	169	227	287
#3	118	149	202	256
#4	98	125	170	217
#5	81	103	141	181
#6	55	70	97	126
#7	29	37	51	68

TABLE E.2.3 - Average of Stations 2+25 to 3+00

Load (kPa)	478	530	732	972
#1 (microns)	173	194	260	329
#2	143	159	212	266
#3	125	139	186	234
#4	102	113	152	193
#5	82	90	122	156
#6	53	56	78	101
#7	25	24	35	46

TABLE E.2 (continued) - 231012 South Freeport Falling Weight Deflectometer Data

TABLE E.2.4 - Average of Stations 3+25 to 4+00

Load (kPa)	378	567	774	1021
#1 (microns)	144	214	281	352
#2	118	174	227	283
#3	102	152	199	248
#4	82	123	162	203
#5	64	97	129	163
#6	38	60	81	104
#7	16	24	34	46

TABLE E.2.5 - Average of Stations 4+25 to 5+00

Load (kPa)	376	569	773	1026
#1 (microns)	135	198	257	323
#2	106	154	199	249
#3	90	132	171	213
#4	71	105	137	171
#5	54	81	106	134
#6	31	47	63	81
#7	11	17	24	31

TABLE E.2.6 - Layer Details

Layer	Material	Mean Thickness (mm)
original surface	dense graded, hot mixed, hot laid AC	30.5
AC layer binder course	dense graded, hot laid central plant mix	210.8
base	crushed stone, gravel, or slag	330.2
subbase	sand	419.1
subgrade soil	n/a	n/a

TABLE E.3 - 233013 Brunswick Falling Weight Deflectometer Data

Section begins 3.64 miles North of Brunswick-Freeport town line.

TABLE E.3.1 - Average of Stations 0+19 to 0+91

Load (kPa)	n/a	570	777	1010
#1 (microns)	n/a	132	182	229
#2	n/a	126	170	215
#3	n/a	123	166	209
#4	n/a	114	154	195
#5	n/a	106	143	181
#6	n/a	90	122	154
#7	n/a	62	83	106

TABLE E.3.2 - Average of Stations 1+19 to 1+92

Load (kPa)	n/a	530	732	972
#1 (microns)	n/a	194	260	329
#2	n/a	159	212	266
#3	n/a	139	186	234
#4	n/a	113	152	193
#5	n/a	90	122	156
#6	n/a	56	78	101
#7	n/a	24	35	46

TABLE E.3.3 - Average of Stations 2+20 to 3+95

Load (kPa)	n/a	568	776	1008
#1 (microns)	n/a	121	167	212
#2	n/a	115	157	199
#3	n/a	112	153	194
#4	n/a	104	142	181
#5	n/a	98	133	169
#6	n/a	83	113	144
#7	n/a	58	79	100

TABLE E.3 (continued) - 233013 Brunswick Falling Weight Deflectometer Data

TABLE E.3.4 - Average of Stations 3+21 to 3+98

Load (kPa)	n/a	565	776	1008
#1 (microns)	n/a	118	163	208
#2	n/a	112	153	195
#3	n/a	109	149	190
#4	n/a	102	139	177
#5	n/a	95	130	166
#6	n/a	81	110	141
#7	n/a	56	77	98

TABLE E.3.5 - Average of Stations 4+21 to 4+74

Load (kPa)	n/a	566	777	1008
#1 (microns)	n/a	118	164	209
#2	n/a	112	153	196
#3	n/a	109	150	191
#4	n/a	101	139	178
#5	n/a	94	129	166
#6	n/a	80	110	140
#7	n/a	55	75	96

TABLE E.3.6 - Layer Details

Layer	Material	Mean Thickness (mm)
original surface	Portland cement concrete	254
base	crushed stone, gravel, or slag	76.2
subbase	gravel (uncrushed)	508
subgrade soil	n/a	n/a

TABLE E.4 - 237023 North Freeport Falling Weight Deflectometer Data

Section begins 5.11 miles South of Durham Road overpass and 2.94 miles South of Freeport-Brunswick line.

TABLE E.4.1 - Average of Stations 0+00 to 1+00

Load (kPa)	388	585	786	1030
#1 (microns)	101	158	216	276
#2	79	123	167	213
#3	74	116	157	200
#4	69	109	147	188
#5	63	100	135	173
#6	51	81	109	140
#7	30	49	66	85

TABLE E.4.2 - Average of Stations 1+25 to 2+00

Load (kPa)	384	579	781	1025
#1 (microns)	104	163	222	284
#2	79	124	168	215
#3	74	116	157	201
#4	69	109	147	188
#5	63	100	136	174
#6	51	81	111	142
#7	32	51	69	89

TABLE E.4.3 - Average of Stations 2+25 to 3+00

Load (kPa)	382	574	779	1026
#1 (microns)	104	161	220	281
#2	78	121	164	210
#3	73	113	153	195
#4	68	105	143	182
#5	62	96	131	167
#6	49	77	105	134
#7	30	47	65	83

TABLE E.4 - 237023 North Freeport Falling Weight Deflectometer Data

TABLE E.4.4 - Average of Stations 3+25 to 4+00

Load (kPa)	384	578	780	1027
#1 (microns)	117	178	240	304
#2	89	136	182	230
#3	83	127	169	214
#4	76	118	157	199
#5	69	107	143	181
#6	55	85	115	145
#7	32	51	69	88

TABLE E.4.5 - Average of Stations 4+25 to 5+00

Load (kPa)	383	577	780	1028
#1 (microns)	102	157	213	273
#2	73	113	153	197
#3	68	106	143	184
#4	63	99	134	172
#5	58	90	123	158
#6	46	72	98	127
#7	27	43	58	26

TABLE E.4.6 - Layer Details

Layer	Material	Mean Thickness (mm)
overlay	dense graded, hot mixed, hot laid AC	30.5
AC layer binder course	dense graded, hot laid, central plant mix	71.1
original surface	Portland cement concrete	203.2
base	crushed stone, gravel, or slag	152.4
subbase	gravel (uncrushed)	609.6
subgrade soil	n/a	n/a

APPENDIX F

SOIL INDEX PROPERTY DATA FOR SIX TYPE 1 SOILS AND NINE TYPE 2 SOILS FROM MAINE (LAW ENGINEERING, 1992)

TABLE F.1 - Soil index property data for six type 1 soils from Maine (Law Engineering, 1992)

SHRP ID	231009 BABG	231009 TPBG55	231012 TPBG56	231028 BA1BS01	237023 BABG-1	237023 BABG-2
Location	Damariscotta	Damariscotta	South Freeport	Bethel	North Freeport	North Freeport
Classification	A-1-a	A-1-a	A-1-a	A-1-a	A-1-a	A-1-a
Material Type	1	1	1	1	1	1
Layer	3	3	2	1	2	2
G_s	2.720	2.652	2.697	2.663	2.662	2.655
γ_{ds} (pcf)	133.1	127.4	111.3	118.9	123.4	121.7
γ_t (pcf)	141	136	126	126	133.6	132.7
γ_{dmax} (pcf)	n/a	135	119	125	130	129
W (%)	5.9	6.7	13.2	6.3	8.3	9.0
W_{opt} (%)	n/a	6	11	6	8	8
S (%)	58.3	59.5	69.5	42.1	63.8	66.2
% pass 3"	100	100	100	100	100	100
% pass 2"	92	97	100	100	99	99
% pass 1 ½"	88	97	100	98	96	96
% pass 1"	83	89	99	98	89	90
% pass ¾"	80	82	99	94	84	86
% pass ½"	74	72	97	85	78	81
% pass 3/8"	70	65	96	79	73	77
% pass #4	59	49	93	67	65	71
% pass #10	45	33	87	54	55	62
% pass #40	20	14	52	16	25	31
% pass #80	11	8	19	4	10	12
% pass # 200	5.6	4.7	3.2	2	4.5	4.6
C_u	26.8	20	4.0	8.2	7.4	13.3
C_c	0.597	1.250	1.0	0.51	0.667	0.533
M_R (psi) σ_{cyclic} =15-psi, σ_3=15-psi	26394	23044	22291	34194*	21664*	31211

- Specimens exceeded maximum strain limit before completing testing sequences. The M_R values shown are at σ_{cyclic} =10-psi and σ_3=15-psi.

TABLE F.2 - Soil index property data for nine type 2 soils from Maine (Law Engineering, 1992)

SHRP ID	231009 TPBS55	231012 TPBS55	231026 TPBS55	233013 BABS-1	233013 BABS-2	233014 BA5BS06
Location	Damariscotta	South Freeport	Wilton	Brunswick	Brunswick	Topsham
Classification	A-4	A-7-6	A-4	A-2-4	A-2-4	A-1-b
Material Type	2	2	2	2	2	2
Layer	1	1	1	1	2	1
G_s	2.728	2.909	2.782	2.685	2.787	2.717
γ_{ds} (pcf)	111.6	105.9	118.5	115.5	114.9	108.7
γ_t (pcf)	117.9	121.4	125.9	126.8	122.0	120.7
γ_{dmax} (pcf)	115	110	120	121	120	n/a
W (%)	5.6	14.7	6.2	9.8	6.2	11.3
W_{opt} (%)	7	12	10	10	6	n/a
S (%)	29.1	59.8	37.1	58.4	33.5	54.4
% pass 3"	100	100	100	100	100	n/a
% pass 2"	99	100	100	100	100	n/a
% pass 1 ½"	99	100	99	100	100	n/a
% pass 1"	97	100	97	99	99	n/a
% pass ¾"	96	100	97	99	98	n/a
% pass ½"	94	100	95	99	96	n/a
% pass 3/8"	93	100	93	98	95	n/a
% pass #4	89	100	90	97	93	n/a
% pass #10	83	100	84	95	87	n/a
% pass #40	59	98	58	74	53	n/a
% pass #80	24	75	32	44	22	n/a
% pass # 200	6	26.1	12.6	32.3	7.1	n/a
C_u	3.5	1.8	5.4	3.5	4.7	n/a
C_c	2.259	0.753	0.989	0.289	1.176	n/a
M_R (psi) σ_d =6-psi, σ_3=4-psi	9977	8956	11283	10036	10085	10304

TABLE F.2 (continued) - Soil index property data for nine type 2 soils from Maine (Law Engineering, 1992)

SHRP ID	233014 BABS	237023 BABS-1	237023 BABS-1
Location	Topsham	North Freeport	North Freeport
Classification	A-1-b	A-2-?	A-2-?
Material Type	2	2	2
Layer	1	1	1
G_s	2.714	2.701	2.706
γ_{ds} (pcf)	107.6	122	115.9
γ_t (pcf)	120.9	129.3	119.2
γ_{dmax} (pcf)	113	128	121
W (%)	12.4	6	2.8
W_{opt} (%)	12	6	3
S (%)	58.4	42.2	16.6
% pass 3"	100	100	100
% pass 2"	100	100	100
% pass 1 ½"	100	100	98
% pass 1"	100	95	95
% pass ¾"	100	94	93
% pass ½"	99	92	91
% pass 3/8"	98	90	88
% pass #4	97	86	82
% pass #10	93	80	73
% pass #40	77	43	33
% pass #80	40	15	12
% pass # 200	11.5	4.6	4.6
C_u	3.1	8.3	5.4
C_c	1.108	2.083	0.857
M_R (psi) σ_d =6-psi, σ_3=4-psi	10234	11302	13740