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Full Length Research Paper

Prediction of subgrade resilient modulus for flexible pavement design

Mehmet Ridvan Ozel[1]* and Abbas Mohajerani[2]

[1]Department of Civil Engineering, School of Engineering and Architecture, Gediz University, Izmir, Turkey.
[2]School of Civil, Environmental and Chemical Engineering, RMIT University, Melbourne, Australia.

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Resilient modulus of subgrade soils is an important input in mechanistic pavement design. The primary objective of this work is to investigate the resilient modulus of four typical Victorian fine-grained subgrade soils under traffic-like repeated loading and to suggest empirical predictive models incorporating physical properties and/or strength of the soils along with the stress state. A repeated load triaxial testing procedure was developed, which is capable of collecting resilient and permanent deformation data from the same specimen. Stress levels for testing were defined as percentages of the confined and/or unconfined soil static strengths. Stress dependency of resilient modulus was studied through the models (such as bilinear model, power model, deviatoris stress model and octahedral stress model) found in the literature and other possible combinations of deviator, confining and octahedral stresses. A semi-logarithmic model was proposed for the prediction of resilient modulus of the fine-grained subgrade soils. Calibration of model constants by soil properties was investigated. An altervative prediction model was also developed based on unconfined compressive strength and deviator stress. Resilient modulus values were back calculated using both the semi-logarithmic model and the model based on unconfined compressive strength and deviator stress. Predicted values were compared with the measured values. Predictive capability of the proposed models were proven for use in flexible pavement design.

Key words: Resilient modulus, fine-grained subgrade soils, repeated loading, flexible pavement design.

INTRODUCTION

Subgrade soil characterization is an important stage of the mechanistic pavement design procedures for flexible pavements (AASHTO, 1993; AUSTROADS, 1992). These procedures are mainly based on analyzing the response of the pavement materials under simulated traffic loads and environmental conditions. The complex elasto-plastic deformational response of pavement materials is studied in two categories in order to simplify the task: resilient strain (ε_r) and permanent strain (ε_p). The resilient deformational response of pavement subgrades under repeated loads is the scope of this study. Resilient response of subgrades can be quantified by resilient modulus (E_r) that is a stress-strain relationship like modulus of elasticity. However, E_r is determined from a repeated load triaxial compression test (RLTT) and is based on only the recoverable portion of the strain (Yoder and Witczak, 1975; Elliot and Thornton, 1988). It is expressed as the ratio of axial repeated deviator stress (σ_d) to the recoverable axial strain (ε_r) and considered as an indication of load-carrying capacity of the subgrades as given in Equation (1) (Seed et al., 1962):

$$E_r = \sigma_d / \varepsilon_r \qquad (1)$$

The Austroads pavement design guide recommends the use of RLTT for determining the "modulus" of subgrades (AUSTROADS, 1992). However, in practice, due to the testing equipment needs and complexities involved in the

*Corresponding author. E-mail: ridvan.ozel@gediz.edu.tr, ridvanozel@gmail.com. Tel: 90-232-355-0000. Fax: 909-232-425-6813.

test, empirical relationships are generally used to estimate the E_r of the pavement subgrade soils. The empirical relationship based on California bearing ratio (CBR), which was first introduced by Heukelom and Klomp (1962), is still the most common tool for pavement engineers. The relationship is given below:

$$E_r = 10\times CBR \text{ (MPa)}. \quad (2)$$

Powell et al. (1984) developed another Er–CBR expression in the Transport and Road Research Laboratory (TRLL) as shown below:

$$E_r = 17.6\times CBR^{0.64} \text{ (MPa)}. \quad (3)$$

Nevertheless, these relationships should be used with caution because of the quasi-static nature of the CBR test and the absence of stress parameters in the expressions. Furthermore, their accuracy has already been examined and questioned in the works of other researchers like Thomson and Robnett (1976, 1979). Predicting a dynamic property of the soils from their CBR values, which is actually a measure of static shear strength of the material, may not always yield the correct results.

The main objective of this study is to investigate the E_r of some Victorian fine-grained subgrade soils under traffic-like repeated loading and to suggest empirical predictive models incorporating physical properties and/or strength of the soils along with the stress state. A laboratory repeated load triaxial testing program for E_r, which also allows collecting permanent strain data, has been developed in this study. Four typical fine-grained subgrade soils are chosen since most local research works have been concentrated on granular soils despite the fact that fine-grained soils have a wide surface coverage across Victoria and Australia (Nataatmadja and Parkin, 1989; Symons and Poli, 1996; Moffatt et al., 1998; Chen, 1999; Lo and Chen, 1999; Nataatmadja and Tan, 2001; Bodhinayake, 2008).

RESILIENT MODULUS PREDICTION

Stress dependency of resilient modulus (E_r) of fine-grained soils has been defined by many researchers and some mathematical models have been suggested. E_r is generally expressed in terms of applied deviator stress (σ_d). Fredlund et al. (1977) adopted a semi-logarithmic model for E_r and σ_d relationship and found consistent results for a fine-grained soil (moraine glacial till) of Canada. The model is expressed as:

$$\log E_r = k_1 - k_2\, \sigma_d, \quad (4)$$

Where k_1 and k_2 are material constants that depend on soil type and soil physical properties. Thompson and Robnett (1979) showed that there is a bilinear relationship between E_r and σ_d. This model is given as follows:

$$E_r = k_3 + k_4\, \sigma_d \quad \text{when } \sigma_d < \sigma_{di}, \quad (5a)$$

$$E_r = k_5 + k_6\, \sigma_d \quad \text{when } \sigma_d > \sigma_{di}, \quad (5b)$$

Where σ_{di} is the deviator stress where two fitted linear lines of the E_r-σ_d graph intersect and k_3, k_4, k_5 and k_6 are material constants. Another form of relationship between E_r and σ_d was proposed in the study of Moossazadeh and Witchzak (1981), in which three fine-grained soils were investigated. The model is known as power model or deviatoric stress model and written as follows:

$$E_r = k_7\, \sigma_d^{k8}, \quad (6)$$

Witczak and Uzan (1988) used octahedral stress attributes for modeling the behavior of some granular soils and later other researchers (Houston et al., 1993; Puppala et al., 1996; Mohammad et al., 1999; Ozel and Mohajerani, 2001) adopted this model in their study and proved its predictive capability for some fine-grained soils. This model, which was originally derived by Shackel (1973), is considered more appropriate than the models incorporating only the deviator stress, since it accounts for both lateral and vertical stresses in three dimensions. The general format of octahedral stress model can be expressed as:

$$E_r = k_9\, (\sigma_{oct})^{k10}\, (\tau_{oct})^{k11}, \quad (7)$$

Where σ_{oct} is octahedral normal stress; τ_{oct} is octahedral shear stress; and k_9, k_{10}, and k_{11} are material constants. Drumm et al. (1990), on the other hand, examined the resilient response of 11 Tennessee fine-grained soils and expressed the E_r as below:

$$E_r = [k_{12} + k_{13}\, \sigma_d] / \sigma_d. \quad (8)$$

All these models take only stress state into account and the model constants need to be calibrated for local soil conditions such as soil type, grading characteristics and moisture-density state.

There are also several investigations in which the E_r (under a specific stress state) is directly modeled in terms of the soil physical and strength properties (Thompson and Robnett, 1979; Thompson and LaGrow, 1988; Elliot and Thornton, 1988; Drumm et al., 1990). In these studies, regression equations were established between E_r and various soil properties such as liquid limit, plasticity index, clay content, organic carbon content, degree of saturation, dry density, percent passing 75 micron sieve, moisture content, California bearing ratio and unconfined compressive strength. Nevertheless, the common drawback of these models is that the stress dependency

Table 1. Basic properties of the experimental soils.

Soil physical properties	S1	S2	S3	S4
Unified soil classification	CH	CL	CH	CH
Clay content (%)	70	46	71	54
Liquid limit (%)	66	34	77	66
Plasticity index	35	11	50	48
Max dry density (mg/m^3)	1.45	1.72	1.32	1.53
Optimum moisture content (%)	29.5	18.5	33	27
Specific gravity	2.75	2.72	2.64	2.61

of E_r is not included.

EXPERIMENTAL DESIGN

Soils tested

Four Victorian subgrade soils were selected for inclusion in the study. These materials were obtained from a local street roadbed in Narre Warren South, where it is known as the Baxter Sandstone of Brighton Group soils (S1); Eastern Freeway extension project, Mitcham (S2); Geelong Road extension project, Altona (S3) and from Pascoe Vale Road duplication project, Broadmeadows (S4). Basic soil tests such as specific gravity, Atterberg limits, particle size analysis and compaction were performed in accordance with the relevant Australian Standards. Compaction characteristics were determined in the laboratory delivering the standard compactive effort. Some of the physical properties of the soils are presented in Table 1.

Testing equipment and sample preparation

A Universal Testing Machine (UTM-5P) was employed to perform repeated load triaxial tests (RLTT). The equipment was a close-loop, fully automatically controlled data acquisition system capable of applying repeated dynamic loads through a shaft of a pneumatic actuator (Figure 1). The test specimens were accommodated in a triaxial pressure cell, which is made of perspex cylinder and suitable for testing specimens having dimensions of 200 mm height by 100 mm diameter. The triaxial pressure cell is fitted for water as a confining medium. The loading pulse duration and rest period were selected 0.5 and 1 s respectively and axial deformations were measured using three linear variable differential transducers (LVDT) mounted on clamps outside the triaxial chamber. Three-stage unconsolidated undrained (UU) triaxial tests (Head, 1982) were performed by using a computer controlled triaxial testing equipment. Tests were conducted on the cylindrical specimens of 38 by 76 mm having the same initial conditions with the RLTT specimens. UTM-5P test machine was also used to determine the unconfined compressive strength (q_u) of the specimens. The specimens for RLTT's and unconfined compressive tests (UCS) were 101.5 mm in diameter and 202 mm in height.

The disturbed soils were processed in accordance with the relevant Australian standard. Representative samples obtained from the processed soil were first mixed with water and allowed to cure for 48 h before compaction, so that the water could completely wet up the clayey soil. Mixtures were kept in sealed plastic bags during waiting periods. All samples were compacted by delivering standard compaction energy. Static triaxial test specimens of Narre Warren soil were produced by extruding out of the soil compacted in a standard proctor mould. Saturated porous stones were used on top and bottom for the RLTT's and UCS tests samples. A rubber membrane was placed over the specimen and secured to the top and bottom with o-rings to fully seal the specimen against pressurized confining water during the RLTT's.

METHODS

The experimental program requires the preparations of 12 RLTT samples for each soil at three different levels of moisture content, which is thought to be the most important physical property affecting the deformational characteristics of fine-grained subgrade soils. Optimum, 2% dry side of optimum and 2% wet of optimum were chosen as testing moisture content levels. As Australian standard for repeated load triaxial test does not cover the testing of subgarde soils, an original procedure was developed in this study. RLTT procedure was designed to allow ε_p and E_r data being collected from the same specimen (Ozel, 2003). This was achieved by extending the conditioning stage up to 10,000 load repetitions, in which the permanent strain data were collected and continuing with the stress-stage test, in which various combination (15 stress levels) of confining (σ_3) and deviator stresses (σ_d) were applied for the investigation of resilient behavior of the materials (Table 2). Three levels of σ_3 were applied. The first conditioning stage of 10,000 is called permanent strain (ε_p) test and the second phase is called resilient modulus (E_r) test here. Some findings of permanent deformation part of the study (the data collected and analyzed during the first 10,000 cycles of the RLTT's) were published elsewhere (Ozel and Mohajerani, 2002). Because different deviator stress levels (DSL) were applied during the ε_p test, a transition stage, which had DSL of 0.7, was placed between the ε_p and E_r tests, in order to bring the specimens to the same deformational status before E_r test begins. Stress-strength ratio in this study is called DSL and can be defined as the ratio of the σ_d of RLTT to the soil strength obtained from UU triaxial or UCS test. There are two main advantages of defining stress levels through the DSL approach: first, keeping the stresses always under the soil failure envelope and the second, testing the compacted soil close to its working stresses. The stress levels developed is given in the Table 2. After completing E_r test, confining pressure released and specimens were tested under quasi-static loading to determine their q_u's (after RLTT q_u values).

Since the σ_d for RLTT in the experimental program are defined by soil strengths, three-stage unconsolidated undrained (UU) triaxial tests (Head, 1982) were first conducted for Narre Warren soil. Soil static confined compressive strengths (σ_{ss}) were determined as the achieved deviator stresses for σ_3 of 15, 30 and 45 kPa. The deviator stresses for RLTT were then calculated as certain percentages of the soil static strengths, which are shown in the Table 2. However, having found not much difference between σ_{ss}

Figure 1. Repeated loading triaxial cell and soil specimen.

(for relatively low values of applied σ_3 values) and q_u values for Narre Warren soil (Table 3), UCS test was decided to employ for Mitcham, Altona and Broadmeadows soils in determining soil strengths and subsequently calculating the σ_d's for RLTT. UCS test results of these soils can be seen in Table 4.

ANALYSIS OF RESULTS

Model for resilient modulus with stress state and soil properties

The influence of stress state on E_r and stress state models for fine-grained soils found in the literature are summarized previously. Regression analyses were performed using these models for the data obtained in this study. Although the models provided reasonably good agreement having coefficient of determination (R^2) ranging from 0.60 to 0.90, different combinations of σ_d, σ_3, σ_{oct} and τ_{oct} with E_r were also investigated before adopting any of the available models for the E_r data of this study. These further analyses have been performed due to the known deficiencies of the models found in the literature, which are the absence of σ_3 effects in the case of σ_d models and collinearity problem for σ_{oct} and τ_{oct} models. The models with the only variable of σ_d (Equations 4, 5 and 6) are criticized elsewhere for not including the effect of lateral pressure on E_r (Houston et al., 1993; Puppala et al., 1996; Muhanna et al., 1998; Ozel and Mohejerani, 2001). These researchers revealed the effect of σ_3 on E_r for fine-grained soils. Therefore this

Table 2. Stress levels for RLTT.

Test name		Stress level σ_3[¶]	DSL[§] (σ_d[†]/σ_{ss}[#])	DSL ($\sigma d/qu$[±])	N[*]
Permanent strain		30	0.3~0.9	0.3~0.9	10,000
Transition stage		30	0.7	0.7	200
	1	45	0.15	0.3	200
	2	45	0.30	0.4	200
	3	45	0.45	0.5	200
	4	45	0.60	0.6	200
	5	45	0.75	0.7	200
	6	30	0.15	0.2	200
	7	30	0.30	0.3	200
Resilient modulus	8	30	0.45	0.4	200
	9	30	0.60	0.5	200
	10	30	0.75	0.6	200
	11	15	0.15	0.1	200
	12	15	0.30	0.2	200
	13	15	0.45	0.3	200
	14	15	0.60	0.4	200
	15	15	0.75	0.5	200

¶ confining stress (kPa); § deviator stress level; † deviator stress (kPa); # confined compressive strength for S1 as given in Table 3 (kPa); ± unconfined compressive strength for S2, S3 and S4 as given in Table 4 (kPa); * number of load repetitions.

Table 3. Soil strengths for S1.

RLTT sample number	Moisture state	UU test		UCS test
		σ_3[±]	σ_{ss}[*]	q_u[#]
01 to 04	Dry	30	330	330
		45	280	270
05 to 08	Optimum	30	255	
		15	220	
09 to 12	Wet	30	190	210

± confining pressure used in UU triaxial test (kPa); * unconsolidated undrained triaxial compressive strength (kPa); # unconfined compressive strength (kPa); all values are the average of three test results.

Table 4. Unconfined compressive strengths, q_u, for S2,S3 and S4 soils.

RLTT sample number	Moisture state	q_u (kPa)		
		S2	S3	S4
01 to 04	Dry	200	220	170
05 to 08	Optimum	175	195	145
09 to 12	Wet	120	165	105

All values are the average of three or four tests.

parameter should be in the model.

The octahedral stress model, which is given in Equation 7, includes the effect of σ_3 through octahedral stresses and the model has been found to be the best in representing the behavior of the fine-grained soils reported in this study. However, the constants of

Table 5. Coefficient of determinations (R^2)based on Equation 9.

Sample number	S1	S2	S3	S4
1	0.94	0.75	0.94	0.81
2	0.93	0.71	0.95	0.82
3	0.86	0.64	0.94	0.96
4	0.94	Failed	0.92	Failed
5	Not available	0.43	0.84	0.91
6	0.93	0.64	0.93	0.89
7	0.93	0.52	0.89	0.95
8	0.97	0.54	0.93	Failed
9	0.97	0.75	0.91	0.66
10	0.99	0.48	0.95	0.89
11	0.93	0.56	0.90	0.99
12	Failed	Failed	0.91	0.88

octahedral stress model are determined by multiple regression analysis, which might be subject to collinearity problem. The collinearity becomes the major concern when the "independent" variables of a multiple regression equation are correlated with each other. In the case of octahedral stress model, normal (σ_{oct}) and shear stress (τ_{oct}) parameters (two variables of regression equation) are both derived from the same major and minor principal stresses, resulting strong correlation between them. This means they are not actually independent variables as assumed. The collinearity is evaluated by variance inflation factor (VIF). Most of the VIF values obtained in this study ranged from 5 to 15 for octahedral model. Possible collinearity is suspected with the VIF values above 4 and VIF of 10 or above indicates that there is a high risk of collinearity (Belsley et al., 1980). Therefore, the adoption of the octahedral stress model for this study has been omitted despite of yielding highest R^2 values among the models in the literature.

Having examined the possible combinations of stresses (σ_d, σ_3, σ_{oct} and τ_{oct}) with E_r, the following model is adopted in this study:

$$\log (E_r/P_a) = k_1 + k_2 (\sigma_d/P_a) + k_3 (\sigma_3/P_a), \quad (9)$$

Where P_a is atmospheric pressure (100 kPa), k_1, k_2 and k_3 are material (model) constants. This semi-logarithmic model is an improved form of Equation (4) with the inclusion of the parameter σ_3 and it has not been used elsewhere to predict E_r. The model was normalized with respect to P_a to make it dimensionally consistent. Having compared the model with the ones in the literature, the model developed in this study has two main superiorities: it includes the effect of σ_3 and it does not subject to the collinearity among the variables (σ_d/P_a and σ_3/P_a) as VIF values were smaller than 4 (statistically meaning no possibility of collinearity). Therefore the predictive model as given in Equation (9) has been adopted to model the resilient behavior of the fine-grained soils in this study on the basis that it overcomes the aforementioned deficiencies of the models found in the literature, namely the absence of σ_3 effect and collinearity problem.

By overcoming these two and having obtained acceptable R^2 values with the proposed model, it has been adopted to model the resilient behavior of the fine-grained soils in this study. Table 5 presents the coefficient of determinations *(R^2)* of multiple regression analyses based on Equation (9).

The constants k_1, k_2 and k_3

Since the need for quantification of the material constants is obvious, an extensive multiple regression analyses were also performed between the material constants and some soil physical properties including moisture content (w), dry density (ρ_d), clay content (C) plasticity index (PI), activity (A) and degree of saturation (S). In regression analyses, the more the variables the higher the coefficient of determination (R^2), however, "the more the variables" also mean less practical and less statistically sound expressions. Therefore, the most important and practical factors were included in the final regression equations. In this regard, for instance, degree of saturation (S) is taken into consideration to represent the combined effect of w- ρ_d condition. Another advantage of the use of S in the multiple regression analyses is that the risk of collinearity would be eliminated as w and ρ_d are interrelated. The range of moisture contents used in the derivation of the equations was 2 percent below and above the optimum. This moisture content range resulted the degree of saturation to have changed from 77 to 98%. It is believed that this falls into the range of *in-situ* moisture conditions of Victorian subgrades. The values from optimum to the upper end of the range represents the equilibrium moisture content of the

Table 6. The regression constants of Equation 9.

Soil name	k_1		k_2		k_3	
	Range	Average	Range	Average	Range	Average
S1	2.95 to 3.15	3.09	-0.27 to -0.07	-0.15	0.16 to 0.37	0.29
S2	2.34 to 2.88	2.66	-0.62 to –0.32	-0.43	0.29 to 0.75	0.43
S3	2.75 to 2.92	2.85	-0.18 to –0.09	-0.14	0.08 to 0.18	0.14
S4	2.72 to 2.86	2.78	-0.43 to –0.18	-0.25	0.09 to 0.40	0.25
All soils	2.34 to 3.15	2.87	-0.62 to –0.07	-0.23	0.08 to 0.75	0.27

subgrade, which is eventually reached in the subgrade depending upon the water table level, rain fall, clay type and drainage conditions, while the lower end of the range could be encountered in the road subgrades of semi-arid region of the state. The soils with more moisture contents (beyond the range used in this study) are unlikely to be found, as good drainage system is desirable for modern roads. The use of following equations for moisture contents other than the ones used in the derivations of the equations is not recommended, as the extrapolations may not be valid.

For the four soils investigated in the study, the material constants k_1, k_2 and k_3 can be best defined in terms of some soil physical properties as follows:

$$\text{Log } k_1=0.019-0.468 \times \log w-59 \times \log C+0.073 \times \log PI, \quad R^2 = 0.70, \text{ SEE} = 0.017, \tag{10}$$

$$k_2=0.16-1.14\times\log S+0.856\times\log C+0.195\times\log PI, \quad R^2 = 0.84, \quad \text{SEE} = 0.056, \tag{11}$$

$$k_3=0.37+0.0023\times S-0.0015\times C-0.0058\times PI \quad R^2 = 0.48, \quad \text{SEE} = 0.11. \tag{12}$$

The minimum, maximum and average values of the constants are also given in Table 6. It should be noted that extreme values of the constants were excluded before establishing Equations10, 11 and 12 and the values in Table 6.

To prove the predictive capability of Equation (9) through the calibration of model constants as given in Equations 10, 11 and 12; material constants (k_1, k_2 and k_3) were first calculated using the measured soil physical properties for a total of 46 samples. Then, E_r values were back calculated for the stress levels of 6, 7, 8, 9 and 10 (Table 2). And the measured and predicted E_r values were averaged for each moisture content group. The results were compared as seen in Figure 2. It is known that the chosen stress levels and corresponding σ_3 of 30 kPa represent a meaningful range of stresses that subgrades are likely to be subjected to during the service.

As can be seen from Figure 2, most values are found close to the line of equality with a coefficient of variation 20. Therefore, it can be concluded that Equation 9 is capable of predicting acceptable values of E_r for the experimental soils for a given stress state through the calibration of model constants with respect to w, S, C and PI as given in Equations 10, 11 and 12.

Model for resilient modulus with unconfined compressive strength

Since UCS test is one of the simple soil strength tests that can be done almost in any soil laboratory, several E_r-q_u correlations have been developed in an attempt to facilitate the E_r prediction by performing one simple UCS test. The relevant studies, for example, can be found in the works of Thompson and Robnett (1979) and Lee et al.(1997). However, the correlations found in the literature were developed for a specific stress level. In other words, the role of stress state on E_r is not included in the models. Here, an E_r - q_u correlation development has been attempted, which also accommodates a stress parameter.

To accomplish this, the measured E_r values under five DSL's were first averaged for each moisture content group within each soil. In other words, the E_r values shown in Table 7 were obtained from the stress level of 6, 7, 8, 9 and 10 during the RLTT's (Table 2). Then, simple regression analyses were carried out between q_u results and each set of average E_r values and the relationships were established in the form of following equation (The results can be seen in Table 8):

$$E_r = a + b\, q_u . \tag{12}$$

To be able to include the stress affect into the model, the relationship between the constants a and b with corresponding DSL's have been examined. Both constants have been found to be greatly dependent on DSL. These correlations were formulated by means of simple regression analyses as follows:

$$a = -26.36 + 29.37\times DSL, \quad R^2= 0.82, \quad \text{SEE}=2.54; \tag{13}$$

$$b = 0.632 - 0.5458\times DSL, \quad R^2= 0.98, \quad \text{SEE}=0.013. \tag{14}$$

Table 7. E_r, UCS and CBR test results.

	S1			S2			S3			S4		
	Dry	Optimum	Wet	Dry	Optimum	Wet	Dry	Optimum	Wet	Dry	Optimum	Wet
E_r @DSL=0.2¶	153	135	91	86	64	38	85	81	68	68	60	50
E_r @DSL=0.3¶	139	120	83	67	42	29	80	76	64	63	54	44
E_r @DSL=0.4¶	119	104	65	55	40	20	74	69	62	59	48	43
E_r @DSL=0.5¶	106	89	54	50	34	19	70	63	57	50	44	39
E_r @DSL=0.6¶	94	80	49	49	33	18	66	60	51	45	39	35
q_u* (kPa)	330	270	210	200	175	120	220	195	165	170	145	105
CBR†	19	9	7	24	7	4	17	12	6	10	7	5

¶Measured Er values (MPa) under the given DSL's; ∗unconfined compressive strength; †unsoaked California bearing ratio.

Table 8. E_r - q_u relationship.

	a	b	R^2	SEE
DSL=0.2	-18.05	0.5186	0.95	7.79
DSL=0.3	-21.15	0.4836	0.91	10.08
DSL=0.4	-14.39	0.4038	0.87	10.43
DSL=0.5	-11.06	0.3504	0.85	9.71
DSL=0.6	-8.41	0.3123	0.86	8.22

If a and b in Equation 12 are replaced with the Equations 13 and 14, Equation 15 given as follows is obtained:

$$E_r = 29.370\times DSL + 0.632\times q_u - 0.546\times\sigma_d - 26.360, \quad (15)$$

Where E_r is resilient modulus in Mpa, q_u is unconfined compressive strength in kPa, σ_d is deviator stress on top of the subgrade (kPa) and DSL is the ratio of σ_d to the q_u. This resultant equation is a very easy way of predicting E_r through two simple parameters, which are q_u and σ_d.

To satisfy the usefulness of the Equation 15, E_r values were back calculated using q_u of each condition and the corresponding testing σ_d's. Figure 3 shows that there is a good agreement between measured and predicted E_r values through the established model. The same coefficient of variance of 20 has been obtained for this comparison. Note that the Equation 15 was derived for four fine-grained subgrade soils whose q_u values range from 145 kPa to 275 kPa at optimum moisture content. The predictive capability of the model might lessen or not be valid for fine-grained soils outside the range of q_u values used in this study.

Resilient modulus – California bearing ratio relationship

As mentioned previously, California bearing ratio tests were also performed for each soil at the same three moisture content levels like RLTT and UCS samples. Table 7 also gives the results of these tests. Since the E_r–CBR expressions (like E_r= 10×CBR in Austroads pavement design guide) are recommended in the pavement design procedures, the veracity of such correlations has been investigated for the soils in this study. Having compared E_r values of various stress levels with their CBR's, it has been found that E_r changes by 2 to 15 times of the CBR values depending on the soil type, moisture content and stress level of the subgrade. If the comparison is limited for the samples at wet side of optimum moisture content, however, E_r values fall into a range where they deviate up to 30 percent from the "10xCBR" prediction. Even in this case, E_r prediction through CBR is quite controversial. Therefore it was concluded from the experimental data that E_r cannot be correlated to the CBR of the soils tested. If the "Er= 10×CBR" expression is utilized as suggested in several pavement design guides and still widely used by practitioners, under- or over-estimation of the E_r values would be inevitable. Having checked in the same manner, Equation 3 has also been found unsatisfactory for E_r prediction.

Conclusions

On the basis of the present study, the following

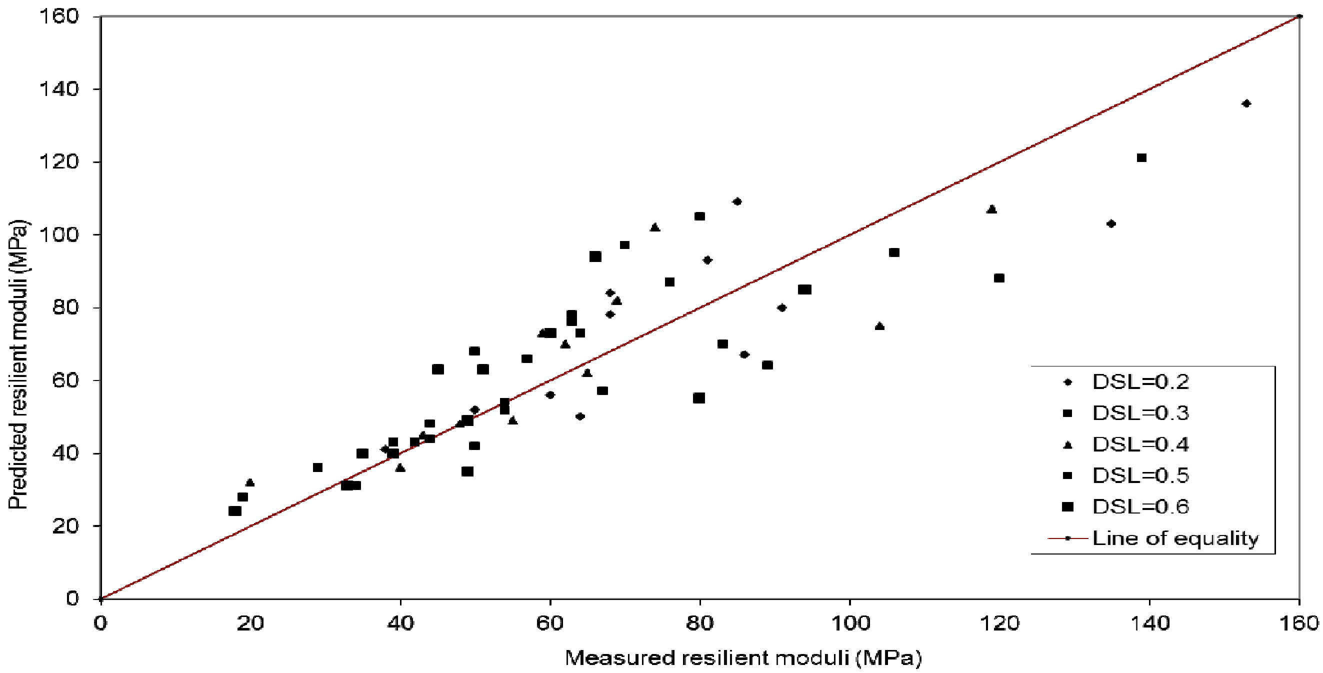

Figure 2. Comparison of measured and predicted resilient modulus values based on Equation 9, 10, 11 and 12.

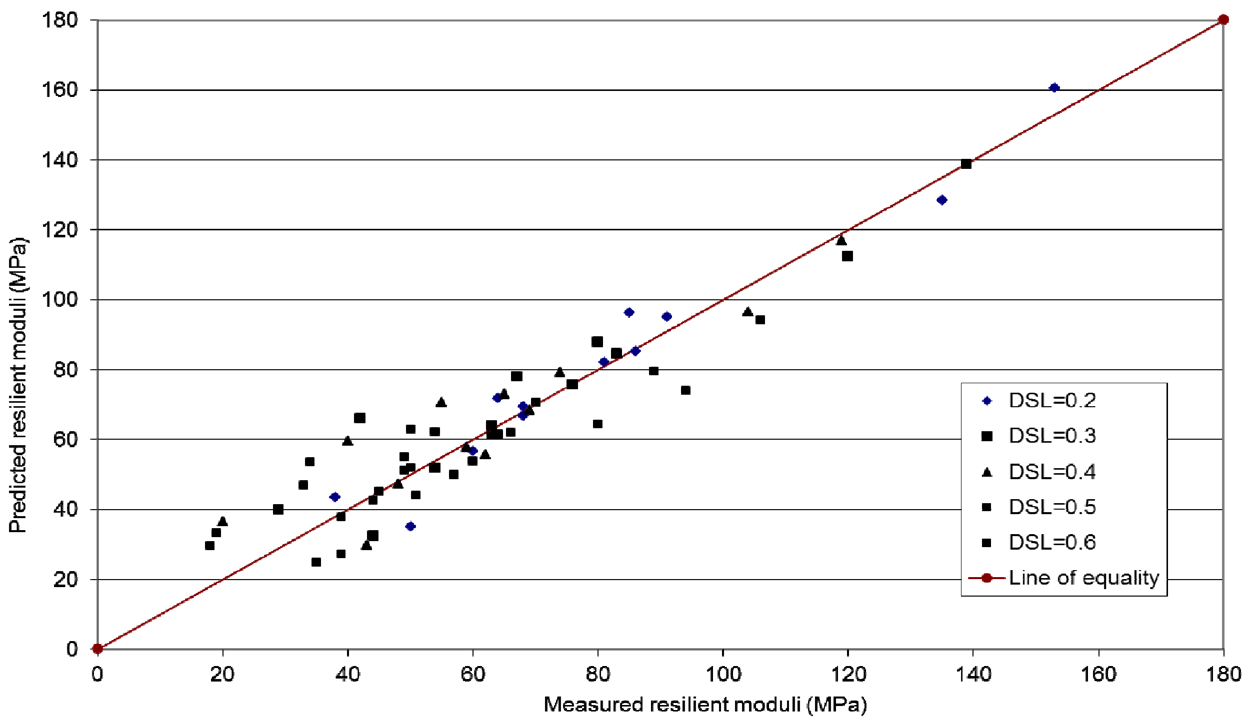

Figure 3. Comparison of measured and predicted resilient modulus values based on Equation 15

conclusions can be drawn through the evaluation of resilient modulus of four Victorian fine-grained subgrade soils:

(i) A new laboratory-testing program for the repeated load triaxial testing of subgrade soils was developed in which stress levels were defined as the ratios of soil strengths (deviator stress level approach).

(ii) Unconsolidated undrained static triaxial (UU) or unconfined compressive strength (UCS) test results can both be used for establishing the stress levels for repeated load triaxial tests as it has not been found much difference between the results of the two tests due to the relatively low confining stresses applied in UU tests.

(iii) Octahedral stress model for the prediction of resilient modulus (E_r) has been found to be subjected to collinearity problem as the variables of the model are interrelated.

(iv) A semi-logarithmic model, which expresses the stress dependency of E_r for fine-grained soils, has been introduced and its predictive capability has been developed. This model incorporates both deviator and confining stresses. The model constants, k_1, k_2 and k_3, can be calibrated with respect to some soil physical properties so that the proposed model can/could be used to estimate the E_r of other fine-grained soils. However, confirmatory repeated load triaxial tests are recommended before adopting the proposed model for other soils.

(v) An alternative prediction model has also been developed, which accommodates deviator stress (σ_d) and unconfined compressive strength (q_u). This model is superior over the earlier "E_r – soil strength" expressions with the inclusion of the deviator stress attribute.

(vi) The comparison of California bearing ratio (CBR) test results with the E_r values indicated that E_r has changed 2 to 15 times the CBR depending upon the stress level, moisture content and the soil type. Therefore, it was concluded that there can not be a unique E_r – CBR expression.

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