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Consideration of the deterioration of stabilised subgrade soils in analytical road pavement design

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ABSTRACT

The stabilisation of road subgrade soil may improve its mechanical properties considerably, however under the combined effect of cumulative traffic load and weathering these materials deteriorate over time and lose performance. However, current road design procedures neglect such deterioration of stabilised soils and consequently their use may result in the under-design of road pavements and as a result unplanned maintenance and/or premature road failure. To address this, this research presents the results of a research programme marrying experimental, analytical and numerical work which was used to develop a methodology which can be used for the first time to design accurately road pavements incorporating stabilised subgrade soils. An extensive experimental programme was carried out consisting of laboratory durability tests to determine the mechanical behaviour of stabilised subgrade soils, in terms of resilient modulus and permanent deformation, under cycles of wetting and drying. Results of the durability tests were used to validate an analytical predictive equation which considers the changes that take place to the material after cycles of wetting and drying. The experimental results show a decrease in the resilient modulus after 25 cycles of wetting and drying cycles for three types of fine grained subgrade soils stabilised with varying amounts of lime-cement. In order to adequately replicate the stress dependency of the performance of the stabilised subgrades for analytical pavement design, two equations were developed that relate the resilient modulus of a stabilised soil with unconfined compressive strength (UCS). The developed equations were utilised with a numerical finite element model of a road pavement to determine the most appropriate road pavement designs, on an engineering basis, for a variety of stabilised soils.

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Introduction

The road pavement is a structural system which is designed, for a predetermined period of time, to withstand the combined effects of traffic and the environment so that the subgrade is adequately protected and that vehicle operating costs and safety are maintained within acceptable limits (McElvaney and Snaith, 2002). When carrying out the structural design of road pavements using an analytical process a numerical model of the pavement structure is used to determine the stresses, strains and deformations at critical locations within the pavement structure. Such models require the characterisation of appropriate resilient modulus values for the materials comprising the road pavement. The critical stresses, strains and deformations so determined are compared with allowable values determined via repeated load laboratory experiments to formulate the design. The resilient modulus and resistance to permanent deformation of many fine-grained subgrade soils however is affected considerably by changes

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in moisture content. As a result these soils often require stabilisation by mechanical or chemical means (Little, 1987; Bell, 1996; Addison and Polma, 2007; Solanki et al., 2010; Rout et al., 2012; Jameson, 2013; Bowers et al., 2013; Rasul et al., 2015). Nevertheless, stabilised soils can still experience notable deterioration with load repetition and weathering (see for example Wu et al., 2011). Therefore when stabilised soils are to be used within a road pavement, it is important to properly characterise their performance so that the road pavement can be designed appropriately (Wu et al., 2011). Hicks (2002) identified three important considerations for the successful design of a stabilised subgrade layer; the structural design, the material mix design and the construction of the stabilised layer. Regarding the structural design, the performance criteria to be used depends on the type of the stabilisation used. These are in three categories in terms of their performance criteria: (i) unbound material; for which the thickness is governed by subgrade strain, this type has no significant tensile strength; (ii) modified material; the design criteria is subgrade strain and modification is carried out to increase the strength and to reduce the moisture and frost susceptibility of fine grained soils; (iii) bound material; the addition of a stabiliser of this type increases the tensile strength of the layer and the performance criteria are fatigue and erosion (Hicks, 2002). Appropriate stabilisation mix design requires the combination of the soils and the stabilisers in the correct proportions to achieve the required strength and durability (Paige-green, 2008).

However, whilst the most widely used and recognised analytical road pavement design procedures, allow for the use of stabilised subgrade layers, they do not take into account the deterioration of the mechanical properties of these layers. Such design procedures include using: USA (ASHTO MEPDG, Texas DOT, Florida DOT and Illinois DOT); (ii) UK design method; French design method and Australian design methods (Queensland DOT, Victoria design method and Roads and maritime services design methods). A useful summary of these design methods to the consideration of stabilised subgrade layers is given by Jameson (2013).

A number of researchers have evaluated the performance of stabilised subgrade soils in terms of the resilient modulus and permanent deformation properties (see for example Chauhan et al., 2008; Abu-Farsakh et al., 2014). However, little research can be found in the literature considering the durability of stabilised subgrade soils subject to cycles of wetting and drying (i.e. weathering) for analytical pavement design. This includes the use of appropriate resilient modulus values to characterise the numerical model and the permanent deformation behaviour for the empirical laboratory based models of material performance.

To address the above issues, this paper describes a novel rigorous approach to the design of road pavements using marginal materials. The approach utilises (i) a suite of laboratory experiments to determine the durability of a number of stabilised soils as a function of cumulative traffic load and weathering, (ii) a method to determine appropriate resilient modulus values for analytical pavement design and, (iii) a novel durability model, (iv) a numerical model of a road pavement. The usefulness and significance of the approach for road pavement design is demonstrated via an example.

Experimental programme

Three fine grained subgrade soils were considered. Their classification as per the AASHTO classification system (AASHTO, M 145) and index properties and particle size distribution are presented in Table 1. The soils were stabilised with different stabiliser ratios, as follows: 2% CC, 4%CC, 2%CC + 1.5%LC and 4%CC + 1.5%LC respectively (CC: cement content and LC: lime content). All the stabilised soil samples were cured for 7 days in a moist cabinet at 100% humidity and a temperature of 21° ± 2°.

Experiments were carried out using the samples to: (i) derive a durability equation based on the resilient modulus and deterioration behaviour of the materials, (ii) develop two equations relating the resilient modulus and unconfined compressive strength (UCS) and, (iii) validate the equations derived in (i) and (ii).

The resilient modulus values of stabilised and unstabilised soils were determined using two procedures. The first method followed the AASHTO T307 procedure (AASHTO, 2006) in which, resilient modulus values of combinations of five deviatoric stress and three confining pressures were determined (i.e. 15 combinations). The five deviatoric stresses used were; 12.4, 24.8, 37.3, 49.7 and 62.0 kPa respectively and the three confining pressures were; 41.4, 27.6 and 13.8 kPa respectively. In the second procedure the resilient modulus values were determined from single and multi-stage permanent deformation tests in which the resilient and permanent strains were separated. The resilient and permanent strains were used to determine the resilient modulus and cumulative permanent deformation respectively. The multi-stage permanent deformation tests consisted of five stages of 10,000 cycles

Table 1
Properties of the three subgrade soils.

Property and test type		A-4	A-6	A-7-5
% Passing	Sieve 5.00 mm	100.00	100.00	100.00
	Sieve 3.35 mm	100.00	91.41	98.97
	Sieve 2.00 mm	100.00	82.00	98.00
	Sieve 1.18 mm	99.95	76.45	97.49
	Sieve 0.600 mm	89.66	71.56	96.97
	Sieve 0.425 mm	85.45	69.81	96.74
	Sieve 0.300 mm	81.59	68.20	96.53
	Sieve 0.212 mm	79.30	67.23	96.35
	Sieve 0.150 mm	77.07	66.18	96.01
	Sieve 0.075 mm	69.27	61.64	93.79
Maximum dry density (gm/cm^3)		1.913	1.889	1.485
Optimum moisture content (%)		10.3	11.0	21.5
Liquid limit (%)		21.0	35.0	51.0
Plasticity index		6.0	14.0	20.0
Specific gravity		2.72	2.71	2.64
Clay content (%)		16	26	52
Silt content (%)		50	34	41
Sand content (%)		34	22	5
Fine gravel content (%)		0	18	2

at five deviatoric stresses of 12.4, 24.8, 37.3, 49.7 and 62.0 kPa respectively. The same confining pressure of 27.6 kPa was used for all stages. In the single stage test the materials were subjected to 50,000 cycles, a deviatoric stress of 62.0 and 120.0 kPa and a confining pressure of 27.6 and 12.4 kPa, respectively. Resilient modulus values in the second procedure were determined from the average of the final five cycles of each stage; i.e. after 10,000 cycles for the multi-stage and 50,000 cycles for the single stage tests, these values were used for road pavement design purposes.

Samples of 100 mm by 200 mm were prepared for the resilient modulus and permanent deformation tests, whilst for the unconfined compressive strength test samples were prepared to dimensions of 50 mm by 100 mm. The maximum dry density and optimum moisture contents of the samples were determined using Proctor tests. For unstabilised and stabilised soils the procedures given in BS 1377–4: 1990 Section 3, methods of test for soils for civil engineering purposes part 4: compaction-related tests and BS 1924–2: 1990 Section 2, Stabilised materials for civil engineering purposes part 2: methods of test for cement-stabilised and lime-stabilised materials, were followed. All samples were compacted at 95% of maximum dry density and at 100% optimum moisture content, see Table 2.

In order to simulate the effect of weathering the materials were subjected to cycles of wetting and drying according to ASTM designation D559, Standard test methods for wetting and drying compacted soil–cement mixtures (ASTM, 2004). The procedure specified in D559 was modified with respect to the number of cycles of wetting and drying (25 cycle were used instead of 12) to represent 25 years of design life of the pavement. Following recommendations of Chittoori (2008), in order to replicate in-situ behaviour the samples were allowed to swell and shrink vertically and horizontally. The changes to the resilient modulus and permanent deformation were assessed instead of the soil–cement losses and moisture and volume changes.

Table 2
Moisture–density relation for the three soils.

Soil type	MDD (gm/cm^3)	OMC (%)	Standard used
Unstabilised			
A-4	1.913	10.3	BS1377-4:1990 Section 3
A-6	1.889	11.0	
A-7-5	1.485	21.5	
Stabilised 2%CC			
A-4	1.853	12.3	
A-6	1.862	13.0	
A-7-5	1.48	23.0	
Stabilised 4%CC			
A-4	1.847	13.2	BS1924-2:1990 Section 2
A-6	1.845	13.5	
A-7-5	1.465	23.5	
Stabilised 2%CC + 1.5%LC			
A-4	1.845	13.0	
A-6	1.847	13.4	
A-7-5	1.472	24.0	
Stabilised 4%CC + 1.5%LC			
A-4	1.838	14.0	
A-6	1.842	14.0	
A-7-5	1.463	24.5	

Durabilty equation

The Mechanistic-Empirical Pavement Design Guide, MEPDG (2004) recommends a minimum unconfined compressive strength (UCS) of 1724 kPa (250 psi) for stabilised sub-bases and subgrade soils for flexible pavements. However, it is preferable to use a mechanical property of the material such as resilient modulus instead.

The ratio of the resilient modulus of a particular soil stabilised with a given amount and type of stabiliser is subject to weathering, MrAWD, to the resilient modulus of the stabilised soil not subject to weathering, M_{rA}, can be written as:

$$F_A = \frac{M_{rAWD}}{M_{rA}} \tag{1}$$

where F_A is the deterioration factors of the material A.

Assuming that the ratios of the deterioration factors of the same soil, each with different amounts of the same stabiliser, is a function of the resilient modulus values of the two materials and can therefore be written as:

$$\frac{F_A}{F_B} = \frac{M_{rA}}{M_{rB}} \tag{2}$$

$$F_B = \frac{M_{rBWD}}{M_{rB}} \tag{3}$$

This (Eq. (3)) has the same meaning as Eq. (1) but for material B.

Combining Eqs. (1)–(3) and rearranging yields:

$$M_{rAWD} = M_{rBWD} * \left(\frac{M_{rA}}{M_{rB}}\right)^2 \tag{4}$$

Accordingly using Eq. (4), the resilient modulus of material A subject to weathering can be determined from the values of the resilient modulus of material A prior to weathering together with the resilient modulus of material B both before and after weathering.

The significance of Eq. (4) is that, by knowing the weathered resilient modulus of a soil with one stabiliser content and type, the weathered resilient modulus values for a range of stabiliser ratios and types can be predicted without carrying out the respective laboratory tests.

To validate the equation the results of resilient modulus and permanent deformation tests were used carried out on three soils at four different stabilisation ratios before and after cycles of wetting and drying, see Tables 3–6. Fig. 1 compares the measured values of resilient modulus versus those predicted using Eq. (4). From Fig. 1 it may be seen that there is a close agreement between the measured and predicted resilient modulus values with associated coefficient of significance (R^2) value of 0.77. Therefore, the equation can be used straightforwardly to determine the deteriorated resilient modulus value or any other properties of lightly stabilised subgrade soils, as demonstrated in the pavement design example shown below.

Table 3
Resilient modulus for stabilised soil and corresponding values after wetting and drying for soil A-4.

Deviatoric stress (kPa)	2%CCT *Mr* (Mpa)	2%CCWD *Mr* (Mpa)	4%CCT *Mr* (Mpa)	4%CCWD *Mr* (Mpa)	2%CC + 1.5%LCT *Mr* (Mpa)	2%CC + 1.5%LCWD *Mr* (Mpa)	4%CC + 1.5%LCT *Mr* (Mpa)	4%CC + 1.5%LCWD *Mr* (Mpa)
12.4	131	70	168	128	105	73	129	117
24.8	160	80	194	143	129	89	165	137
37.3	185	90	214	159	152	103	195	151
49.7	206	101	235	180	176	117	223	166
62	224	112	255	201	198	131	250	182
120	282	160	328	292	295	194	352	255

Table 4
Measured resilient modulus from tests to predicted from Eq. (4) resilient modulus values for soil A-4.

Deviatoric stress (kPa)	2%CC *Mr* (MPa)		4%CC *Mr* (MPa)		2%CC + 1.5LC *Mr* (Mpa)	4%CC + 1.5%LC *Mr* (Mpa)	
	Measured	Predicted	Measured	Predicted	Control	Measured	Predicted
12.4	70	47	128	186	73	117	109
24.8	80	58	143	201	89	137	146
37.3	90	70	159	204	103	151	169
49.7	101	85	180	208	117	166	187
62.0	112	103	201	218	131	182	208
120.0	160	212	292	238	194	255	276

Table 5
Resilient modulus for stabilised soil and corresponding values after wetting and drying for soil A-6.

Deviatoric stress (kPa)	2%CC *Mr* (Mpa)	4%CC *Mr* (Mpa)	4%CCWD *Mr* (Mpa)	2%CC + 1.5%LC *Mr* (Mpa)	2%CC + 1.5%LCWD *Mr* (Mpa)	4%CC + 1.5%LC *Mr* (Mpa)	4%CC + 1.5%LCWD *Mr* (Mpa)
12.4	136	116	90	110	76	115	95
24.8	156	146	103	129	86	149	112
37.3	171	172	117	145	95	171	128
49.7	185	195	132	159	105	190	145
62	198	217	149	172	115	209	164
120	248	309	221	228	167	279	243

Table 6
Measured resilient modulus from tests to predicted from Eq. (4) resilient modulus values for soil A-6.

Deviatoric stress (kPa)	2%CC *Mr* (Mpa)	4%CC *Mr* (Mpa)		2%CC + 1.5%LC *Mr* (Mpa)		4%CC + 1.5%LC *Mr* (Mpa)
	Predicted	Measured	Predicted	Control	Measured	Predicted
12.4	119	90	86	76	95	84
24.8	125	103	109	86	112	113
37.3	133	117	134	95	128	132
49.7	142	132	157	105	145	150
62	153	149	183	115	164	170
120	198	221	307	167	243	249

Resilient modulus nonlinearity

Generally the response of subgrade soils and granular materials to an applied load is dependent on the stress state to which the soil is subjected (Huang, 2004). This can be seen clearly from Figs. 2 and 3 for soils A-4, A-6 and A-7-5 which show the resilient modulus values of stabilised and unstabilised subgrade soils as a function of the number of load cycles from multi-stage permanent deformation tests (the stress levels for each stage are presented on the figures). Fig. 4 shows the permanent deformation test from which these resilient modulus values are determined.

These Figures also show that an increase in deviatoric stress results in an increase in the resilient modulus values of stabilised subgrade soils, and generally the stress decreases with depth of the pavement. Therefore, the resilient modulus of the stabilised subgrade layer can be considered to behave nonlinear especially, when the material has been lightly stabilised.

To account for this nonlinear behaviour a number of authors have suggested various models which relate the resilient modulus to the stress state, a useful summary of which is given by Puppala (2008). The so called k–θ model (Eq. (5)) is widely used to replicate the behaviour of gran-

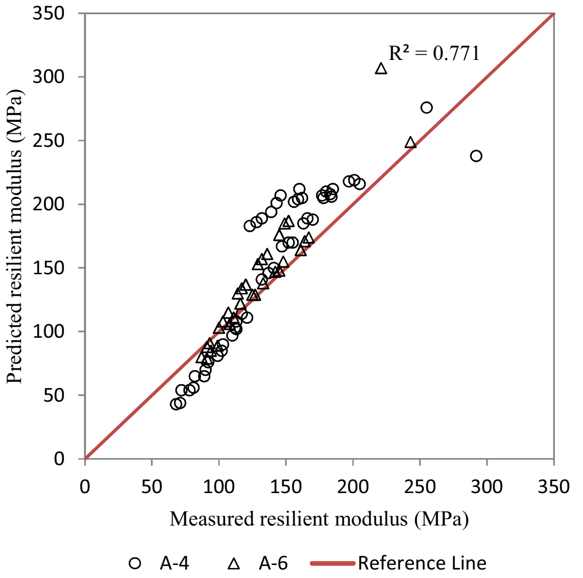

Fig. 1. Relationship between resilient modulus measured from tests and predicted from Eq. (4).

ular materials and a bilinear equation (Eq. (6)) to replicate the behaviour of fine grained materials.

$$Mr = K_1\theta^{K_2} \tag{5}$$

where Mr is resilient modulus, θ is bulk stress or invariant stress; $\theta = \sigma_1 + \sigma_2 + \sigma_3$ or $\theta = \sigma_x + \sigma_y + \sigma_z + \gamma z^*(1 + 2K_o)$ if the normal stresses and surcharge is considered in which γ is average unit weight, z is the depth and K_o is the coefficient of earth pressure.

$$Mr = K_1 + K_3(K_2 - \sigma_d) \tag{6a}$$

$$Mr = K_1 - K_4(\sigma_d - K_2) \tag{6b}$$

where σ_d is the deviatoric stress = $\sigma_1 - \sigma_3$ and K_1, K_2, K_3 and K_4 are material constants.

The universal model proposed by Witczak and Uzan (1988) (Eq. (7)) for subgrade and unbound material includes the octahedral shear stress (τ_{oct}) and bulk stress (θ) to account for the influence of a combination of stresses.

$$Mr = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a}\right)^{k_3} \tag{7}$$

where Mr is resilient modulus, P_a is the atmospheric pressure for the location of the project, θ is bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$, τ_{oct} is octahedral stress = $\frac{\sqrt{2}}{3}(\sigma 1 - \sigma 3)$ for $\sigma_2 = \sigma_3$ and K_1, K_2 and K_3 are regression parameters.

MEPDG (2004) proposes the use of the relationship given in Eq. (8) in which the parameters K_1, K_2 and K_3 are determined from regression analysis of resilient modulus tests carried out in the laboratory.

$$Mr = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3} \tag{8}$$

Herein two relationships were derived (Eqs. (9) and (10)) to take into account findings from the literature, i.e. that resilient modulus is a function of the deviatoric stress. The two developed equations are for stabilised (modified, lightly stabilised) subgrade soils. From the two correlation equations it is possible to find resilient modulus values for a range of stress levels from UCS test results without carrying out the resilient modulus test. The first equation is as follows:

$$Mr = \mathrm{UCS}^{a+b*\sigma_d} \tag{9}$$

where a and b are regression parameters.

In the second equation the bulk stress and octahedral shear stress were also introduced as suggested by Witczak and Uzan (1988), as follows:

$$Mr = \mathrm{UCS}^{\left[a*\left(\frac{\theta}{\sigma_{atm}}\right)^{b}*\left(\frac{\tau_{oct}}{\sigma_{atm}}\right)^{c}\right]} \tag{10}$$

In which θ = bulk stress = $\sigma_1 + 2\sigma_3$, τ_{oct} = octahedral shear stress = $(\sqrt{2/3})(\sigma_1 - \sigma_3)$, σ_{atm} = atmospheric pressure = 101 kPa and a, b and c are regression parameters.

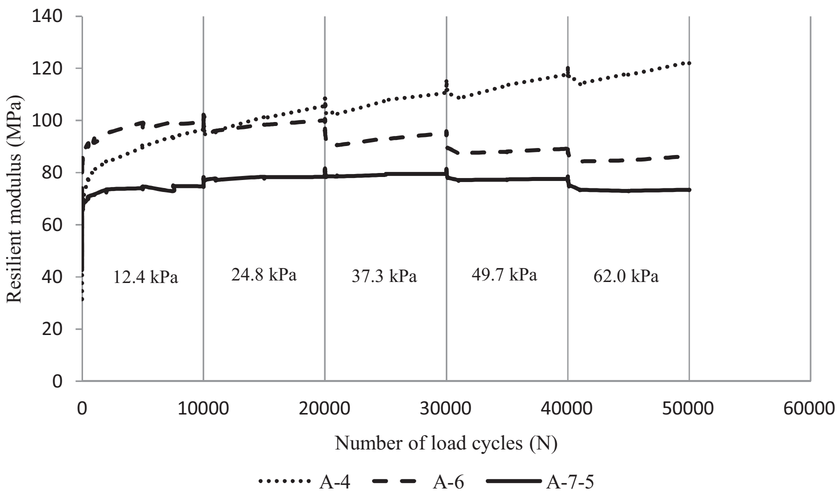

Fig. 2. Resilient modulus values vs. number of load repetitions for different deviatoric stress levels for unstabilised soils.

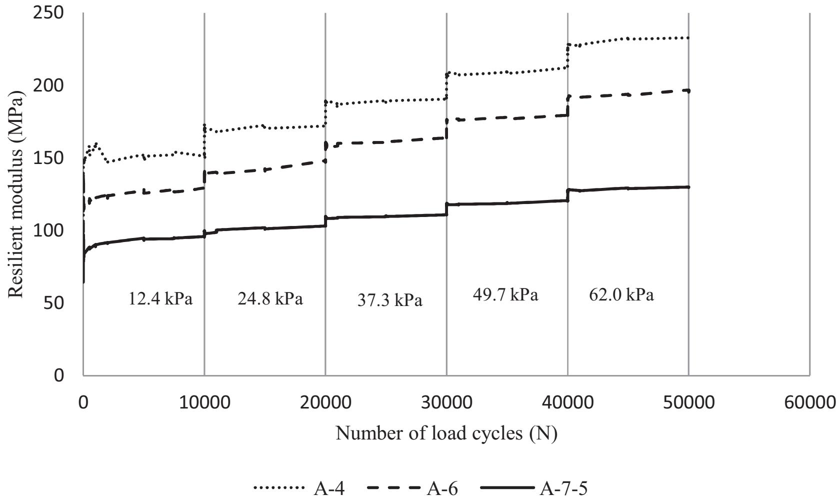

Fig. 3. Resilient modulus values vs. number of load repetitions for different deviatoric stress levels for stabilised soils.

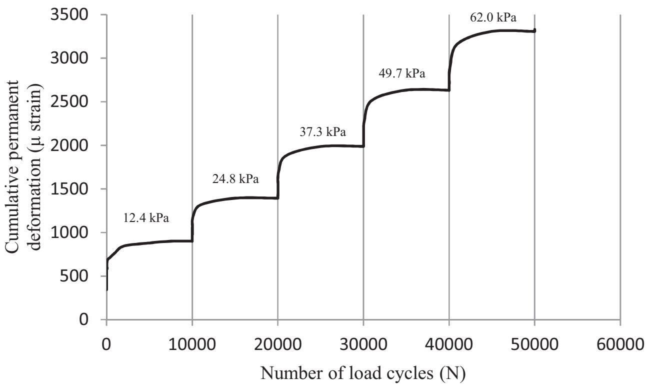

Fig. 4. Relation between accumulative permanent deformation and number of load repetitions in a multi-stage test for unstabilised soil A-4.

Table 7
Unconfined compressive strength results after 7 days curing.

Soil type	Stabiliser content (%)	UCS[*] (kPa)	Soil type	Stabiliser content (%)	UCS (kPa)	Soil type	Stabiliser content (%)	UCS (kPa)
A-4	Unstabilised	197	A-6	Unstabilised	178	A-7-5	Unstabilised	171
	2%CC	580		2%CC	579		2%CC	275
	4%CC	956		4%CC	874		4%CC	357
	2%CC + 1.5%LC	618		2%CC + 1.5%LC	557		2%CC + 1.5%LC	427
	4%CC + 1.5%LC	955		4%CC + 1.5%LC	774		4%CC + 1.5%LC	501

[*] The average of four replicate samples.

The resilient modulus and UCS values given in Tables 3–6 and 7, respectively were used to determine the parameters a, b and c and to validate the models given in Eqs. (9) and (10). For this purpose soil samples stabilised with 2%CC + 1.5%LC and 4%CC + 1.5%LC were used to determine the regression parameters, and samples stabilised with 2%CC and 4%CC were used for validation. From the analysis values of a and b were found to be 0.737 and 0.001 with an $R^2 = 0.791$ for the model given in Eq. (9) and $a = 0.882$, $b = 0.017$ and $c = 0.066$ with an $R^2 = 0.833$ for Eq. (10). The resilient modulus values obtained from the tests for soils A-4 and A-6 with 2%CC and 4%CC were compared with those found from Eqs. (9) and (10) and plotted in Figs. 5 and 6. The corresponding R^2 are 0.733 and 0.821, respectively.

From the above, it may be seen that the relationships described by Eqs. (9) and (10) appear to predict the resilient modulus with satisfactory accuracy and they provide conservative values of resilient modulus for design purposes.

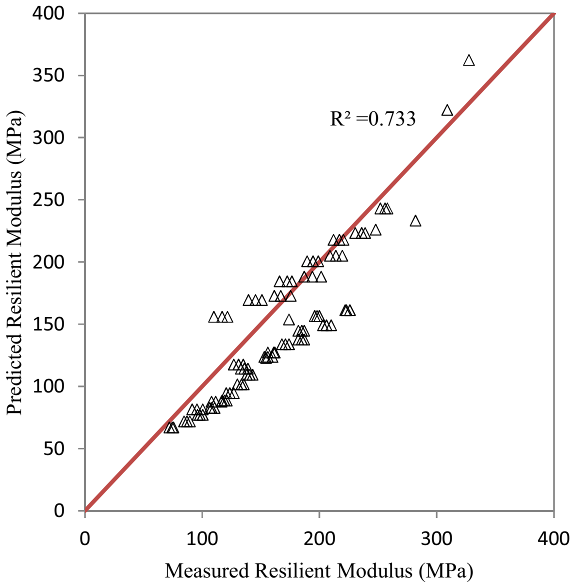

Fig. 5. Measured resilient modulus from tests versus predicted resilient modulus from Eq. (9).

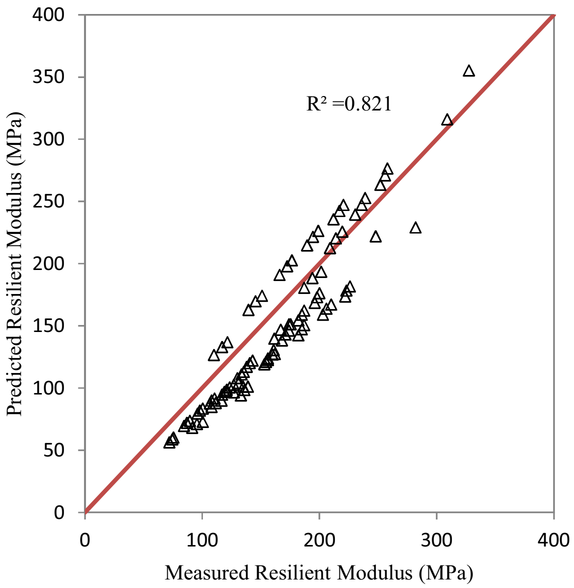

Fig. 6. Measured resilient modulus from tests versus predicted resilient modulus from Eq. (10).

Little and Yusuf (2001) used an Eq. (11), first proposed by Thompson (1970), for lime stabilised soils in mechanistic empirical pavement design procedures.

$$E_R = 0.124(\text{UCS}) + 9.8 \qquad (11)$$

where E_R is resilient modulus in Ksi and UCS is unconfined compressive strength in Psi.

For soils, A-4, A-6 and A-7-5 a comparison was made between the resilient modulus predicated for each soil using Eqs. (9) and (11) together with those determined from the laboratory results described above. The results can be seen in Table A.1. A statistical measure of the similarity, the mean absolute percentage error, MAPE (Hyndman and Koehler, 2006) was used to compare the resilient modulus values obtained from the laboratory and from the two equations. The MAPE when using Eq. (9) is 19, whilst for Eq. (11) it is 25. This suggests that Eq. (9) predicts the value of resilient modulus more closely than Eq. (11) (see Table A.1), see also Table A.2 for Eq. (10) results.

Pavement section analysis

A hypothesised pavement section and a finite element model (FEM) developed by Rasul et al. (2015) were used to determine the compressive strain at the top of the subgrade in order to determine the best of the three soil types for use in an untreated form and when stabilised with 2% CC, 4%CC, 2% + 1.5%LC and 4%CC + 1.5%LC, respectively. The FEM was characterised according to Table 8 and a pressure of 550 kPa and a loading area of 152 mm was applied to simulate a wheel load. The example takes into account the deterioration of resilient modulus with time using a performance model developed by Rasul et al. (2015).

Following a process suggested by Huang (2004), amongst others, an iterative method was developed to determine appropriate modulus values to be used within the FEM. For each analysis, an initial seed value of the resilient modulus was obtained from the relationship between deviatoric stress and resilient modulus values obtained from multi-stage permanent deformation tests. The seed value was used within the FEM to determine the resulting deviatoric stresses at the critical locations of interest. An iterative process thereafter was followed by which the computed deviatoric stresses were used to determine a new resilient modulus value from the results of the laboratory tests. This process was repeated until the computed resilient modulus value and that determined from the laboratory between two iterations converged. Subsequently the resilient modulus values so computed were used for the 30 analysis scenarios described in Table 9 and the compressive strains were calculated at the top of the subgrade.

As mentioned previously, the performance criterion chosen in this research for the modified soils was the compressive strain at the top of the subgrade. Therefore the selection of the stabiliser type and design was taken on the basis of the compressive strain value. However, the variability of subgrade soil type and property encountered in a project makes it problematic to select different stabilisers for different soil types. For example soil A-4 in this

Table 8
Hypothesised pavement section properties.

Layer type	Thickness modulus of elasticity		Poisson's ratio
	(mm)	(MPa)	
Asphalt concrete	100	3000	0.3
Base course	200	300	0.35
Compacted subgrade	200	Variable	0.45
Natural subgrade	_	Variable	0.45

Table 9
Determination of resilient modulus, compressive stress and compressive strain for the pavement section and stabiliser selection.

Soil type	Stabiliser ratio	Start *Mr* (Mpa)	End compressive stress (kPa)	End *Mr* (Mpa)	Compressive strain (μ strain)	End compressive stress (kPa)	End *Mr* (Mpa)	Compressive strain (μ strain)
			Natural subgrade			Compacted subgrade		
A-4	Untreated	187	39	110	346	78	130	579
	Untreated (WD*)	77	38	100	363	70	90	706
	2%CC	282	43	150	277	97	260	365
	2%CC (WD)	160	38	100	367	75	120	606
	4%CC	328	42	150	272	100	300	326
	4%CC (WD)	292	35	99	349	88	240	388
	2%CC + 1.5%LC	295	43	150	276	96	250	376
	2%CC + 1.5%LC (WD)	194	36	99	367	84	150	532
	4%CC + 1.5%LC	352	42	150	270	101	316	312
	4%CC + 1.5%LC (WD)	255	36	100	353	86	210	427
A-6	Untreated	100	38	100	365	72	100	669
	Untreated (WD)	32	28	50	526	48	45	968
	2%CC	248	36	100	352	87	218	416
	2%CC (WD)	198	26	50	513	69	155	533
	4%CC	309	35	100	342	90	260	365
	4%CC (WD)	221	25	50	499	72	180	485
	2%CC + 1.5%LC	228	36	100	357	84	192	449
	2%CC + 1.5%LC (WD)	167	27	50	535	63	110	647
	4%CC + 1.5%LC	279	35	100	347	88	240	388
	4%CC + 1.5%LC (WD)	243	25	50	490	72	177	501
A-7-5	Untreated	46	33	70	440	57	60	856
	Untreated (WD)	8	21	26	706	32	22	1256
	2%CC	155	32	70	448	69	120	610
	2%CC (WD)	87	20	26	757	39	42	1008
	4%CC	174	31	70	441	73	148	541
	4%CC (WD)	110	20	26	751	46	68	839
	2%CC + 1.5%LC	184	31	70	437	75	160	516
	2%CC + 1.5%LC (WD)	123	20	26	707	52	92	730
	4%CC + 1.5%LC	212	30	70	430	77	180	478
	4%CC + 1.5%LC (WD)	163	19	26	713	54	106	685

* Denotes for wetting/drying.

research can be stabilised with 4% cement with a degree of certainty for long term performance where the change in compressive strain after 25 cycles of wetting and drying is from 326 micro-strains to 388 micro-strains. In contrast under similar conditions the compressive strain for soil A-7-5 varies between 541 micro-strains and 839 micro-strains. However, any increase in cement stabiliser content for improving soil A-7-5 for its long term performance may affect the performance of the soil A-4, as the increase in amount of stabiliser may introduce other issues such as reflective cracking that can occur with excessive stabiliser content (Paige-Green, 2008). Therefore the most appropriate choice from this range of stabiliser contents for the three soils could be considered to be stabilisation with 4% cement content plus 1.5% lime content.

Progressive deterioration of resilient modulus

Conventional analytical pavement design procedures use the same resilient modulus value of stabilised layers throughout the design life, whilst the deterioration of the asphalt is accounted for in the design by selecting appropriate resilient modulus values from the laboratory (MEPDG, 2004). The cumulative traffic load to which the road pavement is to be subject (i.e. the design traffic load) is typically based on current traffic loads plus an increment to account for future traffic growth. However, the deterioration of resilient modulus of the stabilised layers and unbound materials are not usually considered.

This deterioration process is illustrated in Fig. 7 in the pavement design example which shows how the resilient modulus value of soil changes with cycles of wetting and drying. To account for this behaviour Rasul et al. (2015) proposed a model given by Eq. (12) which can be used to determine incremental plastic strains as a function of the change in resilient modulus which may be expected seasonally and throughout the life of a road pavement.

$$\sum_{t=1}^{m} \varepsilon_p = a \times \left(\frac{\sigma_{dt}}{M_{rt}}\right) \times N_t^b \qquad \sum_{1}^{m} t = \mathrm{T} \tag{12}$$

where ε_p is accumulated permanent strain in micro strain, σ_{dt} is deviatoric stress in kPa during a period of time t, M_{rt} is resilient modulus in MPa for a period of time t, is the number of load repetitions in the period of time t, a and b are material parameters, T is the design life of the road pavement.

Pavement design example

To illustrate the use of the relationships described above, a hypothetical road pavement section with the dimensions, properties and design parameters shown in

Table 10
Pavement section dimensions and properties.

Soil type	Stabilisation ratio and type	UCS* (kPa)	UCS** (kPa)
Soil properties			
A-7-5	2% CC	275.0	Unknown
	4% CC	357.0	Unknown
	2% CC + 1.5% LC	427.0	Unknown
	4% CC + 1.5% LC	501.0	350.0
Layer type	Thickness (mm)	Resilient modulus (MPa)	Poisson's ratio
Pavement section			
Asphalt concrete	100	2500	0.3
Base course	150	300	0.35
Compacted subgrade	150	Variable	0.35
Natural subgrade	–	47	0.45
Traffic data			
Traffic load for the base year	300,000 heavy trucks		
Tyre pressure	860 kPa		
Loading radius area	152 mm		
Truck growth factor	4%		

* Stabilised soil before the durability test.
** Stabilised soils after the durability test.

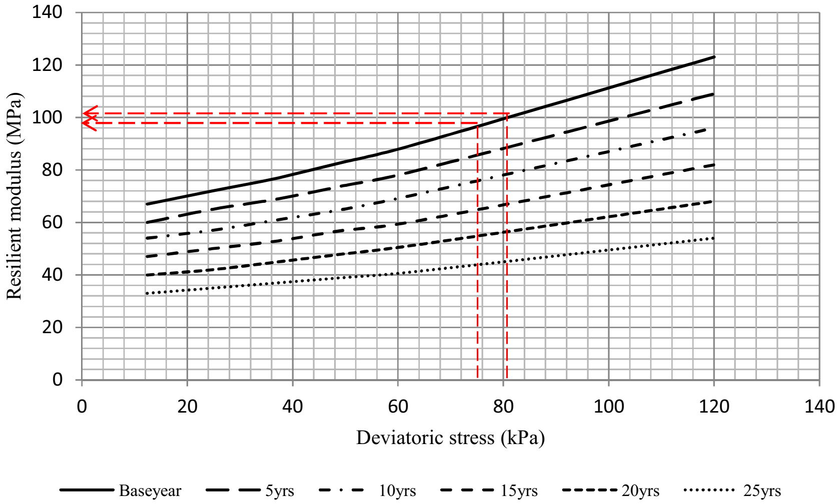

Fig. 7. Iteration analysis of resilient modulus and deviatoric stress convergence for soil A-7-5 stabilised with 2%CC and for five stages of 5 years each.

Table 11
Resilient modulus values for a range of deviatoric stresses from UCS test results.

Deviatoric stress (kPa)	4%CC + 1.5%LC		4%CC + 1.5%LC (WD)		2%CC + 1.5%LC		4%CC		2%CC	
	UCS (kPa)	*Mr* (Mpa)	UCS (kPa)	*Mr* (Mpa)	UCS (kPa)	*Mr* (Mpa)	UCS (kPa)	*Mr* (Mpa)	UCS (kPa)	*Mr* (Mpa)
12.4	501	106	350	81	427	94	357	82	275	67
24.8		114		87		101		88		72
37.3		123		93		109		95		77
49.7		133		100		117		102		83
62.0		144		108		126		110		89
120.0		206		151		180		154		123
160.0		264		191		229		195		154
200.0		339		242		292		247		193

Table 10 was used. The design process may be considered as a number of steps as follows:

Step 1:

Using Eq. (9), the resilient modulus values of the stabilised subgrade soil determined from known UCS values at a variety of deviatoric stresses (see Table 11).

Step 2:

From Eq. (4) the deteriorated (WD) resilient modulus values for soil A-7-5 stabilised with 2%CC + 1.5%LC, 4%CC and 2%CC are determined from the known deteriorated resilient modulus value of stabilised soil with 4%CC + 1.5%LC. To account for the deterioration in resilient

Table 12
Deteriorated resilient modulus for five stages for soil A-7-5 stabilised with 2%CC.

1 Deviatoric stress (kPa)	2 *Mr* before W&D (MPa)	3 *Mr* after WD (MPa)	4 ADF	5 *Mr* after 5 year (MPa)	6 *Mr* after 10 year (MPa)	7 *Mr* after 15 year (MPa)	8 *Mr* after 20 year (MPa)	9 *Mr* after 25 year (MPa)
12.4	67	33	1.38	60	54	47	40	33
24.8	72	35	1.50	65	57	50	42	35
37.3	77	37	1.62	69	61	53	45	37
49.7	83	39	1.76	74	65	57	48	39
62	89	41	1.90	79	70	60	51	41
120	123	54	2.76	109	96	82	68	54
160	154	65	3.56	136	119	101	83	65
200	193	79	4.58	170	147	124	102	79

Table 13
Pavement section analysis for stabilised subgrade soil with 2%CC, 4%CC, 2%CC + 1.5%LC and 4%CC + 1.5%LC, respectively.

	Increment 1	Increment 2	Increment 3	Increment 4	Increment 5
A-7-5_2%CC					
Resilient modulus MDL3 (Mpa)	103	90	79	67	53
Tensile strain beneath L1 (μ strain)	−454	−461	−467	−474	−485
Vertical stress MDL3 (kPa)	82	81	80	79	77
Resilient strain MDL3 (μ strain)	1063	1150	1240	1364	1564
Vertical stress L4 (kPa) Top	59	60	60	60	60
Resilient strain L4 (μ strain) Top	1246	1249	1247	1237	1209
Growth rate (%)	4	4	4	4	4
Years of the stage (years)	5	10	15	20	25
Number of heavy trucks in the base year	300,000	300,000	300,000	300,000	300,000
Growth factor	5.416	12.006	20.024	29.778	41.646
Accumulated number of heavy trucks for the stage	1,624,897	1,976,935	2,405,244	2,926,347	3,560,349
Parameter (*a*)	2205.015	2205.015	2205.015	2205.015	2205.015
Parameter (*b*)	0.038	0.038	0.038	0.038	0.038
Permanent strain MDL3 (μ strain)	3023	3443	3903	4578	5683
Permanent strain MDL3 (mm)	0.453	0.516	0.585	0.687	0.852
Total permanent deformation (mm)	3.09				
A-7-5_4%CC					
Resilient modulus MDL3 (Mpa)	127	113	104	92	80
Tensile strain beneath L1 (μ strain)	−444	−450	−454	−460	−466
Vertical stress MDL3 (kPa)	83	82	82	81	80
Resilient strain MDL3 (μ strain)	939	1006	1056	1135	1231
Vertical stress L4 (kPa) Top	58	59	59	60	60
Resilient strain L4 (μ strain) Top	1230	1241	1246	1249	1248
Permanent strain MDL3 (μ strain)	2481	2776	3039	3418	3912
Permanent strain MDL3 (mm)	0.372	0.416	0.456	0.513	0.587
Total permanent deformation (mm)	2.34				
A-7-5_2%CC + 1.5%LC					
Resilient modulus MDL3 (Mpa)	148	135	124	114	104
Tensile strain beneath L1 (μ strain)	−437	−441	−445	−450	−454
Vertical stress MDL3 (kPa)	84	83	83	82	82
Resilient strain MDL3 (μ strain)	857	906	953	1001	1056
Vertical stress L4 (kPa) Top	57	58	58	59	59
Resilient strain L4 (μ strain) Top	1211	1223	1233	1240	1246
Permanent strain MDL3 (μ strain)	2155	2352	2580	2793	3084
Permanent strain MDL3 (mm)	0.323	0.353	0.387	0.419	0.463
Total permanent deformation (mm)	1.94				
A-7-5_4%CC + 1.5%LC					
Resilient modulus MDL3 (Mpa)	172	158	150	140	130
Tensile strain beneath L1 (μ strain)	−429	−433	−436	−439	−443
Vertical stress MDL3 (kPa)	85	84	84	84	83
Resilient strain MDL3 (μ strain)	783	824	850	886	926
Vertical stress L4 (kPa) Top	57	57	57	58	58
Resilient strain L4 (μ strain) Top	1186	1202	1209	1218	1227
Permanent strain MDL3 (μ strain)	1876	2034	2158	2330	2497
Permanent strain MDL3 (mm)	0.281	0.305	0.324	0.349	0.375
Total permanent deformation (mm)	1.63				

modulus over the life of the pavement, the analysis is divided into a number of stages (increments). For the purposes of this example five stages have used, each of which represents five years of analysis (i.e. 1/5th of the design life). Thereafter the deteriorated resilient modulus for each year is calculated using an annual deterioration factor (ADF) which was determined as follows:

$$\mathrm{ADF} = \frac{\text{Mr before durability test} - \text{Mr after durability test}}{\text{NDC}} \tag{13}$$

Using Eq. (13) the resilient modulus of the specified stage was determined as function of deviatoric stress. Table 12 gives the results obtained for soil A-7-5 at 2%CC. Fig. 7 plots resilient modulus values, for each of these stages.

Step 3:

Step 3 incorporates the iterative procedure described above in which a seed resilient modulus value is chosen and used within the FEM to determine a corresponding computed deviatoric stress at the top of the subgrade. The laboratory determined resilient modulus value corresponding to the computed deviatoric stress is then used again within the FEM to obtain a new deviatoric stress. This process is iterated until the difference between resilient modulus values between successive iterations is within an acceptable limit. This process is shown graphically in Fig. 7 for soil A-7-5. So obtained values of resilient modulus and deviatoric stress are later used in the performance model (Eq. (12)) to determine the incremental accumulation of permanent deformation.

Step 4:

Step 4 involves the determination of the model parameters. For soil A-7-5 the parameters a and b of the performance model (Eq. (12)) were found from regression analysis to be $a = 2205.015$ and $b = 0.038$.

Results

Table 13 shows the results of the pavement section analysis presented above for subgrade soil A-7-5 stabilised with four different stabiliser ratios.

Typically in analytical design procedures it is usual to specify the amount of permanent deformation which occurs in all layers of the pavement structure (including the subgrade). The procedure described here, since it enables the amount of deformation within a stabilised layer to be predicted as a function of stabiliser content, allows the designer to specify the contribution to total deformation to be made by the stabilised subgrade layer (see Table 13) This can enable the designer to trade off lower material performance in the upper layers of the road pavement against the amount of stabilisation required in the subgrade. With reference to the results given in Table 13, should it be decided that the subgrade is to contribute 2 mm of deformation throughout the design life, then subgrades of material of type A-7-5 should be lightly stabilised using 2%CC + 1.5%LC. On the other hand if it was felt that the subgrade should contribute more to the overall deformation (perhaps because of a lack of more durable materials for the upper layers) then A-7-5 stabilised using 2%CC could be used.

Concluding discussion

Stabilisation can improve the performance of the subgrade layers of road pavements. However, in order to account for such improvements in performance within analytical pavement design procedure there is a need for appropriate durability tests and the development of associated relationships to quantify likely in situ soil performance. This approach is lacking in current analytical design procedures and the research paper demonstrated for the first time a rigorous methodology which can be used to take into account the performance of stabilised subgrade layers. To effect this, a research programme marrying experimental, analytical and numerical work was undertaken to develop:

1. A novel relationship which can predict the deteriorated resilient modulus values for different stabiliser contents and types from a deteriorated resilient modulus value of one specified stabiliser content tested for durability.
2. Two correlation equations derived from permanent deformation and unconfined compressive strength tests. The equations predict with an adequate accuracy the resilient modulus from the unconfined compressive strength and the stress state, for three soil types at four different stabiliser contents. The correlation equations can be used to determine a set of resilient modulus values for a series of different stress states.
3. A procedure to take into account the nonlinearity of the stress dependency of the resilient modulus values of stabilised and unstabilised subgrade soils.
4. A performance model for stabilised subgrade soils which can predict with a satisfactory degree of accuracy the incremental accumulation of permanent deformation.

The above procedure was demonstrated within an analytical design procedure which incorporated a FEM. It was also shown how the amount of stabiliser could be varied to facilitate different design options. The results produced are transformative and demonstrate to the highway engineer for the first time the importance in analytical road pavement design of including suitably characterised values of resilient modulus which consider stress dependency and the effects of environmental deterioration.

Acknowledgements

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Appendix A.

Table A.1
Comparison between the prediction capability of Eqs (9) and (11).

Soil type	Stabilisation ratio	Average UCS (kPa)	Deviatoric stress (kPa)	Measured *Mr* (Mpa)	Predicted *Mr* (Mpa) Eq. (9)		Predicted *Mr* (Mpa) Eq. (11)	
A-4	2%CC	580	12.4	131	118	0.101	141	0.074
			24.8	160	127	0.202	141	0.119
			37.3	185	138	0.253	141	0.238
			49.7	206	149	0.277	141	0.318
			62.0	224	161	0.278	141	0.371
			120.0	282	233	0.172	141	0.501
	4%CC	969	12.4	168	173	0.030	189	0.125
			24.8	194	188	0.030	189	0.027
			37.3	214	205	0.042	189	0.118
			49.7	235	224	0.049	189	0.196
			62.0	255	243	0.047	189	0.260
			120.0	328	362	0.107	189	0.423
A-6	2%CC	559	12.4	136	115	0.158	138	0.016
			24.8	156	124	0.208	138	0.116
			37.3	171	134	0.216	138	0.192
			49.7	185	145	0.215	138	0.252
			62.0	198	157	0.208	138	0.302
			120.0	248	226	0.088	138	0.443
	4%CC	845	12.4	116	156	0.344	174	0.494
			24.8	145	170	0.168	174	0.194
			37.3	172	185	0.075	174	0.011
			49.7	194	201	0.033	174	0.107
			62.0	217	218	0.007	174	0.198
			120.0	309	322	0.043	174	0.438
A-7-5	2%CC	275	12.4	74	67	0.090	103	0.391
			24.8	87	72	0.171	103	0.183
			37.3	98	77	0.211	103	0.048
			49.7	108	83	0.234	103	0.050
			62.0	119	89	0.253	103	0.135
			120.0	155	123	0.203	103	0.334
	4%CC	357	12.4	96	82	0.146	113	0.180
			24.8	112	88	0.214	113	0.010
			37.3	124	95	0.234	113	0.086
			49.7	133	102	0.234	113	0.150
			62.0	141	110	0.220	113	0.195
			120.0	174	154	0.115	113	0.350
					Mean Absolute Percentage Error (MAPE)	19	Mean Absolute Percentage Error (MAPE)	25

Table A.2
Results for validation of Eq. (10).

Soil type	Stabilization ratio	Average UCS (kPa)	Confining stress (kPa)	Deviatoric stress (kPa)	θ (kPa)	τ_{oct} (kPa)	Measured *Mr* from tests (MPa)	Predicted *Mr* from Eq. (10) (MPa)	
A-4	2%CC	580	41.4	12.4	95.2	5.8	131	104	0.205
			41.4	24.8	107.6	11.7	161	131	0.188
			41.4	37.3	120.1	17.6	187	151	0.194
			41.4	49.7	132.5	23.4	210	167	0.203
			41.4	62.0	144.8	29.2	226	182	0.196
			27.6	12.4	67.6	5.8	135	101	0.249
			27.6	24.8	80.0	11.7	162	128	0.213
			27.6	37.3	92.5	17.6	185	147	0.203
			27.6	49.7	104.9	23.4	206	164	0.204
			27.6	62.0	117.2	29.2	223	178	0.200
			12.4	12.4	37.2	5.8	127	97	0.238
			12.4	24.8	49.6	11.7	156	123	0.214
			12.4	37.3	62.1	17.6	182	143	0.217
			12.4	49.7	74.5	23.4	203	159	0.216
			12.4	62.0	86.8	29.2	222	174	0.217

(continued on next page)

Table A.2 (*continued*)

Soil type	Stabilization ratio	Average UCS (kPa)	Confining stress (kPa)	Deviatoric stress (kPa)	θ (kPa)	τ_{oct} (kPa)	Measured *Mr* from tests (MPa)	Predicted *Mr* from Eq. (10) (MPa)	
			12.4	120.0	144.8	56.6	282	229	0.187
	4%CC	969	41.4	12.4	95.2	5.8	176	151	0.137
			41.4	24.8	107.6	11.7	202	194	0.039
			41.4	37.3	120.1	17.6	220	226	0.029
			41.4	49.7	132.5	23.4	239	253	0.058
			41.4	62.0	144.8	29.2	258	277	0.072
			27.6	12.4	67.6	5.8	167	147	0.119
			27.6	24.8	80.0	11.7	194	189	0.028
			27.6	37.3	92.5	17.6	214	220	0.030
			27.6	49.7	104.9	23.4	236	247	0.048
			27.6	62.0	117.2	29.2	256	271	0.059
			12.4	12.4	37.2	5.8	162	140	0.134
			12.4	24.8	49.6	11.7	187	181	0.033
			12.4	37.3	62.1	17.6	209	213	0.017
			12.4	49.7	74.5	23.4	231	240	0.039
			12.4	62.0	86.8	29.2	252	263	0.045
			12.4	120.0	144.8	56.6	328	355	0.085
A-6	2%CC	559	41.4	12.4	95.2	5.8	139	101	0.271
			41.4	24.8	107.6	11.7	160	127	0.206
			41.4	37.3	120.1	17.6	174	146	0.159
			41.4	49.7	132.5	23.4	187	162	0.131
			41.4	62.0	144.8	29.2	200	176	0.118
			27.6	12.4	67.6	5.8	136	99	0.275
			27.6	24.8	80.0	11.7	156	124	0.205
			27.6	37.3	92.5	17.6	171	143	0.163
			27.6	49.7	104.9	23.4	185	159	0.140
			27.6	62.0	117.2	29.2	198	173	0.125
			12.4	12.4	37.2	5.8	133	94	0.292
			12.4	24.8	49.6	11.7	153	119	0.220
			12.4	37.3	62.1	17.6	168	138	0.176
			12.4	49.7	74.5	23.4	182	155	0.151
			12.4	62.0	86.8	29.2	196	169	0.139
			12.4	120.0	144.8	56.6	248	222	0.104
	4%CC	845	41.4	12.4	95.2	5.8	122	137	0.128
			41.4	24.8	107.6	11.7	151	174	0.155
			41.4	37.3	120.1	17.6	177	203	0.149
			41.4	49.7	132.5	23.4	199	226	0.138
			41.4	62.0	144.8	29.2	221	247	0.122
			27.6	12.4	67.6	5.8	117	133	0.138
			27.6	24.8	80.0	11.7	146	170	0.168
			27.6	37.3	92.5	17.6	173	198	0.148
			27.6	49.7	104.9	23.4	195	222	0.140
			27.6	62.0	117.2	29.2	217	242	0.117
			12.4	12.4	37.2	5.8	110	127	0.152
			12.4	24.8	49.6	11.7	140	163	0.168
			12.4	37.3	62.1	17.6	166	191	0.151
			12.4	49.7	74.5	23.4	190	215	0.134
			12.4	62.0	86.8	29.2	212	236	0.112
			12.4	120.0	144.8	56.6	309	316	0.023
A-7-5	2%CC	275	41.4	12.4	95.2	5.8	76	60	0.200
			41.4	24.8	107.6	11.7	90	74	0.175
			41.4	37.3	120.1	17.6	101	84	0.167
			41.4	49.7	132.5	23.4	111	92	0.169
			41.4	62.0	144.8	29.2	121	99	0.184
			27.6	12.4	67.6	5.8	75	59	0.209
			27.6	24.8	80.0	11.7	87	72	0.170
			27.6	37.3	92.5	17.6	98	82	0.163
			27.6	49.7	104.9	23.4	108	90	0.165
			27.6	62.0	117.2	29.2	119	97	0.184
			12.4	12.4	37.2	5.8	72	57	0.214
			12.4	24.8	49.6	11.7	85	70	0.174
			12.4	37.3	62.1	17.6	96	80	0.170
			12.4	49.7	74.5	23.4	107	88	0.175
			12.4	62.0	86.8	29.2	117	95	0.189
			12.4	120.0	144.8	56.6	155	121	0.216
	4%CC	357	41.4	12.4	95.2	5.8	101	73	0.273
			41.4	24.8	107.6	11.7	117	90	0.226

Table A.2 (*continued*)

Soil type	Stabilization ratio	Average UCS (kPa)	Confining stress (kPa)	Deviatoric stress (kPa)	θ (kPa)	τ_{oct} (kPa)	Measured *Mr* from tests (MPa)	Predicted *Mr* from Eq. (10) (MPa)	
			41.4	37.3	120.1	17.6	127	103	0.190
			41.4	49.7	132.5	23.4	136	113	0.164
			41.4	62.0	144.8	29.2	143	122	0.145
			27.6	12.4	67.6	5.8	96	71	0.254
			27.6	24.8	80.0	11.7	112	88	0.210
			27.6	37.3	92.5	17.6	124	101	0.184
			27.6	49.7	104.9	23.4	134	111	0.168
			27.6	62.0	117.2	29.2	141	120	0.145
			12.4	12.4	37.2	5.8	92	68	0.254
			12.4	24.8	49.6	11.7	108	85	0.213
			12.4	37.3	62.1	17.6	121	98	0.190
			12.4	49.7	74.5	23.4	130	108	0.168
			12.4	62.0	86.8	29.2	138	117	0.150
			12.4	120.0	144.8	56.6	174	151	0.130
								Mean Absolute Percentage Error (MAPE)	15.0

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