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COMPRESSIVE STRENGTH AND QUASI-ELASTIC MODULUS BEHAVIOURS OF A CEMENTED LATERITIC SOIL

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ABSTRACT: In Thailand, cemented soils are widely used as a base course material for construction of a flexible pavement structure. In the current design approach, the unconfined compressive strength (q_u) of the cemented soil cured for 7 days is chosen as the key parameter, whereas the deformational behaviour of the pavement structure is not taken into account. On the other hand, it is known that the strains mobilised in the layers are significantly related with the design life of a pavement structure. Therefore, it is necessary to evaluate not only the q_u but also the stiffness of a cemented soil. In this study, a lateritic soil was used to prepare test specimens by mixing with various cement contents (C) and cured for 7 days. Unconfined compression (UC) tests were performed to evaluate the q_u and the average secant modulus (E_{50}) as in the current design approach. In addition, triaxial compression (TC) tests, in which the specimen's axial deformation was measured locally, were performed to reliably evaluate the quasi-elastic Young's modulus (E_{eq}) at various stress states. It is found that the q_u and E_{50} significantly increase with increasing C. The E_{eq} value also increases with increasing C and bulk stress (θ). Dependency of E_{eq} on θ is a kind of hypo-elastic stress state-dependent behaviour, which can be explained by the k-θ model as for the resilient modulus (M_r) used in the mechanistic-empirical design of a pavement structure. The E_{eq} value can be mathematically expressed as a function of k, θ and C.

Keywords: Triaxial compression, Elastic modulus, Cement, Lateritic soil, Pavement structure

1. INTRODUCTION

For a pavement structure, most of base layer materials are usually constructed with compacted crushed rock, which is an unbound granular type. However, pavement construction in Thailand encounters the problem of material deficiency for a long time, especially crushed rock products used for granular base layer. In many cases, the material sources are located far away from the construction site, which results in a high construction cost. In addition, the production of crushed rock aggregate involves drilling, blasting, crushing and road haulage, all of which create dust, causing environmental problems. Although the lateritic soil is local natural material abundantly found in many areas in Thailand, it is poor in engineering properties such as high plasticity, low strength, high permeability, and a tendency to retain moisture content [1–3]. Lateritic soils, especially fine-grained lateritic soils, are not suitable for use as the road base layer. Cement stabilization with the lateritic soil has been widely employed to improve the mechanical properties [4, 5] such that it is strong enough to serve as a road base or subbase layer.

The conventional design method of pavement structure in Thailand is based on empirical rules from behaviours observed during service of the pavement structure or of the experimental sections. For the cement-treated soil, it must satisfy the minimum requirement in terms of unconfined compressive strength (q_u). Moreover, in the construction practice, the q_u value is the only strength parameter of cemented soil used for quality control. Department of Highways (DOH), Thailand and Department of Rural Roads (DRR), Thailand specify the minimum value of q_u for soil cement base of not less than 1717 kPa. The thickness of this soil cement base, typically 20 cm, is also specified. However, this design method does not take the deformation behaviours responded from the traffic loading into consideration.

Development of more rational design methods becomes necessary. Analytical or mechanistic design has been developed and is popular among a number of pavement engineers since 1940 and 1960s, respectively. This design uses fundamental material behaviour (linear or nonlinear-elastic, plastic, viscoelastic, etc.), and theoretical model of each pavement material to predict the response of stresses, strains and deflections due to the traffic loads. Using resilient modulus (M_r) as a key parameter in the design has been widely recommended in many design guides [6–8]. Generally, the M_r is determined from repeated load triaxial apparatus for simulating wheel load [9]. Because the M_r is a stiffness of a material responded after many cycles of traffic loading have been applied until there is no irrecoverable deformation developed (therefore, resilient), it is significantly the same as the elastic stiffness. On the other hand, triaxial compression tests, in which small strain-amplitude cyclic loadings were applied, were performed to determine the quasi-elastic Young's modulus (E_{eq}) of geomaterial [10, 11]. In addition, it was shown that the determination of E_{eq} by triaxial compression test can be alternatively used in place of M_r [12].

This study aims to analyse the elastic modulus of cemented lateritic soil and to report its dependency with the stress state and cement content (C) so that a better understanding of the material can be achieved.

2. MATERIAL AND APPARATUS

2.1 Test Materials

A lateritic soil, which did not satisfy the requirements (i.e., gradation, plastic index, and %CBR) for use as a subbase material of ordinary pavement structure specified in DOH and DRR standards, was used in this study. This lateritic soil contains the amount of fines more than the limit indicated in the standards (finer than sieve No.200 over 20%) as shown in Fig. 1. A series of modified Proctor compaction tests (ASTM D1557) were conducted to determine the maximum dry density (MDD) and the optimum moisture content (OMC). Then, CBR tests (ASTM D1883) on specimens prepared with the water content at OMC were performed to determine %CBR for various values of dry density (ρ_d). Table 1 lists its physical and compaction properties. The %CBR at 95% of MDD is equal to 4.26%. This value is significantly lower than the minimum value specified in the standards (DH-S 205/1989 and DRR-S 202/2014) for use as subbase material (i.e., %CBR= 25%).

2.2 Apparatus

A compression machine, consisting of a reaction frame and a precise gear loading system, was used in the present study. The loading system is driven by a computer-controlled servo-motor and is able to perform load reversal with practically no backlash, which is a very important feature for performing precise cyclic loading test [13]. By controlling the displacement to accuracy of less than 1 μm in an automated way, it becomes possible: i) to smoothly switch between displacement and load control loading phases and between sustained loading or stress relaxation stage and a constant strain rate loading or unloading phases; ii) to apply monotonic loading with a very precise controlled displacement rate; and iii) to apply very small amplitude unload/reload cycles to evaluate the elastic properties of test material during otherwise constant strain rate monotonic loading. This gear loading system has a capacity of 50 kN.

Table 1. Physical and index properties of the lateritic soil

Properties	Lateritic soil
Specific gravity, G_s	2.89
Liquid limit, LL (%)	31.3
Plastic limit, PL (%)	14.7
Optimum water content, OMC (%)*	10.79
Maximum dry density, MMD (g/cm^3)*	2.099
CBR at 95% of MMD (%)	4.26

*Modified Proctor compaction test (ASTM D1557)

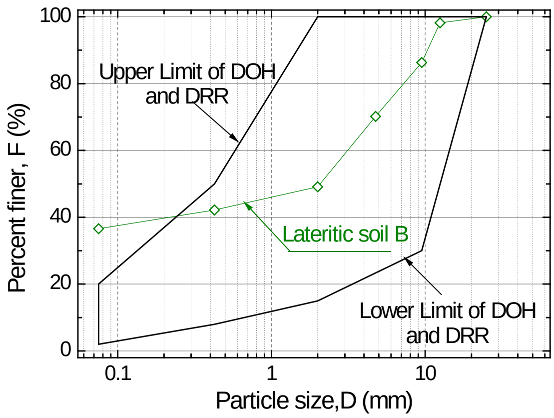


Fig. 1 Gradation characteristics of the lateritic soil used in this study in comparison with DOH and DRR standards.

3. TEST METHODS

3.1 Specimen Preparation

The lateritic soil has been treated with Portland cement type I to improve the mechanical performance for using as the material for base in the pavement structure. The C value was varied at 1%, 2%, 3%, 4%, and 5% by dry weight of the lateritic soil, and then after being compacting in mould, the specimens were cured for 7 days. This curing period is for verification in the construction process, also specified in the DOH and DRR standards (DH-S 204/1990 and DRR-S 244/2013). The cement-treated lateritic soil specimens were wrapped with the plastic film in order to avoid loss of moisture after dissembling the mould and were kept in an incubator for 7 days. For all the tests, the specimens are cylindrical. They are 150 mm high and 70 mm in diameter. Specimens were prepared to achieve the dry density equal to maximum dry density (MDD). The values of water content and wet density of a specimen were controlled not to vary by more than ±1% of OMC and ±3% of the target value, respectively.

3.2 Test Procedure

There were two types of test in this study: i) unconfined compression (UC) test; and ii) triaxial compression (TC) test. The former was performed to evaluate the q_u value and average secant modulus (E_{50}), while the latter the E_{eq} value, of the cemented lateritic soil. Details of these tests are as follows.

3.2.1 Unconfined compression test

Unconfined compression tests were performed by applying continuous monotonic loading (ML) to the test specimens with a constant strain rate (0.0277 %/minute) until failure to obtain the q_u value. Moreover, the top cap and pedestal surfaces were lubricated by a 50-μm layer of high vacuum silicone grease adhering to a 0.3-mm rubber sheet to reduce friction at the specimen ends [14].

3.2.2 Triaxial compression test

A specimen is set on the pedestal and a membrane is put on it. The axial load cell used in triaxial compression (TC) tests performed in the present study was connected in series with the cap inside the chamber so that its reading was free from friction that may be mobilised at the bearing house on top of the chamber. Prior to mould disassembly, suction of -20 kPa was temporarily applied to the specimen via the drainage lines connected to the cap and the pedestal so that the membrane was adhered firmly with the specimen's side surface. Then, a pair of local deformation transducers (LDTs) were installed on the pseudo-hinges firmly glued on the membrane at the opposite diametrical sides, and three clip gauges (CGs) were placed at the height of 1/5, 1/2 and 4/5 of the specimen's initial height, as shown in Fig. 2. These LDTs and CGs were calibrated with a micrometre head mounted with a cross slide roller table. In this paper, the axial strain and the radial strain in TC tests are the averages of readings from two LDTs and three CGs, respectively. In addition, the specimen's axial deformation was also measured externally with a LVDT, which was necessary when the measuring range of LDT was exceeded.

The TC test was performed by applying small strain-amplitude cyclic loadings (CLs) at various $q{:}p$ stress states to determine the E_{eq}. In these TC tests, the E_{eq} value were determined from local axial deformation measured by a pair of local deformation transducers (LDTs, [15]), so the E_{eq} is free from bedding error. These $q{:}p$ stress states are in accordance with AASHTO T307-99 standard [9]. The loading pattern for the TC was presented by Dararat et al. [12]. That is, the sample is first isotropically confined with different values of cell pressure (σ_3). Then it is continuous monotonic loading (ML) sheared by axial compression to a target deviator stress (q) value, while the cell pressure is kept constant to achieve a target $q{:}p$ stress state. After that, sustained loading (SL) is performed for 30 min, holding the stress state at the target, while the sample is allowed to deform (i.e., creep). Next, cyclic loadings (CL), of which the stress-amplitude is equal to 30 kPa, are performed for 10 cycles for evaluating the E_{eq} value. Then, ML shearing is performed to the next target q value, at which 30-min SL and then CLs for 10 cycles are repeated. After finishing CLs at the largest target q value, q is reduced to zero, and then the cell pressure is increased to the next target value under isotropic condition. Then, similar shearing processes as of the first target cell pressure are repeated. In this TC test, the specimen's response was measured at 15 $q{:}p$ stress states by varying five different confining pressures and deviator stresses. More details of this TC apparatus and test procedures can be found at Dararat et al. [12].

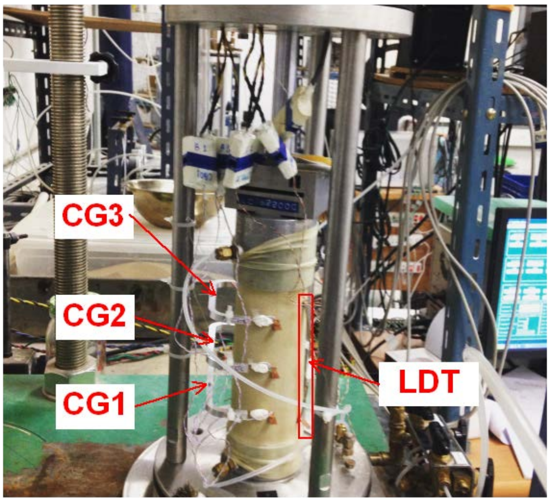


Fig. 2 Installations of LDTs and CGs for measurements of axial and radial deformations free from any bedding errors.

4. TEST RESULTS AND DISCUSSIONS

4.1 Unconfined Compression Test

4.1.1 Unconfined compressive strength (q_u)

Figure 3 shows the relationships between deviator stress (q) and axial strain (ε_a) obtained from unconfined compression tests on lateritic soil treated with five different C values. Three specimens were prepared for each respective C value. Note that the axial strain values (ε_a) presented in Fig. 3 were the ones measured by using LVDT. All the specimens had been cured for 7 days before the start of test. The q_u is defined as the peak value of deviator stress along the respective q-ε_a relationships obtained by monotonic loading test with a constant axial strain rate (i.e., 0.0277 %/min). Then, the q_u values from the three specimens with the same C were averaged. Figure 4 shows the relationship between the averaged q_u and C. It can be clearly seen that with increasing C, the q_u value increases significantly. A function expressed in Equation (1) was best fitted to all data points shown in Figure 4.

$$q_u = 1390.587(C)^{0.454} \tag{1}$$

where q_u and C are in kPa and %, respectively.

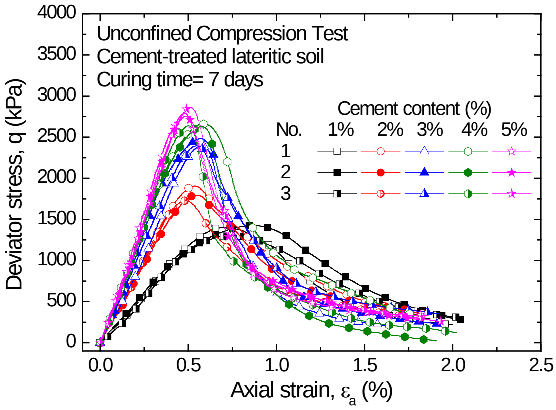


Fig. 3 Relationships between deviator stress (q) and axial strain (ε_a) of lateritic soil treated with cement with different C values

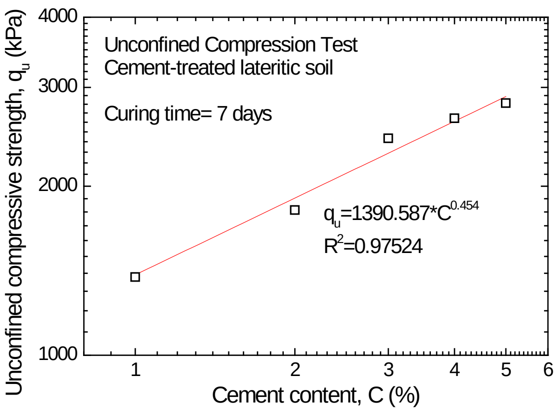


Fig. 4 Relationship between q_u and C

4.1.2 Elastic modulus

Elastic stiffness of the cemented lateritic soil was defined from the E_{50} derived from the q-ε_a relationship obtained by UC test. This E_{50} is defined as the slope of the line passing though the origin and the point along the q-ε_a relationship where q is equal to 50% of q_u. It can be observed that both q_u and E_{50} values increase with increasing C in a similar manner. Thus, the E_{50} values were then plotted with the q_u values for the respectively same C values as shown in Figure 5. It is obvious that the E_{50} increases linearly with the q_u. Therefore, a line expressed with Equation (2) was best fitted to all data points shown in Figure 5.

$$E_{50} = 208.804 q_u \tag{2}$$

In practice, the E_{50} value of a chemically stabilised material, which is obtained from UC test, is used as the E value in the analysis for the responses of a pavement structure. In the case where the E_{50} value is unknown, it is usually estimated from the q_u value at the respective C value, in a similar manner to the expression in Equation (2).

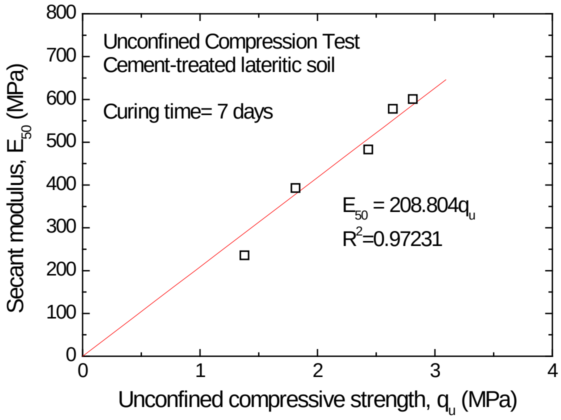


Fig. 5 Relationship between E_{50} and q_u

4.2 Triaxial Compression Test

4.2.1 Stress-strain relationship

As the elastic modulus is the parameter that is of interest in this study, the behaviours at small strain level must be reliably measured, and therefore the local strain measurement was employed by using LDTs. Figure 6 shows the relationship between deviator stress (q) and axial strain (ε_a) from the TC test employing continuous monotonic loading test, intervened by 30-min sustained loadings, after which ten small-strain amplitude unload-reload cycles were performed, with a constant strain rate of 0.0277 %/min, on lateritic soil treated with 3% cement. In addition, Fig. 7 shows a zoomed-up portion of Fig. 6, and only axial strain measured by using a pair of LDTs ($\varepsilon_{a,LDT}$) is presented.

These figures show that at the same deviator stress (q), the value of axial strain (ε_a) measured by LVDT is always greater than that measured by a pair of LDTs, which is due to the measuring errors consisting of system compliances and bedding errors. The axial strain measured by using a pair of LDTs ($\varepsilon_{a,LDT}$) are found to give a sound basis for axial strains measurement at small strains. Moreover, from Fig. 7, it can be readily seen that the q-$\varepsilon_{a,LDT}$ loops during the respective unload-reload cycles are very small, which are consistent with the fact that the residual axial strain developed by these cycles is very small. Therefore, the behaviour during small unload-reload cycles is highly linear-elastic.

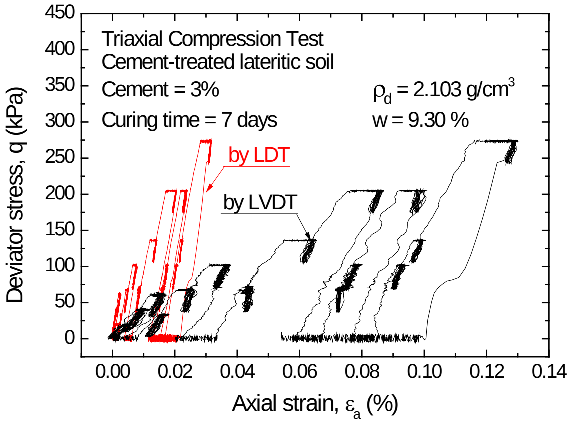


Fig. 6 Relationships between deviator stress (q) and axial strain (ε_a) obtained from small-strain amplitude cyclic loading test on cement-treated lateritic soil with C = 3%.

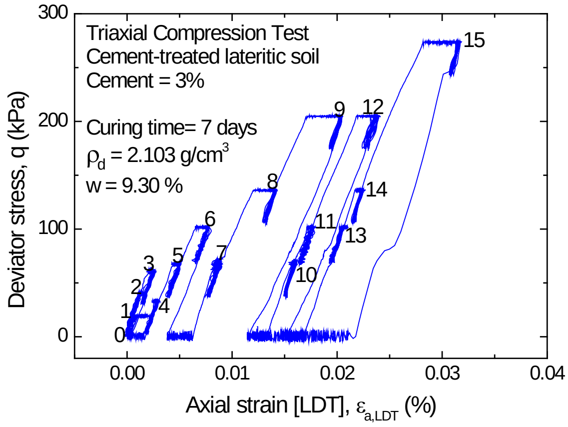


Fig. 7 Close-up of q-$\varepsilon_{a,LDT}$ relationship obtained from small-strain amplitude cyclic loading test on cement-treated lateritic soil with C = 3%.

4.2.2 Quasi-elastic Young's modulus (E_{eq})

Figure 8 shows the unloading q-$\varepsilon_{a,LDT}$ branches for the loop nos. 6-10, by small unload-reload cycles at the sequence no. 5, at which q = 68.9 kPa and p = 57.5 kPa, obtained from the TC tests on lateritic soil treated with C = 3%. It is obvious that the q-$\varepsilon_{a,LDT}$ branches exhibit highly linear-elastic behaviour for the whole vertical deviator stress amplitude (30 kPa). Thus, the E_{eq} was determined from a linear relation fit to the unloading branches presented in Fig. 8. Each E_{eq} value can be defined with a degree of confidence, as confirmed by the values of coefficient of determination (R^2-value) shown in the figure. For each sample, the E_{eq} value is mostly constant among loop nos. 6-10, implying that the behaviour during these unloading branches is significantly linear-elastic. The E_{eq} at the other q:p stress states (shown in Fig. 7) were determined in the same way as of Fig. 8. It is worth noting that as axial strain value is measured locally, and hence free from bedding error, the E_{eq} value defined as shown in Fig. 8 is of the true value.

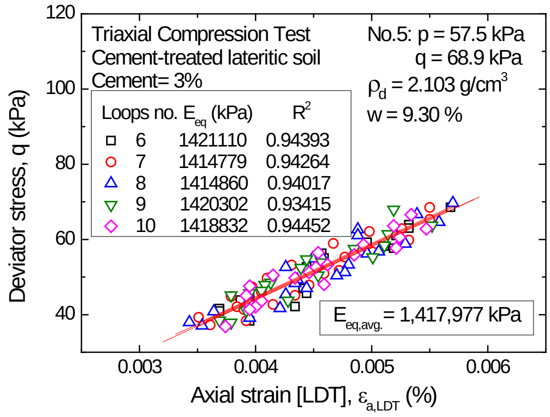


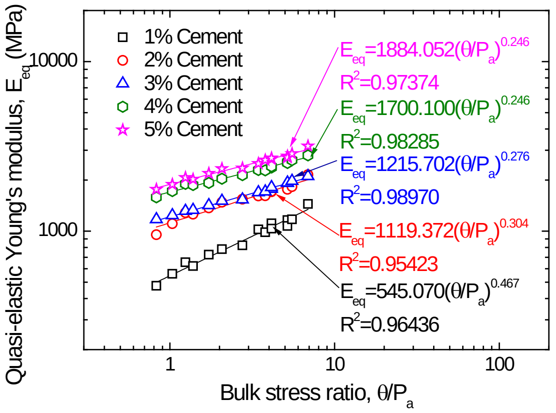
Fig. 8 Relationships between deviator stress (q) and axial strain measured by LDTs ($\varepsilon_{a,LDT}$) during unloading branches to determine the E_{eq} on lateritic soil treated with C = 3%

Figure 9 shows relationships between the average E_{eq} and the bulk stress (θ) in a full-log plot for lateritic soil treated with cement with the C values of 1%, 2%, 3%, 4%, and 5%. The θ value is normalised by the reference pressure (P_a) of 100 kPa. The E_{eq} value increases significantly with an increase in the bulk stress ratio (θ/P_a) value. That is, the E_{eq} of cement-treated lateritic soil is also of hypo-elastic type. Dependency of E_{eq} with θ can be mathematically expressed by Eq. 3.

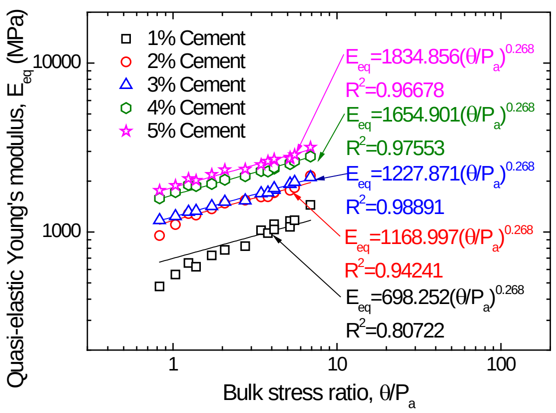
$$E_{eq} = E_0 \left(\frac{\theta}{P_a} \right)^m \quad (3)$$

where E_0 is the value of E_{eq} when $\theta = P_a$ =100 kPa; and m is constant. The lines were best-fitted to the test data points shown in Fig. 9(a). The values of E_0 and m for respective test samples are shown in Fig. 9(a). The functional forms of E_{eq} in Eq. 3 is similar to the k-Theta (k-θ) model [16].

Considering at the m value, it could be seen that the m values obtained from test samples with different C values are quite similar. This implies that the characteristics of increasing E_{eq} with θ for different C values are very similar. For this reason, averaging the m values was attempted and the averaged m value (m_{avg}) of 0.271 was obtained with an exemption that the value of sample for the C = 1% was excluded. Then, regression analysis was re-performed for different C values using Eq. 3, but with the fixed value of $m = m_{avg}$ as shown in Fig. 9(b). The new values of E_0 and m determined from the regression analysis with the fixed value of $m = m_{avg}$ are also shown in Fig. 9(b). Although the data are scattered to some extent, especially for the sample with 1% cement as seen from Fig. 9(b), the E_{eq} value can be defined with a degree of confidence, as confirmed by the coefficient of determination (R^2-value) value shown in the figure.



(a)



(b)

Fig. 9 Relationships between E_{eq} and θ/P_a for lateritic soil treated with various C values using: (a) respective m; and (b) common average m.

The E_0 values by regression analysis with the common average m value are plotted against the C value in full-logarithmic scale as shown in Fig. 10. It seems that the E_0 is rather a function of C. The relationship between the E_0 and C can be fitted using the mathematical form expressed in Eq. 4.

$$E_0 = E_{0,C} C^n \tag{4}$$

where $E_{0,C}$ is the value of E_0 when C=1% (equal to 716.519 MPa); n is constant (equal to 0.582). By combing Eq. 3 with Eq. 4, E_{eq} of cement-treated lateritic soil can be mathematically expressed as follows.

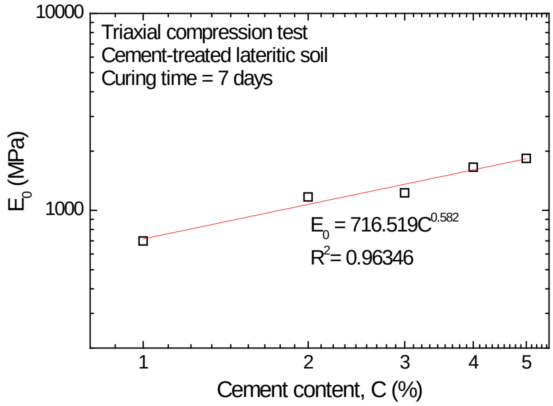


Fig. 10 Dependency of the E_0 value with C.

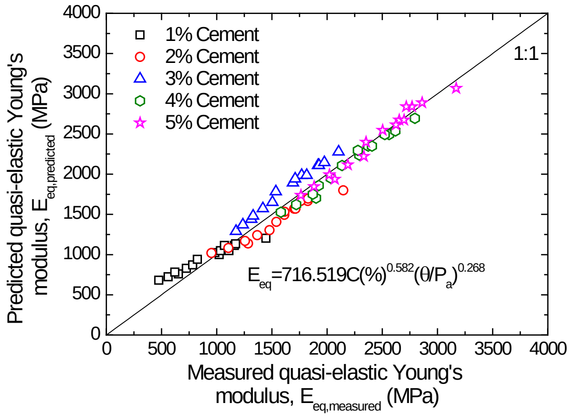


Fig. 11 Comparison between the predicted and measured values of E_{eq}.

$$E_{eq} = E_{0,C} C^n \left(\frac{\theta}{P_a}\right)^m \tag{5}$$

where $E_{0,C}$ is the value of E_0 when C=1% (equal to 716.519 MPa); n is constant (equal to 0.582); m is constant (equal to 0.268); and P_a is the reference pressure (equal to 100 kPa). Fig. 11 shows the comparison between the predicted (by substituting the θ and C into Eq. 5) and the measured E_{eq} values for cement-treated lateritic soil. It can be seen that Eq. 5 gives very satisfactory prediction results as with a good agreement shown in Fig. 11.

5. CONCLUSION

From the test results and analyses performed in this study, the following conclusions can be derived:

1) The compressive strength (q_u) and average secant stiffness (E_{50}) of cement-treated lateritic soil significantly increase with an increase in the cement content (C).
2) The triaxial compression test (TC) employing the small strain-amplitude cyclic loadings can be used to evaluate the true quasi-elastic Young's modulus (E_{eq}) for cement treated lateritic soil by using the local displacement transducers (LDTs) to locally measure the axial strain. The E_{eq} value of test samples exhibited significant dependency on the bulk stress (θ) and the C value.
3) The equation to estimate E_{eq} value from this study can be used as the M_r in the solution for the design and analysis of pavement structure.

6. ACKNOWLEDGMENTS

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