COMPRESSIBILITY AND STRENGTH OF CLAY SUBGRADE OF RAILROAD FOUNDATION IN HIGHLY SATURATED CONDITION

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ABSTRACT

The aligning of railroad crossing lowland of high plasticity clay formation soil may at times become necessary despite it being technically undesirable. This has called for better understanding and extra engineering consideration / measures in the design of railroad foundation with respect to the subgrade strength capacity. This paper describes the laboratory investigation into the effect of cyclic loading produced by contemporary train on the compressibility and strength deterioration of clay subgrade in general, and kaolinite clay in particular. Concept of the lower-bound stability threshold stress is proposed for the characterization of clay subgrade and other fine-grained soil of potential high saturation, in place of the traditional tests and performance standard being used. In conjunction with this, the paper presents the theoretical model which is capable of predicting the lower-bound threshold stress of clay soil based on either the known consolidated state and stress history or the in-situ compacted state, from the fundamental soil properties obtainable in basic testing. Thereby eliminate the need for frequent lengthy testing and cyclic loading laboratory capability.

Key words: Railroad clay subgrade, cyclic triaxial testing, threshold stress, soil compressibility.

1. INTRODUCTION

The running of railroad crossing lowland of clay formation soil may be unavoidable due to topographical constraints or economical consideration. Subgrade soil of clay material has been traditionally looked upon as being unpredictable and problematic in the geo-engineering context. In the design of railroad foundation, a better understanding of its behaviour in the cyclic load / environment interaction is essential if track vertical alignment will to be maintained

The passing of train wheels induces stress of cyclic nature on the subgrade soil beneath railroad tracks that can result in progressive shear failure at a stress level lower than its monotonic strength (Selig and Waters 1994). British researchers had established the existence of 'threshold stress' (or the critical level of stress) in London clay (Heath *et al.* 1972; Waters 1968), where induced stress above such level incurs large deformation, and below which deformation stabilization will be achieved irrespective of the numbers of loading applications (Ansal 1989; Lefebvre 1986; Procter 1984; Sangrey 1969). When subgrade of railway track is being stressed at a level lower than threshold stress of the subgrade soil, stabilization of the track vertical alignment will be achieved.

In general, the formation soil exists beneath future railroad may be in its natural undisturbed state or a compacted state for 'fill' soil. Further compaction of specific effort is normally given at its optimum moisture content, rendering the prepared subgrade in a quasi-saturated compacted state. However, in the case of fine-grained soil, *i.e.*, clay in particular, not only it is difficult to attain homogeneous compaction effort in practice, there can be many changes to the soil physical state that will affect the behavior of compacted clay after the compaction and during service, largely due to the moisture variation both within soil mass and in the longitudinal alignment of the track, interacted with the applied loading. Consequently, characterization of the subgrade soil strength based on conventional tests carried out on the compacted quasi-saturated soil specimen provides little purpose in the design and evaluation of railroad foundation.

This paper presented and discussed the results of the cyclic undrained triaxial tests on fully saturated kaolinite clay, under cyclic loading frequency of 1 Hz, simulating the stress condition at the railroad subgrade due to passing of train wheels / axles at 50 km/hour. Several consolidated series spanning across a wide range of over-consolidated state, *i.e.*, from sub-critical to super-critical region, were investigated. The paper discussed the axial compressibility of kaolinite clay in cyclic condition and described the determination of threshold stress for four different over-consolidated states, interpreted using critical state soil mechanics framework.

The notion of the lower-bound stability threshold is introduced for the characterization of clay subgrade soil in place of the conventional means. In addition, a critical state model and corresponding charts are proposed for the prediction and reassessment of lower-bound threshold stress for general clay subgrade exist in either in-situ compacted state or undisturbed preconsolidated state.

2. EXPERIMENT AND TEST PROCEDURE

2.1 Equipment and Specimen Preparation

Tests were conducted using a commercially available cyclic system automated test unit as shown in Fig. 1. The equipment

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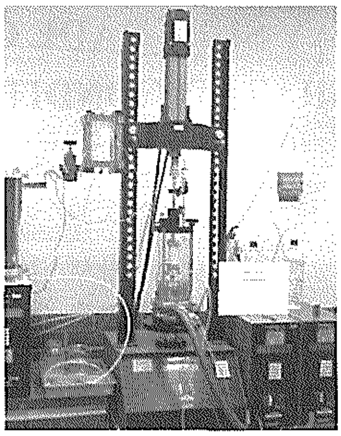

Fig. 1 Cyclic triaxial testing system

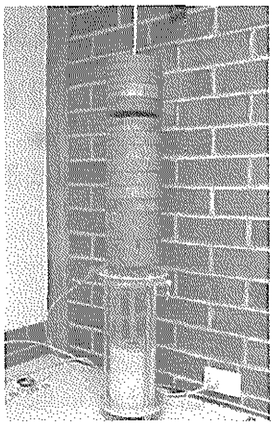

Fig. 2 One-dimensional consolidation of reconstituted slurry

unit consists of a triaxial cell, a load frame, two computer-controlled flow pumps for delivering the cell and the back pressure, a high performance linear servo control electro actuator for applying cyclic loading, and a PC for control the test and data logging. Various transducers are mounted in the system for measuring the axial load, confining pressure, pore water pressure and axial strain.

The clay used in the investigation was saturated kaolinite clay of specific gravity $G_s = 2.65$. Plastic limit of soil $w_p = 26\%$, and Liquid limit of soil $w_L = 53\%$. The coefficient of consolidation $c_v = 2.41$ mm^2/min at an effective isotropic consolidation of 300 kPa. The clay was reconstituted by one-dimensional consolidation (as shown in Fig. 2) of the slurry prepared by the mixing of dried commercial grade kaolinite powder with distilled water. Tubes of 35 mm inner diameter were gently pushed into the consolidated clay, and tube samples were extracted, sealed and stored. The final prepared specimen size was 35 mm in diameter and 70 mm in height.

The re-constituted kaolin clay has the fundamental properties commonly described under critical state soil mechanics framework, as follows:

Slope of the critical state line in p'–q plane,

$M = 0.803$

Volumetric compressibility of normal consolidation,

$\lambda = 0.177$

Volumetric compressibility of recompression line,

$\kappa = 0.068$ and

Specific volume of soil under normally-consolidation at $p' = 1$ kPa, $N = 2.94$

2.2 Test Details

Four series of stress-controlled cyclic tests were carried out on reconstituted saturated kaolinite clay. They consist of an isotropic normally-consolidated series and three isotropic over-consolidated series which have the over-consolidation ratio (OCR) of 1.5, 4 and 20.

The specimens were isotropic-consolidated using a back pressure of 200 kPa. Over-consolidated specimens were prepared by prior consolidation of specimen to a higher pressure, and then allow it to swell under a lower isotropic consolidation pressure. For consistency, during the consolidation or the swelling stage, the cell pressure was maintained for a period equal three times the duration that is required for 100% primary consolidation t_{100} (Head 1986). The final consolidation pressures (or the initial mean effective stress P_i') were: 300 kPa, 200 kPa, 75 kPa and 30 kPa, for normally-consolidated series, over-consolidated series with OCR of 1.5, 4 and 20 respectively.

Prior to carrying out the cyclic compression tests, static axial compression tests were conducted at a control rate of deformation between 0.0035 ~ 0.015 %/min. for each of the tests series. For the cyclic undrained compression tests, soil specimen was subjected to the continuous application of cyclic deviator stress (σ_d or q) of sinusoidal wave form (as shown in Fig. 3) at a loading frequency of 1 cycle per second (1 Hz). Throughout the test, constant observations on the changes in the pore water pressure and the axial deformation were made and captured digitally by the system. The cyclic loading continued until both the excess pore water pressure and the deformation become stabilized, or stopped when the specimen failed with large deformation. The total number of load cycles for all tests were taken to 150,000 cycles and beyond (whenever neccessary). Despite high cyclic loading frequency rate, equalization of pore water pressure within specimen will be taken place, after the deformation or strain has become stabilized (Seed and Martin 1966).

3. TEST RESULTS AND DISCUSSION

3.1 Deformation Characteristics

<u>Cumulative Total Strain</u>

Cumulative total strain is referred as the permanent (irrecoverable or plastic) axial strain, plus the temporary (recoverable or elastic) axial strain.

Under cyclic undrained compression, clay specimen with deviator stress higher than the anticipated threshold stress developed large strain and failed (specimen become squashed) ultimately. However, for deviator stress below the anticipated "threshold stress", permanent strain developed gradually and ultimately become stabilized. Generally, the number of load cycles required for the strain to become stabilized increases with the level of deviator stress. For the normally-consolidated series, cumulative total strain began increasing at a reducing rate after

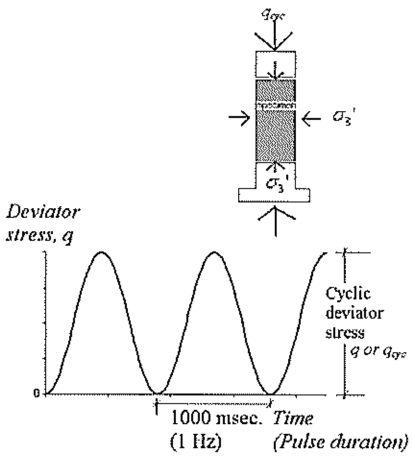

Fig. 3 State of stress during cyclic compression

about 50,000 cycles before it become stabilized (Fig. 4). For the over-consolidated series, the stabilization of the cumulative total strain appeared to occur a lot quicker with respect to the number of load cycles, as shown in Figs. 5 to 7.

The cumulative total strain ε_a which became stabilized can be seen to relate to the stress obliquity q / p' differently for each tests series, as shown in Fig. 8. The relationship may be expressed in the following form:

$$\varepsilon_a = A(q / p')^B \% \tag{1}$$

where A and B are the constants from regression analysis of the tests data

The value of the constant A and B, and the coefficient of determination R^2 of experimental model thus obtained for the respective tests series are summarized and shown in the following Table 1.

Resilient Modulus

Knowledge of the elastic behavior of general foundation soil is often required in the analysis of dynamic response for geo-structure interaction where stress of cyclic nature is prevalent. For railroad subgrade soil, knowledge of its resilient modulus E_r (as defined in Eq. (2) below) is necessary for calculating stresses and deflection in the idealized layered foundation system and for analyzing the system performance using computer software models.

$$E_r = \frac{q}{\varepsilon_r} \tag{2}$$

where q is the cyclic deviator stress, and ε_r is the recoverable or elastic axial strain

Under cyclic undrained compression, the resilient modulus of normally-consolidated clay decreases but become constant as the cumulative total strain become stabilized (see Figs. 9 and 4), whereas the resilient modulus of the over-consolidated clay is very much constant irrespective of the number of loading cycles as shown in Fig. 10.

Brown *et al.* (1975) proposed an improved model to the power model (Moossazadeh and Witczak 1981) which described the relationship between the resilient modulus and the applied stress for saturated over-consolidated soil as follows:

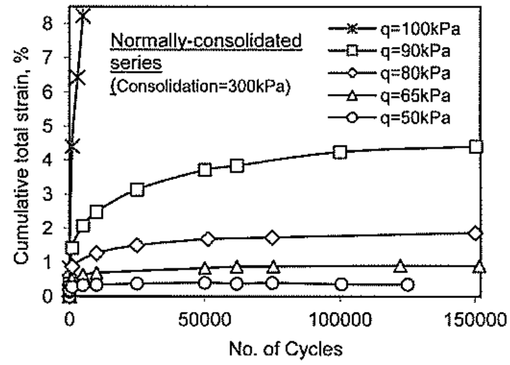

Fig. 4 Development of cumulative total strain over each cycle (Normally-consolidated series)

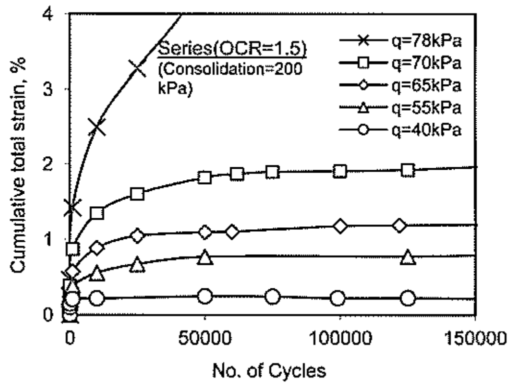

Fig. 5 Development of cumulative total strain for over-consolidated series (OCR = 1.5)

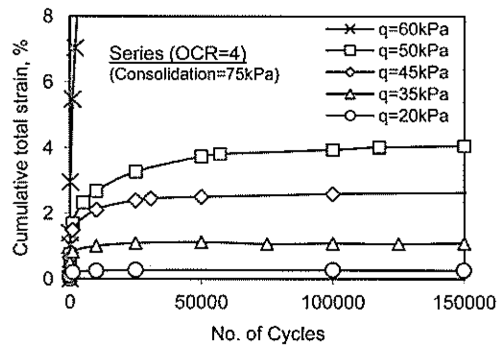

Fig. 6 Cumulative total strain for over-consolidated series (OCR = 4)

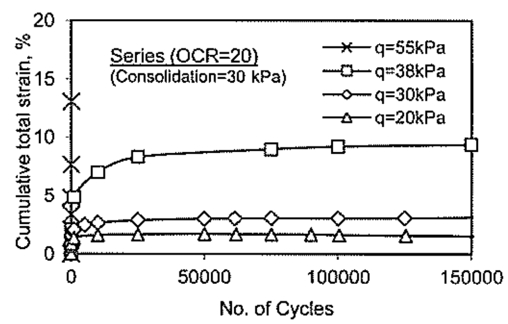

Fig. 7 Cumulative total strain for over-consolidated series (OCR = 20)

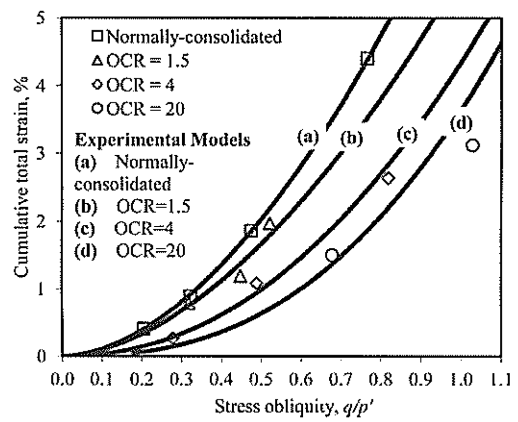

Fig. 8 **The relationship of cumulative total strain with respect to stress obliquity for the various consolidated series**

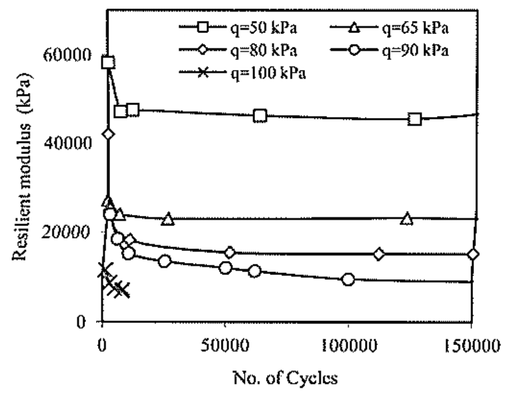

Fig. 9 **Variation of resilient modulus with number of load cycles (kaolinite clay: normally-consolidated at 300 kPa)**

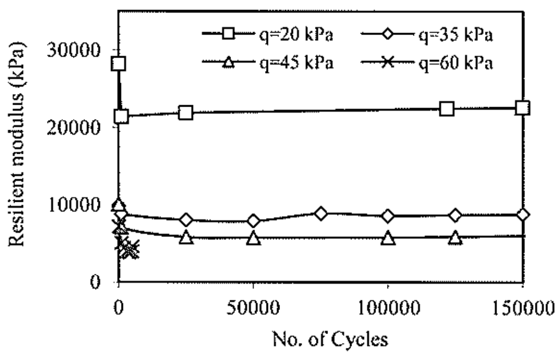

Fig. 10 **Resilient modulus (kaolinite clay; over-consolidated with OCR = 4)**

Table 1 **Constant A and B of experimental model, with coefficient of determination R^2, obtained from regression analysis of tests data for the respective test series. (Re-constituted kaolinite clay)**

State of Consolidation	A	B	R^2
Normally-consolidated	7.11	1.81	1
OCR = 1.5	5.65	1.77	0.93
OCR = 4	4.34	2.13	0.99
OCR = 20	3.64	2.52	0.96

$$E_r = k\left(\frac{\sigma_d}{\sigma_3'}\right)^n \tag{3}$$

where k and n are parameters dependent on soil type and physical state. σ_3' is the effective confining stress.

Using the similar expression, a general correlation can be described of the resilient modulus with respect to the effective stress obliquity q / p' for saturated clay of all consolidated states (with σ_3' replaced by the effective mean stress p') as follows:

$$E_r = k(q / p')^n \tag{4}$$

The correlation as described above appeared to be strong for these tests series, as can be seen from Fig. 11 with higher stress obliquity produces lower resilient modulus. The resilient modulus of the saturated kaolinite clay under cyclic undrained compression may be expressed in the following equation:

$$E_r = 4092(q / p')^{-1.5} \text{ kPa}$$

(for kaolinite clay of all consolidated states, with $R^2 = 0.9$) (5)

3.2 Pore Water Pressure Response and Peak Stress Path

<u>Normally-Consolidated Series</u>

Under continuous application of cyclic deviator stress of sinusoidal form, pore water pressure also varies in the sinusoidal fashion. In the case of normally-consolidated clay series, excess pore water pressure gradually built up over each cycle of deviator stress application, oscillating in the same phase as the cyclic deviator stress. Figure 12 showed the built-up of excess pore water pressure corresponding to the peak of each cyclic deviator stress, for the normally-consolidated tests series. For the tests which attained stabilized total cumulative strain (Fig. 4), excess pore water pressure gradually increased with the total cumulative strain until both become stabilized (Fig. 12), signifying the arrival of a state of resiliency, at which stress within the specimen was in a state of equilibrium and axial strain responded elastically. In the $p' - q$ plot, the term 'peak stress path' is introduced to depict the collection of the peak state of stress of each loading cycle at which the deviator stress is at the maximum in cyclic undrained compression test. Cyclic deviator stress produces peak stress path which represents the path of most critical and vulnerable effective stress state of each load cycle. Figure 13 showed the overall view of the peak stress paths of the cyclic compression tests of various deviator stress level, for the normally-consolidated series. The peak stress paths for all cyclic compression tests (with the deviator stress less than 90 kPa), migrated horizontally towards the left and became stagnated before the critical state line (CSL), after about 100,000 cycles. Upon which, cyclic stress path formed hysteresis loops, signifying the arrival of state of stress equilibrium within the specimen (cyclic stress equilibrium state). However, for the test with cyclic deviator stress q = 100 kPa, peak stress path migrated and reached the CSL at a relatively early stage of cyclic compression test (*i.e.* about 8,000 cycle), thereupon soil specimen failed with large strain.

Figure 14 showed the stress path of static axial test conducted. Comparing it with Fig. 13, it appeared that the stress path of the static axial compression test exists above the state of stresses of the stagnated end of peak stress paths of the cyclic compression tests.

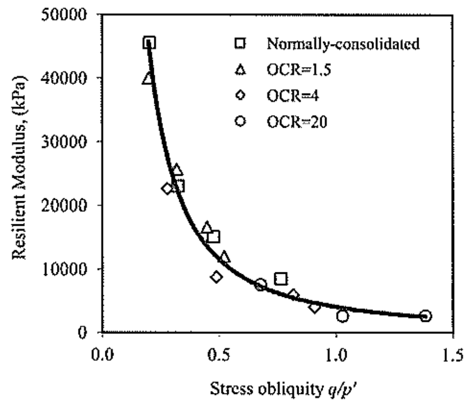

Fig. 11 The variation of resilient modulus with respect to stress obliquity (kaolinite clay)

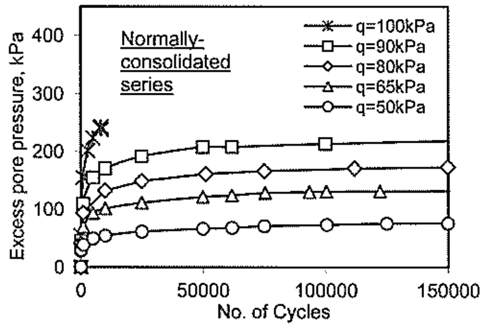

Fig. 12 Accumulation of excess pore water pressure corresponding to the peak deviator stress in the cycle (kaolinite clay; consolidation = 300 kPa)

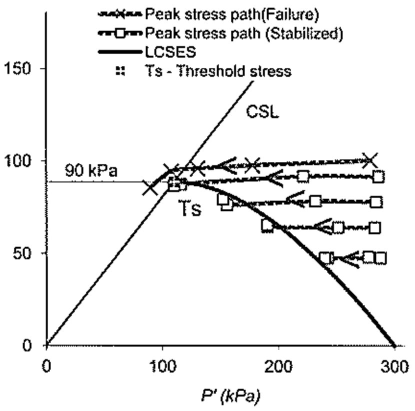

Fig. 13 The peak stress paths for tests of various cyclic deviator stress migrated leftward (kaolin clay; normally-consolidated at 300 kPa)

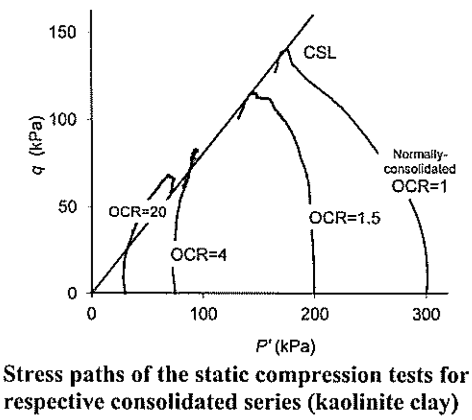

Fig. 14 Stress paths of the static compression tests for the respective consolidated series (kaolinite clay)

Over-Consolidated Series

For the over-consolidated clay series, except for tests with $q > 40$ kPa in the series (OCR = 1.5) where pore water pressure response similar to the normally-consolidated series were observed (Fig. 15), pore water pressure of the over-consolidated series exhibited a response distinct from that of the normally-consolidated series. For tests which arrived at a state of resiliency, excess pore water pressure literally rose by a fraction of the magnitude of deviator stress from the very first load cycle, thereafter oscillated in constant range with little or no accumulation (as shown in Figs. 16 and 17). Peak stress path of each test in these series (OCR = 4, and 20) depicted a short path close to a dot point, which indicated that state of cyclic stress equilibrium within the specimen can only be inherent, irrespective of cyclical loads (Figs.18 to 20). Likewise, similar pore water pressure response was noticed for tests which led to ultimate failure.

For the over-consolidated series, the peak stress path (which resembled as a dot) appeared to be coincide with the stress path of static compression test at the corresponding deviator stress level (q), for the cyclic deviator stress of the lower range (compare Figs. 14, 18 to 20).

3.3 Threshold Stress Identification

Sangrey (1969) reported an effective stress analysis and suggested the straight equilibrium line concept that identifies the critical level of cyclic stress (threshold stress). In this study, using critical state soil mechanics framework, the threshold stress for kaolinite clay of the respective consolidated series can be established by identifying the maximum cyclic deviator stress which can possibly arrive at the state of resiliency. By connecting (or taking the best-fit line of) the locus at the stagnated end of all peak stress paths, the line of cyclic stress equilibrium state (LCSES) was formed (see Figs. 13, 18 to 20). In the case of normally-consolidated series and the over-consolidated series (OCR = 1.5), the LCSES when extrapolated to intercept the CSL at the point "Ts", indicates the level of threshold stress (q_t) (the level of deviator stress (q) at the intercept). For the other over-consolidated series (OCR = 4), the LCSES appeared to be terminated on or slightly above the CSL; whereas, it terminated well above the CSL in the case of series (OCR = 20). In these two over-consolidated series, cyclic deviator stress higher than its threshold stress showed little sign of pore water pressure accumulation or regression, but failed ultimately with large strain. The threshold stress for these two over-consolidated series was established by determined the maximum cyclic deviator stress that will attain the stabilized total cumulative strain, through carrying out additional tests at deviator stress level between the highest cyclic deviator stress of test which had achieved stabilized strain and that which had failed with large strain.

Normalized by the respective pre-consolidation stress P_o', Fig. 21 presented an overall view of the characteristic pattern of a LCSES for the four typical over-consolidated states (including normally-consolidated state where OCR = 1) ranging from the sub-critical to the super-critical region.

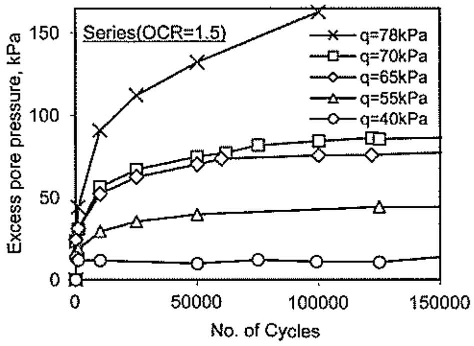

Fig. 15 Accumulation of excess pore water pressure over each load cycle (kaolinite clay; consolidation = 200 kPa: OCR = 1.5)

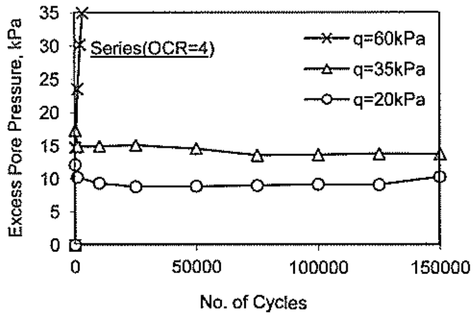

Fig. 16 Accumulation of excess pore water pressure over each load cycle (kaolinite clay; consolidation = 75 kPa: OCR = 4)

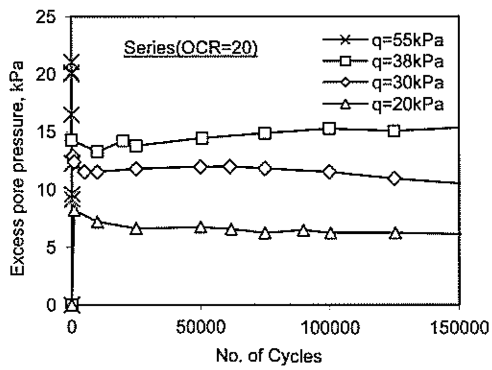

Fig. 17 Accumulation of excess pore water pressure over each load cycle (kaolinite clay; consolidation = 30 kPa: OCR = 20)

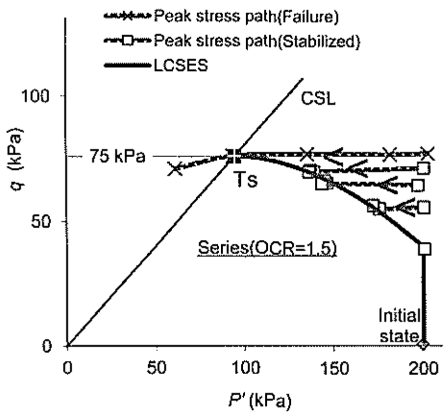

Fig. 18 The peak stress paths and state (kaolinite clay; consolidation = 200 kPa; OCR = 1.5)

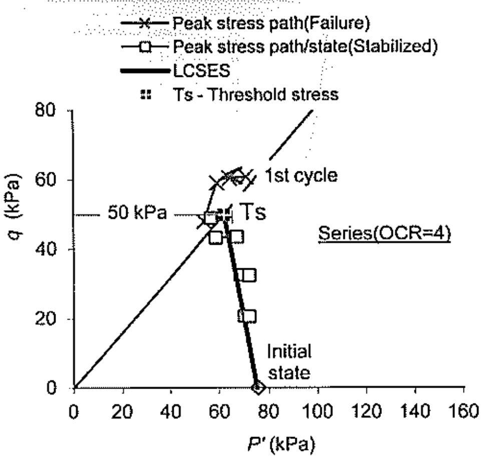

Fig. 19 The peak stress paths / states (kaolinite clay; consolidation = 75 kPa; OCR = 4)

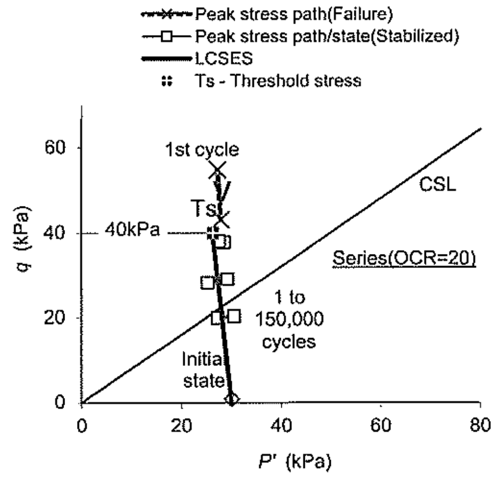

Fig. 20 The peak stress paths / states (kaolinite clay; consolidation = 30 kPa; OCR = 20)

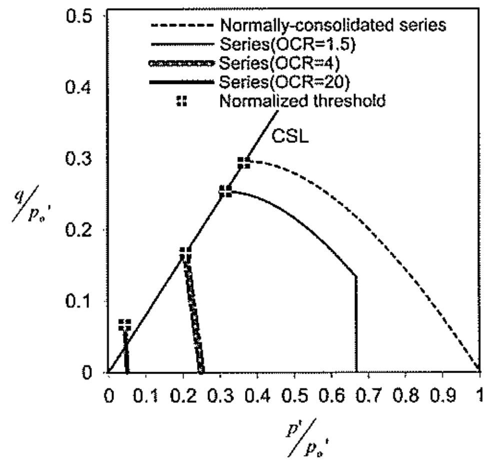

Fig. 21 The line of cyclic stress equilibrium state (LCSES) for the four typical over-consolidated series, normalized by past maximum pre-consolidation pressure (kaolinite clay)

4. CONSTITUTIVE MODELLING AND PRACTICAL APPLICATION

4.1 Characterizing and Managing the Current Lower-bound Threshold Stress

The conventional ballasted railroad, including the modern fixed rail platform, is commonly constructed with timber or concrete sleepers acting like pad footings bearing on the subgrade soil. Subgrade deformation below sleepers generally varies linearly along the track given that subgrade soil can reasonably be assumed homogeneous throughout its influence depth unless otherwise, or except when change of soil foundation stiffness is anticipated, *e.g.*, at the culvert crossing, Differential settlement due to large localized deformation caused by progressive shear failure will be the major problem of track irregularities, assuming overall foundation stability had been satisfied. The objective of the railroad foundation design and track management, therefore, is to ensure that the induced stress at the subgrade soil is controlled and limit by its threshold stress.

Tests of cyclic undrained compression which led to ultimate failure of soil specimen, *i.e.*, for q = 100 kPa in normally-consolidated series, shown that under cyclic loading rate of one cycle per second with the cyclic deviator stress slightly (*i.e.*, 10%) higher than its threshold stress, full excess pore water pressure can be built up in less than an hour that led to large strain failure. In the site condition, clay subgrade though usually quasi- saturated can become highly saturated in wet season. The dissipation of the excess pore water pressure built up in the subgrade will be a slow process due to thicker strata with consequent longer drainage path. Hence, the characterization of subgrade strength simulated by the above tests where fully saturated clay under "undrained" cyclic compression was employed can be justified. The threshold stress determined assuming such extreme condition represents a "lower-bound" but realistic solution.

Notwithstanding the seemingly conservative assumption, under repeated loading application, unlike coarse-grained or well-drained soil that is normally characterized by a single "shakedown" stress (Indraratna and Salim 2005), clay subgrade with its current state normally- to lightly over-consolidated can have increasing "lower-bound threshold stress" over time as excess pore water pressure begins to dissipate. The ability to assess and re-assess the strength of clay subgrade in terms of its lower-bound threshold stress will be a necessity for efficient and proper management of railroad foundation.

4.2 Constitutive Modelling

The constitutive model described in this session is the extension to the original Cam-clay model developed in the sixties by the Cambridge researchers (Schofield and Wroth 1968). The results of the cyclic undrained compression tests on kaolinite clay were used to verify the applicability and authenticity of the extension-model.

Modelling of the Level of Threshold Stress

The level of threshold stress has been identified to be dependent on its strength parameter (M), the current initial consolidated state or the initial effective mean stress (P_i') and the past maximum consolidated stress (P_o') of the soil; moderated by the fundamental response and elasto-plastic behaviour under cyclic loading condition as characterized by its volumetric compressibility indices of the soil (λ and κ).

Based on the notion of volumetric compatibility which involves plastic and elastic volumetric changes within soil mass, the boundary of cyclic stress equilibrium state on the $p' - q$ plane (at constant volume) have a functional relationship identical to the original Cam-clay yield function which can be rewritten as follows:

$$q = Mp' \ln (P_e'/p') \tag{5}$$

where P_e' is the equivalent pre-consolidation pressure on the normally-consolidation line.

Under cyclic undrained compression, soil of various consolidated state, depending on its over-consolidation ratio (OCR) or compacted state, can be divided into either sub-critical region which has a tendency for densification, or super-critical region that tends to dilate. The line of division can be defined in terms of:

$$\mathrm{OCR}_{\mathrm{wet/dry}} = e^{\left(\frac{\lambda}{\lambda-\kappa}\right)} \tag{6}$$

The over-consolidated ratio of soil will simply be,

$$\mathrm{OCR} = \frac{P_o'}{P_i'} \tag{7}$$

In the case of in-situ or existing subgrade soil, the equivalent pre-consolidation pressure P_e' can be computed (ignoring anisotropic effect) using in-situ dry unit weight γ_d (in kN/m^3) (obtained from in-situ dry density) as follows:

$$P_e' = \exp\left(\frac{N - \dfrac{9.81G_s}{\gamma_d}}{\lambda}\right) \tag{8}$$

where G_s is the specific gravity of soil; N is the specific volume of soil under normally-consolidation at p' = 1 kPa.

Together with the designed initial consolidation pressure P_i' (in kPa) and the fundamental properties of the subgrade soil, Eq. (7) can be rewritten as follows:

$$\mathrm{OCR} = \left[\frac{e^{\left(\frac{N-(9.81G_s/\gamma_d)}{\lambda}\right)}}{P_i'}\right]^{\frac{\lambda}{\lambda-\kappa}} \tag{9}$$

For soil or subgrade which is from the sub-critical region, *i.e.*, $\mathrm{OCR} \le \mathrm{OCR}_{\mathrm{wet/dry}}$, the lower-bound threshold stress (q_t) can be identified at the boundary of cyclic stress equilibrium state, when

$$p' = P_e'/e \tag{10}$$

where e is the exponential function of one.

From Eqs. (5) and (10), the lower-bound threshold stress q_t can be derived as follows:

$$q_t = \frac{M}{e} P_o'^{\left(1-\frac{\kappa}{\lambda}\right)} P_i'^{(\kappa/\lambda)} \tag{11}$$

Or it can be directly related to the in-situ dry unit weight γ_d of the soil and its basic properties. From Eqs. (5) and (10),

$$q_t = M \cdot e^{\left(\frac{N-(9.81G_s/\gamma_d)}{\lambda}-1\right)} \tag{12}$$

For soil which belongs to super-critical region (*i.e.* OCR $\geq$ $OCR_{wet/dry}$), the lower-bound threshold stress q_t is determined by identifying (ignoring the anisotropic effect) the q on the boundary of cyclic stress equilibrium state when,

$$p' = P_i' \tag{13}$$

From Eqs. (5) and (13), the lower-bound threshold stress q_t can be rewritten in the following relationship:

$$q_t = M \cdot P_i' \left(\frac{\kappa-\lambda}{\lambda}\right) \ln\left(\frac{P_i'}{P_o'}\right) \tag{14}$$

Similarly, it can also be relate to the in-situ dry unit weight γ_d, the designed initial consolidation pressure P_i' and soil basic properties as,

$$q_t = M \cdot P_i' \left[\frac{N-(9.81G_s/\gamma_d)}{\lambda} - \ln P_i'\right] \tag{15}$$

As a means to aid the design for railroad foundation subgrade of general clay-type, two charts are developed from the above Equations for the determination of the lower-bound threshold stress. Figure 22 shows the relationship between the lower-bound threshold stress q_t and the invert of over-consolidation ratio P_i'/P_o', as given by Eqs. (11) and (14). In order to make the chart dimensionless (which may be applicable for other clay-type), the threshold stress is normalized by both the maximum pre-consolidation pressure P_o', and the strength parameter (slope of critical state line M). In this chart, a number of curves each corresponds to a relative volumetric compressibility for the rebound and the normally-consolidation κ/λ are also depicted.

Figure 23 shows the direct relationship between the lower-bound threshold stress and the current initial consolidation pressure (initial mean effective stress) P_i', normalized by the equivalent pre-consolidation pressure P_e'. The threshold stress is further normalized by the strength parameter M.

Validation and Examination

The comparison between the experimental results on the threshold stress, the predicted level of threshold stress based on the consolidation history of the soil, and that derived from the dry density which represents the in-situ compaction state of subgrade soil is being discussed in this session. Table 2 showed the tabulation of the four series of consolidated kaolinite clay, with its maximum consolidation history and its current initial consolidation pressure (initial mean effective stress), as per the tests procedures previously described. The experimental threshold stress analyzed from the cyclic test results was summarized and presented as the benchmark for comparison in verifying the various model equations. It can be seen that the predicted threshold stress (using Eqs. (11) and (14)) based on the known consolidated state and stress history and the volumetric compressibility indices of the soil predicted the level of threshold stress well for the consolidated states from sub-critical to super-critical region. The slight over-estimate noticed at the heavily over-consolidated region will be improved, by adopting the higher κ value of the re-compression line, in place of the average κ value of the swelling and the re-compression line.

The predicted threshold stress (using Eqs. (12) and (15)) as tabulated was derived based on the compacted state, *i.e.*, dry density, of the soil specimen and the soil fundamental properties. It produced fairly good match with the experimental values. The

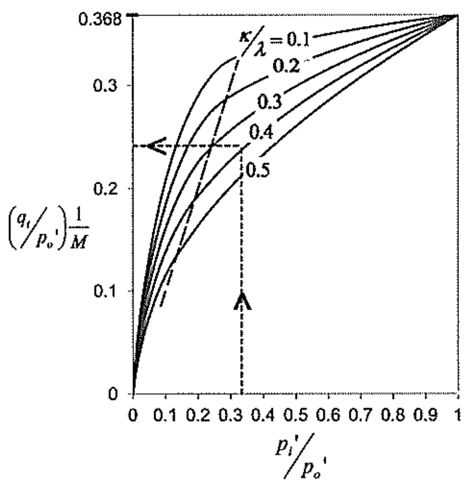

Fig. 22 Chart for the determination of lower-bound threshold stress based on the known consolidated state (Clay material in general) (Loh and Nikraz 2011)

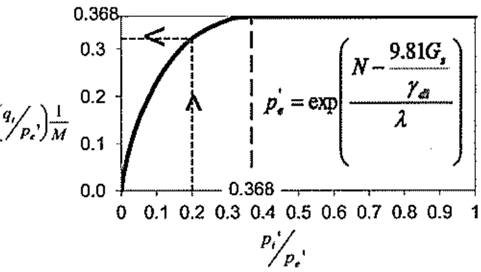

Fig. 23 Chart for the determination of lower-bound threshold stress based on in-situ dry density of clay-material (Loh and Nikraz 2011)

Table 2 Comparison of the model-prediction of lower-bound threshold stress q_t, with the experimental values obtained for the re-constituted kaolinite clay

OCR	OCR $\leq$ $OCR_{wet/dry}$			
	1	1.5	4	20
P_o' (kPa)	300	300	300	600
P_i' (kPa)	300	200	75	30
γ_d (kN/m^3)	13.44	13.22	12.94	13.02
q_t (kPa) (Experimental)	90	75	50	40
q_t (kPa) (Predicted using Equation 11 & 14)	89	76	52	44
q_t (kPa) (Predicted using Equation 12 & 15)	87	73	57	46

sensitivity of the value of dry density on the predicted level of threshold stress means that closer match would be expected if bigger soil specimen is used which minimizes measurement error. In the field, it is not uncommon that the in-situ dry density of the same soil varies amongst samples. Under such circumstance, it is better to be conservative and base the lower-bound threshold stress on the lower range of dry densities obtained. However, as a general rule, to obtain an accurate and representative in-situ dry density for subgrade under existing rail platform, measurement of in-situ dry density (*i.e.* sample volume) should be taken as soon as the sample being taken and specimen is prepared, prior to possible soil dilation taking place.

5. SUMMARY AND CONCLUSION

The elasto-plastic behaviour of a railroad subgrade clay material has been explored by means of cyclic triaxial tests. Four series of undrained cyclic triaxial compression tests on reconstituted saturated kaolin clay of sub- to super-critical region were carried out to provide an insight into the deformation characteristics and strength deterioration due to excess pore water pressure build-up of the railroad subgrade clay material produced by the consecutive passing train wheels during the most vulnerable soil condition.

Under cyclic undrained compression, the stabilized axial strain of clay material is dependent on the effective stress obliquity and the degree of over-consolidation or compacted state. The dynamic stiffness in the form of resilient modulus has a strong correlation with respect to the effective stress obliquity.

Using critical state soil mechanics framework, it has been shown that while failure of clay is generally preceded by the accumulation of excess pore water, it was not the case for the heavily over-consolidated soil where stress equilibrium within soil specimen is inherent under cyclic compression. For clay in the super-critical region, the development of cumulative total strain under various cyclic deviator stresses generally provides good basis for identifying the level of threshold stress.

In this paper, a laboratory-only approach and analysis in determining the lower-bound threshold stresses was first presented. Next, constitutive modeling on the deteriorated strength (threshold stress) of clay material under cyclic undrained compression was described and verification of the models performed using the experimental results and analysis. The theoretical model and analytical charts provide a quick and convenient means by which the lower-bound threshold stress can be determined based on the basic properties of clay material, from the known consolidated stress state or the compacted stress state of the clay subgrade.

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