Modelling Monotonic Behaviour of Unsaturated Compacted Soils in Constant Volume Direct Simple Shear

T. Ishigaki, S.Omoto and N.Nemoto
NIPPO Corporation Research Institute, Tokyo, Japan

A. Iizuka
Kobe University, Kobe, Hyogo, Japan

H. Ohta
Chuo University, Tokyo, Japan

ABSTRACT: The compressibility and simple shear characteristics of unsaturated soil compacted by the application of static pressure and its numerical modelling are investigated. A series of K_0-consolidation, swelling and constant volume direct simple shear tests are conducted on cohesive soil-gravel mixtures by use of a large-scale direct simple shear apparatus. The applicability of soil/water coupled finite element modelling is verified by comparing with the test results under constant volume simple shear. The constitutive model mainly employed in this study is an incremental elasto-plastic constitutive model proposed by Sekiguchi and Ohta (1977) with subloading surface model proposed by Hashiguchi (1989).

KEY WORDS: compacted soil, constant volume simple shear, elasto-plastic model.

1 INTRODACTION

Compacted soil is the essential engineering material for use in construction of pavement subgrade. Prediction of permanent deformation of compacted soil and its application to pavement design may be one of the important subjects in “pavement geotechnical engineering”.

The authors investigate the applicability of soil/water coupled finite element analysis as a tool which predicts the long-term durability of subgrade from the viewpoint of soil elastoplasticity.

In this paper, K_0-consolidation, swelling and constant volume direct simple shear characteristics of unsaturated soil compacted by the application of static pressure and its numerical modelling are investigated. A series of K_0-consolidation, swelling and constant volume direct simple shear tests are conducted on a cohesive soil-gravel mixture by use of large-scale direct simple shear apparatus.

The constitutive model mainly employed in this study is an incremental elasto-plastic constitutive model proposed by Sekiguchi and Ohta (1977) with subloading surface model proposed by Hashiguchi (1989). The procedure of input parameter determination for this model by use of the series of the test results of K_0-consolidation and swelling test and constant volume direct simple shear test are proposed. The applicability of the numerical modeling is verified by comparing with the test results of effective stress path under constant volume simple shear.

2 LABORATORY TEST

2. Apparatus

The material used in this investigation was a cohesive soil-gravel mixture (G_s =2.69, $D_{\max}$ =16.0mm, D_{60} =1.00mm, D_{10} =0.002mm, I_p =18.0, $\rho_{d\max}$ =2.05g/cm^3, w_{opt} =11.0%, CBR=75%). Initial water content of the soil sample was maintained the optimum moisture content of 11.0%. A series of K_0-consolidation and swelling test of unsaturated compacted soil are performed by a newly developed large-scale direct simple shear apparatus. The apparatus and its constrained conditions are shown in Figure 1, which is originally developed by Ohshima et al., (2000) and modified by Ishigaki et al., (2008).

Specimen diameter is 30cm and the height is 12.5cm during shear. Lateral confinement device is a simple shear box which has the laminate structure mainly consisted of 8 steel plates, guide bearing and flat bearings. The device can achieve the K_0-condition during consolidation and swelling stage. Normal and shear loading device are screw driven system controlled by AC servo motor with a closed-loop feedback system. Maximum pressure of normal and shear are 4500kPa respectively.

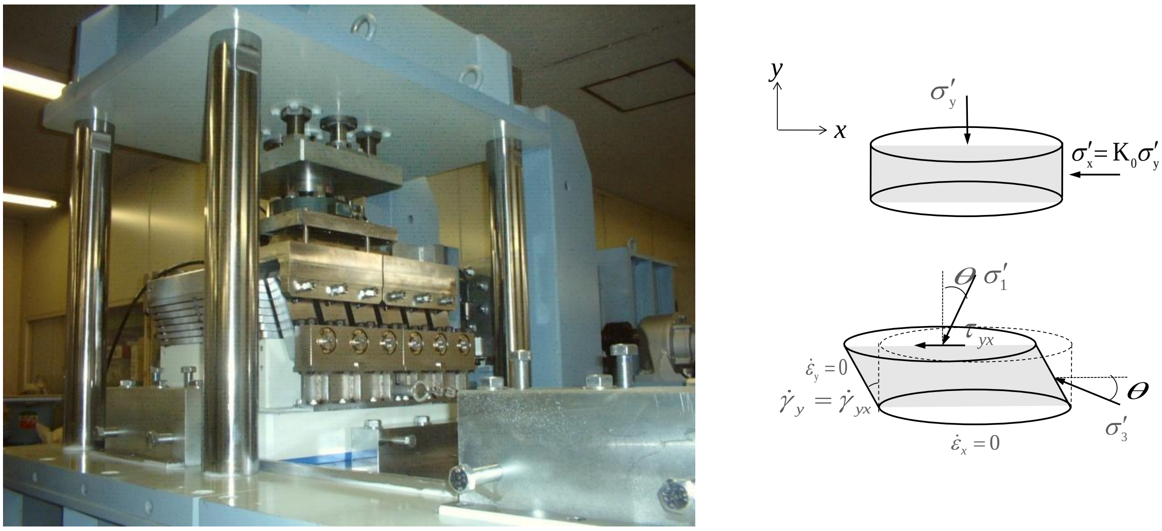


Figure1: Large-scale direct simple shear apparatus and its constrained conditions.

2.2 K_0-consolidation and swelling test

The test results of K_0-consolidation and swelling test in $e - \log \sigma_v'$ diagram are shown in Figure 2. The soil sample was loosely remoulded in a simple shear box not to give any pre-consolidation pressure as possible. Firstly, the specimens were statically consolidated by the application of staged loading. Consolidation time until the end of primary consolidation was about 10 minutes. The test was performed under drained and exhausted condition. Drainage was not observed during the normally consolidation stage. That is, only the pore air in the specimen was exhausted with the increase in consolidation stress. In this test, the consolidation pressure of 3000kPa was needed to get the degree of compaction of 95% based on the Proctor's method. Secondary, the normally consoilidated specimens were swelled without supplying water by the application of staged unloading. Overconsolidation ratios (OCR) were 1.0 (normally consolidated), 2.0, 3.0, 6.0 and 12.0. The material used in this investigation swelled very little during the unloading process. The void ratio of an overconsolidated specimen was nearly equal to that of a normally consolidated one.

Ohta and Hata (1977) proposed a concept of "equivalent pre-consolidation pressure" for unsaturated compacted soil. According to this paper;

(1) The relation between pre-consolidation pressure σ_v' and void ratio e is represented by a set of curves in $e-\log\sigma_v'$ diagram with the initial water content as a parameter.

(2) The shear strength of statically consolidated soil under constant volume shear is a unique function of pre-consolidation pressure regardless of over consolidation ratio, void ratio and of initial water content.

(3) The constant volume shear strength of soil compacted with any kind of dynamic compaction method are roughly predicted from the results of static consolidation and shear test assuming the mechanical behaviour of compacted soil is coincident with that of statically consolidated soil.

Based on this paper, the test results can be interpreted by the concept of effective stress. Compression index Cc and swelling index Cs of unsaturated compacted soil are estimated by $e-\log\sigma_v'$ curve in Figure 2.

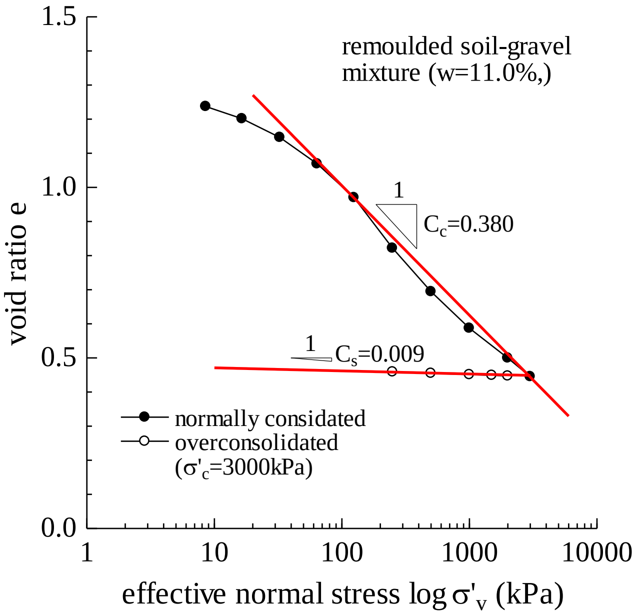


Figure2: K_0-consolidation and swelling test in $e-\log\sigma_{v0}'$ diagram.

2.3 Constant volume direct simple shear test

Figure 3 and 4 show the stress paths of unsaturated compacted soil in constant volume direct simple shear tests using normally consolidated and overconsolidated specimens in $\tau-\sigma_v'$ diagram. These tests were performed under plane strain conditions and the principle stresses continuously rotate during shear. Normal force adjustment device to achieve constant volume was an active height control to prevent changes in the specimen height during shear. Shear displacement rate were slow rate of 1mm/min under drained and exhausted condition, therefore excessive pore water pressure was not generated.

Matsuo and Karube (1966) pointed out that the stress path of unsaturated soil in constant volume shear is equivalent to the effective stress path of saturated soil. Therefore, the change in normal stress is considered to be approximately equal to the change in effective stress of unsaturated compacted soil under plane strain conditions and the principle stresses continuously rotate during shear.

The equivalent pre-consolidation pressure of the normally consolidated specimens was 250, 500, 1000, 2000 and 3000kPa shown in Figure 3. The effective stress paths of unsaturated compacted soil show the phase transformation during shear which firstly occur negative dilatancy and then changes to positive dilatancy. In this study, constant volume simple shear strength of soil is regarded as the state which the stress path is on the phase transformation line. Therefore, the normalized constant volume simple shear strength of τ / σ_c' is 0.244 estimated by Figure 3.

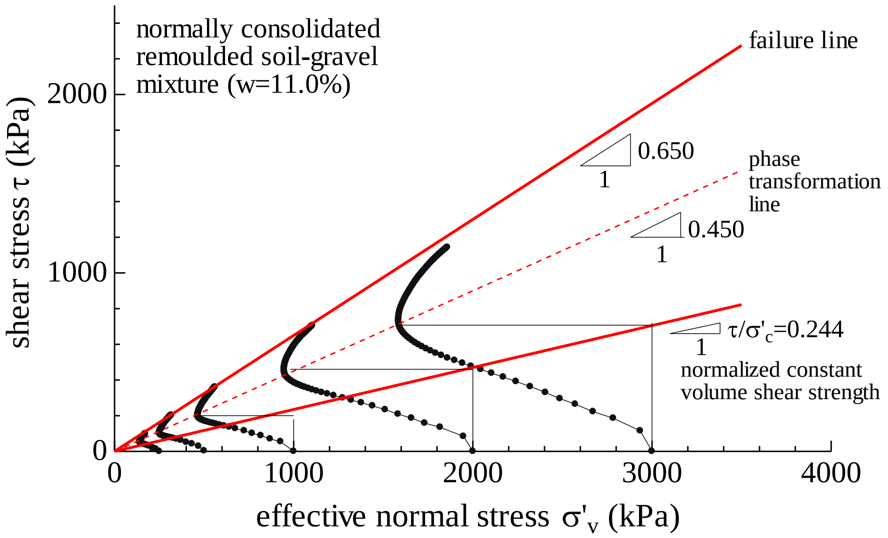


Figure3: Stress path of unsaturated compacted soil in constant volume direct simple shear tests in $\tau - \sigma_v'$ diagram (normally consolidated specimens).

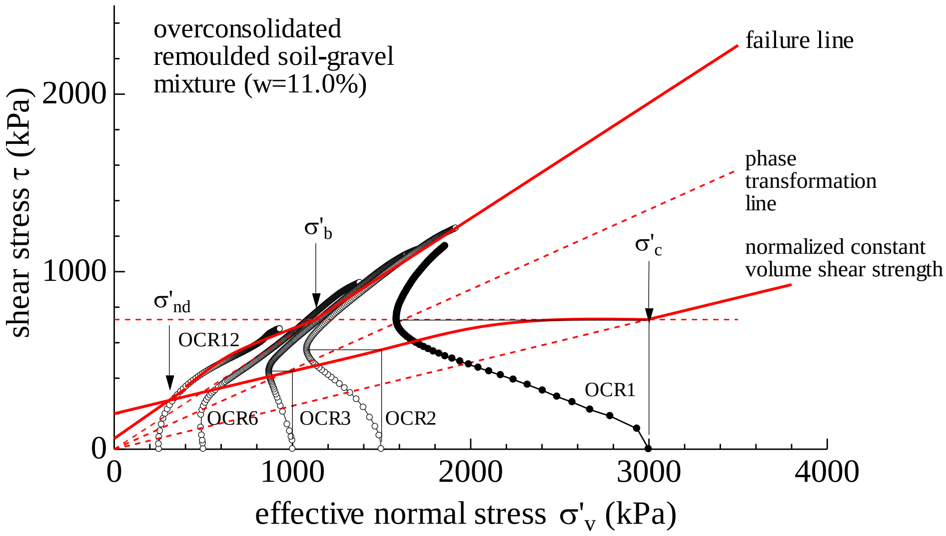


Figure4: Stress path of unsaturated compacted soil in constant volume direct simple shear tests in $\tau - \sigma_v'$ diagram (overconsolidated specimens, σ_c'=3000kPa).

In Figure 4, the equivalent pre-consolidation pressure of the overconsolidated specimens was 3000kPa, which the degree of compaction of the soil specimen was of 95%. Overconsolidation ratios (OCR) were 1.0 (normally consolidated), 2.0, 3.0, 6.0 and 12.0. Failure line is a curve line with cohesion if the effective normal stress is under overconsolidated influence stress of $\sigma_b{}'$. Normalized constant volume shear strength line is also curve line with cohesion if the effective normal stress is under under the equivalent pre-consolidation pressure of $\sigma_c{}'$. The non-dilatant stress of $\sigma_{nd}{}'$ is the effective normal stress which failure line and normalized constant volume simple shear strength line are intersected. The stress path which the initial effective normal stress of specimen is under $\sigma_{nd}{}'$ shows the positive dilatancy, or which the initial effective normal stress is over $\sigma_{nd}{}'$ firstly shows negative dilatancy and then change to positive dilatancy.

3 NUMERICAL MODELLING

3.1 Finite element modeling

The soil-water coupled finite element code used in this analysis is DACSAR coded by Iizuka et al. (1987), which is based on the consolidation theory proposed by Biot (1951) together with the formulation developed by modified Akai and Tamura (1978) by Takeyama (2006, 2007). The formulation is adopted the technique proposed by Christian (1968) and Christian and Boehmer (1970). One element of two-dimensional finite element modelling is employed in this analysis. The constitutive model employed is an incremental elasto-plastic model developed by Sekiguchi and Ohta (1977).

The yield function of elasto-plastic model is defined by:

$$f = MD\ln\frac{p'}{p_0{}'} + D\eta^* - \varepsilon_v^p = 0 \qquad (1)$$

where M is critical state parameter, D is coefficient of dilatancy proposed by Shibata (1963), p' is mean effective stress, $p_0{}'$ is initial mean effective stress, and $\eta^*(=\sqrt{3/2}\|\boldsymbol{\eta}-\boldsymbol{\eta}_0\|)$ is generalized stress ratio proposed by Sekiguchi and Ohta (1977).

Sekiguchi-Ohta model was developed based on a set of assumptions very different from the original cam-clay model by Roscoe et al (1963), but the final mathematical form is essentially the same as the original cam-clay model. The Sekiguchi-Ohta model can also describe the induced anisotropy. In this study, the constitutive model employed is the Sekiguchi-Ohta model with subloading surface model proposed by Hashiguchi (1989). The Sekiguchi-Ohta model which assumes the inside of a yield surface to be an elastic domain has a problem in expression of the mechanical behaviour of overconsolidated soil. Hashiguchi (1989) proposed the subloading surface model which is similar to the yield surface always passed along the present stress point.

The yield function and expanded rule of subloading surface model is defined by:

$$f = f\left(\sigma', Rp_c{}'\right) = 0 \qquad \dot{R} = -\frac{m}{D}\left\|\dot{\varepsilon}^p\right\|\ln R \qquad (2)$$

where, m is constant parameter to control expansion rate of subloadiong surface, R is similarity ratio, and D is coefficient of dilatancy proposed by Shibata (1963).

Yield surface of the Sekiguchi-Ohta model with subloading surface model is shown in Figure 5.

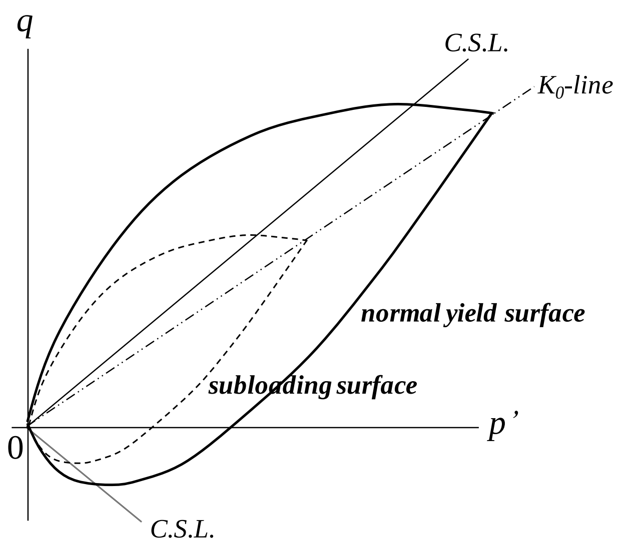


Figure5: Yield surface of Sekiguchi-Ohta model with subloading surface model.

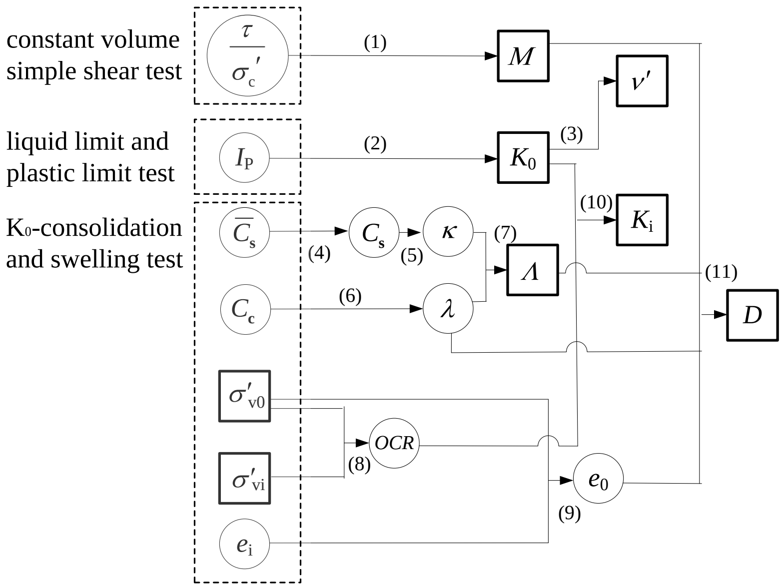
3.2 Input parameter determination

The input parameters needed in the Sekiguchi-Ohta model should primarily be determined through the triaxial test, oedometer test and permeability test. However, these laboratory tests were not performed in this study. Consequently, the procedure of input parameter determination of soil-gravel mixture by K0-consolidation and swelling test and constant volume direct simple shear test follows the flow charts are proposed in Figure 6. The soil parameters in square are needed in the Sekiguchi-Ohta model. Only the constant parameter to control expansion rate of subloadiong surface m of 1.0 is regarded as a fitting parameter. The estimated parameters used in this study are summarized in Table 1.

3.3 Computed results

Computed stress paths and stress-strain curves compared with the laboratory test results are shown in Figure 7. The computed results of the original Sekiguchi-Ohta model are over-estimated the shear strength. The computed results of the Sekiguchi-Ohta model with subloading surface are good agreement with laboratory test results, especially of highly overconsolidated compacted soil. This model also cannot essentially express the phase transformation of unsaturated compacted soil but express the overconsolidated saturated soil well. However, the reason of this good agreement may be that the stress path of unsaturated soil in constant volume shear is approximately equivalent to the effective stress path of saturated soil.

Compacted soil of pavement subgrade are highly over-consolidated soil, therefore it can be concluded that this model is applicable for use in pavement analysis which is needed to simulate the monotonic behaviour and permanent deformation of unsaturated compacted soils under constant volume simple shear condition.



(1) $\frac{\tau}{\sigma_c'} = \frac{(1+2K_0)M\exp(-\Lambda)}{3\sqrt{3}\cosh\beta}$

$\eta_0 = \frac{3(1-K_0)}{1+2K_0}$, $\beta = \frac{\sqrt{3}\eta_0\Lambda}{2M}$

(Ohta et.al., 1985)

(2) $K_0 = 0.44 + 0.0042 I_p$ (Massarsch 1979)

(3) $\nu' = \frac{K_0}{1+K_0}$

(4) $\frac{\overline{C_s}}{C_s} = 1 - \frac{\log\beta}{\log(OCR)}$, $\beta = \frac{1+2K_i}{1+2K_0}$

(5) $\kappa = 0.434 C_s$

(6) $\lambda = 0.434 C_c$

(7) $\Lambda = 1 - \frac{\kappa}{\lambda}$

(8) $OCR = \frac{\sigma_{vo}'}{\sigma_{vi}'}$

(9) $e_0 = e_i - \lambda(1-\Lambda)\ln(\overline{OCR})$

$\overline{OCR} = OCR\frac{1+2K_0}{1+2K_i}$

(10) $K_i = K_0(OCR)^{0.54\exp(-I_p/122)}$ (Alpan 1967)

(11) $D = \frac{\lambda\Lambda}{M(1+e_0)}$ (Ohta 1971)

C_c : compression index, C_s :swelling index of isotropic consolidation, $\overline{C_s}$:swelling index of K_0 condition, D : dilatacy coefficient proposed by Shibata (1963), Λ :irreversibility ratio, M :critical state parameter, K_0 :coefficient of earth pressure at rest, K_i :coefficient of earth pressure at initial state, ν' :effective Poisson's ratio, $\sigma_{v0}'(=\sigma_c')$:equivalent pre-consolidation pressure, σ_{vi}' :effective overburden pressure, OCR: overconsolidation ratio, e_0 : void ratio at rest, e_i : void ratio at initial state, τ/σ_c' :normalized constant volume simple shear strength.

Figure6: Procedure of input parameter determination for Sekiguchi-Ohta model by constant volume direct simple shear test.

Table1: Estimated parameters.

test case	D	Λ	M	ν'	σ_{v0}' (kPa)
CVDSS-OCR1	0.064	0.969	1.735	0.340	3000
CVDSS-OCR2	0.064	0.968	1.735	0.340	3000
CVDSS-OCR3	0.064	0.968	1.735	0.340	3000
CVDSS-OCR6	0.064	0.967	1.735	0.340	3000
CVDSS-OCR12	0.064	0.966	1.735	0.340	3000

test case	K_0	σ_{vi}' (kPa)	K_i	e_0	m
CVDSS-OCR1	0.516	3000	0.516	0.445	1.0
CVDSS-OCR2	0.516	1500	0.712	0.445	1.0
CVDSS-OCR3	0.516	1000	0.860	0.445	1.0
CVDSS-OCR6	0.516	500	1.188	0.445	1.0
CVDSS-OCR12	0.516	250	1.641	0.445	1.0

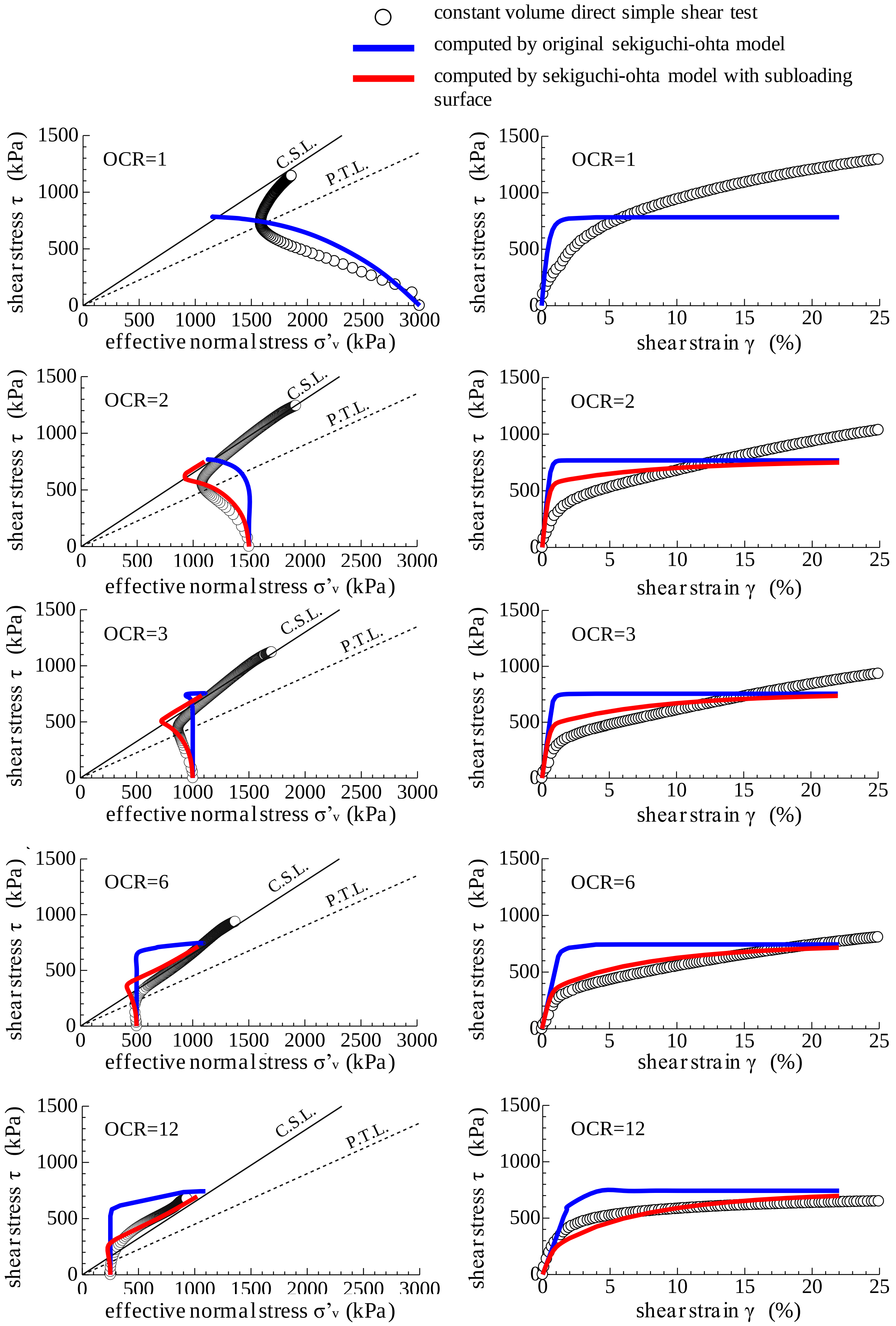


Figure7: Computed stress paths and stress-strain curves compared with the laboratory test shown in Figure 4.

4 CONCLUDING REMARKS

The simple shear characteristics of unsaturated compacted soil and its numerical modelling were investigated. A series of K_0-consolidation, swelling and constant volume direct simple shear tests were performed by a large-scale direct simple shear apparatus. The material used was a cohesive soil-gravel mixture. Constant volume simple shear test was quite useful and practical because the measurements of pore water pressure and pore air pressure are not required. Under constant volume simple shear condition, the effective stress paths showed the positive dilatancy or negative dilatancy in overconsolidated state.

The paper tried to bring the effective stress concept into the analysis of simple shearing characteristics of unsaturated compacted soils by means of constant volume simple shear tests. Based on a concept of equivalent pre-consolidation pressure proposed by Ohta and Hata (1977), the test results were well interpreted by the concept of effective stress.

The constitutive model mainly employed in this study was an incremental elasto-plastic constitutive model proposed by Sekiguchi and Ohta (1977) with subloading surface model proposed by Hashiguchi (1989). The procedure of input parameter determination for this model by the series of the test results of K_0-consolidation and swelling test and constant volume direct simple shear test was proposed in this paper.

The computed results of the Sekiguchi-Ohta model with subloading surface are good agreement with laboratory test results, especially of highly overconsolidated compacted soil.

Compacted soil of pavement subgrade are highly over-consolidated soil, therefore it can be concluded that this model is applicable for use in pavement analysis which is needed to simulate the monotonic behaviour and permanent deformation of unsaturated compacted soils under constant volume simple shear condition.

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