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Sydney Soil Model: (II) Experimental Validation

by

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Abstract

This paper presents simulations of the mechanical behaviour of reconstituted and natural soils using a new model presented in a companion paper and referred to as the Sydney Soil model. It is demonstrated that the performance of the proposed model is essentially the same as that of Modified Cam Clay when describing the behaviour of clays in laboratory reconstituted states. The model has also been employed to simulate the drained and undrained behaviour of structured clays and sands, including calcareous clay and sand. Five sets of conventional triaxial tests and one set of true triaxial tests have been considered. It is demonstrated that the new model provides satisfactory qualitative and quantitative modelling of many important features of the behaviour of structured soils, particularly in capturing various patterns of the stress and strain behaviour associated with soil type and structure. A general discussion of the model parameters is also included. It is concluded that the Sydney Soil model is suitable for representing the behaviour of many soils if their ultimate state during shearing can be defined by an intrinsic and constant stress ratio M* and a unique relationship between mean effective stress and voids ratio, i.e., a unique p' - e curve.

Keywords: clays, sands, calcareous soils, fabric, structure of soil, constitutive relations, plasticity.

1. Introduction

Soils are comprised of particles with wide ranges of mineralogy, size, and shape, arranged to produce different fabrics and with variable bonding between the individual grains. As a consequence, the mechanical properties of materials of such huge diversity are inevitably complex and the development of mechanical constitutive models has been necessarily a progressive process. For example, for the purpose of mechanical modelling, soils are commonly divided into clays and sands and conventionally the behaviour of these two soil types has been modelled separately. For both clay and sand, individual models have been developed, initially to describe the behavior of soil in an artificial state, the laboratory reconstituted state, and more recently to describe natural soils that possess some fabric and structure. However, while much progress has been made most of these models are generally limited in the range of natural soil behaviours that can be simulated.

In Part I of the paper, a new model, the Sydney Soil model (SS), was formulated. In this model the behaviour of soil is described in terms of a reference state and a structured state. The behaviour of the soil at the reference state is based on soil properties at critical states of deformation and the assumption of plastic volumetric deformation-dependent hardening. The influence of soil structure is controlled by the difference in behaviour between the natural soil and the soil at the corresponding reference state. The effects of structure and destructuring on soil behaviour are modelled by a hardening (or softening) process.

In this paper, the Sydney Soil model is evaluated against experimental data. The model is used to simulate the behaviour of six different soils, loaded in both drained and undrained situations. The soil types selected for simulation cover both clays and sands, including calcareous clays and sands. The samples evaluated cover a wide range of densities, possess a variety of different structures, and have been subjected to a very wide range of stresses. Although this validation process is not exhaustive, it is considered sufficiently representative of a wide range of natural soils and loading conditions, to provide an adequate test of the validity and utility of the proposed model.

2. Input data

The Sydney Soil model is defined in terms of eight material parameters, and the specification of three curves, one surface, and the soil type. The eight parameters are the critical state friction angle φ_{cm} and the parameter s^* (used to describe the shape of the critical state failure surface in the π plane), Poisson's ratio ν^*, parameter m for describing soil behaviour within the yield locus, parameters n and γ defining shear destructuring, parameter μ defining the variation of the aspect ratio of the yield surface, and parameter ω used in the flow rule. The three curves are the critical state line in $e-p'$ space, the elastic volumetric deformation function of the soil $E^e(p')$, and the additional voids ratio sustained by the structure of the soil $\Delta E(p'_s)$. The symbols e, p' and p'_s are used here to represent the voids ratio of the soil, the mean effective stress and the size of the structural yield surface, i.e., its intercept on the p' axis, respectively. The surface requiring definition is the initial structural yield surface, defined by the isotropic yield stress (p'_s) and its aspect ratio. Soils are divided into clay and sand types. The only difference between the model for clay and sand types is that for sand-type soils, additional equations are required to model the first loading within the yield locus.

The following methods have been used for identifying the values of the model parameters adopted in the computations described below.

(1) The soil type for all simulations is obtained from conventional classification tests.

(2) The value of the critical state friction angle φ_{cm} has been obtained directly from the test data.

(3) Parameters s^*, μ and ω have been taken as constants, independent of the material type, with values $s^* = 1$, $\mu = 0$ and $\omega = 1$, unless their values could be readily determined from experimental data to be otherwise.

(4) Parameters ν^*, n and m could, in most cases, be determined from the experimental data as explained in Part I (Liu et al., 2009). Where there was insufficient data, their values were assumed to be $\nu^* = 0.25$, $n = 1$, and $m = 1$, as recommended in Part I.

(5) Parameter γ for all the simulations was determined after the values of all other parameters had been identified. Its value was determined by curve fitting the theoretical simulations to the experimental data for the distortional stress and strain relationship.

(6) Reliable identification of the three curves and one surface (the elastic volumetric deformation function $E^e(p')$, the critical state line, the additional voids ratio sustained by the structure of the soil $\Delta E(p'_s)$, and the initial structural yield surface) requires a number of specially designed laboratory tests. For example, tests are required on undisturbed

soils for the determination of the additional voids ratio sustained by the structure of the soil $\Delta E(p'_s)$ and the initial structural yield surface. When there were insufficient data for the accurate determination of these functions, the following techniques were used:

(a) The initial structural yield surface was assumed to be elliptical with its aspect ratio equal to the critical state stress ratio M*, enabling the surface to be determined from a single yield point.

(b) Where the Young's modulus was independent of the confining pressure, the elastic deformation function, $E^e(p)$, was taken as linear in p', and where the Young's modulus increased linearly with p', $E^e(p)$ was taken as linear in $\ln p'$.

(c) The critical state line could be either linear in the $e - p'$ space or linear in the $e - \ln p'$ space. The form which gave the best fit to the available data was adopted.

(d) Where the mathematical form of $\Delta E(p'_s)$ could not be determined directly from the data, the following expression derived by Liu and Carter (1999, 2000) was assumed:

$$\Delta E(p'_s) = a\left(\frac{p'_{y,i}}{p'_s}\right)^b + c \ . \qquad (1)$$

When there were insufficient data the recommendations of Liu and Carter (2000) for the material parameters b and c were assumed, i.e., $b = 1$ and $c = 0$. The symbol $p'_{y,i}$ represents the size of the initial structural yield surface, defined by the intercept of this surface on the mean stress axis and Δe_i is the additional voids ratio at $p'_s = p'_{y,i}$ where virgin yielding commences.

In all computations described below the stress units adopted are kPa (kN/m^2). In presenting the results of the computations, experimental data are represented in the figures of this paper by symbols, and the model simulations are represented by lines.

The tests simulated in this paper are conventional triaxial compression and extension tests with the confining pressure held constant unless the stress path of the test is otherwise specified.

3. Validation

3.1 Reconstituted clay

As a special case, the proposed model reduces essentially to the well-known Modified Cam Clay model developed for clays in a reconstituted state. The performance of the MCC model for laboratory reconstituted clays has been extensively investigated, both for single element tests and for solving boundary value problems, and it is widely accepted that the model captures well the essential features of the behaviour of reconstituted clays (Houlsby et al., 1982; Burland, 1990; Potts and Zdravkovic, 1999). Therefore the proposed SS model should also describe reasonably well the behaviour of most clays in a laboratory reconstituted state.

For completeness, a summary of the basic constitutive equations for the two models is provided here. Further details of the formulation of the Modified Cam Clay model can be found in the text by Muir-Wood (1990). The symbols adopted here have the same meanings as defined in Part I.

For reconstituted clay it can be assumed that there are no effects of structure so that in the Sydney Soil (SS) model the additional voids ratio associated with structure will be zero, i.e., $\Delta e = 0$. In this case the structural yield surface will have the same size as the equivalent yield surface for fully destructured soil, i.e., $p'_e = p'_s$, and the aspect ratio of the yield surface will be identical to the critical state shear stress ratio, i.e., $M = M^*$.

The SS model requires the equation of the critical state line in $e - p'$ space. To be consistent with the MCC model the locus of critical states is given by:

$$e = e_{cs}^* - \lambda^* \ln p' \tag{2}$$

To be consistent with the elastic linear $e - \ln p'$ response, the function controlling the recoverable change in voids ratio in the SS model is defined by:

$$E^e(p') = \kappa^* \ln p' \tag{3}$$

The constitutive equations for virgin yielding in both models may be written as follows:

$$
\begin{aligned}
d\varepsilon_v &= \left(\frac{\kappa *}{1+e}\right)\frac{dp'}{p'} + \left(\frac{\lambda * - \kappa *}{1+e}\right)\frac{dp'_s}{p'_s} \\
d\varepsilon_d &= \frac{2(1+\nu *)}{9(1-2\nu *)}\left(\frac{\kappa *}{1+e}\right)\frac{dq}{p'} + A\left(\frac{\lambda * - \kappa *}{1+e}\right)\frac{dp'_s}{p'_s}
\end{aligned}
\quad . \tag{4}
$$

For the MCC model:

$$
\begin{aligned}
A &= \frac{2\eta}{\overline{M}^2 - \eta^2} \\
dp'_s &= \frac{(2p' - p'_s)\overline{\mathrm{M}}^2 dp' + 2q\,dq}{p'\overline{\mathrm{M}}^2} \\
q &= \frac{1}{\sqrt{2}}\sqrt{\left[(\sigma'_{11} - \sigma'_{22})^2 + (\sigma'_{22} - \sigma'_{33})^2 + (\sigma'_{33} - \sigma'_{11})^2 + 6\left({\sigma'_{12}}^2 + {\sigma'_{23}}^2 + {\sigma'_{31}}^2\right)\right]} \\
\eta &= \frac{q}{p'}
\end{aligned}
\quad . \tag{5}
$$

σ'_{ij} are the Cartesian components of effective stress and $d\varepsilon_v$ and $d\varepsilon_d$ are the increments of volumetric and deviatoric strain respectively. $\overline{\mathrm{M}}$ is the value of the stress ratio, η, at critical state.

For the SS model:

$$
\begin{aligned}
A &= \frac{\left(\frac{3}{2}\right)\sqrt[3]{\eta^\wedge / M *}}{\left|1 - \eta^\wedge / M *\right|} \\
dp'_s &= \frac{(2p' - p'_s)\mathrm{M} *^2\, dp' + 2q^\wedge\, dq^\wedge}{p'\mathrm{M} *^2} \\
q^\wedge &= \eta^\wedge p' \\
\eta^\wedge &= \frac{3\left[f_2 + \sqrt{f_2(f_2 + 27 - 15s*)}\right]}{f_2 + \sqrt{f_2(f_2 + 27 - 15s*)} + 4(27 - 15s*)} \\
f_2 &= \frac{(\sigma'_{11} + \sigma'_{22} + \sigma'_{33})\left[(\sigma'_{11} + \sigma'_{22} + \sigma'_{33})^2 - s*\left({\sigma'_{11}}^2 + {\sigma'_{22}}^2 + {\sigma'_{33}}^2\right)\right]}{\sigma'_1\sigma'_2\sigma'_3} - 27 + 9s*
\end{aligned}
\quad . \tag{6}
$$

The generalized shear stress $q^\wedge$ and the stress ratio $\eta^\wedge$ were formulated to allow the possibility in the SS model of a non-circular section of the failure surface in the π plane, and thus improved model performance for loading along general stress paths. Consequently, different definitions for the shear stress (q and q^), stress ratio (η and η^) and the critical state stress ratio ($\overline{\mathrm{M}}$ and M*) are required. This distinction effectively disappears for large negative values of the parameter s^*, and for practical purposes assuming $s^* = -50$ will effectively recover the volumetric deformation of the MCC model. For distortional deformation, differences in predictions arise because of the difference in the assumed flow rule.

In the SS model non-elastic deformations can occur within the yield locus, however when a large value is assigned to the parameter m, such as $m > 10$, the performance of the SS model is essentially elastic for loading within the yield locus, and almost identical to the MCC model.

A quantitative comparison of the performance of the two models is shown in Figure 1. The values of model parameters used for these computations are listed in Table 1. The size of the initial yield surface is 100 kPa and initial stress state is isotropic. The tests simulated are conventional drained triaxial compression tests with constant confining pressures, for two over-consolidation ratios, OCR = 1 and OCR = 8. As expected the simulations of the two constitutive models for reconstituted clay are essentially the same, although use of the SS model results in slightly reduced peak strength for highly over-consolidated clay, resulting in generally a better match with reality (Schofield, 1980).

3.2 Structured soils

The Sydney Soil model has been evaluated on the basis of comparisons between the model simulations and the corresponding experimental data. The behaviours of six different soils are considered, viz., Nanticoke clay, Bass Strait carbonate sand, Fuji sand, a natural calcarenite, Emmerstad clay, and Toyoura sand. The six soils have been selected because each represents a special stress and strain behaviour pattern. Consequently, the capacity of the proposed model to describe various behaviour patterns of soils is demonstrated qualitatively and quantitatively.

Values of the model parameters for the six soils identified, following the procedures described in section 2, are listed in Table 2.

3.2.1 Nanticoke clay

The first set of simulations includes four drained triaxial compression tests performed by Lo (1972) on samples from a natural deposit of stiff Nanticoke clay. Test specimens were cut from a block of undisturbed soil. In addition to the information in Table 2 the initial voids ratios needed to be estimated, and these were calculated from the known water content of the clay, assuming $G_s = 2.7$. The calculated voids ratio of 0.7 was assigned to the soil at the stress state ($p' = 552$ kPa, $q = 0$), and the voids ratios for the other three tests were calculated according to the elasticity relationship, since the initial stress states were well below the structural yield surface.

A comparison of the simulations and the experimental data is shown in Figure 2. Overall, it is seen that the proposed model gives a reasonably good description of the behaviour of the natural stiff Nanticoke clay. Three particular features of the behaviour of the structured clay are captured by the SS model that cannot be reproduced by simpler models.

(1) Compressive volumetric deformations are predicted during softening. This feature has been widely reported for natural soils (e.g., Bishop et al., 1965; Wong, 1998; Carter et al., 2000).

(2) The volumetric deformation is predicted to change from volumetric expansion to compression with or without initial compression while the soil is softening. The consequences of this feature have been observed in undrained tests (e.g., Lacasse et al., 1985; Georgiannou et al., 1993; Vaughan, 1994).

(3) Much greater compressive volumetric strains are predicted than for reconstituted soils with similar apparent over-consolidation ratios. The greater compression and reduced stiffness of natural clays during virgin shearing has been widely reported (e.g., Graham and Li, 1985; Burland, 1990; Lagioa and Nova, 1995).

3.2.2 Bass Strait carbonate sand

The second set of simulations includes three drained triaxial compression tests performed by Poulos et al. (1982) on Bass Strait carbonate sand. The specimens had been reconstituted in the laboratory and loaded isotropically to the following states at the start of shearing: ($p' = 138$ kPa, $e = 0.607$), ($p' = 276$ kPa, $e = 0.636$), and ($p' = 414$ kPa, $e = 0.597$). It can be seen that the test with the intermediate confining pressure has the highest initial voids ratio. In the SS model this is interpreted as being the result of differences in structure, and the term $\Delta E(p'_s)$ will be different for the three specimens. To determine these terms it has been assumed that the virgin ICL passes through the initial states and a common point ($p' = 20$ MPa, $e = 0.4$), an assumption that has been shown by Liu et al. (2000) to fit the data for many soils. The additional voids ratio functions obtained are $\Delta E = -0.132 - 2.7 \times 10^{-6} p'$ for the test with σ'_3 = 138 kPa, $\Delta E = -0.096 - 5.7 \times 10^{-6} p'$ for the test with σ'_3 = 276 kPa, and $\Delta E = -0.13 - 2.9 \times 10^{-6} p'$ for the test with σ'_3 = 414 kPa.

A comparison of the simulations and the experimental data is shown in Figure 3. It is seen that the proposed model gives a good description of the behaviour of the Bass Strait carbonate sand. Two special features of the behaviour of structured soils are successfully represented.

(1) Expansive volumetric deformation can occur even though the shear stress ratio increases steadily until failure. Similar behaviour has been reported for other structured soils (e.g., Kolymbas and Wu, 1990; Robinet et al*.,* 1999).

(2) The volumetric deformation may either continue to increase, or decrease, with distortional deformations when the resistance of the soil to further shearing is virtually zero. This feature of structured soil behaviour has also been observed for other soils (e.g., O'Rourke and Crespo, 1988; Carter et al*.,* 2000).

3.2.3 Fuji sand

The fourth set of simulations includes five drained true triaxial tests performed by Yamada and Ishihara (1979, 1982) on loose Fuji sand. The five tests involved monotonic loading along linear stress paths in the π plane, with mean effective stress maintained constant (see the insert in Figure 5). The differences between the various tests lay in the value of the Lode angle, θ. The critical state friction angle and the parameter and s^* were measured from experimental data, and a comparison of the final strength, determined experimentally and from simulation using the selected values of φ_{cm} and s^* is shown in Figure 4.

A comparison of the simulations and the experimental data is shown in Figures 5 to 9. It is seen that the proposed model describes satisfactorily the behaviour of the loose Fuji sand for monotonic loading in general principal stress space. As the general patterns of behaviour with respect to variations of the intermediate principal stress in this set of tests are consistent with other experimental data (e.g., Lade and Musante, 1978; Alawi, 1988; Liu and Carter, 2003), it is believed that the SS model has the capacity for representing well the influence of the immediate principal stress on soil behaviour.

3.2.4 Soft natural calcarenite

Results of six conventional drained triaxial compression tests carried out by Lagioia and Nova (1995) on a natural calcarenite have been compared with the model simulations. The natural calcarenite is a coarse-grained material with calcareous inter-particle cement contributing significantly to the material's structure. The soil was formed by marine deposition and has a high degree of uniformity. All specimens tested were considered to possess the same structure.

A comparison of the predicted isotropic compression and the critical state lines with test data is shown in Figure 10. The initial state for the structured soil is defined by $p' = 147$ kPa and $e = 1.148$. The initial voids ratios for all simulated tests were calculated from the predicted ICL. Based on the CSL and ICL, the function for the additional voids ratio was determined. Based on the initial yield points for the soil measured by Lagioia and Nova (1995), the initial structural yield surface can be described by the following expression:

$$\left(\frac{p' - 1200}{1200}\right)^2 + \left(\frac{\sigma_1' - \sigma_3'}{1344}\right)^2 - 1 = 0 \ . \qquad (7)$$

A comparison of the initial structural yield surfaces given by equation (7) and deduced from the experimental data is shown in Figure 11. The initial structural yield surface in the p' - (σ'_1-σ'_3) space is elliptical with an aspect ratio of 1.12.

The test results and the simulations are shown in Figures 12 to 17. Considering the wide range of initial stresses, it is seen that the proposed model gives a very successful simulation of the behaviour of this natural and highly structured calcarenite.

It is also observed in the simulations and in the experimental data (Figures 14 to 16) that both the deviatoric and the volumetric strains increase virtually at constant stress at the moment when virgin yielding commences, and that a large amount of plastic deformation is accumulated at the end of this process. These simulations are consistent with experimental observations of natural soil behaviour where the soils have a very sensitive structure (e.g., Westerberg, 1995, after Rouainia and Muir Wood, 2000; Arces et al., 1998).

For the test with $\sigma'_3 = 3{,}500$ kPa, the initial stress state is much larger than the size of the initial structural yield surface. According to the proposed model, the structure of the soil at $\sigma'_3 = 3{,}500$ kPa is effectively completely destroyed since the soil has a very high destructuring index, i.e., $b = 30$. Thus the soil behaved essentially as a reconstituted material throughout this test. Destructuring of this sample was confirmed by Lagioia and Nova (1995).

3.2.5 Emmerstad clay

The behaviour of a natural sensitive (and therefore highly structured) Norwegian marine clay, Emmerstad clay, in undrained triaxial tests (Lacasse et al., 1985) has been compared with the model predictions.

Following the work by Burland (1990), the ICL* estimated for this clay is:

$$e^* = 0.879 - 0.07\ln p' + 0.00016(\ln p')^3 \;. \tag{8}$$

The additional voids ratio sustained by soil structure has been estimated from oedometer tests as:

$$\Delta E(p_s') = 0.35 + 0.26\left({}^{98}\!/_{p_s'} \right)^{0.4} \;. \tag{9}$$

The initial state of the soil was defined by $\sigma'_1 = 41.5$ kPa, $\sigma'_2 = \sigma'_3 = 23.5$ kPa, $e = 1.155$. The initial structural yield surface was assumed to be an ellipse with the aspect ratio being M^*. The initial vertical yield stress measured from the oedometer data is 110 kPa, which gives $p'_{s,i} = 98$ kPa, based on an empirical expression for K_o suggested by Liu and Carter (2002).

A comparison of the simulations and the experimental data for undrained triaxial shearing is shown in Figures 18 and 19. Overall, it is seen that the proposed model gives a reasonably good description of the behaviour of the natural soft Emmerstad clay, which was sheared from an anisotropic stress state to failure. Laccase et al. (1985) observed that Emmerstad clay is extremely sensitive. This is evident from the simulations, which predict the final strength of the soil as being almost negligible when taken to large strains. However, unlike most very soft clays, Emmerstad clay exhibits a significant amount of negative excess pore pressure, which is usually a feature of stiff clay behaviour (e.g., Vaughan, 1994), before it softens. The SS model is able to simulate this behavior satisfactorily. To achieve this it was necessary for the soil to be much more sensitive to destructuring caused by increasing shear stress than by increasing mean stress.

3.2.6 Toyoura sand

Large amounts of experimental data are available for the mechanical properties of Toyoura sand, enabling the direct determination of all the model parameters. Simulations are reported here for a series of experiments reported by Ishihara (1993). These involved conventional undrained triaxial compression tests at constant confining pressures on specimens prepared by two different methods, dry deposition and moist placement. As reported by Ishihara (1993), the method of preparation had a significant influence on the behaviour, and this was attributed to the different fabric, or structure resulting from the different preparation methods.

The parameters describing the intrinsic properties of Toyoura sand are listed in Table 2. The critical state friction angle was measured from the experimental data, and the value of s^* was estimated from data presented by Matsuoka and Nakai (1982). The CSL for the sand and the ICLs produced by both methods of sample preparation were investigated by Ishihara (1993), who found a unique, intrinsic critical state line and a range of virgin isotropic compression lines depending on the structure of the soil. It has been shown that the CSL and ICLs can be described accurately by the following equation (Liu et al., 2000):

$$e = \Gamma - lp' + B\exp(-rp') \ . \tag{10}$$

where Γ, l, B and r are material constants, and for the ICL these can be related to the initial voids ratio e_i of the sand at $p'_i = 40$ kPa by:

$$\begin{aligned} \Gamma &= 0.9477 + 0.7194 \ln e_i \\ l &= 2.34 \times 10^{-5} e_i^{3.6015} \\ B &= 0.419 e_i^2 - 0.5765 e_i + 0.2174 \\ r &= 0.0372 e_i^2 + 0.0629 e_i - 0.0228 \end{aligned} \ . \tag{11}$$

The theoretical curves determined according to equations (11) for four isotropic compression tests are shown in Figure 20 by solid lines. A good agreement between the simulations and the experimental data is seen. A comparison of the CSL defined by the proposed equation and the experimental data and the initial states of the fifteen tests simulated in this study, which may be seen to be scattered over a wide range of stress levels and voids ratios, are also shown in Figure 20. With the determination of the ICLs, the additional voids ratio $\Delta E(p'_s)$ for Toyoura sand for a given initial state can be defined.

For the samples prepared by dry deposition the function $\Delta E(p'_s)$ has been determined to be:

$$\Delta E = -0.029 + 1.47 \times 10^{-5} p'_s + 0.036 \exp(-0.00372 p'_s) - 0.04 \exp(-0.0015 p'_s) \quad (12)$$

This function is identical for tests A1, A2, A3 and A5 because these were carried out on initially identical samples, and it differs only very slightly for test A4. The values of the model parameters dependent on the type of soil structure are listed in Table 3. A comparison of the model simulations and the experimental data is shown in Figure 21. It is seen that the Sydney Soil model describes the behaviour of the sand prepared by dry deposition reasonably well.

The model parameters describing the shear destructuring were selected to give the best fit over all the tests. However, better fits for individual simulations can be obtained with minor adjustments to the shear destructuring parameters. For example, by assuming $n = 5$ and $\gamma = 0.2$, the simulation for test A4 shown in Figure 22 is obtained. For these samples the reduction of soil strength after the peak is particularly sensitive to the model parameters.

A similar procedure was followed to identify the structure-related parameters for the ten tests prepared by moist placement, and these are listed in Tables 3 and 4.

For comparison purposes, the model simulations and the experimental data were divided into three groups. The first group includes four tests, viz., B1, B2, B3 and B4, and the comparisons for these cases are shown in Figure 23. The second group includes another four tests, viz., B5, B6, B7 and B8, and the comparisons are shown in Figure 24. The third group includes three tests, viz., B1, B9 and B10, and the comparisons are shown in Figure 25. It can be seen that the Sydney Soil model also describes well the different behaviours of Toyoura sand prepared by moist placement, and provides reasonable quantitative predictions. Where the agreement is least satisfactory (Figure 23 test with $\sigma'_{r,i}$ = 1000 kPa) a minor adjustment to the shear destructuring parameters from $n = 0.5$ to 1 and $\gamma = 1$ to 0.1 leads to the improved fit shown in Figure 23.

Different behaviours are shown by Toyoura sand prepared by the dry and moist preparation methods. During undrained tests the samples prepared by dry deposition initially exhibit virgin yielding until reaching the critical state stress ratio, when they harden, basically along the critical state line to failure. By contrast the moistly placed samples exhibit more conventional behaviour, i.e., initial virgin yielding followed by softening to failure. This change of material response has been successfully described by the Sydney Soil model through a rational adjustment of the soil parameters describing the effects of soil structure. To the authors' knowledge, there is no existing constitutive model that is capable of describing two different types of behaviour for the same sand (i.e., for a soil that has the same mineralogy and critical state strength and the same critical state line in $e - p'$ space).

4. Discussion

4.1 Model performance

The Sydney Soil model has been used to simulate the behaviour of six different soils covering clay, sand, calcareous clay, and calcareous sand. The behaviour of very soft and very stiff soil has been considered. The structures of the soils include both naturally formed structures and artificial structures formed by preparing the samples via different methods in the laboratory. The stress levels of the tests also varied widely, e.g., for Toyoura sand from 100 kPa to 3,000 kPa and for a natural calcarenite from 25 kPa to 7,000 kPa. The types of tests include both conventional drained and undrained triaxial compression and extension tests and true triaxial tests. The model parameters have been classified into intrinsic soil parameters and structural soil parameters. In all simulations one set of intrinsic parameters has been selected for each soil and these values are associated with its unique mineralogy. The parameters describing the effect of soil structure are dependent on the geological history and/or the method of sample preparation. For example, the structural parameters for Toyoura sand are strongly dependent on the preparation method, and even for a given method of preparation different structures can be produced by, for instance, varying the height of raining, creating different initial densities, ICLs and $\Delta E(p'_s)$.

It has been shown that overall the Sydney Soil model provides a satisfactory description of the behaviour of all six different soils. It has also been shown that the following special features of the behaviour of structured soils can be successfully captured by the model.

(1) The structured soil may exhibit monotonic volumetric compression during softening, as observed in conventional triaxial tests.

(2) During softening, the volumetric deformation of a structured soil may change from initial expansion to compression.

(3) Structured soil may exhibit volumetric expansion when the soil "hardens" steadily from an isotropic stress state to failure at the critical state of deformation.

(4) Unlike the behaviour of laboratory reconstituted clay, the volumetric deformation for structured soils may either continue to increase or decrease at distortional deformations where the resistance of the soil to further shearing is virtually zero.

(5) For soils with a very sensitive structure the deviatoric and the volumetric strains may be produced at nearly constant stress from the moment when virgin yielding occurs, and a large amount of plastic deformation may be accumulated at the end of this process.

(6) Soil samples of a given mineralogy may exhibit different material behaviour depending on their initial structure.

4.2 Soil properties

The Sydney Soil model allows flexibility in specifying soil properties. For example, in describing the elastic volumetric deformation, instead of assuming a linear $e - \ln p'$ relationship, as adopted in the MCC model, any appropriate mathematical form can be adopted if it better describes the soil behaviour. Three functions describing the volumetric behaviour of the soil have to be specified. They are the equations describing $E^e(p')$, the CSL and $\Delta E(p'_s)$.

4.3 Type and degree of soil structure

In studying naturally occurring soils, it is seen that soil structure is characterized by two features, the type and degree of structure. The type of soil structure is dependent on the mechanical and chemical constraints imposed during the formation of the structure, and the degree or magnitude of soil structure is dependent on the amount of deformation (and the amount of the chemical reaction) that has occurred since its formation.

The modelling of soil structure may also be studied in terms of the type and degree of soil structure. Parameters used in the Sydney Soil model reflecting the influence of soil structure are n, γ, μ, ω, $p'_{s,i}$ (or the initial structural yield surface), ICL and $\Delta E(p'_s)$. It appears that parameters n, γ, μ, and ω are mainly dependent on the type of soil structure. Parameters $p'_{s,i}$, ICL and $\Delta E(p'_s)$ depend on both the type and degree of soil structure, while the mathematical forms of the initial structural yield surface, ICL and $\Delta E(p'_s)$ are dependent on the type of soil structure. The absolute magnitude of the yield surface, ICL and $\Delta E(p'_s)$ are dependent on the degree of soil structure. It is well known that the initial structural yield surfaces of soil samples of similar mineralogy usually possess the same shape but may have different sizes (e.g., Ladd et al., 1977; Diaz-Rodriguez et al., 1992; Desia et al, 1986; Liu and Carter, 1999; Desai, 2001; Liu et al, 2003). Therefore, it may be expected that natural soils from the same deposit will generally possess the same type of structure but of varying degrees.

Six sets of numerical simulations were presented in this paper. For five sets, the soil specimens in each set possessed more or less the same structure. They are natural Nanticoke clay, Bass Strait carbonate sand, Fuji sand, a natural calcarenite, and Emmerstad clay. It was seen that the behaviour of all five types of soil could be simulated successfully with one set of structure parameters describing the type of structure (i.e., n, γ, μ, and ω), but with different values for the different degrees of structure (i.e., $p'_{s,i}$, ICL and $\Delta E(p'_s)$). For Toyoura sand, two different structures were manufactured in the laboratory by different means of sample preparation. Two independent sets of structure parameters were identified. Satisfactory numerical simulations were achieved when different sets of soil parameters were selected to represent the different methods of sample preparation.

Based on the performances of the model in these simulations, the following conclusions may be tentatively drawn.

(1) The influence of soil structure on soil behaviour can be modelled sufficiently well through macro-parameters which are determined from the macro-response of the soil to stress variations.

(2) Soil structure may be characterized by its type and degree, and the parameters in the SS model that reflect the influence of structure may be similarly categorized.

4.4 Identifying parameter values

A significant number of parameters are required to enable the Sydney Soil model to capture the many types of soil behaviour and the transitions between them. The patterns of soil behaviour in the model are dependent on many factors, such as the position of the stress path, the elastic deformation properties, the position of the critical state line in e - p' space, the additional voids ratio in the e - p' space, and the destructuring function. Generally, there may not be a one-to-one correspondence between the parameter values and the predicted soil response. Consequently, injudicious selection of model parameters may result in large discrepancies between theoretical predictions and experimental data.

Experience with this model has revealed the following observations in regard to parameter determination.

(1) There is generally no difficulty in assessing parameter values provided sufficient experimental data is available. This data must be sufficient to determine the critical state strength, M*, the position of the critical state line and virgin isotropic compression lines in $e - p'$ space, from which the additional voids ratio function ΔE is determined, and an estimate of the initial yield surface. Without good quality experimental data to identify these parameters, simulations made by the model are likely to be of little value.

(2) Generally, the more parameters which have to be determined by curve fitting of simulations to experimental measurements, the greater the difficulty in identifying their values accurately and reliably. This is a problem shared in common by many constitutive models for soils: a one-to-one relationship between parameter values and model simulations is not guaranteed.

(3) Obtaining parameters from fully drained loading is relatively simple. However, for undrained loading small changes in the relationships between the four volumetric deformation components (i.e., the elastic component and the three plastic components arising from the intrinsic soil response and the effects of mean stress and stress ratio change on plastic volume strain) can radically alter the stress path that must be followed in p' - q^ space in order to maintain constant volume deformation, making parameter identification difficult and simulation less reliable.

(4) Parameter r for all the six sets of simulations presented was determined by curve fitting. The determination of r by fitting is feasible if all other parameters are known (or their range of variation is known). For drained tests, the adjustment of the parameter may be made by examining the volumetric deformation during virgin yielding. For undrained tests, the adjustment of the parameter may be made by examining the p' - q^ stress path during virgin yielding.

(5) For many applications constant values may be conveniently assumed for three

parameters, i.e., $s^* = 1$, $\mu = 0$ and $\omega = 1$.

(6) Data for natural clays (e.g., Diaz-Rodriguez et al., 1992) show initial structural yield surfaces that differ significantly from the shape of the assumed reference, unstructured, yield surface. Preliminary studies have shown that modification of the model to allow for this can significantly improve the quantitative performance of the model, and is the subject of ongoing work.

5. Conclusion

The ability of the Sydney Soil model to represent many aspects of the behaviour of reconstituted and natural soils has been demonstrated in this paper. It was shown that overall the Sydney Soil model gives satisfactory descriptions of the behaviour of six quite different soils for a wide range of stress levels, stress paths and soil types and it can provide satisfactory qualitative and quantitative modelling of many important features of the behaviour of structured soils.

The central assumption of the Sydney Soil model is the existence of an ultimate critical state of deformation for soil when sheared sufficiently, a characteristic demonstrated by many natural soils. The model demonstrates that accurate allowance for the volumetric deformation behaviour is required to simulate the wide range of soil responses. This is accommodated in the Sydney Soil model by using functions to capture more accurately this feature of the soil behaviour.

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