THE BEHAVIOUR OF THREE CALCAREOUS SOILS IN MONOTONIC AND CYCLIC LOADING

by

XIAOYAN MAO

A thesis submitted for the degree of Doctor of Philosophy at The University of Western Australia

Department of Civil and Resource Engineering September 2000
To
My Parents, Brothers and Sister
Abstract

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Calcareous sediments exist in many different forms, each possessing different characteristics. This thesis is aimed at (1) investigating the fundamental behaviour of un cemented calcareous soils covering a wide range of particle sizes in undrained monotonic and cyclic loading, and (2) characterising the observed behaviour using both empirical and theoretical methods.

Three calcareous sediments – muddy silt, silt and sand – recovered from areas off the North-West Coast of Australia are included in this research. A method of reconstituting samples of a fine-grained calcareous silt in such a way as to replicate the behaviour of in situ material is developed. The monotonic behaviour of the three soils is investigated through a series of undrained triaxial and simple shear tests, while the cyclic behaviour is investigated through a series of undrained simple shear tests. The monotonic responses of the three soils are different: the calcareous muddy silt behaves like soft clays, while the calcareous sand and silt behave like sands. However, the cyclic responses of the three soils in simple shear tests are very similar.

The test data obtained from the experimental work are synthesised and analysed. The effect of cyclic shear stress is expressed by cyclic strength curves. The effect of mean shear stress on the cyclic strength is characterised by using a model (the “modified Gerber” model) expressed in terms of equal damage contours in mean stress versus cyclic stress space. A model for characterising pore pressure generation in both symmetric and non-symmetric cyclic loading tests is developed. On the basis of these empirical equations and strain hardening behaviour in respect of shear stress ratio versus shear strain curves, a framework of simulating cyclic simple shear behaviour is proposed, and reasonable predictions for the behaviour of the calcareous muddy silt and the calcareous sand are obtained.

An elasto-plastic effective stress model (the MIT-S1 model) is applied in predictions of the monotonic behaviour of the calcareous muddy silt, calcareous silt and calcareous sand. The model gives good predictions for the monotonic behaviour of the calcareous muddy silt and the calcareous sand. Application of this model to the calcareous silt encounters difficulties due to no clear Limiting Compression Curve (LCC) regime being observed in compression. When a pseudo-LCC regime is assumed, the model predictions capture the basic trend of the behaviour of the calcareous silt in both triaxial and simple shear tests. The predictive capability of the model for cyclic loading is investigated. Poor prediction of cyclic behaviour is observed when the original mapping functions are used, which is not surprising since these were developed on the basis of the observed monotonic behaviour of over-consolidated clays. However, with modified mapping functions the model predicts the behaviour of the calcareous muddy silt qualitatively well for undrained triaxial and simple shear tests under symmetric and non-symmetric cyclic loading.
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Declaration

I hereby declare that, except where specific reference is made in the text to the work of others, the contents of this thesis are original and have not been submitted to any other university.

Xiaoyan Mao
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Introduction

1.1 Background

Calcareous sediments are widely distributed on continental shelves in temperate and tropical areas of the world, including regions such as the Arabian Gulf and the coasts of Australia and India, where there are substantial petrochemical reserves. All calcareous sediments have a similar basic chemistry, but the methods by which they are formed vary among three processes (Fookes 1988):

(a) Skeletal carbonate body, such as shells, coral reefs and similar but broken organic material, eventually become cemented to form a rock of mainly biological origin (bioclastic).

(b) Particulate (clastic) material, such as fragments of older limestones, are transported and eventually cemented to form a rock of mechanical origin.

(c) Material precipitated from water as a result of the pressure, temperature and concentration of the solution, eventually forms a rock or cement of chemical origin.

More than one process may occur in any environment. Even at the same stratigraphical level, calcareous sediments differ, according to the particular environment at their time of deposition and the large variety of possible post depositional (diagenetic) changes. The diversity of formation processes and origins of sediment creates a wide range of soil and rock types, including well-cemented limestone and coral, moderately-cemented and lightly-cemented sands and silts, and completely uncemented muddy silt. Hence, calcareous sediments exist in many different forms, and each possesses different characteristics.

Semple (1988) summarised that the calcareous sediments, especially of the troublesome bioclastic soils, presented three important features, i.e. (1) the individual grains are extremely angular and weak, (2) they exhibit variable cementation, and (3) the materials are variable in particle type and size distribution as well as in degree of cementation. Due primarily to the weak nature of calcareous soil grains and their relatively low densities, calcareous sediments are highly compressible. This causes low skin friction to be mobilised on driven piles, lower bearing capacity and greater settlements under footings than would be expected from a conventional settlement analysis. Traditional
practices have successfully been used for other types of material, but have been proved to be ineffective for offshore foundations on calcareous sediments. Poor understanding of their behaviour, along with improper selection of design tools, has produced many design pitfalls (McClelland 1988, Alba & Audibert 1999). Due to difficulties and failures associated with the foundations of offshore structures founded on calcareous sediments, the behaviour of calcareous sediments has received increased attention over the past two decades. Much of the earlier studies on calcareous sediments is documented in the 1st and 2nd International Symposium on Engineering for Calcareous Sediments held respectively in Perth (Jewell & Andrews 1988, Jewell & Khorshid 1988) and in Bahrain (Shafei 1999).

The majority of previous studies on uncemented calcareous sediments have focused on their compression and monotonic shearing behaviour. The studies of their cyclic behaviour, which is the main concern in the design of offshore foundation, have been limited (e.g. Kagawa 1988, Airey & Fahey 1991, Joer et al. 1995 and Hyodo et al. 1998), indeed, these studies mainly concentrate on the behaviour of calcareous sands. Experiences in the waters off the North West Coast of Australia show that new types of calcareous sediments are constantly being encountered (Fahey 1998), ranging from very fine-grained (in clay size) muddy silt, mid-sized grained silts to coarse sands. Knowledge of the behaviour of fine-grained calcareous sediments in undisturbed condition with high void ratios is limited. Little attention has yet been paid to the comparison of the behaviour of different calcareous sediments covering a wide range of particle sizes.

The use of analytical methods requires extensive knowledge of the basic properties of calcareous sediments. It is, therefore, important to investigate the fundamental behaviour of uncemented calcareous sediments covering a wide range of particle sizes in both monotonic and cyclic loading, which forms the first goal of this study. The second goal of this study is to characterise the monotonic and cyclic behaviour of the soils by use of both phenomenological (empirical) and theoretical methods.

1.2 Scope of the Study

This study consists of two parts according to the above two goals. The first part is to construct a database from laboratory element tests and to investigate the fundamental
behaviour of uncemented calcareous sediments covering a wide range of particle sizes. The second part involves the characterisation of the behaviour of calcareous sediments taking into account experimental evidence through both phenomenological and theoretical methods.

In the first part, three calcareous sediments with particle sizes in the ranges of clay, silt and coarse sand are selected. A method of reconstituting samples of fine-grained calcareous silt in such a way as to replicate the behaviour of in situ materials is developed. The monotonic behaviour of the three calcareous sediments is then investigated through a series of undrained triaxial and simple shear tests. A series of undrained cyclic simple shear tests, which is arguably the most suitable test in assessing soil behaviour beneath offshore foundations, is conducted on the selected calcareous sediments to investigate their cyclic behaviour. Different kinds of loading patterns are involved to investigate different aspects of cyclic behaviour, such as the effects of cyclic shear stress amplitudes and the effects of mean shear stress.

In the second part, the test data obtained from the experimental results are synthesised and analysed. Based on the laboratory test data, empirical models for predicting cyclic response are developed, with particular emphasis on the development of shear strains and reduction of effective stress (increase of pore pressure) under cyclic loading as well as the effect of cyclic shear stress and mean shear stress. The potential of a constitutive model (MIT-S1 model) for simulating the monotonic response of a broad range of calcareous sediments is assessed. The model predictive capability for cyclic loading is also examined.

1.3 Outline of Thesis

This thesis is divided into nine chapters. A brief outline for each chapter is described as follows.

In Chapter 2, methods of characterising soil behaviour are reviewed; both empirical and constitutive methods are included. Empirical models for characterising the behaviour of soils are surveyed, with emphasis on the most important aspects, such as stress-strain response, generation of pore pressure, and the effects of mean shear stress on cyclic behaviour. Various constitutive models for clays and sands under both monotonic and cyclic loading in the literature are also reviewed.
The experimental work is explained in Chapter 3. Firstly, the triaxial and simple shear apparatus together with their measuring and recording systems are described. Three calcareous sediments: calcareous muddy silt, calcareous silt and calcareous sand, used in this study are then introduced, together with the description of the methods used to prepare the reconstituted silt and sands, while the method for preparing the reconstituted muddy silt is described in Chapter 4. Thirdly, triaxial and simple shear testing procedures are described. Finally, the testing programmes carried out to investigate the behaviour of three carbonate sediments in undrained triaxial and simple shear tests are reported.

Chapter 4 outlines a method of preparing reconstituted calcareous muddy silt with the aim of simulating the behaviour of undisturbed samples that possessed large void ratios and showed strain softening behaviour. The detailed procedures used in this preparation are described, including the use of a synthetic flocculant and curing procedure. The mechanical behaviour and the microstructure of the reconstituted samples prepared in this way are then compared with those of undisturbed samples.

Chapter 5 presents the results of the undrained monotonic triaxial and simple shear tests on the three calcareous sediments, and their behaviour is examined individually through effective stress path and stress-strain responses, together with their normalised behaviour in both undrained triaxial and simple shear tests. The shear behaviour of all three calcareous sediments is then linked with their compression behaviour and analysed in the framework of critical state soil mechanics.

The behaviour of the three calcareous sediments in undrained cyclic simple shear tests is reported in Chapter 6. The cyclic responses of the three soils are presented individually in the order of calcareous muddy silt, calcareous silt and calcareous sand. Typical responses of each soil subjected to symmetric cyclic loading are presented. The effects of cyclic shear stress levels, mean shear stress levels, consolidation conditions, and confining stress are examined. Cyclic responses of the three soils are then compared. Two common features are also characterised in respect of effective stress paths and stress ratio-shear strain relationships.

The cyclic response data discussed in Chapter 6 are synthesised and analysed in Chapter 7. The expression of cyclic strength is discussed first, followed by a comparison of
cyclic strengths of different calcareous sediments. A model expressing the effects of non-zero mean shear stress on cyclic strength is then presented. Thirdly, a model for characterising pore pressure generation in undrained cyclic loading is presented. Finally, on the basis of the strain hardening behaviour in respect of shear stress ratio-shear strain curves, a framework of simulating cyclic simple shear behaviour is then presented.

In Chapter 8, an elasto-plastic effective stress model (MIT-S1 model) is evaluated for predicting the monotonic behaviour of the calcareous muddy silt, calcareous silt and calcareous sand. The formulation of the constitutive model and the parameters required by the model are described. After determining the model-input parameters for each soil, the model is evaluated for predictions of the monotonic shear behaviour of each soil. The model capability for predicting cyclic behaviour is also examined.

In Chapter 9, a summary of this thesis is presented, major conclusions obtained from this study are summarised, and future work in this area is recommended.
2
Review of the Methods for Characterising Soil Behaviour

2.1 Introduction

The methods for characterising the behaviour of soils can be classified into two categories, namely, phenomenological and theoretical methods. The former denotes those based on empirical equations derived by curve-fitting. The latter refers to constitutive relationships, which are grounded in the theory of elasticity, plasticity and/or viscosity. Section 2.2 explains the terminology commonly used in cyclic loading. Section 2.3 reviews the empirical models for characterising soil behaviour, including stress-strain response, pore pressure generation in undrained cyclic loading tests, and the effects of mean stress on cyclic behaviour. Some constitutive soil models for monotonic and cyclic behaviour are investigated in Section 2.4.

2.2 Terminology Used in Cyclic Loading

The term ‘cyclic loading’ suggests a system of loading which exhibits a degree of regularity both in its magnitude and in its frequency (O'Reilly & Brown 1991). Although both the magnitude and frequency of waves, winds and storms are irregular, cyclic loading tests with constant amplitude and frequency are mostly used in investigating cyclic behaviour of soils.

In order to investigate the cyclic behaviour of soils, two types of cyclic loading tests are commonly used: (1) constant strain amplitude cyclic loading tests and (2) constant stress amplitude cyclic loading tests. The majority of these tests have been conducted under constant frequency. Constant stress amplitude cyclic loading tests are the most frequently used in relation to geotechnical engineering practices. In this type of cyclic loading test, the terms ‘cyclic stress’ and ‘mean stress’ are defined respectively as the amplitude of cyclic loading and the average of the applied stress around which cyclic loading is applied, as depicted in Figure 2-1.

Depending on the level of mean stress, four possible patterns of cyclic loading tests are shown in Figure 2-2. The term ‘2-way’ cyclic loading is often used to denote cycling in such a way that zero stress is crossed, i.e. cycling from negative to positive values of
stress (e.g. cases in Figures 2-2a and b). By contrast, the term ‘1-way’ cyclic loading is denoted as cycling in a range in which no zero stress is crossed (e.g. cases in Figures 2-2c and d). ‘Symmetric’ cyclic loading is a 2-way case with zero mean stress (Figure 2-2a), which is also called zero mean stress cyclic loading. ‘Non-symmetric’ cyclic loading denotes cycling around non-zero mean stress and is also called non-zero mean stress cyclic loading. The examples shown in Figures 2-2b, c and d belong to this latter category.

As a consequence of cyclic loading, strain will develop with increasing number of cycles applied to the sample, which may be depicted schematically in Figure 2-3. If the maximum strain induced in a cycle is defined as total strain shown in Figure 2-3, the total strain induced in each cycle can be divided into two components: (1) mean strain, which is represented by the average value of strain within a cycle, and (2) cyclic strain, which is the amplitude of strain within a cycle.

2.3 Empirical Models for Characterising the Behaviour of Soils

One of the main concerns in offshore foundation engineering is the cyclic loading applied to offshore foundations by waves and storms. Prediction of the behaviour of saturated soils underneath offshore foundations subjected to cyclic loading is an important aspect of design. Considerable efforts have been made in the 1960s and 1970s in understanding the behaviour of soils under cyclic loading associated with the study of soil liquefaction induced by earthquakes (Seed & Lee 1966, Seed & Idriss 1967, Castro 1975, Castro & Poulos 1976, etc.). The most important aspects of undrained cyclic behaviour are strain development and pore pressure generation, which are directly linked to failure of offshore foundations subjected to waves and storms. This section reviews the empirical models related to the aspects of stress-strain behaviour and pore pressure generation response. Methods accounting for the effects of symmetric and non-symmetric cyclic loading are also surveyed.

2.3.1 Modelling Stress-Strain Behaviour

In a cyclic loading test, the stress-strain behaviour of soils is non-linear and hysteretic. Typical stress-strain behaviour of a constant cyclic stress amplitude test during the first cycle is shown schematically in Figure 2-4. In this figure the associated initial
backbone curve is also constructed, which is defined as the initial loading curve for the positive part and an extension of the initial loading curve into the negative domain.

The initial backbone curve is the basis for characterising the stress-strain behaviour for hysteretic loops. Idriss et al. (1978) summarised that the backbone curve could be expressed in a variety of ways, which included the bilinear (Thiers & Seed 1968), the multilinear (Joyner & Chen 1975), the hyperbolic (Kondner 1963, Kondner & Zelasko 1963, Hardin & Drnevich 1972), the Ramberg-Osgood (Streeter et al. 1974, Richart 1975, Macky & Saada 1984), and the Davidenkov (Seed et al. 1992) formulations. The most commonly used functions are hyperbolic, Ramberg-Osgood and Davidenkov models, which are described as follows:

Hyperbolic model
\[
\tau = \frac{G_0 \gamma}{1 + \frac{G_0 \gamma}{\tau_f}}
\]

In which \(\tau\) is the shear stress at shear strain amplitude \(\gamma\), \(G_0\) is the initial maximum tangent modulus, and \(\tau_f\) is the asymptotic value of \(\tau\).

Davidenkov model
\[
\tau = G_0 \gamma \left[ 1 - H \left( \frac{1}{n} \gamma \right) \right]
\]

In which \(H\) is a function that describes the basic shape of the stress-strain relationship, the coefficient \(n\) is equal to 1 for initial loading and 2 for unloading and reloading.

Ramberg-Osgood model
\[
\gamma = \frac{1}{G_0} \tau \left[ 1 - H \left( \frac{1}{n} \tau \right) \right]
\]

It can be seen that the Ramberg-Osgood model is in the same format as the Davidenkov model but gives the shear strain in terms of the shear stress.

The main difference between the hyperbolic and Davidenkov (or Ramberg-Osgood) models is that the stress-strain curve described by the hyperbolic model is bounded by the asymptotic value of stress, while the stress-strain curve described by the Davidenkov model (or Ramberg-Osgood model for strain-stress curve) is unbounded.
In the description of hysteretic loops, it is widely accepted that two rules, originally proposed by Masing (1926; quoted by Pyke 1979), can be used as a basis for modelling the cyclic non-linear stress-strain behaviour of soils. These two rules are usually stated as: (1) the shear modulus on each loading reversal assumes a value equal to the initial tangent modulus for the initial loading curve, and (2) the shape of the unloading or reloading curves is the same as that of the initial loading curve, except that the scale is enlarged by a factor of two. In the study of the behaviour of soils subjected to irregular cyclic loading, Pyke (1979) proposed two additional rules to supplement the two Masing rules. Pyke called the two supplementary rules the extended Masing rules. The two extended Masing rules are stated as: (3) the unloading and reloading curves should follow the initial backbone (loading) curve if the previous maximum shear strain is exceeded, and (4) if the current loading or unloading curve intersects the curve described by a previous loading or unloading curve, the stress-strain relationship follows that previous curve.

It is evident that the above four rules (the two original and the two extended Masing rules) are applicable to the behaviour of a non-degrading material only, i.e. the initial backbone curve is followed by subsequent hysteretic loops. Such non-degrading behaviour and the above four rules have been successfully modelled by the mechanical model presented by Iwan (1967) and by the incremental theory of plasticity proposed by Prevost (1977).

However, it has been found that the stress-strain behaviour of most soils in undrained cyclic loading presents degrading behaviour, i.e. the initial backbone curve is not followed by the succeeding stress-strain hysteretic loops. The degrading behaviour is schematically illustrated in Figure 2-5.

In order to simulate the degrading behaviour of soils, degraded backbone curves have been adopted (e.g. Idriss et al. 1978, Vucetic 1990, Matasovic & Vucetic 1993). In simulating the degrading behaviour of soft clays, Idriss et al. (1978) proposed an empirical model, in which the stiffness reduction is represented as a function of the number of cycles and a degradation parameter, depending on shear strain amplitude. Vucetic (1990) used a family of degraded backbone curves and families of degraded reloading and unloading stress-strain curves to describe the shear stress-shear strain behaviour of normally and over-consolidated clays, and proposed a fifth rule for
describing the hysteretic loop with regard to the degraded behaviour. Matasovic & Vucetic (1993) proposed a method for characterisation of liquefiable sands, in which the concept of dynamic backbone curve was used in conjunction with the Masing rules to construct unloading and reloading branches of cyclic loops.

Having found that cyclic degradation under undrained simple shear loading conditions could be attributed mainly to decrease in the mean effective stress, Puzrin et al. (1995) normalised the shear stress and shear strain by the current mean effective stress values, and a non-degrading backbone curve (in terms of $\tau/p'-\gamma/p'$) was obtained. The mean effective stress was used as a fatigue parameter, and it was found possible to describe degradation in terms of the parameters of Iwan’s mechanical model. Pradhan (1989) showed that stress ratio-shear strain ($q/p'-\gamma$ for triaxial and $\tau/\sigma_a'-\gamma$ for torsional shear) curves for a silica sand presented strain hardening behaviour. He then described the stress ratio-shear strain behaviour by using the two original Masing rules with the second Masing rule being slightly modified.

### 2.3.2 Models for Characterising Pore Pressure Generation in Cyclic Loading

Many empirical models have been developed for characterising the accumulation of pore pressure during undrained cyclic loading tests. These models range from the simple (e.g. Bjerrum’s linear function) to more complicated (e.g. Ishibashi’s function). Some of these models are discussed below.

(a) **Bjerrum’s Linear Function**

A linear relationship between normalised pore pressure rise and cycle number was proposed by Bjerrum (1973) as:

$$\beta(\%) = \frac{\Delta u}{\sigma'_v N} \times 100$$

In the above equation $\Delta u$ is the excess pore pressure after $N$ cycles, and $\sigma'_v$ is the consolidation effective vertical stress in a cyclic simple shear test. The parameter $\beta$ represents the rate of pore pressure ratio increase. However, it is observed that in most undrained cyclic loading tests the pore pressure rise in the first few cycles is much higher than for subsequent cycles. Therefore, the application of Bjerrum’s linear function is limited.
(b) Berkeley Trigonometric Function

From the study of soil liquefaction at the University of California, Berkeley, Seed et al. (1975) related pore pressure ratio to the cyclic ratio through characteristic curves expressed as the following:

\[ \frac{\Delta u}{\sigma'_{v0}} = \frac{2}{\pi} \sin^{-1} \left[ \left( \frac{N}{N_1} \right)^{\frac{1}{2}} \right] \]

where \( \Delta u = \) excess pore pressure, \( \sigma'_{v0} = \) consolidation effective vertical stress, \( N = \) cycle number, \( N_1 = \) number of cycles to initial liquefaction, \( \theta = \) an empirical constant depending on type of soil and cyclic stress ratio.

This function has been widely used in analyses of earthquake and wave loading-related problems (e.g. Rahman et al. 1976, 1977). However, since the function is derived using pore pressure variations at zero shear stress, its application is limited to cyclic stress conditions where the shear stress crosses zero during some stage of cyclic loading. Therefore, it cannot be used for conditions where the shear stress does not cross zero.

(c) Sarma’s Dynamic Pore Pressure Parameter

Sarma (1976) presented a dynamic pore pressure parameter, \( A_n \), to predict the pore pressure generation in undrained cyclic loading tests as follows:

\[ r_u = A_n r_d \]

where \( r_u = \) pore pressure ratio, \( (\Delta u/\sigma'_{rc}) \) for cyclic triaxial test conditions, or \( (\Delta u/\sigma'_{v0}) \) for cyclic simple shear test conditions; \( r_d = \) cyclic stress ratio, \( (q_{cy}/\sigma'_{rc}) \) for cyclic triaxial tests, and \( (\tau_{hv}/\sigma'_{v0}) \) for cyclic simple shear tests.

Sarma and Jennings (1980) utilised the results from cyclic simple shear tests, shaking table tests, and triaxial cyclic loading tests, and proposed an expression for the same dynamic pore pressure parameter, \( A_n \), for loose sands:

\[ \sqrt{A_n} = \sqrt{A_1} + \beta_1 \log N \]

In which \( A_1 \) is the dynamic pore pressure parameter corresponding to the cycle number \( N=1 \), \( \beta \) is a parameter constant for a given material and state of testing.
From the results of cyclic simple shear tests (Martin et al. 1975), Tsatsanifos and Sarma (1982) expressed the parameter $A_1$ by a hyperbolic relationship as:

$$A_1 = \frac{C_1}{1 - C_2 r_d}$$

Parameter $\beta_n$ is assumed to be a linear function of stress level $r_d$ (Tsatsanifos and Sarma 1982) and is expressed as:

$$\beta_n = C_3 + C_4 r_d$$

Combining Equations 2-6a, b, c and d, the pore pressure generation function can now be expressed as:

$$r_u = \left( \left( \frac{C_1}{1 - C_2 r_d} \right)^{1/2} + (C_3 + C_4 r_d) \log N \right)^2 r_d$$

Parameters $C_1$, $C_2$, $C_3$ and $C_4$ can be determined from a series of three to four constant stress undrained cyclic loading tests under different stress ratios. $C_1$ and $C_2$ are computed from the hyperbola representing the pore pressure ratio versus stress ratio for the first cycle. $C_3$ and $C_4$ are obtained by plotting the slope of Equation 2-6b with respect to the stress ratios.

This function has three limitations: (1) four parameters without physical meaning are needed, (2) as pointed out by Tsatsanifos and Sarma (1982), the parameters depend on the type of soil as well as on the mode of testing, which means they must be determined from tests with conditions representative of the expected field conditions, and (3) the effect of mean stress on pore pressure behaviour is not considered.

**Ishibashi's Pore Pressure Function**

Ishibashi et al. (1977) assumed that the increment in the residual pore pressure during the $N^{th}$ cycle is a product of the stress history $H$, number of cycle effects $\bar{N}$, and applied stress intensity function $I$.

$$\Delta U_N = H \cdot \bar{N} \cdot I$$
where $\Delta U_N = (u_N - u_{N-1})/\sigma'_c$ is the normalised increment in residual pore pressure during the $N^{th}$ cycle, $u_N$ and $u_{N-1}$ are the residual pore pressures at the end of the $N^{th}$ cycle and $(N-1)^{th}$ cycle respectively, $\sigma'_c$ is the initial effective confining pressure, $N$ is the cycle number.

The stress history ($H$), number of cycle effects ($\overline{N}$) and applied stress intensity function ($I$) are expressed by:

\[ H = 1 - U_{N-1} \quad 2-7b \]
\[ \overline{N} = \frac{C_1 \cdot N}{N^{C_2} - C_3} \quad 2-7c \]
\[ I = \left( \frac{\tau_N}{\sigma'_N-1} \right)^{a} \quad 2-7d \]

where $U_{N-1} = u_{N-1}/\sigma'_c$ is the normalised total residual pore pressure at the end of the $(N-1)^{th}$ cycle, $\tau_N$ is the applied shear stress in the $N^{th}$ cycle, $\sigma'_{N-1}$ is the effective confining stress, parameters $\alpha$, $C_1$, $C_2$, and $C_3$ are material constants. Therefore, the normalised incremental residual pore pressure can be represented as:

\[ \Delta U_N = \left(1 - U_{N-1}\right) \left( \frac{C_1 \cdot N}{N^{C_2} - C_3} \right) \left( \frac{\tau_N}{\sigma'_N-1} \right)^{a} \quad 2-7e \]

By using this function, it is possible to predict the pore pressure rise under a given uniform cyclic loading corresponding to any given number of cycles.

In order to predict pore pressure under non-uniform cyclic loading, Ishibashi et al. (1977) introduced an equivalent number of cycles $N_{eq}$ which is expressed as:

\[ N_{eq} = \sum_{i=1}^{N} \left( \frac{\tau_i}{\tau_N} \right)^{2.4} \quad 2-7f \]

where $\tau_i$ is the applied shear stress amplitude corresponding to the $i^{th}$ cycle (where $1 \leq i \leq N$), and $\tau_N$ is the shear stress amplitude at the $N^{th}$ cycle. When the $N_{eq}$ value, instead
of N, is introduced in Equation 2-7e, then the pore pressure rise for non-uniform cyclic loading can be expressed as:

$$\Delta U_N = (1 - U_{N-1} \left( \frac{C_1 \cdot N_{eq}}{N_{eq} \cdot C_t - C_3} \right) \left( \frac{\tau_N}{\sigma_{N-1}} \right)$$

This function is based on undrained cyclic tests subjected to high shear stress levels and failing within 100 cycles. Both uniform and non-uniform cyclic loading tests are symmetrical around zero shear stress. The effect of mean stress on pore pressure behaviour is not considered. There are four parameters that must be determined by curve fitting.

(e) *Datta’s Function*

Datta et al. (1980) proposed a pore pressure generation function based on a study of cyclic triaxial tests on a dense reconstituted carbonate sand. The cyclic stress conditions were wholly compressive (from zero to maximum deviator stress).

$$\frac{\Delta u_m}{\sigma_{ec}} = C + K \cdot \log N$$

where $\Delta u_m$ is the mean pore pressure in a cycle, $\sigma_{ec}$ the initial effective confining pressure, N the cycle number, C the value of the ratio $\Delta u_m/\sigma_{ec}$ after the first cycle, and K the cyclic pore pressure parameter, equal to the slope on the $\Delta u_m/\sigma_{ec} \cdot \log N$ graph. Both C and K are affected by stress level and confining pressure, and therefore they are not material constants.

(f) *Modified Berkeley Function*

In studying the cyclic behaviour of calcareous sands in symmetric and non-symmetric cyclic triaxial tests, Kaggwa (1988) normalised pore pressure with ‘limit excess pore pressure’ rather than consolidation pressure, and modified the Berkeley model as follows:

$$\frac{\Delta u}{\Delta u_1} = \frac{2}{\pi} \sin^{-1} \left[ \beta_p \left( \frac{N}{N_f} \right)^{1/2} \right]$$
where $\Delta u_1$ is the limiting pore pressure defined as the maximum excess pore pressure that can be achieved in a cyclic loading test, $N_f$ is the number of cycles required to develop the limiting pore pressure, and $\theta$ is an empirical constant depending on type of soil and cyclic stress ratio. This model has been developed based on undrained cyclic triaxial tests on calcareous sands.

2.3.3 Models Accounting for the Effect of Mean Stress

When an offshore gravity foundation is subjected to cyclic wave loading, the underlying soil elements experience both symmetric cyclic loading and non-symmetric cyclic loading depending on their locations (Andersen 1976, 1991). The damaging effect of cyclic loading depends not only on the amplitude of the cyclic stress but also on the mean stress.

The effects of non-zero mean stress on the fatigue life of materials and structures have long been an important aspect of the study of fatigue problems. The general method used to compare cycling around non-zero mean stresses with cycling around zero mean stress is to introduce equal damage contours in stress space. In the aspect of metal fatigue life study, several empirical models have been proposed to account for the effect of non-zero mean stress on the fatigue life of metals (Collins 1981, Benham et al. 1996). The most widely used are the Goodman and Gerber models, which are presented as follows:

Goodman model:

$$\sigma_{eq} = \frac{\sigma_{cyc}}{1 - \left(\frac{\sigma_m}{\sigma_u}\right)}$$

Gerber model:

$$\sigma_{eq} = \frac{\sigma_{cyc}}{1 - \left(\frac{\sigma_m}{\sigma_u}\right)^2}$$

where $\sigma_{cyc}$ and $\sigma_m$ are cyclic and mean stress level respectively, $\sigma_{eq}$ is the cyclic stress level equivalent to zero-mean stress loading, $\sigma_u$ is the ultimate (tensile) strength of the material.

The diagram of equal damage contours represented by the two models is shown in Figure 2-6. The solid curves are Gerber's parabolic relation, the dotted lines are
Goodman’s linear relationship, and the thick line represents the one cycle failure line. By using this diagram or Equations 2-10 and 2-11, the fatigue life of metal for any desired combination of mean stress level and cyclic stress amplitude can be represented by equal-failure contours.

The equal damage contour method used in the fatigue life of metals has been applied in the behaviour of soils subjected to cyclic loading by several authors (Poulos 1988, Malek et al. 1989, Joer et al. 1995, McCarron et al. 1995). Poulos (1988) presents the idea of a cyclic stability diagram as a mean of defining the response of a pile to various combinations of mean and cyclic load. Malek et al. (1989) presents contours of equal numbers of cycles to failure from undrained cyclic direct simple shear tests performed on reconstituted Boston Blue Clay. Joer et al. (1995) used the Gerber model to relate one-way cyclic triaxial and direct simple shear tests to equivalent symmetric cyclic tests. A critical level of repeated loading was obtained by McCarron et al. (1995) in the same stress space from direct simple shear tests performed on Beaufort Sea clay.

2.4 Review of Constitutive Models for Soils

As the finite element or finite difference methods have become established tools in the static and dynamic analysis of complex geotechnical engineering problems, the need for more accurate simulated soil behaviour has increased, which has motivated the development of a large variety of constitutive soil models over the last several decades. The choice of an appropriate constitutive model in numerical analysis will ultimately determine whether the solution obtained is realistic and meaningful. This section reviews constitutive models for soils (clays and sands) under both monotonic and cyclic loading.

The summaries on constitutive models for soils given by Pande & Pietruszczak (1986), although somewhat outdated, provide a good overview of the major categories of model formulations available. Depending on the fundamental theories on which they are based, the various constitutive models can be classified into four categories: (1) Non-linear elasticity models, (2) Elasto-plasticity models, (3) Elasto-visco-plasticity models, and (4) Endochronic-plasticity models. Among these categories, the elasto-plasticity theory is the most widely used in predicting soil behaviour (Bianchini et al. 1991). It is believed that the models based on the theory of elasto-plasticity are the most versatile and have the highest predictive capabilities (Penda & Pietruszczak 1986).
In general, the stress-strain behaviour of uncemented soils can be explained within the framework of Critical State Soil Mechanics (Roscoe et al. 1958, Schofield & Wroth 1968, Wood 1990). The concept of critical state was firstly incorporated into a constitutive soil model, the so-called original Cam Clay Model (Roscoe & Schofield 1963). Since then, the theory of critical state soil mechanics has been incorporated in many comprehensive models owing to the advantage of ready comprehensibility and generally a small number of material parameters.

In the framework of elasto-plasticity and critical state soil mechanics, many constitutive models have been developed to simulate the behaviour of both clays and sands under monotonic loading. The Cam Clay models (Roscoe & Schofield 1963, Roscoe & Burland 1968) predict well the behaviour of isotropically consolidated clays but not that of anisotropically consolidated clays. In order to predict the behaviour of anisotropically consolidated clays, Prevost (1978a) developed a model combining the properties of isotropic and kinematic hardening by introducing the concept of a field of plastic moduli. The model presented by Whittle & Kavvadas (1994), which uses isotropic and rotational hardening, describes the anisotropic and strain softening behaviour of clays very well. By incorporating bounding surface plasticity, this model also shows good prediction in the behaviour of over-consolidated clays.

There are many models for the behaviour of sands available in the literature (e.g. Bardet 1986, Jefferies 1993, Crouch et al. 1994, Pestana & Whittle 1999, etc.). The limitation of Bardet’s model is that model-input parameters are dependent on void ratio. The models of Jefferies (1993) and Crouch et al. (1994) overcome this drawback by including the void ratio as a state parameter. However, these models describe well the behaviour of isotropically consolidated sands but not that of anisotropically consolidated sands. A recently reported model (Pestana & Whittle 1999), which overcomes the above mentioned limitations, is capable of predicting the rate independent, effective stress-strain-strength behaviour of uncemented soils (both clays and sands) over a wide range of confining pressure and densities. This model will be evaluated for prediction of the behaviour of three calcareous sediments in Chapter 8.

The simulation of the cyclic behaviour of soils seems more complicated by the need for descriptions of non-linearity during unloading and for descriptions of induced anisotropy caused by reversal of loading. However, there are still many constitutive
models in the literature that have shown their predictive capabilities for cyclic behaviour qualitatively well.

On the basis of the levels of sophistication, Norris (1986) divided the models for cyclic loading into two categories: (1) simple models, which are fitted to experimental data at only one point in each cycle, and (2) sophisticated models, which are based on complete stress-strain histories of cycles. The models in the first category include those proposed by Carter et al. (1982) and Van Eekelen (1982). The models in the second category are those which use stress-reversal surfaces to incorporate the dependence of material behaviour on stress path history.

The existing models using stress-reversal surfaces to describe the effect of stress path history on the cyclic behaviour fall into two groups: (1) multi-surface models and (2) two-surface models. The multi-surface models are based on the work of Iwan (1967) and Mroz (1967). Although Iwan and Mroz presented their models independently, the two models serve the same purpose. These models have been used for soils and are known as the nested yield surface or multi-surface plasticity model. Prevost (1978b) extended the Iwan/Mroz model for the undrained behaviour of clays under monotonic and cyclic loading conditions. In this development, the basic work-hardening rule is still of the kinematic type but a simultaneous isotropic hardening (or softening) is allowed. In simulating the undrained behaviour of clays, Houlsby (1999) uses multiple yield surfaces within the framework of work-hardening plasticity theory to take into account the non-linear behaviour of soil at small strains as well as the effects of stiffness on recent stress history. One of the major disadvantages of multiple surface models, as pointed out by Houlsby (1999), is that they require specification of a large number of material parameters corresponding to each yield surface.

There are many two-surface models, which are also called 'bubble' models (Tabbaa & Wood 1989), for describing the cyclic behaviour of soils. Different names are given to the outer surface in these models such as bounding surface (Dafalias & Popov 1975), limiting surface (Krieg 1975), consolidation surface (Mroz et al. 1979), memory surface (Tseng & Lee 1983), and bond strength envelope (Kavvadas & Amorosi 1998). Although the different two-surface models possess specific features in addition to the main feature of a distance dependent plastic modulus, all of them fall within the general analytical framework presented by Dafalias (1986), who also claimed that the multi-
surface formulation is a particular case of the bounding surface general formulation, considering a piecewise mapping rule instead of a continuous one, and a piecewise variation of the plastic modulus instead of a continuous one.

The two-surface plasticity (or usually called bounding surface plasticity) has provided a simple and flexible theoretical framework to assemble the proposed models and has been widely used by many researchers in a variety of plasticity constitutive models for the behaviour of both clays and sands. Dafalias & Herrmann (1982) applied the bounding surface plasticity for monotonic and cyclic behaviour of clays under isotropic consolidation conditions. In simulating cyclic behaviour of normally and over-consolidated clays, Mroz et al. (1978, 1979 and 1981) developed two-surface models for soils, using a full, effective stress vector and a boundary surface founded on the complete critical state ellipse. By using bounding surface plasticity, Bardet (1986) constructed a constitutive equation to simulate the non-linear behaviour of loose and dense sands subjected to various types of loading. Whittle (1987) incorporated bounding surface plasticity into an incrementally linearised elasto-plasticity model, which he used to predict the behaviour of normally consolidated Boston Blue clay in undrained cyclic simple shear tests as well as the behaviour of over-consolidated Boston Blue clay. The above mentioned models have a common feature that the operation of the two-surface (or bounding surface) models takes place in the deviator stress - mean effective stress plane (or q-p' plane in the case of triaxial loading). Qualitative agreements between predicted and real soil behaviour have been observed in the applications of these models. However, none of the above models can simulate the phase transformation behaviour (i.e. from contractive to dilative) observed in the stress paths of many undrained cyclic loading tests as shown in Figure 2-7, which is also called a 'butterfly' shaped stress path.

In order to describe the behaviour of sands under rotational shear, Wang et al. (1990) constructed a constitutive model by combining the bounding surface and hypoplasticity (Dafalias 1986) and the creation of a new loading surface upon reverse loading (Mroz and Zeinkiewicz 1984) in a novel way. This model was modified later by Li (1997) and Li et al. (1999) to incorporate the basic premises of critical-state soil mechanics and cover both dense and loose sand behaviour. The original (Wang et al. 1990) and modified (Li et al. 1999) bounding surface hypoplasticity models operate the bounding
surface in the deviatoric stress plane. Both models show the capability of simulating the feature of the effective stress paths shown in Figure 2-7.

In general, many constitutive models exist, which are able to simulate the behaviour of both clays and sands under monotonic and cyclic loading conditions. However, all these models have certain inherent advantages and limitations, which depend to a large degree on their particular applications. It is pointed out by Chen (1989) that no one mathematical model can completely describe the complex behaviour of real soils under all conditions. Each soil model is aimed at a certain class of phenomena, captures their essential features, and disregards what is considered to be of minor importance in that class of applications.

2.5 Summary

Empirical and constitutive models in the literature for characterising soil behaviour have been reviewed in this chapter. The hysteretic stress-strain behaviour of soils in cyclic loading tests can be described by an initial and degraded backbone curves and two original and two extended Masing rules. The most commonly used functions for describing backbone curves are hyperbolic, Ramberg-Osgood and Davidenkov models. Empirical models for characterising pore pressure generation range from very simple (Bjerrum’s linear function) to complex (Ishibashi’s four-parameter function). All these models are developed from the results of cyclic tests on sands and most of them under symmetric cyclic loading conditions. Goodman and Gerber models have been widely used in fatigue studies for characterising the effects of non-zero mean stress on cyclic behaviour.

There are many constitutive soil models for characterising the stress-strain and failure behaviour under monotonic loading conditions. All these models have certain intrinsic advantages and limitations, which depend on their particular applications. The widely used models for simulating cyclic behaviour are those that are based on elasto-plasticity and can be classified into two categories: (1) multi-surface models and (2) two-surface models. Each of these models can simulate one or more aspects of cyclic behaviour qualitatively well.
3
Experimental Work

3.1 Introduction

The behaviour of calcareous sediments subjected to undrained monotonic and cyclic loading at the single element level is usually investigated through laboratory element testing, mainly triaxial and simple shear tests. In this chapter, triaxial and simple shear apparatuses used in this study are described. Three calcareous sediments: muddy silt, silt and sand, which were recovered from offshore areas of the North West of Australia, are then introduced. For each of the soils, microscopic views of particles and particle size distribution are presented. Sample preparation methods for the silt and sand are described. In the third part, the triaxial and simple shear testing procedures are explained. Finally the testing programmes for the three soils are presented.

3.2 Testing Equipment

3.2.1 Triaxial Test Apparatus

The triaxial test apparatus, manufactured at The University of Western Australia (UWA), is shown in Photo 3-1. The apparatus tests a cylindrical sample confined in a latex membrane, within a pressurised cell. The triaxial apparatus consists of six components comprising a loading system, a displacement measurement system, a cell pressure controlling system, a back-pressure system, a signal amplifier box and a controlling computer. A schematic diagram of the triaxial apparatus is shown in Figure 3-1.

The loading system consists of an electric motor connected to a ball-and-screw drive through a reduction gearbox. This system is attached directly to the top of triaxial cell. A load cell, which is connected to the loading system, can measure axial load within the cell, eliminating problems with ram friction. The capacity of the load cell may be chosen to give reasonable resolution depending on the strength of the sample. The displacement rate can be selected using a motor and a gearbox driving the loading ram at a constant displacement rate. A constant rate of loading can also be achieved by
feedback from the computer control system. The control program selects the displacement rate of the machine and the displacement direction.

Axial displacements are measured using an external LVDT (Linear Variable Differential Transformer) attached to the loading ram in such a way that it measures the relative movement between the loading ram and the top of the triaxial cell.

A cylindrical sample confined in a latex membrane is placed in the centre of the cell, which is then filled with water. The water in the cell is pressurised using an air-water interface cylinder, with the air pressure being computer controlled using a pressure controller manufactured at UWA. The maximum allowable cell pressure in the triaxial apparatus is 3 MPa.

Drainage lines are attached to the top cap and base pedestal, so that drainage can occur from both ends of the sample. Alternatively, the bottom drainage tap can be shut off during consolidation, allowing the pore pressure at this end to be monitored. This allows consolidation tests to be carried out in which the coefficient of consolidation (c_v) is determined from pore pressure response as well as from height change or volume change. A Geotechnical Digital Systems (GDS) pressure controller is used to apply back-pressure and measure volume change. The maximum controlling pressure of the GDS is 1.5 MPa.

The electric signals from the load cell, LVDT (or LDTs), cell pressure and back-pressure transducers are amplified through a signal amplifier box, and are then sent to the computer. The progress of a test may be viewed on the computer monitor as the test proceeds. Triaxial tests with different stress paths can be conducted using a computer control programme.

3.2.2 Simple Shear Test Apparatus

Two simple shear apparatuses, namely "old" and "new" simple shear apparatuses shown in Photos 3-2 and 3-3, are used in this study. The principles of the two simple shear apparatuses are similar except for loading-control systems and sizes of samples. Two drawbacks of the old simple shear apparatus are: (1) a pneumatic controlled loading system, (2) fixed sample size; only 50 mm diameter sample can be tested. The former results in difficulty in controlling the horizontal displacement during cyclic loading, especially when the sample is close to failure, i.e. overshooting of cyclic shear stress
often occurs towards the end of cyclic loading tests. The latter constrains the size of the samples. In order to overcome the two drawbacks of the old simple shear apparatus, the new simple shear apparatus has been manufactured ‘in house’ at UWA.

Schematic diagrams of the old and new simple shear apparatuses are shown in Figures 3-2a and 3-2b respectively. The principle of the new simple shear apparatus will be discussed in the following with the differences from the old simple shear apparatus being mentioned whenever required. In the following discussion, “simple shear apparatus” will indicate the new simple shear apparatus, if not specified.

The simple shear apparatus consists of a loading system, a displacement measurement system, a cell pressure controlling system, a back-pressure system, a signal amplifier box and a controlling computer. The sample is confined in a latex membrane, within a pressurised cell, much like a triaxial sample.

The simple shear apparatus is a displacement-type machine. As shown in Figure 3-2, the horizontal and vertical-loading rams are connected to two motors and gearboxes and constant displacement rates can be achieved. A constant rate of loading can also be achieved by feedback from the computer control system. The displacement rate and direction are selected by the control program. Vertical force is measured using three cells supporting the base pedestal and a load cell mounted between the loading ram and the top cap. Horizontal force is measured using a load cell incorporated into the lateral loading ram.

The displacements of both the vertical and horizontal loading rams are measured internally using LVDTs (external LVDTs are used for the old simple shear apparatus). The cell pressure is applied using compressed air, with the air pressure being computer controlled using a pressure controller manufactured at UWA. Drainage lines are attached to the top cap and base pedestal, so that drainage can occur from both ends of the sample. A GDS pressure controller is used to apply the back-pressure to the sample and measure the volume change.

The test is controlled, and the data logged, using a computer. The progress of the test may be viewed on the computer monitor as the test proceeds. Data can be logged at rates exceeding one complete set of data per second.
The feedback system allows the total vertical stress to be kept constant during the shearing phase (monotonic or cyclic) while maintaining a constant sample height. This is achieved by locking the vertical loading ram, and using the feedback system to vary the cell pressure to maintain the total vertical stress constant. Since the height and volume are both constant (tests are undrained), the cross-sectional area must remain constant (on average) as well.

3.3 Material Used and Sample Preparation Methods

Calcareous sediments exist in many different forms and possess various characteristics due to their different origins and formation processes. This study focuses on the behaviour of uncemented calcareous sediments covering a wide range of particle sizes. It investigates three uncemented calcareous sediments, namely calcareous muddy silt, calcareous silt and calcareous sand. Details of each soil are discussed in the following sections.

3.3.1 Calcareous Muddy Silt

The calcareous muddy silt was originally recovered from 18-100 m depth beneath the seabed at the Gorgon site on the shelf close to the edge of the North West Shelf of Australia. The soil conditions are similar at all depths down to about 100 m, apart from one thin layer of sandier soil. The water depth at the site is about 200 m. The undisturbed samples were obtained as part of a site investigation for a proposed gas production platform.

This soil is composed of over 95 % calcium carbonate, with X-ray diffraction tests indicating that over 60 % of this are in the form of aragonite, with the remainder being calcite. With reference to the ESEM (Environmental Scanning Electronic Microscope) image of the reconstituted soil shown in Figure 3-3, the individual aragonite crystals may be discerned to be elongated needle-like in shape, with a length in the order of 2-5 μm, and a diameter in the order of one-tenth of the length.

Figure 3-4 shows the grading curves of samples from various depths down to 100 m and this reveals that approximately 60 % of particles in this soil are in the range of clay size (< 2 μm). It should be noted that none of the particles is clay mineral. The specific gravity (Gs) lies between that of calcite (2.7) and that of aragonite (2.9). The liquid limits lie between 40 and 50, with plastic limits of about 30, giving plasticity indices (Ip)
of 10-20. According to the classification system proposed by Fookes (1988), the soil would be designated calcareous (or carbonate) muddy silt.

The soil is uncemented, with a very high in situ void ratio (typically between 1.6 and 1.8 and up to 2.0 in some cases), which is attributed to its highly structured fabric (this issue will be discussed in Chapter 4). The undisturbed samples were used for a series of undrained monotonic and cyclic triaxial and simple shear tests associated with a preliminary study for the platform project. The left-over material, including material used in these tests (some of this material had been oven dried), was collected with the intention of investigating the fundamental behaviour of this type of soil by use of reconstituted samples. Of eight samples reconstituted by sedimentation, and having been subjected to a vertical consolidation stress equivalent to a depth of about 20 m, the void ratios of seven of them were found to be between 1.23 and 1.31, with the other having a void ratio of 1.39. The shearing behaviour of the reconstituted samples was also found to be quite different from that of the undisturbed samples in undrained monotonic triaxial and simple shear tests. Therefore, there was a necessity to develop an alternative method of preparation of reconstituted samples, which would show the same behaviour as undisturbed samples, in order that the fundamental behaviour of this type of soil could be investigated.

A method using flocculation was found to satisfy this purpose and will be described in detail in Chapter 4. The samples prepared by this method are called ‘floc soil’ in this thesis. The compression and shearing behaviour of floc soil has been compared with that of undisturbed samples through compression tests and a series of undrained simple shear and triaxial tests. The microstructure of floc soil has also been compared with that of the undisturbed sample through ESEM. It was found that the microstructure and mechanical behaviour of floc samples are very similar to those of undisturbed samples (details in Chapter 4). Therefore, the floc samples were used to explore the fundamental behaviour of the calcareous muddy silt.

The floc samples used in triaxial tests were 50 mm in diameter and with the ratio of height to diameter greater than 2. The simple shear samples were 50 mm in diameter and 18 mm in height. The simple shear tests on the calcareous muddy silt were undertaken using the old simple shear apparatus.
3.3.2 Calcareous Silt

The second soil studied in this project is a seabed material recovered from the surface of the seabed at the Gorgon site (same site as the first soil recovered) at the North West shelf of Australia. The motivation for recovery of this seabed material was to conduct centrifuge model tests in which large soil mass was required. The original seabed material was first dried in an oven and then sieved through a 150 μm mesh. This work was carried out outside the campus by an independent company. The microscopic view of the calcareous silt is shown in Figure 3-5, which indicates that the soil consists of shell fragments, with some of them rounded in shape. The grading curves shown in Figure 3-6 suggests that 70 % of the soil particles belong in the silt range with 30 % in the range of fine sand. This seabed material after sieving is designated as calcareous silt.

Preliminary study on the calcareous silt showed that in undrained monotonic triaxial and simple shear tests the soil presented dilative behaviour. However, the purpose of the centrifuge model tests was to simulate the behaviour of foundations on the deep material (the calcareous muddy silt), which has large void ratios and presents strain softening behaviour in undrained triaxial compression tests. Two methods have been used in order to increase void ratio and diminish the dilative behaviour as follows: (1) A commercially produced carbonate powder named 'OMYA CARB' was chosen to blend with the natural soil as filler, and (2) Silicon oil was used as pore fluid instead of water in order to reduce $c_v$, the coefficient of consolidation. The grading curve of OMYA CARB is shown in Figure 3-6 together with that of the calcareous silt. The specific gravity is 2.77 for the calcareous silt and 2.71 for OMYA CARB.

The samples used in simple shear and triaxial tests were prepared from slurry. First a mixture of 85 % dried calcareous silt and 15 % OMYA CARB were combined together, then 70 % silicon oil by weight of total dry soil mass was added to this mixture. After being thoroughly mixed and de-aired, the soil-oil slurry was ready to be used for reconstituting samples.

In simple shear tests (conducted in the 'old' apparatus), samples are easily handled due to the small sample size (50 mm in diameter and 18 mm in height). The soil-oil slurry was poured into a tube of 72 mm diameter and was consolidated under vertical effective stress of 100 kPa for 3 days. The consolidated sample was pushed out of the tube and
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cut to the size of 50 mm in diameter and 18 mm in height using a thin-walled ring cutter. The sample was then placed on the base pedestal of the simple shear apparatus. The membrane and the top cap were fitted and the sample was then ready to be placed into the cell for testing. The simple shear tests on the calcareous silt were conducted using the "old" simple shear apparatus.

In preparing triaxial samples, a split mould of 72 mm diameter was used. This mould was attached to the lower platen of the cell whilst the membrane was maintained in position inside the mould by a vacuum. The soil-oil slurry, prepared as mentioned above, was then poured into the mould from the top. After levelling the surface, a filter paper and a porous disc were placed on top of the slurry. One-dimensional pre-consolidation was then performed on the slurry with drainage allowed at both the top and bottom. After the sample was consolidated under vertical effective stress of 100 kPa, the two-segment sample former was removed leaving a sample ready for testing. The diameter of the triaxial samples was 72 mm with height in the range of 150 to 165 mm depending on the amount of soil-oil slurry added to the consolidation mould.

In the following chapters, 'the calcareous silt' implies the mixture of the seabed material after sieving, OMYA carbonate powder and silicon oil.

3.3.3 Calcareous Sand

The third soil is a seabed sand recovered from the seabed at the Legendre field in the North West Shelf offshore area of Australia. The seabed material was first dried in an oven and then sieved through a mesh of 2.4 mm. The particles less than 0.6 mm were removed from the soil in order to create a uniform coarse sand sample with the intention of making samples as loose as possible. The seabed sand is composed of small shells of round disk-like shapes (Figure 3-7). The grading curve of this calcareous sand is shown in Figure 3-8, and suggests that the sand is a uniform coarse sand, therefore, this material is designated as coarse calcareous sand. In the following discussion 'coarse' will be omitted for brevity.

Simple shear tests were carried out using the "new" simple shear apparatus. In preparing simple shear samples, the calcareous sand was soaked in water and the mixture was then de-aired. A membrane was attached to the bottom cap of the simple shear by use of two 'O-rings'. A two-segment sample former, with diameter of 82 mm
and height of 32 mm, was then positioned. The de-aired saturated sand was then placed with the aid of a teaspoon layer by layer until the height was just above the top of the mould. During this procedure, the sand was maintained below the water surface and with zero falling distance to produce samples in as loose a state as possible. The sand above the top of the mould was then trimmed and the top cap was then placed on top of the sample. Next the membrane was pulled on the top cap and two ‘O-rings’ were placed on. The sample together with the two-segment sample former was then placed into the simple shear apparatus. After the top and bottom caps were fixed, the two-segment sample former was removed leaving a sample ready for testing.

Samples for triaxial tests were prepared in a split mould with 72 mm diameter by pluviation of the dry sample through air. The split mould was attached to the lower platen of the cell and the membrane held to the inside of the mould by a vacuum. The dry sand was then poured into the mould from the top to ensure samples were as loose as possible. After levelling the surface, a filter paper and a porous disc were then placed on top. Water was flushed through the sand from top to bottom. This allows the sample to become slightly dense and to have some cohesion to stand when the two-segment sample former was removed. The ratios of sample height to diameter of all the triaxial samples were greater than 2.

3.4 Testing Procedures

3.4.1 Testing Procedures in Triaxial Tests

After setting up the sample in the manner described in Section 3.3, the drains were connected to the GDS back-pressure apparatus, and saturation of the sample was then commenced. A back-pressure of 1000 kPa was used in triaxial tests, while cell pressure of 1010 kPa was applied during saturation. In order to determine the degree of saturation, a B value test was carried out, and this consisted of closing drainage valves, and applying a cell pressure increment of 100 kPa, while measuring the pore pressure. A B value of 95% or greater was achieved in the triaxial tests conducted on the three soils.

For normally consolidated tests on all three soils, samples were consolidated either isotropically or anisotropically. For isotropic consolidation, the cell pressure was gradually ramped up, controlled by a computer program, until the target value was
reached. For anisotropic consolidation, the consolidation stresses were applied by slowly ramping the deviator stress to the required target value, and adjusting the cell pressure to ensure the required consolidation stress ratio ($\sigma'_{ho}/\sigma'_{vc}$). The ramping time was generally in the order of 2-3 hours for the calcareous muddy silt and calcareous silt, 20-30 minutes for the calcareous sand. Following ramping, sufficient time was allowed for creep to occur. The total time allowed for consolidation and creep was 15 hours for the calcareous muddy silt and the calcareous silt and 3 hours for the calcareous sand.

For over-consolidated tests on the calcareous muddy silt, samples were consolidated isotropically and/or anisotropically. For isotropic consolidation, the cell pressure was gradually ramped up until the target value ($p'_c$) was reached. This pressure was maintained overnight (at least 12 hours), then the cell pressure decreased gradually to the required cell pressure ($p' = p'_c/OCR$). The new pressure was maintained for 5 hours. For anisotropic consolidation, one-dimensional consolidation (i.e. 'true $K_0$' condition) was applied on the sample, i.e. during consolidation, axial strain ($\varepsilon_a$) was maintained equal to volumetric strain ($\varepsilon_v$). The consolidation stresses were applied by ramping the deviator stress slowly (at a displacement rate of 0.1 mm/min), and adjusting the cell pressure to ensure $\varepsilon_v = \varepsilon_a$. In the case of $\varepsilon_a > \varepsilon_v$, the axial displacement was stopped until $\varepsilon_a < \varepsilon_v$. After the required consolidation stress ($\sigma'_{vc}$) was reached, at least 12 hours were allowed for creep to occur. The deviator stress was then decreased slowly, and the cell pressure was adjusted to ensure $\varepsilon_v = \varepsilon_a$ until the required vertical effective stress ($\sigma'_{v} = \sigma'_{vc}/OCR$) was reached.

Conventional undrained triaxial tests, i.e. constant radial total stress ($\Delta \sigma_3 = 0$) path, were conducted on the three soils for both monotonic and cyclic shear tests. In the monotonic tests, constant axial displacement rate of 0.1 mm/min was applied, while in cyclic tests, a constant frequency of 0.1 Hz was used.

3.4.2 Testing Procedures in Simple Shear Tests

After the sample had been set up in the apparatus and the cell closed, the saturation phase was started. Using the old simple shear apparatus, a back-pressure of 200 kPa was applied with a cell pressure of 210 kPa. For the tests in the new simple shear apparatus, 400 kPa back-pressure was applied with cell pressure of 410 kPa. A B-value
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A testing programme consisting of three individual programmes for calcareous muddy silt, calcareous silt and calcareous sand was designed to investigate the fundamental behaviour through a series of undrained triaxial and simple shear tests.

In this thesis, the testing condition of Consolidated Isotropically and Undrained shearing is denoted as ‘CIU’, while the testing condition of Consolidated Anisotropically and Undrained shearing is denoted as ‘CAU’ for both triaxial tests and simple shear tests.
3.5.1 Shearing Tests on the Calcareous Muddy Silt

The shearing tests conducted on the calcareous muddy silt include undrained monotonic triaxial, undrained monotonic simple shear and undrained cyclic simple shear tests, which are summarised in Table 3-1a, b and c respectively. The testing programme can be classified into four series, each designed to examine a particular facet of the undrained behaviour of the soil under monotonic and cyclic loading conditions.

Series 1 included undrained monotonic triaxial and simple shear tests to investigate the monotonic behaviour under different testing conditions. The triaxial tests were conducted on both normally consolidated and over consolidated samples (Table 3-1a). Two CIU tests and six CAU tests are included in this testing group. The CIU tests were conducted with over consolidation ratios, OCR (= p'/p), of 1 and 2 under an initial consolidation mean effective stress, p'_c, of 150 kPa. Preliminary study showed that the effective stress ratio "at rest", K0 (= σ'ho/σ'vo), was 0.4. This K0 value was used in the consolidation phase in the CAU tests on normally consolidated samples of this soil. The over-consolidated behaviour of the soil were also investigated through CAU tests with OCR (= σ'vc/σ'v) of 2, 4 and 7 under initial consolidation vertical effective stress σ'vc of 150 kPa. The simple shear tests (see Table 3-1b) were conducted on normally consolidated samples, including one CIU test with vertical effective consolidation stress σ'vc of 125 kPa and four CAU tests with σ'vc of 100, 150, 175, and 250 kPa. The σ'ho/σ'vc ratio of 0.4 was used in the consolidation phase for the CAU simple shear tests.

Series 2 studied the influence of cyclic shear stress τcyc on undrained cyclic behaviour. Tests in this series were performed under symmetric cyclic loading (i.e. mean shear stress τm = 0). Tests from "FlocSS06" to "FlocSS20" in Table 3-1c belong to this category. A total of eleven tests were performed under CAU conditions, and a total of four tests were undertaken under CIU conditions. A range of cyclic shear stress levels with τcyc/σ'vc ratios from 0.1 to 0.25 were involved to cover the whole spectrum of the number of cycles to failure. Both CIU and CAU tests were conducted under σ'vc of 150 kPa, and the σ'ho/σ'vc ratio of 0.4 was used for CAU tests.

Series 3 investigated the effects of mean shear stress τm on cyclic behaviour through a series of tests with a biased mean shear stress (τm > 0). A total of eleven tests were conducted in this category (tests from "FlocSS21" to "FlocSS31" in Table 3-1c). These
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tests were conducted under CAU conditions with $\sigma'_{vc}$ of 150 kPa and a $\sigma'_{hc}/\sigma'_{vc}$ ratio of 0.4. The relative values of $\tau_m$ and $\tau_{cyc}$ can be classified into three groups. (1) Two-way cyclic tests with biased mean shear stress: $0 < \tau_m < \tau_{cyc}$. Four tests were conducted under $\tau_m = 0.4\tau_{cyc}$ with varying $\tau_m$ values (tests from “FlocSS21” to “FlocSS24”). (2) True 1-way cyclic loading: $\tau_m = \tau_{cyc}$, under which four tests were conducted (tests from “FlocSS25” to “FlocSS28”). (3) One-way cyclic loading with high mean shear stress $\tau_m > \tau_{cyc}$, including two tests with $\tau_m = 2.5\tau_{cyc}$ and one test with $\tau_m = 5\tau_{cyc}$ (tests from “FlocSS29” to “FlocSS31”). The combinations of cyclic shear stress and mean shear stress are shown in Figure 3-9, in which the symmetric tests ($\tau_m = 0$) discussed in Series 2 are also included.

Series 4 studied the effect of consolidation conditions (CIU or CAU) on the monotonic and cyclic behaviour of the soil from the results of the tests included in the first three series.

3.5.2 Shearing Tests on the Calcareous Silt

Undrained triaxial and simple shear tests conducted on the calcareous silt (mixture of seabed material under 150 µm, OMYA CARB and silicon oil) are summarised in Table 3-2a and b respectively. The testing programme for this soil comprised three parts, each of which was intended to investigate an aspect of undrained shear behaviour of this soil subjected to monotonic and cyclic loading.

The first part examined the undrained monotonic behaviour of the soil in triaxial and simple shear tests. The triaxial tests include three CIU tests and four CAU tests on normally consolidated samples (Table 3-2a). Two CIU compression tests were conducted under $p'_c$ of 150 and 1500 kPa, and one CIU extension test was carried out under $p'_c$ of 150 kPa. The effective stress ratio “at rest” $K_0$ was measured as 0.4 from preliminary testing, and this value was used as the consolidation stress ratio ($\sigma'_{hc}/\sigma'_{vc}$) in all CAU triaxial and simple shear tests. Three CAU compression tests were performed under $p'_c$ of 90, 180, and 270 kPa, and one CIU extension test was carried out under $p'_c$ of 90 kPa. One CAU monotonic simple shear test was undertaken under $\sigma'_{vc}$ of 150 kPa (test “OsiltSS01” in Table 3-2b).
The second part studied the influence of $\tau_{cyc}$ on undrained cyclic behaviour. A total of seven tests were performed under symmetric cyclic loading conditions ($\tau_m = 0$) with a range of cyclic shear stress $\tau_{cyc}$ (tests from "OsiltSS02" to "OsiltSS08" in Table 3-2b). The tests were performed under CAU conditions with $\sigma'_{vc}$ of 150 kPa and a $\sigma'_{hc}/\sigma'_{vc}$ ratio of 0.4.

The third part explored the effect of mean shear stress $\tau_m$ on undrained cyclic behaviour of the soil. Tests from "OsiltSS09" to "OsiltSS15" in Table 3-2b belong to this category. The tests in this series were performed under CAU conditions with $\sigma'_{vc}$ of 150 kPa and a $\sigma'_{hc}/\sigma'_{vc}$ ratio of 0.4. Three combinations of $\tau_m$ and $\tau_{cyc}$ are included in these tests: (1) two-way cyclic loading with a biased mean shear stress: $0 < \tau_m < \tau_{cyc}$, (2) true one-way cyclic loading: $\tau_m = \tau_{cyc}$, (3) one-way cyclic loading with a high mean shear stress: $\tau_m > \tau_{cyc}$.

3.5.3 Shearing Tests on the Calcareous Sand

The undrained monotonic triaxial, monotonic simple shear and cyclic simple shear tests conducted on the calcareous sand are summarised in Tables 3-3a, b and c respectively. The aim of this testing programme was to investigate undrained behaviour in the following four aspects: (1) undrained monotonic behaviour in triaxial and simple shear tests under different confining pressures; (2) effect of cyclic shear stress levels on cyclic behaviour; (3) effect of initial confining pressures on cyclic behaviour; (4) effect of consolidation conditions (CIU and/or CAU) on cyclic behaviour.

Undrained monotonic triaxial and simple shear tests were conducted under a range of confining pressures. Three CIU monotonic triaxial compression tests were conducted under $p'_c$ of 75, 500 and 1500 kPa (Table 3-3a). Two CIU and two CAU monotonic simple shear tests were performed under $\sigma'_{vc}$ of 75 and 300 kPa (Table 3-3b). A $\sigma'_{hc}/\sigma'_{vc}$ ratio of 0.4 was used for all CAU tests (monotonic and cyclic) on this soil.

A series of symmetric cyclic CIU and CAU simple shear tests were conducted under two vertical effective consolidation stresses $\sigma'_{vc}$ of 75 and 300 kPa as summarised in Table 3-3c. Four CAU (from "SandSS05" to "SandSS08") and three CIU tests (from "SandSS09" to "SandSS11") were conducted under $\sigma'_{vc}$ of 75 kPa, and Four CAU (from "SandSS12" to "SandSS15") and three CIU tests (from "SandSS16" to
"SandSS18") were carried out under $\sigma'_v c$ of 300 kPa. In each case, different $\tau_{cy/\sigma'_v c}$ ratios were involved in order to investigate the effect of cyclic shear stress level on cyclic behaviour. The effect of confining pressure and the consolidation stress ratio on the cyclic behaviour of this soil were also examined through studying the results of these tests.
4 Preparation of Reconstituted Calcareous Muddy Silt Using a Synthetic Flocculant

4.1 Introduction

As discussed in Section 3.3.1, it was intended to investigate the behaviour of the calcareous muddy silt (the aragonite soil) through triaxial and simple shear tests on reconstituted samples. However, the samples reconstituted by sedimentation showed much lower void ratios than those of undisturbed samples, resulting in different undrained shear behaviour. Therefore, it was necessary to develop an alternative method of preparing reconstituted samples, which would show the same behaviour as that of undisturbed samples.

This chapter describes a method of preparing reconstituted samples by using a synthetic flocculant. The principle of the flocculant is illustrated first, and the various stages in developing this method of sample preparation are then described. These include choosing an effective flocculant from five different synthetic polyacrylamides, comparing the shearing behaviour of reconstituted samples with undisturbed samples, and developing a curing procedure to make the behaviour of reconstituted samples as similar as possible to that of undisturbed samples. Finally, the reconstituted samples are compared with undisturbed samples in respect of their void ratios, compression behaviour, shearing behaviour and microstructure.

Most of the material in this chapter has been published by Mao & Fahey (1999), and much of the text is taken verbatim from that paper.

4.2 Material used

4.2.1 Soil

The soil is an aragonite-rich calcareous muddy silt, with about 80% of particles in the range of clay size (< 2 μm). The composition, grading curves and physical properties of the soil have been discussed in Section 3.3.1.
4.2.2 Flocculant

The flocculants used were provided by Allied Colloids (Australia) Pty Ltd, and are widely used in the mineral processing industry in Australia. The flocculants are polyacrylamides, with high molecular weights, anionic character and long-chain molecular structures, and are sold under the general trademark Magnafloc®. Specific details of the chemistry of the flocculants were not available. However, there is a considerable body of literature that describes the way in which these types of flocculants behave. A most useful review has been provided by Akers (1975). Akers distinguishes between ‘coagulation’ and ‘flocculation’, using the former term to describe aggregation of colloidal particles brought about by adding ions to reduce the thickness of the repulsive electrical double layer around the particles, whereas the latter term describes aggregation brought about by soluble high molecular weight polymers that act in some other way than charge modification. Thus, the flocculation due to the addition of positive ions to a clay suspension, as described in most soil mechanics textbooks, would be called coagulation according to this terminology.

The type of synthetic polymetric flocculants used in this study was described by Ruehrwein & Ward (1952) as producing ‘bridging flocculation’, which is schematically illustrated in Figure 4-1. In this process, the long-chain molecules of the flocculant (which may be randomly coiled) adsorb onto the surfaces of a number of soil particles, binding them together physically to form strong stable flocs. The adsorption is generally thought to be by charge effects, dispersion forces or hydrogen bonding, and the strength of the floc increases with the increase in the molecular weight (i.e. the chain length) of the flocculant (Akers 1975).

4.3 Procedures of Sample Preparation

4.3.1 Preliminary Trials Using Five Flocculants

The idea of using flocculants to produce higher void ratios was first investigated in an initial screening study. In this study, five different types of Magnafloc flocculants were used. These are designated Mg 800HP, Mg E10, Mg 338, Mg 336 and Mg 919. According to Allied Colloids, the anionic character increases in the order given in this list. The flocculants were supplied pre-mixed at a concentration of 2.5 g/l, with a recommendation being given to adjust the concentration to give a final concentration in
the slurry of 0.25 g/l for the trials. Some care was required in the mixing process, because it was found that using a high-speed ‘milkshake’ mixer resulted in severe reduction in effectiveness, probably due to breakdown in the structure of the long-chain molecules. This was evidenced by a reduction in the gel-like nature of the water-flocculant solution.

The soil was mixed as a thick slurry, and then poured slowly into a tall cylinder containing water and flocculant. After a thorough shaking, the mixture was poured into a PVC cylinder, and allowed to settle for several hours. After the mixture had fully settled, a multi-stage compression test was carried out on the sample. This involved placing a top cap with porous disk very carefully onto the surface of the settled soil, and gradually increasing the load on the top cap, allowing sufficient time for the soil to reach equilibrium before applying the next increment. The final height was determined for each stage, and from this the void ratio at each stage was determined.

One such test was carried out using each of the five flocculants. A similar test was also carried out without flocculant. The results of these tests are presented in Figure 4-2 as plots of void ratio \( e \) versus effective vertical stress \( \sigma'_v \). This figure shows that the void ratio at any effective vertical stress increases with the increasing anionic character of the flocculant.

It should be noted that these tests were carried out on samples with high height-to-diameter ratios (final values close to unity). Thus the ‘silo effect’ (the reduction in vertical stress with depth due to side-wall friction) probably results in higher void ratios than would be the case in an oedometer, where the height-to-diameter ratio tends to be much lower.

In spite of the relative crudeness of these tests, the results suggested that the technique appeared to give the increase in void ratio required. Because of the ‘silo effect’ issue mentioned above, the flocculant that gave the highest void ratio, i.e. Mg 919, was chosen for further more rigorous trials, even though in these trials it appeared to give void ratios that were higher than required.
4.3.2 Investigations using Mg 919

Having established that the void ratio could be increased by addition of the flocculant, the next step was to compare the mechanical properties of the reconstituted soil and the undisturbed samples.

Undisturbed samples to be used for triaxial or simple shear testing are generally X-rayed while still in the sample tube to ascertain the sample quality. For a description of this process, refer to Carter et al. (2000). Only those samples that showed no evidence of disturbance on the radiographs were subsequently tested. Further evidence of the quality of the sampling operation was obtained by sectioning the tube longitudinally to obtain a thick ‘slab’, which was also examined by X-radiography. This showed that disturbance could be identified only in a very thin layer along the tube wall. Hence, it is believed that the samples as received represent the in situ conditions very well.

The results of a series of triaxial and simple shear tests on the undisturbed samples were available from the testing carried out for the Gorgon site investigation. Many of the strength tests were carried out on samples consolidated anisotropically to $\sigma'_v$ of 150 kPa with a $K_0$ of 0.4. This value of $K_0$ was determined from consolidation tests conducted under ‘true $K_0$’ conditions in a triaxial cell. This involves adjusting the confining pressure during consolidation such that the volumetric strain is always equal to the vertical strain, which ensures that the radial strain is zero (on average).

Most of the undisturbed samples tested were from depths between about 18 and 23 m below the seabed. For an average submerged unit weight of about 6 kN/m$^3$, the in situ vertical effective stress would be less than 140 kPa. However, the available evidence suggests that the soil in situ may be slightly overconsolidated, and hence these samples were consolidated to $\sigma'_v \geq 150$ kPa to ensure that the results would represent normally consolidated conditions. Some of the undisturbed samples were consolidated to $\sigma'_v = 250$ kPa, with the additional 100 kPa representing the extra stress caused by a notional gravity platform.

Because of this database from the tests on the undisturbed samples, it was decided to reconstitute the soil to similar effective stress states, and carry out identical tests. The results would then be directly comparable to those from the undisturbed samples. In
this section, it is demonstrated that further refinement of the sample preparation technique was required to achieve identical behaviour.

Since an aqueous solution of a Magnafloc flocculant can be stored only for 3-5 days at normal temperatures in the absence of direct sunlight, a supply of Mg 919 in powder form was obtained from Allied Colloids for all further work. This was mixed with water at a concentration of 2.5 g/l as required, being further diluted to 0.25 g/l when added to the soil slurry. The mixing had to be carried out without severe agitation, to avoid breaking the long-chain molecules.

The soil slurry was poured slowly into the flocculant solution and mixed gently before being allowed to sediment. After some time, thick slurry settled to the bottom of the container. The liquid on top of this slurry was then removed and slurry was placed into a cylinder of 72 mm in internal diameter and 300 mm in length (a tube with a diameter of 50 mm was used in later preparation in order to save the rather scarce material, since once the material had been mixed with flocculant it could not be reused). Drainage was permitted from both top and bottom of the tube.

In this series of tests, the vertical effective stress was increased in increments up to a maximum of 150 kPa, the final target value in most of the strength tests in this series. However, in later tests (i.e. in all but the first simple shear test on the ‘cured floe soil’), the maximum applied in this stage was only 100 kPa. This means that when these samples were reconstituted in the triaxial or simple shear apparatus, the final stage of consolidation would definitely be in the normally consolidated range. It appears also that when the initial consolidation stress was equal to that applied in the test, a lower void ratio resulted than when the initial consolidation stress was lower than the final value.

The final loading increment was left in place for at least 24 hours. The sample was then removed from the tube, and set up in either the triaxial or simple shear apparatus and reconsolidated to the final target value.
4.3.3 Simple Shear Tests on the ‘Floc Soil’

For this stage, five simple shear tests (comprising two monotonic and three cyclic tests) were carried out on this soil reconstituted with flocculant. These samples are denoted as ‘floc soil’. The test details are given in Table 4-1.

The initial void ratios given in Table 4-1 are the values after consolidation in the cylinder but before transfer of the sample to the simple shear apparatus. The final void ratios given are those after reconsolidation in the testing apparatus ($\sigma'_{vc} = 150$ kPa with $K_0 = 0.4$). These values were obtained by measuring the water content and the total dry weight of material at the end of the test. For these samples, the average of the initial and final void ratios were 1.67 and 1.49, respectively. The difference between these averages illustrates the point made earlier that re-applying the same consolidation stress results in a significant reduction in void ratio. It could also be an indication of the severity of the ‘silo effect’ in the long tube in which the samples were first consolidated, which would result in the final effective stress in the lower part of the tube being less than that applied at the surface.

The corresponding values for the undisturbed samples were 1.74 and 1.61, i.e. similar to, but slightly higher than, the floc soil values. The difference between the initial and final void ratios for the undisturbed samples is a reflection of the fact that the effective vertical consolidation stress ($\sigma'_{vc}$) was slightly higher than the in situ $\sigma'_v$ value. Note also that samples reconstituted without flocculant had initial void ratios between 1.23 to 1.31, and final values (after consolidation) between 1.08 and 1.13.

The results of the tests on the floc soil are summarised in Figures 4-3 and 4-4. In Figure 4-3, the maximum shear stress obtained in the two monotonic tests is plotted against the vertical effective consolidation stress $\sigma'_{vc}$. In this (and other) plots, the maximum shear stress is designated $\tau_{peak}$, but this does not imply that any post-peak strain softening occurred (the symbol $\tau_{max}$ is used to designate the maximum shear stress in a cyclic loading test). Also shown are the results of comparable tests on undisturbed samples. These show that the reconstituted samples are slightly stronger than undisturbed samples.
Figure 4-4 shows the results of the cyclic tests, plotted as normalised cyclic shear stress (\(\tau_{cyc}/\sigma'_{vc}\)) versus the number of cycles \(N\) to reach a total shear strain \(\gamma_t\) of 15%. The results of similar tests on undisturbed samples are also illustrated. In all cases, the tests were symmetrical cyclic tests, in which the horizontal shear stress was cycled about a mean of zero, to limits of \(\pm \tau_{cyc}\). In all cases, the shear strain also increased almost symmetrically about the initial zero state. The monotonic tests are also shown in Figure 4-4, plotted on the ‘\(N = 1\)’ axis.

Though there is some scatter in the results for the undisturbed samples, there appears to be a clear differentiation between the undisturbed and floc soil, with the latter exhibiting somewhat higher cyclic strength. The significance of the difference can be appreciated by considering the horizontal separation between the trend-lines for the two sets of tests, indicating at least one, or possibly two, orders of magnitude difference in the number of cycles to failure at comparable levels of normalised shear stress.

This set of results indicates that the floc soil, while having an initial void ratio comparable to that of undisturbed samples, appears to have a somewhat higher strength, both in monotonic and cyclic loading. This higher strength was attributed to the presence of the flocculant, given the statements by Akers (1975) and others about the strength of flocs produced by high molecular weight polymetric flocculants, such as Mg 919.

### 4.3.4 Cured Floc Soil

In order to make progress in rectifying this problem, the analogy with Norwegian marine ‘quick’ clays was considered. The salt water conditions in which these clays were sedimented produced the flocculated structure of these soils. This structure remained intact when the salt was leached out of the soil with time after geostatic uplift of the land. Thus, the salt was required to form the structure, but not to maintain it. Therefore, it was surmised that in this case also, the flocculant was no longer required for maintaining the soil structure once the structure had been ‘locked into place’ by consolidation, and that removing it in some way might reduce the strength back to that of natural soil.

Various methods were therefore considered for treating the consolidated soil in order to either leach out the flocculant, or break it down, without affecting the structure of the
soil. Exposing the flocculant-water mixture to sunlight for prolonged periods was known to break down the structure, but this would not have been possible for the flocculant in the sample. However, it was also found that heating the mixture achieved the same effect, in a shorter time. Experiments with flocculant-water mixture heated to various temperatures showed that a temperature just under boiling point appeared to achieve full breakdown in a very short time. This was manifested for the mixtures by a reduction in the viscosity of the mixture back to that of water.

This technique was therefore applied to the floc soil. The sample was prepared as before. After consolidation in the tube, the sample was extruded into a larger-diameter tube (for ease of handling) and immersed in water. The temperature of the water was then gradually increased to between 80°C and 90°C using an electric immersion heater. The sample was kept in the water maintained at this temperature for at least 12 hours. The temperature of the water was then gradually decreased back to room temperature. The sample was then removed from the tube and trimmed to the correct size for simple shear tests or for triaxial tests. The samples subjected to this curing process are denoted 'cured floc soil'.

4.4 Comparison of Cured Floc Soil and Undisturbed Soil

4.4.1 Comparison of Behaviour in Compression Tests

Before carrying out any strength tests on the cured floc soil, the effect of the curing process on the consolidation behaviour was investigated by carrying out two oedometer tests on the floc soil and cured soil. The method of sample preparation was identical to that already described. After preparation, the sample was trimmed to size and mounted in the oedometer. Drainage was permitted from both top and bottom of the sample. Seven stages of loading and two stages of unloading were applied. The final height and moisture content, from which the void ratio can be calculated, were measured after the test.

The results of the tests are shown in Figure 4-5 as plots of void ratio $e$ versus $\sigma'$ (on a logarithmic scale). The results for the undisturbed samples are also shown in this figure. The range of behaviour indicated by the two undisturbed tests is greater than the range of behaviour indicated by the floc soil and cured floc soil samples. Thus, the floc
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soil and cured floc soil appear to reproduce the compression behaviour of the undisturbed material equally well.

The values of $\lambda$ and $\kappa$ (the gradients of the compression and swelling lines on $e$-$\ln \sigma'$ plots) are shown for each sample in Table 4-2. The $\lambda$ values are typical of those for other calcareous soils. For example, Carter et al. (1988) report a range of $\lambda$ values between 0.18 and 0.24 for soils from the North Rankin 'A' site on the North West Shelf of Western Australia. The high $\lambda/\kappa$ ratio (the value ranges from about 7 to 14) is also typical of calcareous soils.

4.4.2 Comparison of Behaviour in Simple Shear tests

A series of undrained simple shear tests (four monotonic and nine cyclic) was conducted on cured floc soil, with the results being compared with similar tests on undisturbed samples.

The test details are presented in Table 4-3. All samples were reconsolidated anisotropically to a $K_0$ of 0.4, and with $\sigma'_{vc}$ of 150 kPa for most tests, but with some higher values of $\sigma'_{vc}$ also used, as shown in the table. The initial void ratio values shown are the values after initial consolidation, and the final values are those measured at the end of the consolidation phase of the simple shear test, prior to the commencement of the undrained shearing phase. The average initial and final void ratio values are 1.91 and 1.55 respectively. These are higher than the corresponding values for the floc soil (1.67 and 1.49 respectively). However, the initial consolidation pressure used in the preparation of the floc soil samples was 150 kPa, while a value of 100 kPa was used for all but one of the cured floc soil samples. This partly explains the higher values. In addition, a slight expansion of the samples was observed after the curing process. To assess this, a floc soil sample was split in two prior to curing. The void ratio of one-half was determined at that stage to be 1.61. Following curing of the second half, the void ratio was determined to be 1.65, an increase of about 2.5%. The reason for this increase is not known, though as will be shown later, some change in structure occurs in the curing process.

The results of the monotonic simple shear tests on the cured floc soil are summarised in Figure 4-6, together with the results of corresponding tests on undisturbed samples. The
results are plotted as maximum shear stress $\tau_{\text{peak}}$ versus effective vertical consolidation stress $\sigma'_{\text{vc}}$. Comparing Fig 4-6 and 4-3, the curing process appears to have had some effect in reducing the monotonic strength, such that the strengths shown in Figure 4-6 are similar to those for the undisturbed samples.

All the cyclic simple shear tests were symmetric tests (i.e. cycling about a mean horizontal shear stress of zero). The results of the cyclic tests on the cured floe soil are compared with those for the undisturbed samples in Figure 4-7. The results are plotted as normalised cyclic shear stress ($\tau_{\text{cyc}}/\sigma'_{\text{vc}}$) versus number of cycles $N$ to 15% total shear strain $\gamma$. Comparing Figs 4-7 and 4-4, it is clear that the curing process has resulted in a much closer agreement with the results of the tests on the undisturbed samples. Evidently, the curing process has removed the strengthening effect of the flocculant, and hence has probably broken down the flocculant completely, with only a slight effect on the void ratio at the end of reconsolidation (the effect was a slight increase).

The monotonic simple shear behaviour of undisturbed soil, reconstituted soil without flocculant, floe soil and cured floe soil are compared in Figures 4-8 to 4-10. Figure 4-8 gives the stress paths in terms of normalised shear stress $\tau/\sigma'_{\text{vc}}$ against normalised vertical effective stress $\sigma'_{\text{vc}}/\sigma'_{\text{vc}}$, Figure 4-9 shows plots of $\tau/\sigma'_{\text{vc}}$ versus shear strain $\gamma$ for the four tests, while Figure 4-10 presents plots of normalised excess pore pressure ($Au/\sigma'_{\text{vc}}$) versus $\gamma$.

These four samples were all normally consolidated to $\sigma'_{\text{vc}}$ of 150 kPa, except for the test on reconstituted soil without flocculant, where $\sigma'_{\text{vc}}$ was 80 kPa (and also normally consolidated). The void ratio after consolidation for this sample was 1.08, compared to the values of 1.61, 1.45 and 1.68 for the undisturbed sample, floe soil sample and cured floe soil sample, respectively. Figure 4-8 shows that the reconstituted soil shows a dilative-type response, the floe soil shows a slightly dilative response, whereas the other two samples show no such tendency. Due to dilative response, the reconstituted sample exhibited a higher absolute strength than the other samples, and a correspondingly much higher normalised strength ($\tau_{\text{peak}}/\sigma'_{\text{vc}}$) as shown in Figure 4-9. It can be also observed that the floe soil presents higher normalised strength than that of the undisturbed soil because of the strengthening effect of the flocculant. In all the three plots, i.e.
normalised stress paths, stress-strain and pore pressure curves, the agreement between the undisturbed and cured floc soil is very good, indicating that all aspects of the monotonic response of the undisturbed soil are re-created in the cured floc soil.

The dilative-type response of the reconstituted sample in Figures 4-8 to 4-10 is typical of the response of many of the calcareous sands and silts from the North West Shelf, especially when reconstituted. This is often the case even when these soils are normally consolidated. Just as with silica sands, it is the initial void ratio prior to consolidation, as much as (or more than) the consolidation stress history, that dictates whether the soil will show a dilative or a contractive response. In this case, even preparation by sedimentation in water results in the initial void ratio of the reconstituted sample being significantly lower than the in situ value, and this results in a very different response.

Figure 4-11 shows the pore pressure response in a cyclic simple shear test on a cured floc soil sample, compared with a similar test on an undisturbed sample, plotted as normalised excess pore pressure versus number of cycles. The tests show that both failed at about the same number of cycles, with about the same cyclic shear stress level. Figures 4-12a and 4-12b show the shear strain development with number of cycles for the same two tests (two separate figures are required in this case, for clarity). The overall pattern is the same for the two, though the strain amplitude in the early stages is slightly greater for the cured floc soil than for the undisturbed sample.

### 4.4.3 Comparison of Behaviour in Triaxial Tests

Two undrained triaxial compression tests with mean effective consolidation stress $p'_c$ of 150 kPa and a $K_0$ of 0.4 were conducted on the cured floc soil. Details of the two tests are listed in Table 4-4. The results of the two tests are compared with the tests on undisturbed samples under the same testing conditions in Figures 4-13 and 4-14.

The stress paths of the two samples shown in Figure 4-13 show a good agreement between the cured floc soil and undisturbed soil. The stress path of the undisturbed sample is very close to that of the first test on cured floc soil, with the second test demonstrating a slightly higher value of peak strength.

The stress-strain curves of the three tests are presented in Figure 4-14. The second and third peaks observed in the test on the undisturbed sample were caused by changes in
the loading rate during the test. Good agreement is observed in this figure. All the peak strengths in the three tests occurred at $\varepsilon_a < 0.1\%$, followed by a slightly strain softening response. A slight difference between cured floc soil and undisturbed samples was observed when the axial strain was greater than 5%, after which the deviator stress in the test on the cured floc soil tended to increase. However, in the test on the undisturbed sample, a decrease in deviator stress to large axial strain was observed. This discrepancy between the two samples may be produced by the slight difference in microstructure of the two samples, which is discussed in the following section.

4.4.4 Comparison of Microstructures of ‘Cured Floc Soil’ with Undisturbed Samples

The microstructure of the various categories of material – undisturbed samples, reconstituted samples without flocculant, reconstituted ‘floc soil’ and reconstituted ‘cured floc soil’ – was examined using the Environmental Scanning Electron Microscope (ESEM) in the Centre for Microscopy and Microanalysis at UWA.

The principles of the ESEM are outlined by Danilatos (1993). In a standard SEM (Scanning Electron Microscope), high vacuum must be applied to the sample chamber, so samples must be dried completely before testing. This process would destroy the structure of soils such as that being considered here. In the ESEM, a positive pressure of a few torr (about 0.5 kPa) can be applied to the chamber. Combining this with a low temperature (between 0 and 5°C), saturated vapour pressure conditions are maintained in the chamber, preventing any evaporation from the sample under examination.

The examination was conducted on the faces exposed when the samples, in their saturated consolidated states, were broken in half. The resulting micrographs are shown in Figures 4-15 to 4-19. In each case, the scale is indicated by the white bar near the bottom of the image. The magnification is shown in the information below each image. The chamber pressure (in torr) is shown in the bottom right of each image (e.g. $P = 5.4$ T for the two images in Figure 4-15).

The first pair of images (Figure 4-15) show the reconstituted sample (without flocculant) at two different magnifications ($\times2000$ and $\times4000$). This case is presented first, since it shows the basic shape of the disaggregated individual crystals of aragonite. The packing of the crystals appears random, but relatively dense, with small voids between individual particles. In contrast, the two micrographs for undisturbed material
(Figure 4-16) show a highly aggregated structure. Individual crystals are still discernible, but they appear to be organised into distinct dense aggregated structure, with larger voids between the aggregates. There appears to be a distinct amorphous filling between the aragonite crystals in the aggregates, but this could possibly be water held by capillary attraction. The overall void ratio of the undisturbed soil is greater than about 1.7, while that of the reconstituted sample is less than about 1.3.

Two micrographs for the uncured floe soil are shown in Figure 4-17. The structure shown here is similar to that for the undisturbed soil, though the aggregation is somewhat less distinct. (The quality of the image at x4000 magnification is also poorer than that of the equivalent image in Figure 4-15).

Two micrographs for the cured floe soil are shown in Figure 4-18. These show a very well-aggregated structure, but with the individual crystals being much more clearly discernible than in the equivalent images for the undisturbed soil (Figure 4-16). The aggregates appear larger and the micro-voids are certainly larger than for the undisturbed soil.

Comparing Figures 4-16 and 4-18 in more detail, it appears that while the geometry of the structure is similar, the scales of the structure are different. This point becomes clearer when the micrograph at x500 magnification for cured floe soil (Figure 4-19) is compared with the micrograph at x2000 magnification for the undisturbed sample (Figure 4-16(a)). Now both the scale and geometry of the structures of the two samples appear similar. This suggests that the scale of the structure for the cured floe soil is about four times greater than that for the undisturbed soil, i.e. that the sizes of the aggregates and of the macropores are about four times greater for the former than for the latter.

Since the overall void ratios of the two materials are similar at all effective stresses, this difference in scale of macropores would be expected to lead to a much higher permeability for the cured floe soil than for the undisturbed soil. The coefficients of consolidation (c_v) obtained from oedometer test for the floe soil are compared with those for the undisturbed soil in Figure 4-20. It can be seen that no systematic difference is found between the values of the two soils. Therefore, this apparent
difference in scale of the structure appears to have no significance for the mechanical behaviour of the materials.

4.5 Conclusions

This chapter has described a method of preparing aragonite soil samples that produces reconstituted samples with void ratios similar to the high void ratios found to considerable depths in undisturbed samples of this soil. The method includes three major steps: mixing soil slurry with flocculant solution, consolidation in a consolidometer and curing in hot water. The mechanical behaviour (compression and shear behaviour) of the soil reconstituted using this process has been found to be almost identical to those of undisturbed samples of the soil, and the microstructures of the two samples are found to be very similar. Thus, the method can be used to produce reconstituted samples with consistent properties that are practically identical to those of the natural structured material. For simplicity, 'cured' will be omitted and samples prepared by this method will be called 'floc samples' in the following chapters.
5
Monotonic Responses of Three Calcareous Sediments

5.1 Introduction

This chapter presents the undrained monotonic behaviour of the three calcareous sediments. The behaviour is examined through effective stress paths, stress-strain and stress ratio-strain curves as well as normalised stress paths and plots of normalised stress versus shear strain. The undrained monotonic shearing behaviour of the three soils is then linked with their compression behaviour and analysed within the framework of critical state soil mechanics. In order to apply this framework, ideally, hydrostatic compression tests are required. However, in this study, one-dimensional compression tests were used instead to obtain the compression behaviour. The compression behaviour of the three soils is also discussed in this chapter.

5.2 Terminology Relating to Stress-Strain Behaviour and Stress Paths

Some concepts used in describing shearing behaviour in the thesis are explained here. The stress-strain behaviour of a soil in either drained or undrained monotonic tests can be classified as ‘strain hardening’ or ‘strain softening’. The former implies that extra shear stresses are required to achieve further strain, while the latter implies that shear stresses decrease with increasing strain. Generally, the terms ‘strain hardening’ and ‘strain softening’ can be used to describe the responses of changes in general physical measurements associated with strain, e.g. responses of stress-strain, stress ratio-strain, moment-strain, etc. In this thesis, the terms ‘strain hardening’ and ‘strain softening’ are used to describe both stress-strain and stress ratio-strain responses.

In the description of monotonic shearing behaviour of soils, two terms: ‘dilative’ and ‘contractive’ (or ‘dilation’ and ‘contraction’), are widely used associated with volumetric changes in drained shear tests or pore pressure (or mean effective stress) responses in undrained shear tests. In describing soil behaviour in the drained shear tests, there is no confusion about the two terms; ‘dilation’ and ‘contraction’ indicate respectively the increase and decrease of volume during drained pure shearing (i.e. no mean effective stress change). However, the two concepts are less clear relating to
undrained shear behaviour in the literature. Chern (1985) defines ‘contractive’ as “a behaviour associated with loss of shear resistance after the occurrence of a peak” and ‘dilative’ as “strain hardening behaviour with no loss of shear resistance” as shown in Figure 5-1a. It is obvious that he used the terms dilative and contractive to describe respectively strain hardening and strain softening behaviour in undrained shear tests. Hyodo et al. (1998) defined the response of stress paths (in q-p' space) moving towards the right (increase of mean effective stress) as ‘dilative’ behaviour and towards the left as ‘contractive’ behaviour (see Figure 5-1b). Regarding these two concepts, Been et al. (1991) stated that “in undrained tests the volumetric strain is zero, and dilation and contraction are used in a loose manner to describe negative and positive rates of pore pressure change”. The confusion arises because of the increase in mean effective stress that occurs in a standard triaxial test. If triaxial tests were always performed with a vertical total stress path in q-p space, this confusion would not arise. In this thesis, dilative and contractive behaviour will be defined as negative and positive rates of pore pressure change, respectively, as used by Been et al.

5.3 Behaviour of the Calcareous Muddy Silt

The behaviour of the calcareous muddy silt was investigated through a series of triaxial and simple shear tests on undisturbed samples and samples reconstituted using the flocculation procedure (floc samples) described in detail in Chapter 4, in which it has been demonstrated that the floc samples are able to reproduce very well the behaviour of the undisturbed samples. The series of triaxial and simple shear tests on the undisturbed samples of the calcareous muddy silt has been carried out in the Geomechanics group at UWA, associated with a preliminary study for a platform project (the author conducted all of the simple shear testing for this project). The testing programme on the floc samples was aimed at investigating aspects not covered by the tests on the undisturbed samples. The test results discussed in this section include those of both the undisturbed samples and the floc samples.

5.3.1 Behaviour in Undrained Triaxial Tests

(a) Behaviour of Anisotropically Consolidated Samples

Table 5-1 summarises the undrained CAU monotonic triaxial tests on the undisturbed samples of the calcareous muddy silt normally consolidated under a range of mean
effective stress $p'_c$ values and a consolidation stress ratio, $\sigma'_{hc}/\sigma'_{vc}$, of 0.4. The tests include six compression tests with $p'_c$ of 90, 108, 126, 150, 186 and 231 kPa and two extension tests with $p'_c$ of 48 and 90 kPa. All these tests were conducted using the same procedures as described in Section 3.4.1.

The results of the CAU monotonic triaxial tests on undisturbed samples are presented in Figure 5-2. The effective stress paths (ESP) of these tests are plotted in Figure 5-2a, and a total stress path (TSP) is also shown for one test. It can be observed that critical state points for the compression tests are scattered with the friction parameter $M$ (which is the gradient of critical state line in q-$p'$ space) ranging from 1.50 to 1.94. The friction angle $\phi'_{cs}$ corresponding to the average value of $M$ (1.72) is 42°. The values of $M$ and $\phi'_{cs}$ corresponding to the extension phase are -1.18 and 47° respectively.

The ESPs of these compression tests lie between the consolidation line and the critical state line (CSL). It can be observed that the ESPs of compression tests are separated into two ranges by a line that links the peaks of all compression tests, radiating from the origin of the q-$p'$ space. This line is named the peak strength line (PSL) in this thesis. In the first range (between the consolidation line and the PSL), both mean effective stress $p'$ and deviator stress $q$ increased with the process of shearing until the PSL was reached. The gap between the TSP and the ESP of the test with the highest confining pressure shown in Figure 5-2a indicates that excess pore pressure is still induced in this range. After the stress path crosses the PSL, both $q$ and $p'$ decrease until the CSL is reached. The reduction in shear resistance (i.e. decrease of $q$) indicates that a strain softening response occurred.

This strain softening behaviour can be observed clearly in the stress-strain curves shown in Figure 5-2b. For each compression test, the peak strength was reached quickly at axial strains $\varepsilon_a \leq 0.1\%$. After reaching the peak strength, the deviator stress decreased rapidly with the development of axial strain.

The strain softening and contractive behaviour in stress-strain curves and stress paths shown in Figures 5-2a and b are very like those of reconstituted Magnus clay (Burland 1990) and reconstituted Boston Blue clay (Sheahan 1991). However, the friction angle mobilised at critical state for calcareous muddy silt (42°) is much higher than that for
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Magnus clay ($\phi'_{es} = 30^\circ$) and for Boston Blue clay ($\phi'_{es} = 33.5^\circ$). This is the cause of angularity of aragonite particles of the calcareous muddy silt. The peak strength of the calcareous muddy silt from undrained triaxial compression tests (in terms of $S_{uTc}/\sigma'_{vc}$ ratio) is in the range of 0.36-0.39, which is higher that those for reconstituted Magnus clay (0.31) reported by Burland (1990) and reconstituted Boston Blue clay (0.32) reported by Sheahan (1991).

Figure 5-2c presents the plots of stress ratio ($q/p'$) with $\varepsilon_a$. In these plots, there is neither peak strength nor strain softening behaviour observed. Smooth curves with strain hardening behaviour are observed. The curves of $q/p'-\varepsilon_a$ fall into the hyperbolic-type of relationship.

The normalised stress paths ($q/p'_c-p'/p'_c$) and plots of normalised stress ($q/p'_c$) versus axial strain ($\varepsilon_a$) of the six undrained triaxial compression tests are presented in Figure 5-3. The normalised stress paths of the six compression tests in Figure 5-3a are almost coincident especially before the peak strength is reached. The $q/p'_c-\varepsilon_a$ curves of the six compression tests (Figure 5-3b) converge together in the initial stage but scatter in the intermediate and final stages of these tests. The scattered data reflect the fact that the undisturbed samples were recovered from different depths below the seabed (see Table 5-1) and therefore these samples were not identical with respect to their "fabric". Overall, it can be said that when stresses are normalised by $p'_c$, the stress paths and stress-strain curves of the calcareous muddy silt in undrained triaxial tests are very similar. The behaviour is described as being "normalised behaviour" when similar shearing responses (i.e. stress paths, stress-strain curves, pore pressure-strain curves, etc.) are obtained from different tests when stresses are normalised by the consolidation stress $p'_c$.

The over-consolidated behaviour was investigated through CAU triaxial tests on the floc samples of the calcareous muddy silt initially consolidated to $\sigma'_{vc}$ of 150 kPa under one-dimensional compression, followed by one-dimension swelling to the required vertical effective stress ($\sigma'_v = \sigma'_{vc}/OCR$) as described in Section 3.4.1. Figure 5-4 presents the results of four tests on floc samples with over-consolidation ratios (OCR) of 1, 2, 4 and 7. The effective stress paths (ESP) of these tests are shown in Figure 5-4a. The stress path of the normally consolidated sample presents contracted behaviour,
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while stress paths of over-consolidated samples show dilative behaviour. It can be observed that the higher the OCR values the stronger the dilation that the soil presents. All ESPs move towards the CSL. The two friction parameters $M$ and $\phi'_{cs}$ are 1.86 and 45° respectively, which is in the CSL range of CAU triaxial tests in shown Figure 5-2a.

The shapes of the stress-strain ($q$-$\varepsilon_a$) curves of the four tests shown in Figure 5-4b are quite different. The stress-strain curve for OCR = 1 shows an obvious initial peak strength at $\varepsilon_a$ of about 0.1 % followed by strain softening. The stress-strain curve for OCR = 2 presents an indistinct initial peak strength at $\varepsilon_a$ of about 0.4 % followed by a slight strain softening. The stress-strain curves for OCR = 4 and 7 show no initial peak strength. It can be seen that initial stiffness ($q/\varepsilon_a$) decreases with increase in OCR values. The four stress-strain curves converge together at an axial strain $\varepsilon_a$ of about 10 %. The ESPs and $q$-$\varepsilon_a$ responses of this soil under different OCR shown in Figure 5-4 are very similar to those of other clays under similar conditions, e.g. Boston Blue Clay (Fayad 1986, Sheahan 1991), and Lower Cromer Till (a low plasticity sandy clay reported by Gens 1982).

(b) Behaviour of Isotropically Consolidated Samples

The behaviour of isotropically consolidated calcareous muddy silt can be viewed through the results of two CIU triaxial compression tests on the floe samples with OCR of 1 and 2 shown in Figure 5-5. The ESP of the test on a normally consolidated floe sample (Figure 5-5a) shows no initial peak strength, and the mean effective stress $p'$ decreases after the start of shearing and moves towards the critical state line. Slightly dilative behaviour is observed in the final stage of the test. The ESP of the over-consolidation test (OCR = 2) moves straight up to critical state line. The stress-strain curves of the two tests tend to converge after axial strain equal to 5 %. The behaviour of the calcareous muddy silt in the CIU triaxial test on a normally consolidated sample is different from that in the CAU tests on normally consolidated samples in that a strain hardening response is observed in the CIU test.

5.3.2 Behaviour in Undrained Simple Shear Tests

Four CAU monotonic simple shear tests were conducted on the floe samples normally consolidated under vertical effective stress $\sigma'_{vc}$ of 100, 150, 175 and 250 kPa and
The results of the four tests are presented in Figure 5-6. As has been discussed in Chapter 3, in undrained simple shear tests constant sample height is maintained by locking the loading ram, and at the same time, vertical total stress $\sigma_v$ is maintained constant by increasing the cell pressure during shearing. Typical responses of total vertical and horizontal stresses ($\sigma_v, \sigma_h$) as well as effective vertical and horizontal stresses ($\sigma'_v, \sigma'_h$) in this type of simple shear tests can be observed in Figure 5-6a, which shows plots of $\sigma_v\gamma, \sigma_h\gamma, \sigma'_v\gamma$, and $\sigma'_h\gamma$ for a test with $\sigma'_v\gamma$ of 150 kPa. It can be seen that $\sigma_v$ is maintained constant while $\sigma_h$ increases quickly and eventually equals $\sigma_v$, and $\sigma'_v$ and $\sigma'_h$ also tend to equalise at the end of shearing.

The stress paths in $\tau-\sigma'_v$ space for the four simple shear tests are shown in Figure 5-6b. It can be seen that $\sigma'_v$ decreased during shearing until the critical state line was reached. The shapes of stress paths of the four tests are similar with sizes increasing with the increase in $\sigma'_v\gamma$. The stress-strain ($\tau-\gamma$) and shear stress ratio $t/\sigma'_v$ versus shear strain $\gamma$ responses in these tests are plotted in Figures 5-6c and d respectively. The $\tau-\gamma$ curves did not show the type of softening response that was observed in the CAU monotonic triaxial tests. Shear stresses increased quickly within an initial 2% shear strain. Then the rate of increase decreased in the range of $\gamma = 2-5\%$. After reaching 5% shear strain, little further increase in shear stress occurred. The $t/\sigma'_v-\gamma$ curves show strain hardening behaviour and can be represented by a hyperbolic-type relationship.

In Figure 5-7, stress paths and stress-strain curves of the four tests are plotted with shear stress $t$ and effective vertical stress $\sigma'_v$, normalised by consolidation effective vertical stress $\sigma'_v\gamma$. The normalised stress paths and $t/\sigma'_v\gamma$ curves of the four tests move together with relatively little scatter observed. Overall, when stresses are normalised by $\sigma'_v\gamma$, the stress paths and stress-strain curves of the soil in undrained simple shear tests are very similar, suggesting that the soil (in a normally consolidated state) exhibits normalised behaviour in undrained simple shear tests.

The effective stress paths of the four simple shear tests are plotted in terms of $q-p'$ space and compared with those of the triaxial tests in Figure 5-8. The calculation of $q$ and $p'$ for the simple shear tests is based on the assumption that there is a complementary shear
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stress on the vertical planes of the simple shear sample, so that the principal normal effective stresses ($\sigma'_1$ and $\sigma'_3$) can be obtained from the following equations:

$$\sigma'_i = \frac{\sigma'_v + \sigma'_h}{2} + \sqrt{\left(\frac{\sigma'_v - \sigma'_h}{2}\right)^2 + \tau^2}$$

$$\sigma'_3 = \frac{\sigma'_v + \sigma'_h}{2} - \sqrt{\left(\frac{\sigma'_v - \sigma'_h}{2}\right)^2 + \tau^2}$$

where $\sigma'_v$ and $\sigma'_h$ are vertical and horizontal effective stresses respectively, and $\tau$ is the horizontal shear stress.

Although the ESPs of both simple shear and triaxial tests start from the same consolidation line, the ESP experienced in simple shear tests are quite different from those in triaxial tests. However, if it is assumed that critical state was reached at the end of both triaxial and simple shear tests, the critical state obtained from the simple shear tests approached the critical state range determined from the triaxial tests, indicating that critical state is an intrinsic parameter for a soil regardless of the type of shearing mode.

5.4 Behaviour of the Calcareous Silt

5.4.1 Behaviour in Undrained Triaxial Tests

A total of seven undrained monotonic triaxial tests, consisting of three CIU tests and four CAU tests, have been conducted on the calcareous silt normally consolidated under a range of consolidation stresses. The three CIU tests include two compression tests with consolidation mean stresses $p'_c$ of 150 and 1500 kPa and one extension test with $p'_c$ of 150 kPa. The four CAU tests include three compression tests with $p'_c$ of 90, 180 and 270 kPa and one extension test with $p'_c$ of 90 kPa. The consolidation stress ratio used in the CAU tests is 0.4.

The stress paths of the seven tests are presented in Figure 5-9. It can be seen that the stress paths of the CIU compression tests are separated into contraction and dilation ranges by the phase transformation (PT) state line, which was initially defined by Ishihara et al. (1975) as a state for sand at which the behaviour changes from contractive to dilative. In the contractive range, the mean effective stress $p'$ decreases as a
consequence of shearing until the PT state is reached. There is no peak deviator stress found in this range. The stress paths of the CAU compression tests consist of three ranges separated by initial peak strength line (IPSL) and the PT state line. In the first range, the mean effective stress $p'$ increases slightly with the increase of the deviator stress $q$. Very little shear strain ($\varepsilon_a < 0.1\%$) is induced in this range. After the initial peak deviator stress, both $p'$ and $q$ start to decrease until the PT state is reached. Contraction and strain softening response occur in this range. Having achieved the PT state, both the mean effective stress $p'$ and the deviator stress $q$ increase until failure is reached. This behaviour is very similar to that of anisotropically consolidated dense quartzitic silt in undrained triaxial compression tests reported by Zdravković & Jardine (2000).

The ESPs of the CIU tests are quite different from those of the CAU tests in that there is no initial peak strength and no strain softening response in the ESPs of the CIU tests. However, a unique PT state line is observed for both the CIU and the CAU compression tests, and a unique PT state line is also observed in the extension side for all tests. In the extension tests, a second transformation phase can be seen where the behaviour becomes contractive again, and this is believed to be caused by necking of the samples, which has been observed in the extension tests. If the stresses at the end of shearing ($\varepsilon_a > 20\%$) are assumed to be close to the critical state, the critical state line is found to be almost coincident with the PT state line. The friction parameters $M$ and $\phi'_{cs}$ are 1.67 and $41^\circ$ corresponding to compression phase, and are $-0.75$ and $25^\circ$ corresponding to extension phase.

The $q$-$\varepsilon_a$ curves of the CIU compression tests (Figure 5-10a) show strain hardening behaviour. The PT state, represented by solid dots in these plots, is reached at axial strains $\varepsilon_a$ of 2% to 4% for all three tests. Although the stress-strain responses of the two CIU compression tests differ, their stress ratio-axial strain ($q/p'$-$\varepsilon_a$) curves (plotted in Figure 5-10b) are similar, and very slight peaks are observed in the $q/p'$-$\varepsilon_a$ curves.

The $q$-$\varepsilon_a$ curves of the CAU compression tests shown in Figure 5-11a present different responses from those of the CIU compression tests (Figure 5-10a). The initial peak strengths were reached within axial strain $\varepsilon_a = 0.1\%$, and subsequently, all three tests showed strain softening until the PT state was reached. The axial strain $\varepsilon_a$
corresponding to the PT state for three tests was between 2.5 % and 4 %, after which the deviator stress $q$ increased with increasing axial strain, indicating that strain hardening occurred. Figure 5-11b shows the $q/p' - \varepsilon_a$ curves for the three CAU tests, in which strain hardening behaviour is observed for axial strains below 8 %, after which strain softening response was observed.

The normalised stress paths and the plots of normalised deviator stress versus axial strain ($q/p'_c - \varepsilon_a$) of the two CIU compression tests are plotted in Figure 5-12. Strong dilative behaviour is found in the test with lower confining pressure ($p'_c = 150$ kPa), while relatively weak dilative behaviour is associated with the test under higher confining pressure ($p'_c = 1500$ kPa), indicating that the normalised responses of the soil in CIU triaxial tests are different. The normalised stress path and $q/p'_c - \varepsilon_a$ curves of the three CAU compression tests are presented in Figure 5-13. The stress paths and stress-strain curves show narrow bands for the three CAU test results, suggesting that in the narrow range of confining stress ($p'_c = 90 - 270$ kPa in this case), the normalised responses of the soil are similar.

5.4.2 Behaviour in Undrained Simple Shear Tests

The effective stress path, stress-strain and stress ratio-shear strain curves from an undrained monotonic simple shear test on the calcareous silt consolidated under vertical effective stress $\sigma'_vc$ of 150 kPa and $\sigma'_ho/\sigma'_vc = 0.4$ are illustrated in Figures 5-14a, b and c respectively.

The effective stress path in $\tau-\sigma'_v$ space consists of contractive and dilative ranges, separated by the PT state line. The stress-strain ($\tau-\gamma$) curve presents strain hardening behaviour, and the PT state (represented by a solid dot) in the $\tau-\gamma$ curve was reached at shear strain $\gamma$ of about 5 %. A hyperbolic-type relationship can be observed in the stress ratio-shear strain curve ($\tau/\sigma'_v-\gamma$).

Figure 5-15 compares the simple shear test results with those of triaxial tests with respect to stress paths in $q-p'$ space. Since the same consolidation stress ratio ($\sigma'_ho/\sigma'_vc = 0.4$) was used for both triaxial and simple shear tests, the effective stress paths of triaxial and simple shear tests started from the same consolidation line, but from then
on, they follow quite different paths. The PT state line is observed to be unique for these stress paths, indicating that the PT state line is independent of the shearing mode.

### 5.5 Behaviour of the Calcareous Sand

#### 5.5.1 Behaviour in Undrained Triaxial Tests

Three CIU monotonic triaxial tests were conducted on the calcareous sand normally consolidated under three mean effective stresses $p'_c$ of 75, 500 and 1500 kPa, details of which have been given in Table 3-3a.

The effective stress paths (ESPs) of the three tests and the total stress path (TSP) of the test with $p'_c = 1500$ kPa are shown in Figure 5-16a. The ESPs of the three tests consist of two ranges, i.e. contractive and dilative, separated by the PT state line. If the states at the end of tests ($\varepsilon_a > 15\%$) are assumed to be the critical state, the critical state line is found to be coincident with the PT state line, and the friction parameters $M$ and $\phi'_c$ in the compression phase are 1.61 and 39.4° respectively.

It can be also observed that the mean effective stress $p'$ increased to some extent in the initial stage of the three tests. The gap between the TSP and ESP for the test with $p'_c = 1500$ kPa indicated that there were excess pore pressures generated in this range. This behaviour is not observed for most other sands, e.g. tailings sand and Ottawa sand (Chern 1985), Toyoura sand (Ishihara 1993), Dog's Bay sand (Coop 1990), and Shirasu sand (Hyodo et al. 1998). Such behaviour may be caused by the soil fabric structure created by the shape of the particles: in this case, most of them are round flat disc-like shells (ref. Figure 3-7). During air pluviating used in preparing triaxial samples for this soil (see Section 3.3.3), it is quite possible that soil particles (round flat disc-like shells) settled with flat side in horizontal so that an anisotropic particle matrix structure might be formed.

The $q_{-\varepsilon_a}$ curves of the three tests shown in Figure 5-16b present strain hardening behaviour. It can be seen that as confining pressures increase, the axial strains corresponding to the PT state (represented by solid dots) increase. The $q/p'_-\varepsilon_a$ curves shown in Figure 5-16c reveal hyperbolic-type relationships similar to those of the calcareous muddy silt and silt discussed previously. The normalised stiffness of this soil decreases as the confining stress $p'_c$ increases from 75 to 1500 kPa indicating the
usual effect of the confining pressure on the shear behaviour of sand. The greater the confining pressure, the lower the position of the $q/p'_c-\varepsilon_a$ curves.

The normalised stress paths and $q/p'_c-\varepsilon_a$ curves are presented in Figures 5-17a and 5-17b, in which a strong effect of stress level may be observed. It can be seen that as the confining pressures increase, the sand shows less dilative behaviour. This behaviour is consistent with those of Toyoura sand (silica sand reported by Ishihara 1993), a calcareous sand (Ismail 2000), and the calcareous silt discussed above. However, Hyodo et al. (1998) observed the opposite response for very loose Shirasu sand (a crushable volcanic soil) subjected to CIU monotonic triaxial tests with $p'_c$ of 50, 100 and 300 kPa, although the density (or void ratio) of each sample after consolidation was not mentioned in the paper. The behaviour of Shirasu sand may be caused by the crushable nature of this soil. The density (or void ratio) at the start of shearing might have been very different, i.e. an initially very loose sample may be still very loose at low $p'_c$, but may be very dense at high $p'_c$ after isotropic compression.

5.5.2 Behaviour in Undrained Simple Shear Tests

Four undrained monotonic simple shear tests including two CIU and two CAU tests have been carried out on the calcareous sand. The two CIU tests were conducted under consolidation mean effective stress $p'_c$ of 75 and 300 kPa, whilst the two CAU tests were conducted under consolidation vertical stress $\sigma'_{vc}$ of 75 and 300 kPa and a consolidation stress ratio of 0.4. The results of these tests are presented in Figures 5-18 to 5-23.

Figure 5-18 shows the effective stress paths ($\tau-\sigma'_v$ space) of the four simple shear tests. A unique PT state line is found for the four tests, and the overall responses of the four tests are very similar. Each stress path consists of contractive and dilative ranges separated by the PT state line. The stress paths of the two CAU tests are noted as being below those of the two CIU tests. This may be due to the slightly higher void ratios found in the two CAU tests compared to those of the CIU tests at the same $\sigma'_{vc}$, which is a result of the lower mean effective stress ($p'$) applied in the CAU tests than in the CIU tests at equivalent stress levels ($\sigma'_{vc}$) (referring to Table 3-3b).
The τ-γ curves for the CIU and the CAU tests are presented in Figures 5-19a and 5-20a respectively. Again the τ-γ responses for the CIU and the CAU tests are similar, and strain hardening behaviour is observed in these plots. The shear strains corresponding to the PT state (represented by solid dots) for these tests are between 2 and 4.5 % and are found to increase with the increase of confining pressure. The stress ratio-shear strain (τ/σ'γ) behaviour shown in Figures 5-19b and 5-20b present hyperbolic-type relationships for both CIU and CAU tests. It is noted that the τ/σ'γ curves for tests with the lower consolidation stress σ'vc of 75 kPa lie above those for the tests with σ'vc of 300 kPa. This observation is similar to the q/p'-εa response observed in Figure 5-16c for the same soil in triaxial tests as discussed above.

Figures 5-21 and 5-22 show normalised responses for the CIU and the CAU tests respectively. The normalised stress paths within the contractive range of the two CIU tests are almost coincident (Figure 5-21a). In the dilative range, the normalised stress paths are still identical but the extent of the stress paths is different. The τ/σ'vc-γ curves of the two CIU tests (Figure 5-21b) before reaching the PT state points are similar with the PT state points being reached at different shear strains. After the PT state has been passed, the τ/σ'vc-γ curves of the two CIU tests diverge from each other. Similar observations were also obtained for the CAU tests shown in Figures 5-22a and b.

The effective stress paths of the four simple shear tests are plotted in q-p' space in Figure 5-23, together with the effective stress paths of two triaxial tests with confining pressures p'_c of 75 and 500 kPa. The triaxial test with p'_c of 1500 kPa is not plotted in this figure. The figure reveals that in the q-p' space the CIU simple shear and the CIU triaxial tests experience similar stress paths, while CAU simple shear tests undergo different stress paths. A unique phase transformation line is obtained for all these tests.

5.6 Characteristics of Undrained Monotonic Behaviour

It is widely known that the undrained shear response of an uncemented soil depends primarily on the initial soil state: void ratio, confining pressure and consolidation stress ratio (σ'hc/σ'vc). Stress-induced anisotropy ("fabric") can also have an effect for K₀-consolidated samples, and this is also likely to be true in this case. Nevertheless, this
“fabric” is ignored in this section, and the responses of the three calcareous sediments are analysed through the framework of critical state soil mechanics.

5.6.1 Critical State Soil Mechanics Framework for Clays and Sands

The important contribution of critical state soil mechanics is the correlation of shear behaviour of soils to their compression behaviour. The compression and shearing behaviour of clays was first linked by the pioneering work of Roscoe et al. (1958) and Schofield & Wroth (1968). This made it possible to estimate shearing behaviour through soil states in e-log $p'$ space. The compression behaviour of sands, depending on both void ratios and confining pressures, makes the shearing behaviour of normally consolidated sands significantly different from that of normally consolidated clays. Been and Jefferies (1985) simplified the description of sand behaviour through unifying the effect of void ratios and confining pressures with a state parameter $\psi$, which is defined on a plot of $e$ versus log $p'$ as the vertical distance between the current state and the critical state (steady state) line ($\psi = e_0 - e_{cs}$) as shown in Figure 5-24.

It is well known that most sands behave much like a clay at high OCR in that they tend to dilate under undrained shearing, although some sand deposits may be sufficiently loose to be contractive under undrained shearing like a clay at low OCR. Been et al. (1988) presented a direct relationship between the OCR and the state parameter ($\psi$) as follows:

$$\log(OCR) = \log 2^{1-\kappa} + \psi(\kappa - \lambda)$$

$$OCR = \frac{p_{\text{max}}'}{p_0'}$$

5-2

where $\lambda$ and $\kappa$ are respectively compression and swelling indices in e-log $p'$ space, $p_0'$ and $p_{\text{max}}'$ are respectively current stress and maximum compression stress experienced (Figure 5-24). Therefore, the state parameter for sand will play a similar role as OCR in describing the character of clay response under given loading conditions. Hence, responses of clays and sands are related through the state parameter $\psi$ in the framework of critical state soil mechanics. Contractive behaviour is expected when the state parameter $\psi >> 0$. Strong dilative behaviour is anticipated, when $\psi < 0$, and when $\psi$ is slightly positive, dilative behaviour is still likely in undrained shear. If the normal
consolidation line is parallel to the critical state line (i.e. \( \psi \) is constant in the whole range of stress), the soil will show normalised behaviour (i.e. when stresses are normalised by consolidation stress level, shearing responses are almost identical, as seen in shearing tests on normally consolidated clays). However, if \( \psi \) is not constant in the range of stress, the normalised responses of the soil will be different (i.e. when stresses are normalised by consolidation stress level, the stress paths and stress-strain curves will be different, as observed in shearing tests on sands).

Stress induced anisotropic behaviour is another important aspect of soil behaviour, which can not be expressed by use of the state parameter. The previously discussed stress-strain behaviour of the muddy silt and silt shows that strain hardening behaviour was observed for the isotropically consolidated samples as shown in Figures 5-5b and 5-10a, while strain softening was observed for anisotropically consolidated samples \( (\sigma'_{hc}/\sigma'_{vc} = 0.4) \) as shown in Figures 5-2b and 5-11a. Several researchers have reported the effect of stress anisotropy on the undrained behaviour of clays and sands. Ladd & Varallyay (1965) showed that Boston Blue Clay presented strain hardening (in stress-strain curves) in CIU triaxial tests and strain softening after a peak strength in CAU triaxial tests. Strain softening behaviour in CAU triaxial tests has also been presented by Burland (1990) for both natural and reconstituted soft clays. The effects of consolidation stress ratio on the undrained monotonic behaviour of two siliceous sands reported by Chern (1985) showed that the sand behaviour changed from strain hardening to strain softening as the consolidation stress ratios changed from 1 (CIU) to 0.5 (CAU). Vaid & Eliadorani (1998) observed the same behaviour in undrained triaxial tests on a siliceous sand. The results of two calcareous sands (of high angularity) subjected to undrained triaxial tests, presented by Ismail (2000), revealed the same effect of the consolidation stress ratio \( \sigma'_{hc}/\sigma'_{vc} \) on the undrained monotonic behaviour. Therefore, the stress anisotropy (i.e. \( \sigma'_{hc}/\sigma'_{vc} \)) is another parameter that plays an important role in determining soil behaviour in undrained triaxial tests.

The undrained monotonic behaviour of soils can be qualitatively described by the state parameter (\( \psi \)) and consolidation stress ratio \( (\sigma'_{hc}/\sigma'_{vc}) \). The state parameter governs the soil shear behaviour with respect to the degree of dilation and contraction in effective stress paths. Whether or not a soil presents normalised behaviour is governed by the state parameter. The effect of stress anisotropy on shear behaviour is reflected by the
consolidation stress ratio \((\sigma'_h/\sigma'_c)\). The stronger the initial stress anisotropy (or the smaller the \(\sigma'_h/\sigma'_c\) ratio), the greater is the possibility of strain softening occurring. The undrained shearing behaviour of a soil is governed by both \(\psi\) and \(\sigma'_h/\sigma'_c\), which represent soil state in \(q-p'-e\) space.

5.6.2 Characteristics of Undrained Monotonic Shear Behaviour for the Three Calcareous Soils

In order to link the shearing behaviour of the three soils to their compression behaviour, the compression curves of the three soils are plotted together with their critical state points obtained from shear tests in Figures 5-25, 5-26 and 5-27. In doing this, the following two assumptions have to be made.

1. The term ‘critical state’ (or ‘steady state’ as defined by Poulos 1981) is defined as the state at which the soil ‘continues to deform at constant stress and constant void ratio’ (Roscoe et al. 1958). In undrained monotonic triaxial or simple shear tests, it is usually easy to reach the critical state for contractive soils (e.g. clays and loose sands). For dilative soils (e.g. dense sands), however, it is difficult to measure the critical state conditions in undrained monotonic shear tests. In this case, it is assumed that the critical state is reached at the end of any test in which more than 20 % (or 15 % in two cases) axial strain (for triaxial tests) or shear strain (for simple shear tests) has been attained. Although this assumption might be not true for dilative soils (e.g. silt and sand), the state at the end of each shearing test should be close to the true critical state. Thus, the critical state points shown in the three figures are the data at the end of each shearing (triaxial or simple shear) test.

2. The compression tests conducted on the three soils were either oedometer tests or one-dimensional constant strain rate tests performed in a rigid-walled (oedometer-type) device, in which the lateral stress could not be measured. The results of these compression tests were available in \(e-log \sigma'_v\) space. The critical state points obtained from shear tests are in the space of \(q-p'-e\). In order to compare the critical state with compression behaviour, the compression curves should be plotted in \(e-log p'\) space. It has been observed that the isotropic compression and one-dimensional compression curves of a soil are qualitatively similar. There is evidence that for calcareous sand (Coop 1990) and non-calcareous sand (Hendron 1963), the
Monotonic Responses of Three Calcareous Sediments

A compression curve from a one-dimensional compression test on a soil is approximately parallel to that from an isotropic compression test for the same soil in a high pressure range. Therefore, for a qualitative comparison of critical state and normal compression curves, transformation from e-log $\sigma'_v$ to e-log $p'$ by assuming a constant $K_0$ is reasonable at least for the normal consolidation stage.

For the calcareous muddy silt and calcareous silt the measured lateral stress ratios at rest are 0.4 and 0.45 respectively. For the calcareous sand, however, no measured value is available. An empirical equation $K_{ONC} = 1 - \sin\phi'_c$ (Jaky 1944, Mesri & Hayat 1993) is widely used for describing the relationship between $K_{ONC}$ and the internal friction angle at constant volume ($\phi'_c$). Sutherland (1999) showed that this relationship is applicable for a silica sand and three calcareous sands at low compression stresses ($\sigma'_v < 850$ kPa), and a narrow range of $K_{ONC}$ values (0.32-0.34) was obtained for three calcareous sands. However, Coop & Lee (1993) reported one-dimensional compression data that shows that $K_{ONC} = 0.51$ and 0.57 at elevated stress level for Dog's Bay sand (a calcareous sand) and Ham River Sand (a silica sand) respectively. In this case, a value for $K_{ONC}$ is required for relatively high stress levels, hence, $K_{ONC}$ is chosen as 0.49 for this calcareous sand.

(a) The Calcareous Muddy Silt

The results of three oedometer tests on the calcareous muddy silt (two of them on undisturbed samples and one on a floc sample) are presented in Figure 5-25. The compression curves of the calcareous muddy silt shown in Figure 5-25 reveal that a straight normal compression line (NCL) is obtained in $e$-ln $p'$ space, with an average $\lambda$ value of 0.20. All shear tests shown in this figure were conducted on normally consolidated samples. The critical state points obtained from undrained triaxial and simple shear tests plotted in this figure show a scattered pattern. However, These points form a band which is below the NCL, suggesting that the initial values of state parameter $\psi$ for all shearing tests on normally consolidated samples were positive. The soil should therefore show contractive behaviour, which is in agreement with that observed in Figures 5-2a and 5-6.
(b) The Calcareous Sand

It is widely known that, except for soil intrinsic properties (such as grain size distribution, mineralogy, grain shape, mineral to mineral friction angle etc.), the compression behaviour of freshly prepared sand samples is affected by the formation void ratio, and all the compression curves for samples at different formation void ratios converge to a unique void ratio-effective stress line, which has been called a Limiting Compression Curve (LCC) by Pestana and Whittle (1995). Therefore, unlike a clay, the normal compression curves of a sand are a group of compression curves.

A one-dimensional constant-strain-rate (CSR) compression test was conducted on the calcareous sand with a loading rate of 0.5 mm/min. The result of this test is plotted as $e$ versus log $p'$ in Figure 5-26, together with the points obtained from the end of the shearing tests. The compression curve represents the typical compression behaviour of sands, in which three ranges of behaviour are observed, i.e. an initial pseudo-elastic range, an elasto-plastic transition range, and a plastic range (or LCC).

The end points (taken to be critical state points) obtained from undrained monotonic triaxial and simple shear tests on the calcareous sand form a curved line in the $e$-log $p'$ space. Ishihara (1993) demonstrated that the critical state line for Toyoura sand (a silica sand used as a Japanese standard sand) at low to medium confining pressures ($p' < 15$ MPa) is a curve, which is not parallel to the NCL in $e$-log $p'$ space. Miura et al. (1984) showed that at high confining pressure ($p' \geq 20$ MPa), the critical state line of the same sand is almost parallel to the LCC. Evidence of a curved CSL for a sand was also presented by Been et al. (1991). For sand with its initial state in the LCC region, shear behaviour is expected to be similar under different confining stresses (therefore different void ratios) since the CSL is parallel to the LCC in that regime. However, within an engineering interest range of stress levels, which is not in the LCC regime for most sands, the undrained behaviour is expected to depend on the state parameter $\psi$ (void ratio and confining stress), which is not constant and varies from positive to negative in that region. The compression curve of the calcareous sand exhibits similar features as those observed in a non-calcareous sand. The confining pressures used in shear tests on the calcareous sand are not in the LCC region and both void ratio and confining pressure are expected to affect its undrained shear behaviour. As the state
parameter varies from negative to positive, the dilative behaviour shown in the
undrained shear tests changes from strong to weak. There should be a critical value for
the state parameter of the soil, above which the soil presents contractive behaviour.
However, there are insufficient data to establish this critical value.

(c) The Calcareous Silt

The compression behaviour of the calcareous silt was examined through one-
dimensional compression tests. In order to investigate whether or not formation void
ratios have an effect on the compression behaviour of the silt, samples were prepared in
different ways. It is difficult to prepare samples with varying formation void ratios by
vibration such as in the preparation of sand samples because of the very fine grained
powder (OMYA CARB) contained in the samples. In order to achieve different
formation void ratios, the same proportions of the calcareous silt and OMYA CARB
was used and prepared by the following three methods: (1) mixed with water, (2) mixed
with silicon oil, (3) dry sample without any fluid. The results of these tests are
summarised in Figure 5-27. It can be seen that the water mixture gave the lowest
formation void ratio, the mixture without fluid produced the highest formation void
ratio, while the oil mixture gave the medium value. A slow loading rate (0.2 mm/min)
was applied in these tests. Although the different viscosity of pore fluids (i.e. water,
silicon oil and air) may affect the rate of pore fluid moving, the trend of the CSR
compression curves clearly reveals that the formation of void ratios of silt does affect its
compression behaviour. It can be seen that as confining pressure becomes high, the
compression curves tend to converge to a unique line. However, no LCC regime is
clearly observed in the plots since the fine grain size of the soil particles requires much
higher pressure to achieve the LCC.

The critical state points obtained from the end of undrained triaxial and simple shear
tests are also plotted in this figure (note that all triaxial and simple shear tests were
conducted on samples mixed with oil). It can be seen that these points straddle the two
"oil" compression curves in the pressure range where the shear tests were conducted.
The initial soil state parameter $\psi$ in the shear tests varied from negative to positive,
giving rise to the undrained monotonic behaviour of the calcareous silt discussed above,
which shows the dependency on both void ratio and confining pressure. A strong
dilative response was observed associated with a negative value of $\psi$, while a weak dilative response was observed associated with a positive value of $\psi$.

5.7 Conclusions

This chapter has presented the undrained monotonic behaviour of three calcareous sediments, from clay-sized muddy silt to coarse sand, subjected to triaxial and simple shear tests. After the discussion of the individual behaviour of the three soils, the framework of critical soil mechanics was applied to them, and the following conclusions have been inferred.

1. The monotonic responses of the three soils are remarkably different in respect to the effective stress paths and stress-strain responses. The behaviour of the calcareous muddy silt is much like that of soft clays in that a strain softening response is observed in CAU tests and when stresses are normalised by consolidation stress the stress paths and stress-strain curves are very similar. The behaviour of calcareous sand, depending on both void ratio and confining pressure, reveals a similarity to that of non-calcareous sand. The behaviour of the calcareous silt is similar to that of non-calcareous silt, and it depends on both void ratios and confining pressures.

2. A unique critical state line is found in q-p' space regardless of testing conditions and shear modes for the calcareous muddy silt. For the calcareous silt and for sand, the PT state line is found to be unique regardless of soil state, testing conditions and shear modes. The PT state line for the silt and sand is also found to be coincident with the CSL if stresses at the end of shearing ($\varepsilon_a > 15\%$) are assumed to be critical state for each test. The friction angles at the critical state for the muddy silt, silt and sand are 42°, 41° and 39.4° respectively.

3. Although the three calcareous soils present different stress-strain responses, similar stress ratio-shear strain responses are observed, which show strain hardening responses and can be represented by a simple relationship (e.g. hyperbolic-type relationship).

4. The behaviour of the three calcareous soils has been analysed through the framework of critical state soil mechanics. Like non-calcareous soils, the undrained shearing behaviour of the three calcareous soils could be explained by use of the state parameter $\psi$ and initial stress anisotropy (or consolidation stress ratio $\sigma'_h/\sigma'_v$).
6 Cyclic Responses of Three Calcareous Sediments

6.1 Introduction

This chapter presents the cyclic behaviour of the three calcareous sediments subjected to undrained simple shear tests reported in Chapter 3. The cyclic response data of the three soils are presented individually in the order of calcareous muddy silt, calcareous silt and calcareous sand. For the calcareous muddy silt and the calcareous silt, the typical cyclic response is presented initially, followed by a discussion of the effects of cyclic shear stress levels and mean shear stress levels. In the case of the calcareous muddy silt, the effect of consolidation stress ratios is also considered. For the calcareous sand, the effects of consolidation conditions, cyclic shear stress levels and confining stress levels are examined after the description of typical cyclic responses. The cyclic responses of all three soils are then characterised in respect of their effective stress paths and stress ratio-shear strain behaviour.

6.2 Cyclic Response of the Calcareous Muddy Silt

As explained in Chapter 3, a series of undrained cyclic simple shear tests was carried out on the floe samples of the calcareous muddy silt aimed at investigating the effects on cyclic behaviour of: (1) consolidation stress ratio $\sigma'_\text{hc}/\sigma'_\text{vc}$; (2) magnitude of cyclic shear stress $\tau_{\text{cyc}}$; (3) mean shear stress $\tau_m$. The results of these tests are discussed in this section.

6.2.1 Typical Response in Symmetric Cyclic Simple Shear Tests

A total of eleven symmetric CAU cyclic simple shear tests (i.e. $\tau_m = 0$) were performed on the floe samples of the calcareous muddy silt under consolidation vertical stress $\sigma'_\text{vc}$ of 150 kPa and consolidation stress ratio, $\sigma'_\text{hc}/\sigma'_\text{vc}$ (the ratio of consolidation horizontal stress to vertical stress) of 0.4. Figures 6-1a to 6-1f show the results of a typical test in which the cyclic shear stress level (i.e. cyclic stress normalised by consolidation vertical stress) $\tau_{\text{cyc}}/\sigma'_\text{vc}$ was 0.17.
The effective stress path of the test in the space of shear stress ($\tau$) and vertical effective stress ($\sigma'_v$) is shown in Figure 6-1a, together with the effective stress path of a monotonic test, which indicates the critical state line in this plot. Since the simple shear tests were conducted under constant total vertical stress ($\sigma_v$), the total stress paths for both the monotonic and cyclic tests are vertical at $\sigma'_v = 150$ kPa. Note, however, that in order to maintain $\sigma_v$ constant, it was necessary in all tests to increase the cell pressure (i.e. increase $\sigma_h$), so that in $\tau$-$p$ space, the total stress path would be inclined to the right. In the cyclic stress path in Figure 6-1a, the vertical effective stress ($\sigma'_v$) continues to decrease, as a consequence of cyclic loading, until the stress path crosses the critical state line, which acts as a 'boundary' for the stress paths. However, stress paths may exist above this boundary due to the dilative properties of the soil as discussed below.

The monotonic stress path in Figure 6-1a demonstrates a contractive response (i.e. increasing pore pressure giving reducing $\sigma'_v$). However, the cyclic stress path reveals a dilative response after an initial contractive response. This can be clearly observed in the 20th cycle, in which the vertical effective stress decreases before point 'A' and increases after this point until point 'B' is reached. In the unloading process (or reloading in the opposite direction), a contractive response occurs until point 'C' is reached, after which a dilative response appears, and the whole process forms a loop shaped like a butterfly. As the stress path moves towards the critical state line, the 'butterfly'-shaped loop becomes more and more obvious, indicating that a stronger dilation is occurring at that stage. The points 'A' and 'C', which separate cyclic stress paths into contractive and dilative phases, are defined as a phase transformation state in cyclic loading ($PT_{cyc}$) in this thesis. This definition is similar to that given to phase transformation in an undrained monotonic shearing test by Ishihara et al. (1975). The features of the $PT_{cyc}$ state will be further discussed in Section 6.5.

The stress-strain ($\tau$-$\gamma$) curves presented in Figure 6-1b show a significant cyclic degradation response, i.e. stiffness of each cycle decreases as a consequence of cyclic loading. The shape of the $\tau$-$\gamma$ curve also changes as accumulated shear strain increases, and an 'S-shape' $\tau$-$\gamma$ curve is gradually developed suggesting a very complicated relationship between $\tau$ and $\gamma$. In some cases (e.g. Figure 6-1b), the loop looks deeper than it should because of insufficient data points being recorded. Note that this is one of the flaws of the "old" simple shear apparatus discussed in Section 3.2.2. However, tests
conducted in the "new" simple shear apparatus show clear 'S-shape' \( \tau - \gamma \) curves and will be discussed later in this chapter (e.g. Figure 6-20b).

The plot of stress ratio \( (\tau/\sigma'_{v}) \) versus shear strain \( (\gamma) \) curve shown in Figure 6-1c shows strain hardening behaviour, i.e. extra shear stress ratio is required to make a shear strain increase. The hysteretic loops in this figure show a nesting pattern. The features of the \( \tau/\sigma'_{v} - \gamma \) response will be further discussed in Section 6.6.

Figure 6-1d plots the minor principal effective stress \( (\sigma'_{3}) \) versus cycle number \( (N) \), where \( \sigma'_{3} \) is calculated in the usual way from \( \sigma'_{v}, \sigma'_{h} \) and \( \tau_{vh} \) (this assumes that the complementary shear stress \( \tau_{hv} \) exists on all vertical planes). When the shearing phase is started, the cell pressure increases quickly to maintain the total vertical stress constant, which gives rise to the increase of minor principal effective stress in the first cycle. After the 1st cycle, the minor principal effective stress decreases with the subsequent cycles. At the 44th cycle, the minor principal effective stress crosses zero. This phenomenon is very similar to that in undrained cyclic triaxial tests on saturated sands observed by Seed and Lee (1966) and Castro (1975), who found that effective stresses momentarily drop to zero for some short period of time during cyclic loading. This phenomenon is called 'partial liquefaction' by Seed and Lee and 'initial liquefaction' by Castro. The term 'initial liquefaction' is adopted for this phenomenon in this thesis. The significance of initial liquefaction in describing the undrained cyclic behaviour can be seen in the following.

The excess pore pressure plot shown in Figures 6-1e reveals a cumulative increase after shearing has started. The critical state is reached at about 90 % of maximum generated pore pressure. Having reached the critical state (at the 30th cycle), the rate of pore pressure generation decreases distinctly in the succeeding cycles. Initial liquefaction occurred at 44th cycles and no more excess pore pressure was generated after that. After initial liquefaction, the pore pressure varied in a range reflecting the changes of effective stress in the 'butterfly'-shaped stress path in Figure 6-1a.

The shear strain induced as a consequence of cyclic loading is shown in Figure 6-1f. Initially, shear strain developed at a relatively slow rate. However, after the initial liquefaction was reached, the developing rate of shear strain increased, and the shear strain induced in each subsequent cycle became progressively larger. Failure is defined
as a large shear strain induced during a cyclic loading test (in this case total shear strain $\gamma_t = \pm 30\%$).

6.2.2 Effects of Consolidation Conditions (CIU and CAU)

Four CIU (i.e. $\sigma'_h/\sigma'_v = 1$) undrained symmetric cyclic simple shear tests, under consolidation effective vertical stress $\sigma'_v = 150$ kPa, were performed on the floc samples of the calcareous muddy silt. Typical results of a CIU test with cyclic stress level $\tau_{cyc}/\sigma'_v = 0.2$ are shown in Figures 6-2a to 6-2f.

By comparing the results shown in Figure 6-2 with those in Figure 6-1, it can be seen that the responses of the CIU test are remarkably similar to those of the CAU test in respect of stress path, stress-strain behaviour, patterns of pore pressure generation and shear strain development. The consolidation stress ratios have little effect on the general responses of the soil subjected to cyclic simple shear test, as was also the case in the monotonic simple shear tests. However, consolidation stress ratios do have an effect on the strength of the soil, since the consolidation effective mean stress level in a CIU test is greater than that in a CAU test under the same consolidation vertical effective stress. This can be observed in Figure 6-3, in which cyclic shear stresses normalised by effective vertical consolidation stress ($\tau_{cyc}/\sigma'_v$) are plotted against number of cycles to failure ($\gamma_t = 15\%$, in this case) for both CIU and CAU tests. It can be seen that the trend line of CIU tests (represented by solid triangles) is above that of CAU tests (represented by circles) in the whole range except for lower cycle number ($N < 40$), in which very few tests are conducted. This observation means that at the same cyclic stress level, more cycles are required to fail a CIU test than a CAU test.

6.2.3 Effects of Cyclic Shear Stress $\tau_{cyc}$

The effect of cyclic shear stress ($\tau_{cyc}$) on cyclic behaviour was studied from eleven CAU symmetric cyclic simple shear tests under $\sigma'_v = 150$ kPa and $\sigma'_h/\sigma'_v = 0.4$ with a wide range of $\tau_{cyc}$ values. The results of three typical tests are plotted in Figure 6-4 to 6-7 in terms of stress paths, minor principal effective stress versus cycle number ($\sigma'_3$-N), pore pressure generation, and shear strain development respectively. The following summarises the events observed.
1. The significant effect of cyclic shear stress amplitude on cyclic response is that, as the amplitude increases, the number of cycles needed to attain a certain shear strain level decreases.

2. As a consequence of cyclic loading, vertical effective stress decreases, and stress paths move toward the critical state line. However, the vertical effective stress does not reach zero in all the three tests. Stress paths of the three tests with different cyclic shear stress levels show a similar pattern (Figure 6-4).

3. The three tests show similar patterns in their effective stress paths, $\sigma'_3$-N curves, and pore pressure generation curves. The three pore pressure generation curves (Figure 6-6) show similar shapes if the scale of cycle number is ignored, indicating that a unique pore pressure generation function exists between generated pore pressure and cycle number normalised by a reference cycle number. This issue will be further discussed in Chapter 7.

4. A symmetrical shear strain development pattern is observed for the three tests. In the test with high cyclic shear stress level $\tau_{cyc}/\sigma'_vc = 0.25$ (Figure 6-7a), shear strain developed at an almost constant rate and quickly reached $\pm 30\%$ in the 17th cycle. During the test with low cyclic stress level $\tau_{cyc}/\sigma'_vc = 0.11$ (Figure 6-7c), shear strain initially accumulated slowly and then the rate accelerated after the critical state was reached. The non-symmetric shear strain in the last stage of the test shown in Figure 6-7c is due to the sample slipping to one side caused by the overshooting of cyclic shear stress. This is the first flaw of the old simple shear apparatus as discussed in Section 3.2.2.

5. The critical state line plays an important role in the symmetric cyclic simple shear tests. Before attaining the critical state, pore pressure is generated continually and shear strain develops at a low rate. After reaching the critical state, the shear strain generated in each subsequent cycle becomes progressively larger and only a very small amount of extra pore pressure is generated, which is very similar to the results obtained on reconstituted Boston Blue Clay by Malek et al. (1989). Initial liquefaction is reached very soon after the critical state is achieved in the 2nd and 3rd tests (Figure 6-5b and c). In the test with high cyclic shear stress level (Figure 6-5a), failure ($\gamma_t$ of 30 %) was reached quickly (within 15 cycles), without initial liquefaction being reached.
6.2.4 Effects of Mean Shear Stress $\tau_m$

The effects of the mean shear stress level ($\tau_m$) on cyclic response were investigated by a series of undrained cyclic simple shear tests with a consolidation condition of $\sigma'_{vc} = 150$ kPa and $\sigma'_{ho}/\sigma'_{vc} = 0.4$ as reported in Chapter 3. The relative values of $\tau_m$ and $\tau_{cyc}$ fall into three groups: (1) $\tau_m < \tau_{cyc}$; (2) $\tau_m = \tau_{cyc}$; and (3) $\tau_m > \tau_{cyc}$. In the first case, the shear stress crosses zero during cyclic loading; for the second case, shear stress is applied from zero to $2\tau_{cyc}$; and in the third case, the shear stress is always above zero. One typical test from each group is presented here. The results are shown in Figures 6-8 to 6-11 as plots of effective stress paths, minor principal stress curves, pore pressure generation, and shear strain development. The following observations are inferred from these results.

1. The effective vertical stresses in the stress paths of the three tests (Figure 6-8) continue to decrease as a consequence of cyclic loading. The phase transformation state in cyclic loading ($PT_{cyc}$), which has been defined as the points separating cyclic stress paths into contractive and dilative phases in Section 6.2.1, is observed in the loops of stress paths of the tests presented in Figure 6-8a and 6-8b. In Figure 6-8c, in which the test was conducted with high $\tau_m$ value, there is no $PT_{cyc}$ state observed in the stress path. The stress path of a corresponding monotonic test is also plotted in each figure. The discrepancy between the monotonic stress path and the cyclic stress path of the first cycle in each figure reveals the effect of strain rate, which is 0.1 mm/min in the monotonic test and is approximately twenty times this value in the cyclic tests, as reported in Chapter 3.

2. The plots of minor principal effective stresses versus cycle number ($\sigma'_3$-N) for the three tests in Figure 6-9 reveal that no initial liquefaction ($\sigma'_3 = 0$) occurred.

3. The overall patterns of pore pressure generation (Figure 6-10) are similar. However, as the mean shear stress ($\tau_m$) becomes higher, the initial curvature of the pore pressure generation curve becomes greater. This issue will be discussed again in Chapter 7.

4. The shear strain development of the three tests (Figure 6-11) is significantly different from those in symmetric tests shown in Figure 6-7, in which a symmetrical pattern of shear strain development is observed. Among these three tests, significantly different
patterns of shear strain development can be observed. The total shear strain $\gamma$ induced in each cycle can be divided into two components: (1) mean shear strain $\gamma_m$, which is represented by the average value of shear strain within a cycle, and (2) cyclic shear strain $\gamma_{cyc}$, which is the amplitude of shear strain within a cycle. In the test with $\tau_m < \tau_{cyc}$ (Figure 6-11a), both $\gamma_m$ and $\gamma_{cyc}$ were accumulated as a consequence of cyclic loading. In the test with $\tau_m = \tau_{cyc}$ (Figure 6-11b), $\gamma_m$ developed quickly while the $\gamma_{cyc}$ increased at a slow rate. In the test with $\tau_m > \tau_{cyc}$ (Figure 6-11c), $\gamma_m$ increased significantly while very little $\gamma_{cyc}$ was induced during the cyclic loading. Therefore, in non-symmetric cyclic tests failure is best identified by $\gamma_t$, which is the combination of $\gamma_m$ and $\gamma_{cyc}$.

The stress path, stress-strain behaviour, pore pressure generation and shear strain accumulation patterns of the calcareous muddy silt discussed above are qualitatively very similar to those of reconstituted Boston Blue clay in undrained cyclic simple shear tests for all combinations of $\tau_m$ and $\tau_{cyc}$ (Melak et al. 1989).

6.3 Cyclic Response of the Calcareous Silt

The cyclic behaviour of the calcareous silt was studied through a series of undrained cyclic simple shear tests as described in Chapter 3. The aim of this section is to present the typical cyclic behaviour of this soil and the effects of amplitude of $\tau_{cyc}$ and $\tau_m$ on cyclic behaviour.

6.3.1 Typical Response in Symmetric Cyclic Simple Shear Tests

A total of six symmetric CAU cyclic simple shear tests have been performed on the calcareous silt. The results of a typical undrained cyclic simple shear test with cyclic shear stress level $\tau_{cyc}/\sigma_{vc}' = 0.13$ are presented in Figures 6-12a to 6-12f. The following are summarised from the events observed.

1. Overall, the responses of the calcareous silt are very similar to those of the calcareous muddy silt shown in Figure 6-1.

2. The cyclic stress path exhibited in Figure 6-12a resembles that of the cyclic test on the calcareous muddy silt as shown in Figure 6-1a, although the stress paths from the monotonic tests on the two soils are different. The effective vertical stress in the cyclic test continually decreases as a consequence of cyclic loading even though the PT state
line is crossed. After the effective vertical stress decreased to some extent, the cyclic stress path shows a dilative behaviour, indicating the existence of a PT\textsubscript{cyc} state.

3. The stress-strain curve ($\tau$-$\gamma$) exhibited in Figure 6-12b shows a complicated relationship, whilst the stress ratio-shear strain curve ($\tau/\sigma_{vc}^\prime$-$\gamma$) presented in Figure 6-12c reveals a strain hardening response. Nested patterns are observed in the $\tau/\sigma_{vc}^\prime$-$\gamma$ curve.

4. Pore pressure (Figures 6-12e) is generated quickly during the initial stage, then the generation rate decreases. The pore pressure generation rate increases again when the PT state is reached. This generation rate is maintained until initial liquefaction ($\sigma_3' = 0$) occurs.

5. The phase transformation (PT) state line obtained from the undrained monotonic test plays an important role in the shear strain development in the cyclic test (Figure 6-12f). Very little shear strain is induced before the PT state line is reached, but after the PT state line is reached, shear strain quickly develops at a constant rate until failure (a large shear strain) occurs. It can also be observed in Figure 6-12 that initial liquefaction ($\sigma_3' = 0$) occurs in a few cycles after the PT state line is reached.

6.3.2 Effects of Cyclic Shear Stress $\tau_{cyc}$

The results of three CAU symmetric ($\tau_m = 0$) cyclic simple shear tests with cyclic stress level $\tau_{cyc}/\sigma_{vc}^\prime = 0.2, 0.14$ and 0.12 are presented in Figures 6-13 to 6-16 as plots of effective stress path, plots of minor principal effective stress versus cycle number, pore pressure generation and shear strain development curves. The following observations are made.

1. As the cyclic stress level increases, the number of cycles to reach cyclic failure (i.e. a certain level of shear strain) decreases.

2. Similar stress path patterns are observed for the three tests (Figure 6-13). After vertical effective stress decreased to some extent, PT\textsubscript{cyc} state points can be seen in each stress path loop.

3. Initial liquefaction occurred in the three symmetric cyclic loading tests (Figure 6-14).
4. The pore pressure generation curves for the tests with $\tau_{cyc}/\sigma_{vc}' = 0.14$ and 0.12 (Figures 6-15b and 6-15c) are found to be of the same shape. In the test with high cyclic shear stress levels, the sample failed quickly, and the pore pressure generation curve showed a hyperbolic relationship.

5. Symmetric shear strain generation curves are observed in Figure 6-16 though some asymmetry develops towards the end of tests in Figures 6-16b and c. During the test with highest cyclic shear stress level, the sample failed within a few cycles and shear strain developed at a constant rate. The shear strain curves of the tests with lower cyclic shear stress levels (Figures 6-16b and c) show that reaching the PT state line (defined from the monotonic test) results in a significant change in shear strain response. Very small shear strain accumulated before the PT state was reached. The point corresponding to the PT state line marks a turning point, after which shear strain developed rapidly towards a larger value.

6. In the three tests, initial liquefaction occurred very soon (a few cycles) after the PT state line was reached. The dramatic change in shear strain development was marked by reaching the PT state line.

6.3.3 Effects of Mean Shear Stress $\tau_m$

Undrained cyclic simple shear tests with non-zero mean stress were conducted on the calcareous silt as explained in Chapter 3. Two typical tests with $\tau_m < \tau_{cyc}$ and $\tau_m = \tau_{cyc}$ are presented in Figures 6-17 to 6-19. The following summarises the observations from these results.

1. Overall, the effects of mean shear stress on the cyclic response of the calcareous silt are similar to those on the calcareous muddy silt presented in Figures 6-8 to 6-11.

2. Pore pressure generation patterns for the two tests shown in Figure 6-18 are similar.

3. Shear strains in the two tests show quite different behaviour (Figure 6-19). Both the mean shear strain and cyclic shear strain contribute to failure. As the mean shear stress increases, failure is dominated by the mean shear strain.
6.4 Cyclic Response of the Calcareous Sand

The cyclic response of the calcareous sand was investigated through a series of undrained simple shear tests as reported in Chapter 3. The results of these tests are presented in this section.

6.4.1 Typical Response in Symmetric Cyclic Simple Shear Tests

Figure 6-20 shows the results of a typical CAU cyclic simple shear test on the calcareous sand consolidated under $\sigma'_vc = 75$ kPa and $\sigma'_hc/\sigma'_vc = 0.4$. For comparison purposes, the results of a typical CIU cyclic simple shear test under $\sigma'_vc = 75$ kPa are shown in Figure 6-21. The following inferences are reached from these tests.

1. The overall cyclic response of the calcareous sand shown in Figure 6-20 is remarkably similar to those of calcareous muddy silt (Figure 6-1) and calcareous silt (Figure 6-12) discussed in previous sections.

2. Dilative behaviour can be observed in cyclic stress path loops after the effective vertical stress has decreased to some extent.

3. The stress-strain ($\tau$-$\gamma$) curve (Figure 6-20b) reveals a complicated relationship, while the stress ratio-shear strain ($\tau/\sigma'_v$-$\gamma$) relationship (Figure 6-20c) presents a strain hardening behaviour. Nested pattern is observed in the $\tau/\sigma'_v$-$\gamma$ curve.

4. Reaching the PT state line has a noticeable effect on the cyclic shear strain development curve (Figure 6-20f). A very small amount of shear strain occurs before the PT state is reached. After reaching the PT state line, the rate of shear strain development increases considerably.

5. In general, the responses of a CIU test shown in Figure 6-21 are noticeably similar to those of the CAU test in Figure 6-20. This observation is similar to the comparison of the CIU and the CAU simple shear tests on the calcareous muddy silt in Figures 6-1 and 6-2.

The stress-strain, pore pressure generation and shear strain accumulation responses of the calcareous sand are very similar to those of a quartz sand in undrained cyclic simple shear tests reported by Peacock and Seed (1968).
6.4.2 Effect of Cyclic Shear Stress $\tau_{\text{cyc}}$

The results of three typical symmetric CIU cyclic shear tests with $\sigma'_{\text{vc}} = 300$ kPa are presented in Figures 6-22 to 6-25 as plots of stress paths, minor principal effective stress curves, pore pressure generation and shear strain development. The observations from these results are summarised as follows.

1. The overall responses of the soil in these tests are shown to be very similar to those of the calcareous muddy silt (Figures 6-4 to 6-7) and the calcareous silt (Figures 6-13 to 6-16).

2. As the cyclic stress level increases, the number of cycles to reach cyclic failure decreases.

3. The PT state line acts as a characteristic 'boundary', below which pore pressure is continually generated and very little shear strain develops, but once reached very little pore pressure is generated and shear strain increases rapidly towards a large value.

4. Initial liquefaction is reached in a few cycles after the PT state has been reached. Following initial liquefaction, pore pressure changes significantly during each cycle, reflecting the contractive/dilative response that occurs in each cycle.

6.4.3 Effect of Confining Pressure $\sigma'_{\text{vc}}$ on Cyclic Response

The effects of confining pressure on cyclic responses were investigated through a series of tests with consolidation vertical effective stresses $\sigma'_{\text{vc}} = 75$ and 300 kPa on both isotropically and anisotropically consolidated samples. For the CAU tests, a consolidation stress ratio ($\sigma'_{\text{h}}/\sigma'_{\text{vc}}$) of 0.4 was applied. The effects of consolidation stress level on pore pressure and shear strain responses are compared in Figures 6-26 to 6-29. Two CIU tests (with the same $\tau_{\text{cyc}}/\sigma'_{\text{vc}}$ ratio of 0.23) under $\sigma'_{\text{vc}} = 75$ and 300 kPa are presented in Figures 6-26 to 6-27 respectively, while two CAU tests (with $\tau_{\text{cyc}}/\sigma'_{\text{vc}}$ ratio of 0.20) under $\sigma'_{\text{vc}} = 75$ and 300 kPa are presented in Figure 6-28 to 6-29 respectively. The observations are summarised as follows.

1. It can be seen from Figures 6-26 and 6-27 that the shapes of the normalised pore pressure generation and the shear strain development curves of the two CIU tests are very similar. A shear strain of 15% is reached in the first test ($\sigma'_{\text{vc}} = 75$ kPa) after 55
cycles, whilst in the second test ($\sigma'_{vc} = 300$ kPa), the same shear strain was reached at around 100 cycles.

2. In Figures 6-28 and 6-29, the shapes of the normalised pore pressure generation and shear strain development curves for the CAU test with $\sigma'_{vc} = 75$ kPa are almost identical to those for the CAU test with $\sigma'_{vc} = 300$ kPa. After 30 cycles in the test with $\sigma'_{vc} = 75$ kPa, the shear strain reaches 15%, while the same amount of shear strain is reached in the 26th cycle in the tests with $\sigma'_{vc} = 300$ kPa.

In order to examine the effect of consolidation pressure on the number of cycles to produce the same shear strain (so-called 'cyclic strength' as will be discussed in Chapter 7), the normalised cyclic shear stress ($\tau_{cyc}/\sigma'_{vc}$) for CIU tests is plotted against the number of cycles to $\gamma_t$ of 5% and 15% in Figures 6-30a and b respectively. In each figure, the circles represent the results of tests under $\sigma'_{vc}$ of 75 kPa, and the triangles represent the results of tests under $\sigma'_{vc}$ of 300 kPa. It can be seen that the trends of the two series test data come together, indicating that consolidation stress level has no effect on the normalised 'cyclic strength' curve. Similar behaviour is also observed in Figures 6-31 for the CAU tests.

Therefore, after normalisation by consolidation stress level ($\sigma'_{vc}$), the cyclic responses of the calcareous sand in undrained simple shear tests under two consolidation effective vertical stresses of 75 and 300 kPa are very similar. A contrary conclusion was reached by Finnie et al. (1999) in studying symmetric cyclic simple shear tests on a carbonate sand under two different consolidation stress levels ($\sigma'_{vc} = 75$ and 400 kPa). They reported that under these two consolidation stress levels, shear strain development and pore pressure generation of the soil in symmetric cyclic loading are different.

6.5 Characterising Undrained Stress Paths for the Three Soils: PT_{cyc} State Surface

A key finding in the undrained cyclic stress paths of the three soils, described in the preceding sections, is that similar dilative responses are observed for the three soils regardless of the different responses observed in their corresponding monotonic stress paths. Phase transformation from contractive to dilative response in undrained cyclic stress paths has been defined as the PT_{cyc} state in Section 6.2. The aim of this section is to characterise the PT_{cyc} state for the three soils.
6.5.1 \( P T_{cyc} \) State Surface

In order to examine the \( P T_{cyc} \) state in a 3-dimensional stress space, the stress paths of a symmetric cyclic simple shear test and a symmetric cyclic triaxial test on the calcareous silt are presented in Figures 6-32a and b respectively. Figure 6-32a also shows the stress path of a monotonic test under the same consolidation condition as the cyclic test shown in the same figure. In Figure 6-32a, if the \( P T_{cyc} \) state points in the cyclic stress path in one shearing direction are linked together, a straight line radiating outwardly from the origin of \( \tau - \sigma'_v \) space is obtained. Similarly, a straight line is obtained in the opposite shearing direction. It can be seen from Figure 6-32a that the phase transformation (PT) line obtained from the monotonic test locates above the \( P T_{cyc} \) state line in the same direction. The corresponding shear stress ratios \((\tau/\sigma'_v)\) for the PT line (in the monotonic test) and the \( P T_{cyc} \) line in a positive shearing direction in \( \tau-\sigma'_v \) space are 0.54 and 0.22 respectively. For the cyclic triaxial test shown in Figure 6-32b, a pair of \( P T_{cyc} \) lines can also be obtained by using the same method. It can be inferred from the two figures that the \( P T_{cyc} \) state exists as a conical-shaped surface in a 3-dimensional stress space with the origin of stress space as the apex of the conical surface. The \( P T_{cyc} \) state surface is similar to the critical state surface in stress space but with a smaller apex angle.

6.5.2 Characteristics of \( P T_{cyc} \) State

(a) Uniqueness of \( P T_{cyc} \) State Regardless of Testing Conditions

Figure 6-33 presents the effective stress paths of the calcareous muddy silt in six cyclic simple shear tests, including two CIU tests and four CAU tests. The two CIU cyclic simple shear tests were conducted under symmetric cyclic loading with cyclic shear stress levels \((\tau_{cyc}/\sigma'_v)\) of 0.25 and 0.2 (Figures 6-33a and b). The four CAU cyclic simple shear tests were conducted with four different conditions consisting of (1) \( \tau_m = 0 \), (2) \( \tau_m < \tau_{cyc} \), (3) \( \tau_m = \tau_{cyc} \), and (4) \( \tau_m > \tau_{cyc} \) (Figures 6-33c, d, e and f).

A pair of \( P T_{cyc} \) state lines is obtained by linking \( P T_{cyc} \) points of the stress path loops in Figure 6-33a. The corresponding shear stress ratio \((\tau/\sigma'_v)\) for the \( P T_{cyc} \) line in a positive shearing direction is 0.18 for this soil, and the pair of lines in the two shearing directions are symmetric in \( \tau-\sigma'_v \) space. These \( P T_{cyc} \) state lines are then superimposed on the
stress paths of other tests shown in Figures 6-33b, c, d, e and f. Only one PT_{cyc} line was
drawn in Figures 6-33e and f, as the effective stress paths of these two tests exist only in
one direction. It can be observed that the pair of PT_{cyc} lines obtained from the first test
(Figure 6-33a) passes through all the PT_{cyc} points in Figures 6-33b, c, d and e. The only
exception is observed in the test shown in Figure 6-33f, in which there exists no PT_{cyc}
state point due to the stress path being above the PT_{cyc} state line. It can be concluded
that the PT_{cyc} state line is not affected by testing conditions, such as cyclic stress level,
mean shear stress level and consolidation conditions (CIU and/or CAU).

(b) Uniqueness of PT_{cyc} State Regardless of Shearing Modes

The effective stress paths of the calcareous silt plotted in Figure 6-34 are in two groups.
Group 1 includes Figures 6-34a, b and c, which show the stress paths of three cyclic
simple shear tests including two symmetric cyclic tests and one non-symmetric cyclic
test. A pair of PT_{cyc} state lines is drawn by linking PT_{cyc} points in the first test (Figure
6-34a), and the ratio of \tau/\sigma'_{v} corresponding to the PT_{cyc} line in \tau-\sigma'_{v} space is 0.22 for
this soil. Both PT_{cyc} lines are then drawn in the stress paths shown in Figures 6-34b and
c, which provide evidence for the independence of PT_{cyc} state lines on cyclic shear
stress levels and mean shear stress levels for this soil.

Group 2, consisting of Figures 6-34d, e and f, includes plots of stress paths from cyclic
triaxial tests. Figure 6-34d presents the effective stress path of a CIU cyclic triaxial test
on the calcareous silt. A PT_{cyc} state line, represented by a dotted line, can be drawn by
linking the points where the mean effective stress path changes from contractive to
dilative in the direction with positive q. This straight line is then superimposed in
Figure 6-34e, in which the first four cycles of the test shown in Figure 6-34a are re-
plotted in q-p' space. It can be seen from Figure 6-34e that the PT_{cyc} line (obtained
from the triaxial test in Figure 6-34d) passes through the turning points of the stress path
of the simple shear test plotted in q-p' space.

A further step is to examine whether the PT_{cyc} lines in q-p' and \tau-\sigma'_{v} spaces are the
same. This is fulfilled by inspecting the first four cycles of the stress path of the test
shown in Figure 6-34a plotted in terms of q-p' space (Figure 6-34e) and in terms of \tau-
\sigma'_{v} space (Figure 6-34f). The flat bars seen in the stress paths represent the data
recorded during the test. Five solid dots in these two figures represent the PT_{cyc} state
points observed in $\tau-\sigma'_v$ space (Figure 6-34f). It can be seen that in $q-p'$ space, the $PT_{cyc}$ state line (dotted line), which was originally obtained from the triaxial test, passes through the five solid dots, despite two of them being positioned slightly below the $PT_{cyc}$ line due to the existence of data recording gaps. This observation indicates that the $PT_{cyc}$ state is not affected by different shearing modes (i.e. triaxial and simple shear).

(c) Uniqueness of $PT_{cyc}$ State Regardless of Loading Rate

Figure 6-35 examines the $PT_{cyc}$ state lines for the calcareous sand. The first five graphs (Figures 6-35a to e) present the effective stress paths of five simple shear tests under both CIU and CAU conditions with two consolidation vertical effective stresses of 75 and 300 kPa. A pair of $PT_{cyc}$ state lines obtained from the first test (Figure 6-35a) with corresponding $\tau'/\sigma'_v$ ratio of 0.41 in $\tau-\sigma'_v$ space in positive shearing direction, are superimposed in the stress paths of other four tests (Figures 6-35b, c, d and e). It can be seen that the pair of $PT_{cyc}$ state lines pass through all the state points in these figures. Therefore, the $PT_{cyc}$ state lines are not affected by cyclic stress level, consolidation conditions (CIU and CAU), as well as consolidation vertical stress level.

The last graph (Figure 6-35f) represents the stress path of a monotonic simple shear test, in which unloading-reloading loops were carried out at a constant loading rate of 0.1 mm/min. In cyclic simple shear test, however, a constant frequency of 0.1 Hz is applied as discussed in Chapter 3. This frequency is equivalent to an initial loading rate of 1.5-3 mm/min in cyclic simple shear tests depending on the amplitude of shear stress. As shear strain increases, the loading rate speeds up (adjusted by a programme) in order to keep the frequency constant. Therefore, compared to loading rates imposed during a cyclic test, the loading rate used in a monotonic test is much smaller. The pair of $PT_{cyc}$ state lines obtained from Figure 6-35a are superimposed in Figure 6-35f. It can be seen that the two $PT_{cyc}$ state lines pass through the turning point in the stress path, indicating that change of loading rate does not affect the $PT_{cyc}$ state.

It can therefore be concluded that $PT_{cyc}$ state lines (or surface in a 3-dimensional stress space) are unique regardless of soil state, consolidation condition (CIU and CAU), loading rate and shearing mode (simple shear or triaxial). The $PT_{cyc}$ state is a function of soil intrinsic properties such as particle size, angularity, crushability etc. The
mobilised friction angles corresponding to the $PT_{cyc}$ state lines in $\tau-\sigma'_v$ space for the calcareous muddy silt, calcareous silt and calcareous sand are $10.1^\circ$, $12.3^\circ$ and $22.1^\circ$ respectively, indicating the effect of particle size on the apex angle of $PT_{cyc}$ state surface. In the tests on the three soils, the $PT_{cyc}$ state is reached after the effective vertical stress decreases to some extent. Thus, the $PT_{cyc}$ state is triggered by the tendency for the soil particles to densify during cyclic loading.

6.6 Characterising Shear Strain Behaviour for the Three Soils: Strain Hardening Behaviour in Stress Ratio-Shear Strain Relationships

The shear stress-shear strain ($\tau-\gamma$) curves of the three calcareous soils shown in Figures 6-1b, 6-12b and 6-20b demonstrate very complicated relationships, i.e. gradual development of “S-shape” $\tau-\gamma$ curves as shown in Figure 6-36 for a symmetric test on the calcareous muddy silt. This feature makes it difficult to characterise the shear strain behaviour of the three soils. However, the stress ratio-shear strain ($\tau/\sigma'_v-\gamma$) curves for the three soils shown in Figures 6-1c, 6-12c and 6-20c exhibit strain-hardening behaviour. The responses observed in the $\tau/\sigma'_v-\gamma$ curves make it easy to characterise shear strain responses in undrained cyclic loading tests. This section aims at characterising the strain hardening behaviour in respect of the $\tau/\sigma'_v-\gamma$ relationships.

6.6.1 Features of Backbone Curve

Typical $\tau/\sigma'_v-\gamma$ responses of symmetric cyclic simple shear tests on the calcareous muddy silt, calcareous silt and calcareous sand are shown in Figures 6-37a, b and c respectively, and strain hardening behaviour is observed. If the apexes of the loops are connected, a backbone curve may be obtained in the $\tau/\sigma'_v-\gamma$ curve for each test. As discussed in Chapter 2, a backbone curve can be expressed in a variety of ways. Among them, the hyperbolic function has been widely used (Kondner 1963, Hardin & Drnevich 1972, Finn et al. 1977, Pyke 1979, Pradhan 1989) due to two main advantages: (1) only two parameters with clear physical meaning are required to describe a backbone curve, and (2) it can be expressed either as shear stress in terms of shear strain or as shear strain in terms of shear stress. The hyperbolic model is adopted herein for describing the stress ratio in terms of shear strain response as follows:
where $\chi = \tau/\sigma'_v$ is the stress ratio, $\gamma$ is the shear strain, $G^*_i$ is the initial gradient of the $\tau/\sigma'_v$-$\gamma$ curve, and $\chi_f$ is the asymptotic value of the stress ratio $\chi$.

By applying this function, a backbone curve is drawn for each of the three soils (Figures 6-37a, b and c). The values of $G^*_i$ and $\chi_f$ for each soil are shown in each figure. The hyperbolic function describes quite well the backbone curves of the three soils. It must to be noted that, due to shortcomings of the pneumatic loading control system in the old simple shear apparatus (discussed in Chapter 3), in which simple shear tests on the calcareous muddy silt and calcareous silt were conducted, the apices of cycle loops were not properly recorded (Figures 6-37a and b). However, in the $\chi$-$\gamma$ curve for the calcareous sand, on which tests were carried out in the new simple shear apparatus (Chapter 3), a much clearer well-defined backbone curve is obtained (Figure 6-37c).

In Figures 6-37a, b and c, a monotonic test result for each soil has also been plotted, together with cyclic test results. The fact that the monotonic loading curve is not the backbone curve of the cyclic loading test reveals that the shear strain rate has an effect on the backbone curve. This can be demonstrated through the results shown in Figure 6-38. In Figure 6-38a, the $\chi$-$\gamma$ curve of a symmetric CAU cyclic simple shear test on the calcareous sand consolidated under $\sigma'_vc = 75$ kPa and $\sigma'_h/\sigma'_vc = 0.4$ is plotted together with that of a monotonic test conducted under the same consolidation conditions. In this case, the backbone curve lies well above the monotonic curve. Figure 6-38b plots the $\chi$-$\gamma$ curve of the monotonic 'loops' test, which has been shown in Figure 6-35f, together with the result of the same monotonic test as shown in Figure 6-38a. It can be observed that in this case the $\chi$-$\gamma$ curve of the monotonic test perfectly represents the backbone curve of hysteretic loops. This shows that the backbone curve is affected by cyclic loading rate, and only corresponds to the monotonic test if the shearing rate is similar.
Apart from the shear strain rate, the backbone curve is also affected by other factors such as initial confining pressure, void ratio and strain history (Pyke 1979). The two basic parameters $G^*$ and $\chi_f$ can be made functions of these factors if necessary.

### 6.6.2 Features of Hysteresis Loops

Having defined the backbone curve, a common method for describing symmetric hysteresis loops from the backbone curve is to assume that the soil behaviour satisfies the Masing rules (Masing 1926). Pyke (1979) supplemented the Masing rules by two extended rules after studying simple shear behaviour of a soil subjected to irregular cyclic loading. The two original and two extended Masing rules have been described in Chapter 2 (Page 2-4) and will not be repeated here.

These rules are examined through the results just presented in Figures 6-37 and 6-38 as well as those for non-symmetric cyclic loading tests on the calcareous muddy silt (Figure 6-39). It can be observed that the 1st and 2nd rules are well supported by the symmetric cyclic loading tests. The 3rd rule is supported by the results of the non-symmetric cyclic tests presented in Figure 6-39, whereas the 4th rule cannot be proved due to lack of irregular cyclic loading tests conducted on these soils.

The $\chi$-$\gamma$ response of the three calcareous soils can be expressed by use of a backbone curve and the Masing rules describing hysteretic loops. The values of $G^*$ and $\chi_f$ characterise the cyclic resistance of a soil.

### 6.7 Conclusions

The cyclic behaviour of the three calcareous sediments – calcareous muddy silt, calcareous silt and calcareous sand – subjected to undrained cyclic simple shear tests has been discussed in this chapter. The following conclusions are drawn.

1. The undrained cyclic responses of the calcareous muddy silt are qualitatively similar to those of reconstituted Boston Blue clay, whilst the undrained cyclic responses of the calcareous sand are similar to those of a quartz sand. Similar undrained cyclic responses are observed for the three soils in respect to effective stress path, stress-strain behaviour, stress ratio-shear strain behaviour, pore pressure generation and shear strain development.
2. Failure in undrained cyclic simple shear tests occurs when the total shear strain induced in cyclic loading reaches a certain level. In symmetric cyclic tests, shear strains develop symmetrically. The critical state (for contractive soil) or phase transformation state (for dilative soil) marks a turning point in shear strain development, before and after which the rates of shear strain development are very different. In non-symmetric cyclic tests, the total shear strain can be divided into mean shear strain and cyclic shear strain. As mean shear stress increases, shear strains are dominated by mean shear strain.

3. In symmetric cyclic tests, the shapes of pore pressure generation curves corresponding to different cyclic shear stress levels are the same. In non-symmetric cyclic tests, the curvature of the pore pressure generation curve, as plotted in normalised pore pressure versus normalised cycle number, increases with an increase in mean shear stress.

4. The general cyclic responses in CIU and CAU cyclic simple shear tests are very similar. However, under the same vertical consolidation stress level and same cyclic stress amplitude, the number of cycles to failure in a CIU test is greater than that in a CAU test.

5. The cyclic responses (normalised by $\sigma'_{vc}$) of the calcareous sand subjected to undrained simple shear tests under two consolidation effective vertical stresses of 75 and 300 kPa are very similar.

6. The $PT_{cyc}$ state surface separates stress space into contractive and dilative phases for undrained cyclic loading testing. The $PT_{cyc}$ state surface is unique for a particular soil in spite of soil state, consolidation stress ratios ($\sigma'_h/\sigma'_{vc}$), loading conditions (symmetric and non-symmetric), shearing modes (triaxial or simple shear). The $PT_{cyc}$ state surface is a function of the intrinsic properties of a soil.

7. Strain hardening is found in the relationships of stress ratio-shear strain ($\chi-\gamma$) for the three soils subjected to undrained cyclic simple shear tests. The $\chi-\gamma$ responses of the three soils can be described by a backbone curve, which is a hyperbolic function, and two original and the first extended Masing rules which describe the hysteretic loops. The second extended Masing rule cannot be proved due to lack of irregular cyclic loading test data.
7
Characterisation of Cyclic Responses

7.1 Introduction

The behaviour of many calcareous sediments recovered from offshore in the North West Shelf of Australia has been investigated through undrained triaxial and simple shear tests associated with either commercial testing projects or research projects in the past decade in the Geomechanics Group at UWA. These calcareous sediments include a wide range of particle sizes, components and fabric structure, ranging from fine-grained muddy silt to coarse sands. Some of the soil samples were undisturbed and others were reconstituted or remoulded. The data from cyclic loading tests on one very lightly-cemented and four uncemented soils have been collected together with the test data for the three calcareous soils studied in the preceding chapters in this thesis in order to characterise the cyclic behaviour of the uncemented calcareous sediments. In this chapter, the behaviour of these soils in undrained cyclic simple shear tests is characterised through phenomenological (empirical) methods. This chapter consists of the following four parts:

1. The effect of cyclic shear stress level on cyclic behaviour is expressed by way of a cyclic strength curve. The cyclic strengths of a broad range of calcareous sediments are compared and characterised.

2. The effect of mean shear stress on the cyclic strength is characterised using the method of equal damage contours. A model (the Gerber model) is modified and then applied to two other calcareous soils.

3. A model for pore pressure generation behaviour under both symmetric and non-symmetric cyclic loading tests is developed. This model is then applied to two other calcareous soils.

4. Through combining cyclic strength curves, the equal damage contour model, the pore pressure generation model and strain hardening behaviour, a framework for simulating cyclic simple shear behaviour is proposed.
7.2 Effect of Cyclic Shear Stress: Cyclic Strength Curve

7.2.1 Expression of Cyclic Strength

The cyclic strength of a soil can be defined as the cyclic stress, $\tau_{\text{cyc}}$, required to produce a given level of total shear strain, $\gamma_t$, after a specific number of cycles, $N$. This can be expressed by plotting the cyclic stress as a function of the number of cycles to produce a certain total strain. In the literature, cyclic shear stress normalised by the consolidation stress, $\tau_{\text{cyc}}/\sigma'_v$, is commonly used since undrained strengths of most soils are proportional to the consolidation stress level (Peacock & Seed 1968, Finnie et al. 1999).

The plots of $\tau_{\text{cyc}}/\sigma'_v$ versus $N_{15}$ for the calcareous muddy silt and the calcareous silt discussed in preceding chapters are presented in Figures 7-la and b respectively (In this context $N_5$, $N_{15}$, $N_{30}$ etc. signify the number of cycles required to reach $\gamma_t = 5\%$, $15\%$, $30\%$ etc. $N_f$ signifies number of cycles to failure, which could be defined as $N_f = N_5$, $N_f = N_{15}$, or $N_f = N_{30}$). Both symmetric and non-symmetric cyclic loading data are included in these figures. The number next to each non-symmetric cyclic loading test indicates the mean shear stress level normalised by consolidation vertical effective stress, $\tau_m/\sigma'_v$. It can be observed that the mean shear stress has a significant effect on the cyclic strength curves. The test data for non-symmetric cyclic loading tests on the calcareous muddy silt shown in Figure 7-la reveal that the higher the mean shear stress, the lower the position of the cyclic strength curve. Because of this, it is difficult to compare the cyclic strengths of non-symmetric cyclic loading tests with those of symmetric cyclic loading tests by a plot that takes no account of $\tau_m$. This issue will be discussed in Section 7.3.

In Figure 7-2, only the symmetric cyclic test data from Figure 7-1 are replotted. The cyclic strength curves in this figure may be represented by the power law used by Finn et al. (1978), which is of the following form:

$$\frac{\tau_{\text{cyc}}}{\sigma'_v} = cN_f^{-b} \quad 7-1$$

where $\tau_{\text{cyc}}$ is the cyclic stress amplitude for symmetric cyclic loading, $\sigma'_v$ is consolidation vertical effective stress, $N_f$ is number of cycles to failure (i.e. a certain level of total shear strain), and parameters $c$ and $b$ are constants relying on soil
properties and testing conditions. Parameter c represents the intersection of the cyclic strength curve with the $\tau_{cyc}/\sigma'_{vc}$ axis, while parameter b reflects the rate of degradation in cyclic strength with increasing $N_f$. Note that Equation 7-1 implies that the cyclic strength ratio $\tau_{cyc}/\sigma'_{vc}$ continues to reduce towards zero with increasing numbers of cycles. In reality, there may be a "cyclic threshold" – a cyclic strength ratio below which failure never occurs. Such a threshold (at $\tau_{cyc}/\sigma'_{vc} = 0.1$) might be inferred from the data in Figure 7-2a, but this point has not been further investigated here.

It can be seen from Figure 7-2 that the test data for symmetric cyclic loading are well represented by this power function. However, these test data reveal that it is difficult to compare cyclic strengths of different soils by using this normalisation, because of the variation of both intersection with the $\tau_{cyc}/\sigma'_{vc}$ axis and the rate of degradation of the cyclic strength curves.

An alternative way of comparing the cyclic strengths of different soils is by normalising the cyclic stress level by the undrained monotonic strength, $\tau_{peak}$, which is the maximum value of shear stress in an undrained monotonic simple shear test under the same consolidation stress level as in cyclic loading tests. The normalised cyclic stress may then be plotted as a function of the number of cycles to failure (or to a specified shear strain level), to give the so-called S-N curve, in which $S$ is the normalised cyclic stress (in this case, $S = \tau_{cyc}/\tau_{peak}$), and $N$ is the number of cycle. The S-N curve is widely used in metal fatigue studies and is also used as cyclic strength curves for soils (e.g. Joer et al. 1993). In this normalisation, the intersection of the S-N curve with the normalised cyclic stress ($\tau_{cyc}/\tau_{peak}$) axis should be equal to 1 if soil behaviour is rate independent or shearing rates used in monotonic and cyclic tests are same. The S-N curves of different calcareous soils will be discussed in the following section.

7.2.2 Cyclic Strengths of Calcareous Soils

A total of eight calcareous sediments are considered in this section, all from the offshore area of the North West Shelf of Australia. In order to simplify discussion in the following sections, the eight soils are named in a series from Soil #1 to Soil #8, which are summarised in Table 7-1, and briefly described in the following.
Soils #1, #2, #3 and #4 were recovered as undisturbed samples from various depths beneath the surface of the seabed at different sites. Soil #1 is the calcareous muddy silt discussed in the preceding chapters and was originally recovered from a depth of 18-30 m below the surface of the seabed. Soil #2 is also a fine-grained silt and was recovered from 20-30 m beneath the seabed surface. Soil #3, nominally undisturbed, is a very slightly-cemented silty sand recovered from the seabed. Soil #4 is a calcareous silt and was recovered from 5-30 m below the seabed. Soils #5, #6, #7 and #8 were recovered from the surface of the seabed at different sites and remoulded or reconstituted samples of these soils were used in corresponding tests. Soil #5 is the mixture of a calcareous seabed silt, OMYA CARB and silicon oil, which has been discussed in the preceding chapters. Soils #6 and #7 are two calcareous sands. Soil #8 is the coarse calcareous sand which has been studied in the preceding chapters.

The undrained monotonic and cyclic simple shear tests on seven of these soils (all except Soil #2), were conducted under constant vertical total stress and constant sample height by adjusting lateral confining stress as discussed in Chapter 3. The tests on Soil #2 were conducted under constant sample height and constant lateral confining stress and as a consequence, the total vertical stress reduced during shearing.

Figure 7-3 shows a summary of the cyclic strength curves (i.e. S-N curves) corresponding to $\gamma_t$ of 30 % for the eight calcareous sediments under undrained symmetric cyclic simple shear tests. For Soils #1, #2 and #8, more than one testing condition was applied and all the results are presented in this figure. Two bands of data are evident in this figure. The upper band consists of all the data of undisturbed samples (Soils #1, #2, #3 and #4), which show non-dilative behaviour in undrained monotonic shear tests, while the lower band consists of the data of seabed silt and sands (Soils #5, #6, #7 and #8), which show dilative behaviour in undrained monotonic shear tests. The separation of dilative soils (the lower band) from non-dilative soils (the upper band) indicates the difficulties of normalising cyclic shear stress using peak strength measured in a monotonic test for a dilative soil.

The lower position of cyclic strength curves of dilative soils is due to the high peak strength of such soils mobilised at the end of monotonic tests (see Figures 5-9 and 5-16a). In Chapter 5, the phase transformation (PT) state, which separates the dilative phase from the contractive phase, has been observed to be unique for a specific soil
regardless of testing conditions. The PT state is reached at a shear strain $\gamma$ of about 5% for the calcareous silt and 2.5-5% for the calcareous sand. It was noted in Chapter 6 that large shear strains began to occur after the cyclic effective stress path had crossed the PT state line. Therefore, the shear stress mobilised at the PT state, $\tau_{PT}$, appears to be a more useful reference strength than the peak strength for dilative soils. The cyclic response of the calcareous silt and the calcareous sand discussed in Chapter 6 reveals that the PT state line is crossed at shear strains of about 5%, and this is followed by a rapid increase in shear strain. Therefore, total shear strain of 5% seems to be a better failure criterion for dilative soils.

In Figure 7-4, the cyclic shear stress normalised by reference shear stress ($\tau_{cyc}/\tau_{ref}$) is plotted against the number of cycles to reach a total shear strain of 5% ($N_5$). These are the same tests as shown in Figure 7-3. The reference shear stress, $\tau_{ref}$, here denotes $\tau_{PT}$ for dilative soils and $\tau_{peak}$ for non-dilative soils. A single band of data consisting of both dilative and non-dilative soils is now evident in this figure. The same set of data (and normalisation) is shown in Figure 7-5, but in this case the number of cycles to cause a total shear strain of 1% is plotted. A reasonably narrow band of data is again observed.

The cyclic strength curves shown in Figure 7-4 may be quantified by expressing normalised shear stress level ($\tau_{cyc}/\tau_{ref}$) as a function of the number of cycles ($N$) to failure, as follows:

$$\frac{\tau_{cyc}}{\tau_{ref}} = N_f^{-r}$$

where parameter $r$, which can be obtained through curve fitting, represents the rate of degradation in cyclic strength with increasing $N$. The best-fit line through the data corresponds to $r = 0.14$, and the data are bounded by lines with $r = 0.08$ and $r = 0.22$. Note that in this case the S-N curve is in terms of $\tau_{cyc}/\tau_{ref}$ versus $N$, i.e. S is the ratio of $\tau_{cyc}/\tau_{ref}$ rather than $\tau_{cyc}/\tau_{peak}$ as referred previously.

One of the advantages of this type of normalisation can be observed from the plots in Figures 7-6 and 7-7. Figure 7-6 shows a plot of $\tau_{cyc}/\sigma'_{vc}$ versus $N_5$ for the calcareous sand discussed in Chapter 6, while Figure 7-7 shows a plot of $\tau_{cyc}/\tau_{ref}$ versus $N_5$ for the
same soil. The scattered data in Figure 7-6 demonstrate that normalisation by consolidation stress can not eliminate the effects of different values of the state parameter (or the combined effects of void ratios and confining pressures) and consolidation conditions (CIU and CAU) on cyclic strength curves. However, as shown in Figure 7-7, normalising by using the $\tau_{PT}$, which is a characteristic strength in the monotonic test, produces a very narrow band of data with the exception of a single point with a high cyclic shear stress level. Similar results are reported by Hyodo et al. (1998), who used phase transformation strength $q_{PT}$ in undrained monotonic triaxial tests to normalise cyclic deviator stress $q_{cyc}$ and found that the effects of void ratio and confining stress on the cyclic strength of crushable soils subjected to undrained triaxial tests were eliminated.

The main points of this section can be summarised as follows:

1. Non-zero mean shear stress has a significant effect on the cyclic strength curves. The cyclic strength of a soil subjected to non-symmetric cyclic loading can not be compared directly with that in symmetric cyclic loading tests by use of the S-N curves.

2. The cyclic strength of a soil subjected to symmetric cyclic loading can be expressed by normalised cyclic shear stress $\tau_{cyc}/\sigma_{vc}'$ versus number of cycles $N$ to a specified shear strain $\gamma$. However, this expression makes it difficult to compare cyclic strength of different soils.

3. The best way to compare the cyclic strengths of different soils is to normalise cyclic shear stress by a reference strength which is the peak monotonic strength for contractive soil and the phase transformation strength for dilative soil. By using this normalisation, a unique cyclic strength curve is obtained for a specific soil and the cyclic strength degradation curve is characterised by a single parameter $r$. The cyclic strengths of eight calcareous sediments are compared using this method, and this showed that the parameter $r$ is in the range 0.08 to 0.22, which indicate that cyclic strength curves of these soils vary in a large range.

7.3 Effect of Mean Shear Stress: Equal Damage Contours

The cyclic strength of a soil in non-symmetric cyclic loading tests has been found not to be comparable with that in a symmetric cyclic test, as shown in Figures 7-1a and b. In this section, methods of taking the effects of the mean cyclic stress into account are
investigated. The aim is to determine how results from symmetric cyclic tests can be used to predict the behaviour in a non-symmetric test. The practical application of such a procedure would be that only symmetric tests would need to be carried out to fully characterise the cyclic strength of a particular soil.

7.3.1 Proposed Model

The patterns of accumulation of shear strain in tests with different combinations of \( \tau_m \) and \( \tau_{cyc} \) discussed in Chapter 6 are different, as illustrated in Figure 7-8. In the cyclic loading test with \( \tau_m = 0 \), failure is caused by accumulation of cyclic shear strain \( \gamma_{cyc} \) (Figure 7-8a). In contrast, in the cyclic loading test with \( \tau_m > \tau_{cyc} \), very little cyclic strain was induced, and failure results from the increasing mean shear strain \( \gamma_m \) (Figure 7-8d). Failure for these tests is usually defined as total shear strain \( \gamma_t \) (sum of cyclic shear strain amplitude \( \gamma_{cyc} \) and mean shear strain \( \gamma_m \)) reaching some value (e.g. Andersen 1976). In this case, the failure criterion is chosen as \( \gamma_t \) of 30 % in order to reflect complete features of shear strains (both \( \gamma_m \) and \( \gamma_{cyc} \)) in different types of cyclic loading tests.

As reported in Chapter 3, a series of cyclic simple shear tests was carried out on the calcareous muddy silt (floc samples), including tests with different \( \tau_{cyc}-\tau_m \) combinations, in order to investigate the concept of "equal damage contours" applied to this soil (ref. Figure 3-9). Figure 7-9 summarises these test results plotted in \( \tau_m-\tau_{cyc} \) space with both \( \tau_m \) and \( \tau_{cyc} \) normalised by monotonic shear strength \( \tau_{peak} \). Each symbol in this figure represents a single cyclic simple shear test. The number next to each symbol is the number of cycles to reach \( \gamma_t \) of 30 % in that test. The two existing models, the Goodman and Gerber models, are applied in this figure, represented by a series of straight dotted lines and a series of parabolic solid curves, respectively. It can be seen that the Gerber model matches the data in the lower region (where \( \tau_{eq}/\tau_{peak} < 0.5 \)) reasonably well, but overestimates the data in the upper region (where \( \tau_{eq}/\tau_{peak} > 0.5 \)). By contrast, the Goodman model matches the data in the upper region but underestimates the data in the lower region.

In Figure 7-9, the Gerber contours lie above the one-cycle failure line when \( \tau_{eq}/\tau_{peak} > 0.5 \), leading to overestimation in the upper region in the \( \tau_{cyc}/\tau_{peak}-\tau_m/\tau_{peak} \) space. Ideally,
all the test data points should be on or below this limit. However, due to the different rates at which monotonic and cyclic tests are usually performed, it is also possible for the cyclic data points to be above this one cycle failure line (e.g. Malek et al. 1989, McCarron et al. 1995). This phenomenon reflects the effect of strain rate on soil responses, particularly for clays. The occurrence of strain rate effects on soil behaviour has been reported by many researchers either in stress-controlled tests or strain-controlled tests (e.g. Wood 1982, Bjerrum 1973, Vucetic 1990). However, the data in Figure 7-9 all plot below the one cycle failure line.

In order to correct the overestimation by the Gerber model in the upper region in $\tau_{cyc}/\tau_{peak}-\tau_m/\tau_{peak}$ space, this model is modified by defining the equivalent shear stress in symmetric cyclic loading $\tau_{eq}$ as:

\[
\tau_{eq} = \frac{\tau_{cyc}}{1 - \left(\frac{\tau_m}{\tau_{peak}}\right)^a}
\]

where $a = \min\left[\frac{\tau_{peak}}{\tau_{eq}}; \frac{1}{2}\right]$

and $\tau_m$, $\tau_{cyc}$ and $\tau_{peak}$ are mean shear stress, cyclic shear stress and maximum monotonic shear strength, respectively. The parameter $a$ varies from 1 at the one-cycle-failure line to 2 after $\tau_{eq} \leq 0.5\tau_{peak}$. This equation provides a smooth transition from the Goodman model for $\tau_{eq} = \tau_{peak}$ ($a = 1$) to the Gerber model when $\tau_{eq} \leq 0.5\tau_{peak}$ ($a = 2$). Since $\tau_{eq}$ appears on both sides of this equation, it cannot be solved analytically, but is readily solved numerically.

The equal damage contours represented by the modified Gerber’s model are plotted in Figure 7-10, together with the same set of cyclic simple shear test data shown in Figure 7-9. It can be seen that the proposed model matches the data better than either model in Figure 7-9. The test data are plotted on an S-N fatigue diagram in Figure 7-11, in which the solid and the open circles in this figure represent the symmetric and non-symmetric tests respectively. Figure 7-11a reveals the S-N plots of symmetric and non-symmetric cyclic simple shear test results before equalisation. The same data are presented in Figure 7-11b but in this case, the non-symmetric tests have been converted into the
equivalent value for a symmetric test using the modified Gerber model. This equalisation process brings the sets of data quite close together.

7.3.2 Application of the Modified Gerber Model to Other Soils

The modified Gerber model is applied to the results of cyclic simple shear tests on two other calcareous soils, Soils #2 and #3, which have been discussed previously in Section 7.2.2.

Both symmetric and non-symmetric (with $\tau_m = \tau_{cyc}$) cyclic simple shear tests were conducted on the two soils. The results for Soil #2 are presented in Figure 7-12. Figure 7-12a summarises the test results in $\tau_{cyc}/\tau_{peak}$-$\tau_m/\tau_{peak}$ space, together with the equal damage contours of the modified Gerber model. The number next to each test denotes the number of cycles needed to reach $\gamma_t$ of 15%. The S-N curves are plotted in Figure 7-12b, with the non-symmetric test results converted to the equivalent symmetric S value using the modified Gerber equation. Similar plots for Soil #3 are shown in Figures 7-13a and b. It can be observed that the modified Gerber model is useful for these two soils.

7.3.3 Application of the Modified Gerber Model to Different Shear Strains

In the above discussion, the equal damage contours for the calcareous muddy silt are applied with the number of cycles corresponding to $\gamma_t$ of 30% (Soil #1), and 15% (Soils #2 and #3). In practice, a different failure criterion might be used. To investigate the applicability of the modified Gerber model to other shear strain levels, the S-N curves of Soil #1 with numbers of cycles to $\gamma_t$ of 15, 10, 5 and 1% are plotted in Figures 7-14a, b, c and d respectively.

It can be seen that as the total shear strain reduces, the scatter increases, indicating that the equalisation process does not work well at low strain levels. This is due to the fact that the overall shapes of the strain development curves differ for the different combinations of $\tau_m$ and $\tau_{cyc}$, as shown previously in Figure 7-8. Therefore, having defined the failure criterion as $\gamma_t$ of 30%, and having used this to derive an equalisation equation, this equation would not be suitable for other shear strain levels. However, in spite of this, the equation still provides a reasonable equalisation at $\gamma_t$ of 15%, and a
somewhat less successful equalisation at $\gamma_t$ of 10% and 5%. The equalisation does not work at all for $\gamma_t$ of 1%, mainly because this strain level is reached or exceeded in the first cycle in many cases, so no equalisation technique could work.

The non-applicability of this model to other shear strain levels implies that the parameter $a$ in Equation 7-3 is not a material constant (e.g. in Goodman and Gerber models) or fixed-type expression (e.g. in the Modified Gerber model), but is a function of different shear strain levels.

### 7.4 Pore Pressure Generation Model

Excess pore pressure generated in undrained cyclic loading is often used as an important damage criterion in offshore and seismic related foundation design. Some of the existing pore pressure models have been reviewed in Chapter 2. On the basis of the cyclic simple shear data for the calcareous muddy silt discussed in Chapter 6, a generalised pore pressure generation model is proposed, in which cyclic tests with symmetric and non-symmetric cyclic loading are considered. The proposed model is then applied to other soils.

#### 7.4.1 Proposed Pore Pressure Generation Model

Pore pressure generation curves measured in four cyclic simple shear tests with $\sigma'_{vc}$ of 150 kPa and different cyclic shear stress amplitudes are presented in Figure 7-15 for the calcareous muddy silt. The effects of cyclic shear stress amplitude on the pore pressure generation behaviour may be observed: the rate of generation of excess pore pressure increases with the cyclic shear stress amplitude, and as a consequence, the number of cycles to reach maximum excess pore pressure decreases with increase in cyclic shear stress amplitude. In order to make pore pressure curves comparable amongst different tests, some normalisation must be adopted.

Many researchers have normalised pore pressure by consolidation stress level ($\sigma'_{vc}$) when symmetric cyclic loading tests were studied (Bjerrum 1973, Seed et al. 1975, Datta et al. 1980). However, this type of normalisation is not suitable for non-symmetric cyclic loading tests. Considering the stress paths of the two tests illustrated schematically in Figure 7-16, the excess pore pressure in the non-symmetric test at failure $\Delta u_f$ is much less than the initial confining stress $\sigma'_{vc}$, while the excess pore
pressure in the symmetric test at failure is almost equal to the consolidation stress level. Therefore, the maximum pore pressure generated at failure in cyclic loading tests is used instead of the consolidation vertical effective stress in normalising pore pressures. This type of normalisation has been employed by Kagawa (1988) for normalising pore pressure responses in symmetric and non-symmetric cyclic triaxial tests on calcareous soils, and also used by Malek et al. (1989) in studying undrained simple shear behaviour of reconstituted Boston Blue clay. The number of cycles to failure $N_f$ is employed to normalise cycle number $N$ as used by Seed et al. (1975), Kagawa (1988) and Malek et al. (1989).

Since failure is defined as a certain level of total shear strain $\gamma_t$ (which is a combination of accumulated mean shear strain $\gamma_m$ and cyclic shear strain $\gamma_{cyc}$), in practice $\Delta u_f$ is determined as the value of $\Delta u$ corresponding to a given value of $\gamma_t$, and $N_f$ is chosen as the number of cycles required to reach this value of $\gamma_t$.

The normalised pore pressure curves of the four symmetric cyclic simple shear tests (which have been shown in Figure 7-15) are presented in Figure 7-17. In these plots, failure is defined as $\gamma_t = 30\%$ (i.e. $\Delta u_f$ and $N_f$ correspond to $\Delta u$ and $N$ at $\gamma_t = 30\%$). It can be seen that the normalisation process produces pore pressure generation curves for these tests that are practically identical.

The failure criterion can be chosen to be a different level of total shear strain. Figures 7-18 and 7-19 present the normalised pore pressure curves with failure criteria of $\gamma_t$ of 15 % and 5 % respectively. It can be observed from these figures that for these two failure criteria, the normalisation process works reasonably well for these tests. One exception is observed in Figure 7-19, in which the test with the highest cyclic shear stress amplitude does not merge with the other three owing to the fact that the test reached failure (here defined as $\gamma_t = 5\%$) at the 3rd cycle.

This normalisation method can also be applied to non-symmetric cyclic loading tests. The results of three tests with $\tau_m = 0$, $\tau_m = \tau_{cyc}$ and $\tau_m > \tau_{cyc}$ for the calcareous muddy silt are plotted in Figure 7-20 in terms of $\Delta u/\Delta u_f$ versus $N/N_f$. Three failure criteria used are $\gamma_t$ of 30 %, 15 % and 10 % in Figures 7-20a, b and c respectively. These figures
show that the curvatures of the normalised pore pressure generation curves increase with increase in $\tau_m$.

The effect of mean shear stress on the pore pressure generation behaviour may be re-examined through four groups of tests plotted in Figures 7-21a, b, c and d for the $\tau_m/\tau_{cyc}$ ratios of 0, 0.4, 1 and 2.5 respectively. In these plots both pore pressure and cycle number were normalised by the reference values $\Delta u_f$ and $N_f$ to $\gamma_f$ of 30%. At least two tests with different cyclic shear stress levels are included in each figure. It can be observed that the normalised pore pressure curves for the tests with the same $\tau_m/\tau_{cyc}$ ratio coincide with each other.

The pore pressure generation curves for symmetric and non-symmetric cyclic simple shear tests can be represented by the following equations:

\[
\frac{\Delta u}{\Delta u_f} = \left[ 1 - \left( 1 - \frac{N}{N_f} \right)^m \right]^{\theta_0} \tag{7-4a}
\]

\[
\theta = \theta_0 + p \left( \frac{\tau_m}{\tau_{cyc}} \right) \tag{7-4b}
\]

where: $m$, $\theta_0$ and $p$ are constants. Parameter $m$ controls the curvature of the end part of the pore pressure generation curve, parameter $\theta_0$ represents the initial slope of the curve for a symmetric cyclic loading test, and parameter $p$ controls the rate of increase in initial slope with increase in $\tau_m/\tau_{cyc}$ ratio. This equation assumes that the initial slope of the curve increases linearly with increase of $\tau_m/\tau_{cyc}$.

The parameters, $m$, $\theta_0$ and $p$, can be determined through one symmetric cyclic loading test and at least one but preferably two non-symmetric cyclic loading tests (i.e. where $\tau_m/\tau_{cyc} > 0$). Pore pressure curves obtained from Equation 7-4 are plotted as heavy dashed lines in Figure 7-21. For the test results in Figures 7-21a and c, the simulated curves are drawn by using Equation 7-4a with parameters, $m$ and $\theta$, determined by curve fitting. For the symmetric test results shown in Figure 7-21a, values of $m$ and $\theta_0$ ($= \theta$ for symmetric tests) are found to be 2 and 1.3 respectively. For the tests with $\tau_m/\tau_{cyc} = 1$ (Figure 7-21c), $m$ and $\theta$ are found to be 2 and 2.7. Hence, parameter $p$ can
be calculated as 1.4 from the relationship between \( \theta \) and \( \tau_m / \tau_{cyc} \) expressed by Equation 7-4b. Therefore, parameters \( m, \theta_0 \) and \( p \) for the calcareous muddy silt are determined as 2, 1.3 and 1.4 respectively. For the tests in Figures 7-21b and d, parameter \( \theta \) is calculated from \( \theta_0, p \) and \( \tau_m / \tau_{cyc} \) as 2 and 4.8 respectively, and the simulated pore pressure curves are drawn by using Equation 7-4a. It can be seen that the model gives a good representation of the pore pressure generation behaviour for the various values of \( \tau_m \). In order to determine parameter \( p \) with reasonable confidence, non-symmetric cyclic tests with two different \( \tau_m / \tau_{cyc} \) ratios are required.

Possible shapes of pore pressure generation curves that can be obtained from Equation 7-4 are plotted in Figure 7-22. The curves obtained using a range of different values of \( m \) with fixed \( \theta \) (\( \theta = 3 \) in this case) are plotted in Figure 7-22a, whilst the curves obtained using varying values of \( \theta \) with fixed \( m \) (\( m = 2 \) in this case) can be seen in Figure 7-22b. These figures cover a wide range of shapes, which could be encountered in the pore pressure generation behaviour of different soils.

7.4.2 Application of the Proposed Pore Pressure Model to Other Soils

The proposed pore pressure model developed using results for the calcareous muddy silt is now applied to simulate the pore pressure behaviour of several other calcareous soils. Various soils described in Section 7.2.2 were subjected to cyclic loading tests under non-symmetric cyclic loading. Soil #4 and Soil #5 have been subjected to non-symmetric cyclic loading tests with more than two levels of \( \tau_m / \tau_{cyc} \) ratio, as well as symmetric cyclic loading tests. Hence, the test results of these two soils can be used to evaluate the proposed pore pressure model.

Figure 7-23 presents the normalised pore pressure behaviour of Soil #4 subjected to undrained cyclic simple shear tests, with \( \gamma_f = 30 \% \) being taken as 'failure'. The \( \tau_m / \tau_{cyc} \) ratios for the tests in Figures 6-23a, b and c are 0, 1 and 3 respectively. Figure 7-24 shows the normalised pore pressure behaviour of Soil #5 in similar tests, again with \( \gamma_f \) taken to be 30 \%. In this case, the \( \tau_m / \tau_{cyc} \) ratios for the tests in Figures 7-24a, b, c and d are 0, 0.48 and 0.67 and 1.11 respectively. In these plots, a maximum of two test results are shown in each case for clarity, even though in some cases more than two tests had been conducted at the same \( \tau_m / \tau_{cyc} \) ratio. It can be seen that the tests with the same
\( \tau_m / \tau_{\text{cyc}} \) ratio follow the same trend, and the initial curvature of the normalised pore pressure curves increases with the increase in \( \tau_m / \tau_{\text{cyc}} \) ratio. In Figures 7-23 and 7-24, the values of the three model constants \( m, \theta_0 \) and \( p \) shown for each soil were determined using the method described above. The thick dashed line in each figure represented the model output. It can be observed that the model performs very well in reproducing the pore pressure behaviour for these two soils.

7.5 Simulation of the Cyclic Behaviour in Undrained Simple Shear Tests

Several empirical models for characterising different aspects of cyclic behaviour of calcareous soils in simple shear tests have been discussed in the preceding sections. In this section, these empirical models are assembled together with the strain hardening behaviour which has been discussed in Chapter 6, and a framework for simulating the behaviour of soils in undrained cyclic simple shear tests is then developed.

7.5.1 Framework of Simulating Cyclic Simple Shear Behaviour

In Chapter 6, it has been demonstrated that the hysteretic loops of \( \chi - \gamma \) curves for the three calcareous sediments can be described by a backbone curve, combined with two original and two extended Masing rules. The backbone curve can be represented by a hyperbolic model as described by Equation 6-1. According to the two original Masing rules as stated in Section 2.3.1, the stress ratio-shear strain curve in any unloading or reloading curve can be expressed by the following equation:

\[
\chi - \chi_0 = \frac{G_i^* (\gamma - \gamma_0)}{1 + \frac{G_i^* (\gamma - \gamma_0)}{\alpha \chi_f}}
\]

where \( \chi \) is stress ratio defined as \( \tau / \sigma' \), \( G_i^* \) is the initial gradient of the backbone curve (i.e. the initial \( \chi - \gamma \) curve), \( \chi_f \) is the asymptotic value of the stress ratio \( \chi \), \( \gamma_0 \) and \( \chi_0 \) are respectively the values of \( \gamma \) and \( \chi \) at the reversal point, and parameter \( \alpha \) equals 1 for initial loading curve and 2 for unloading / reloading curves. For the initial loading curve (i.e. before unloading/reloading), both \( \gamma_0 \) and \( \chi_0 \) are equal to zero and parameter \( \alpha \) equals 1, therefore, Equation 7-5 is identical to Equation 6-1. Parameters \( G_i^* \) and \( \chi_f \) can be determined through curve fitting.
Characterisation of Cyclic Responses

Equation 7-5 reveals that the relationship between \( \chi \) and \( \gamma \) in any unloading-reloading loop may be calculated if the stress reversal point in the loop can be determined. The schematic plot of a stress path of an undrained cyclic simple shear test shown in Figure 7-25 demonstrates that the stress ratio at the two reversal points (A and B) in the \( i^{th} \) cycle can be expressed by the following equations:

\[
\chi_{0A} = \frac{\left( \tau_m + \tau_{\text{cyc}} \right)}{\sigma'_{vc} - \Delta u_{iA}} \\
\chi_{0B} = \frac{\left( \tau_m - \tau_{\text{cyc}} \right)}{\sigma'_{vc} - \Delta u_{iB}}
\]

where \( \tau_m \) and \( \tau_{\text{cyc}} \) are mean shear stress and cyclic shear stress respectively, \( \sigma'_{vc} \) is the consolidation vertical effective stress, and \( \Delta u_{iA} \) and \( \Delta u_{iB} \) are the generated pore pressure corresponding to the two reversal points A and B in the \( i^{th} \) cycle.

The pore pressure generation behaviour of the three calcareous sediments discussed in Chapter 6 shows that before initial liquefaction occurs, the pore pressure in each cycle varies in a very small range, particularly for the tests with a large number of cycles to failure. It is reasonable to assume that the generated pore pressure in the \( i^{th} \) cycle can be represented by the average value in that cycle. Therefore, in Equation 7-6a and 7-6b, \( \Delta u_{iA} = \Delta u_{iB} = \Delta u_i \), where \( \Delta u_i \) is the average excess pore pressure in the \( i^{th} \) cycle, which can be calculated through the proposed pore pressure model expressed by Equation 7-4.

In the calculation of \( \Delta u_i \) by using Equation 7-4, the number of cycles to failure \( N_f \) and the generated pore pressure at failure \( \Delta u_f \) need to be determined. \( N_f \) can be calculated using Equation 7-2 for symmetric cyclic loading after parameter \( r \) being determined as described in Section 7.2.2. For non-symmetric cyclic loading, the equalised symmetric cyclic shear stress \( \tau_{eq} \) is first calculated using Equation 7-3. The number of cycles to failure is then determined by Equation 7-2.

From the schematic stress paths shown in Figure 7-16, it may be deduced that the maximum excess pore pressure \( \Delta u_f \) for an undrained cyclic simple shear test can be expressed as:

\[
\Delta u_f = \sigma'_{vc} - \frac{\left( \tau_m + \tau_{\text{cyc}} \right)}{\chi_{cs}}
\]
where $\sigma'_{vc}$ is the consolidation vertical pressure, and $\chi_{cs}$ is the slope of the CSL in $\tau-\sigma'_{y}$ space for a contractive soil or the slope of the PT line in $\tau-\sigma'_{y}$ space for a dilative soil. This definition of failure has been used by Malek et al. (1989) in studying undrained simple shear behaviour of reconstituted Boston Blue clay.

It must be pointed out that the determination of $\Delta u_f$ and $N_f$ are dependent on different failure criteria. The criterion in calculating $\Delta u_f$ is that the stress path reaches the CSL or PT line, while the criterion for calculating $N_f$ is the point on the S-N curve corresponding to a certain level of total shear strain. In order to make the failure criterion for determining $N_f$ consistent with that of $\Delta u_f$, the shear strain level in the S-N curves should be chosen as that when the CSL or PT line is touched in $\tau-\sigma'_{y}$ space. It has been observed in Chapter 6 that the PT line (for dilative soils) or the CSL (for contractive soil) is reached at about $\gamma_t$ of 5% for symmetric cyclic loading in simple shear tests. Therefore, 5% total shear strain is chosen as the failure criterion in determining $N_f$ in this modelling.

After $\Delta u_f$ and $N_f$ have been determined, the reversal stress ratio $\chi_0$ corresponding to the $i^{th}$ cycle can be calculated through Equation 7-6. The reversal shear strain $\gamma_0$ may then be calculated using Equation 7-5. Following the two original and two extended Masing rules, the hysteretic loops of the $\chi-\gamma$ response in a cyclic simple shear test can, therefore, be simulated.

Since it is assumed that the pore pressure generated in the $i^{th}$ cycle can be represented by the average value of the generated pore pressure, the effective vertical stress $\sigma'_v$ is also assumed to be constant for the $i^{th}$ cycle. Using the relationship between $\tau$ and $\sigma'_v$ (i.e. $\tau = \chi \sigma'_v$) the shear stress $\tau$ can also be worked out, and hence the shear stress-strain behaviour can be obtained.

The framework for simulating cyclic simple shear behaviour, which consists of empirical models described by Equations 7-2 to 7-7, is depicted schematically in Figure 7-26. These equations are summarised in Table 7-2, and the parameters used in these equations are listed. The physical meaning of each parameter has been explained when the corresponding model was introduced, and is also listed in this Table. The determination of these input parameters is based on curve fitting in each case.
7.5.2 Application of the Framework to Simulation of Cyclic Simple Shear Behaviour

Table 7-3 lists input parameters for the calcareous muddy silt, which were determined on the basis of the results of the tests on this soil discussed in Chapters 5 and 6. By using these input parameters, the framework is applied to simulate a symmetric cyclic simple shear test on the calcareous muddy silt under $\sigma'_{vc} = 150$ kPa and $\tau_{cyc} = 20$ kPa. The simulated results are shown in Figure 7-27. Comparing the simulated with the measured behaviour shown in Figure 6-1, it can be seen that the simulated results capture the general responses of the soil. The stress path moves to the critical state line, and the stress-strain curve shows degradation behaviour. The simulated shear strain development and pore pressure generation curves reproduce the typical response of the soil in undrained symmetric cyclic simple shear tests.

Figures 7-28, 7-29 and 7-30 present simulated behaviour of the calcareous muddy silt subjected to non-symmetric cyclic loading simple shear tests with three conditions: (1) $\tau_m < \tau_{cyc}$, (2) $\tau_m = \tau_{cyc}$, and (3) $\tau_m > \tau_{cyc}$ respectively. Comparing the simulated results with the measured behaviour in the corresponding tests (Figures 6-8, 6-10 and 6-11), it can be seen that these simulations capture well the effect of $\tau_m$ on the behaviour in respect of stress path, shear strain development and pore pressure generation.

Table 7-4 lists the input parameters for the calcareous sand determined on the basis of the test data of this soil discussed in previous chapters. These parameters are used to simulate the behaviour of the calcareous sand subjected to a symmetric cyclic loading simple shear test under $\sigma'_{vc} = 75$ kPa and $\tau_{cyc} = 20$ kPa. The simulated responses are presented in Figures 7-31. Through comparison of the simulated with the measured behaviour (Figure 6-20), it can be seen that the general behaviour of this soil in symmetric cyclic loading is also captured well by the simulations. It can be also observed that the shear strains (Figure 7-31d) develop at a very low rate in the early stages of the test followed by a rapid increase in rate, which typically represents the cyclic behaviour of this sand, which is different from that of the muddy silt (Figure 7-27d).

The limitation of this empirical method can be observed in the simulated stress paths (e.g. Figures 7-27a and 7-31a) and stress-strain curves (e.g. Figures 7-27b and 7-31b). The simulated stress paths do not provide the 'butterfly'-shaped stress paths observed in
the behaviour of the three soils (Figures 6-1a, 6-12a and 6-20a) and do not capture the strain softening response observed in the stress-strain relationships shown in Figures 6-1b, 6-12b and 6-20b. This limitation is attributed to the simplification that the average value of pore pressure in a cycle is used in the calculation of the reverse stress ratio. However, this is only a problem after initial liquefaction occurs (see Figure 6-12 for example).

7.6 Conclusions

This chapter has demonstrated empirical methods of characterising the behaviour of a calcareous soil in undrained cyclic simple shear tests, and has applied these methods successfully to several other calcareous soils. Since these soils are reasonably representative of the range of (uncemented) calcareous soils encountered in offshore areas (of Australia and probably elsewhere), these methods are likely to be applicable to all such soils. The following conclusions are obtained.

1. Cyclic strength of a soil can be expressed as the normalised cyclic shear stress ($\tau_{\text{cyc}}/\sigma_{ve}^\prime$) required to failure after a specific number of cycles. However, it is difficult to compare cyclic strengths of different soils by using this expression. In order to compare cyclic strengths of different soils, an S-N curve, in which $S$ is the cyclic shear stress normalised by a reference monotonic strength (i.e. $S = \tau_{\text{cyc}}/\tau_{\text{ref}}$) is used. This reference monotonic strength is the peak strength for contractive soils and the phase transformation strength for dilative soils. In this way, cyclic strengths of eight calcareous sediments are compared, and a reasonably narrow band of cyclic strength curves is obtained.

2. The general method used to compare a non-symmetric loading test with a symmetric cyclic loading test is to introduce equal damage contours in $\tau_{\text{cyc}}-\tau_m$ space. The widely used models, the Goodman and Gerber models, are found not suitable for the behaviour of the calcareous muddy silt. The Gerber model is, therefore, modified. The modified Gerber model shows good performance in applications to other two calcareous soils (Soils #2 and #3). However, this model is found to be not applicable for small levels of shear strains.

3. A new pore pressure generation model has been developed for both symmetric and non-symmetric cyclic simple shear tests. The model shows good performance in application to other two calcareous soils (Soils #4 and #5).

4. A framework for simulating cyclic simple shear behaviour has been proposed, which consists of several empirical relationships for characterising cyclic strength,
equal damage contours, pore pressure generation and hysteretic stress-strain behaviour. Application of this framework to simulating the behaviour of both the calcareous muddy silt and sand show that the basic behaviour of the two soils in symmetric and non-symmetric cyclic simple shear tests can be replicated reasonably well before initial liquefaction is reached. After initial liquefaction, the framework fails to predict the 'butterfly'-shaped stress paths and strain softening behaviour observed in a stress-strain curve.
8 Evaluation of a Constitutive Model for Three Calcareous Sediments

8.1 Introduction

In this chapter, an effective stress constitutive soil model, the MIT-S1 model, which is able to simulate the rate independent monotonic behaviour of clays and sands, is evaluated for simulating the behaviour of the three calcareous soils discussed in the preceding chapters after the model has been implemented into a computer program. In the first part of this chapter, the formulation of the MIT-S1 model, and the model input parameters are described. The model is then applied to simulate the monotonic behaviour of the three calcareous soils in triaxial and simple shear tests. For each soil, the selection of model input parameters is described, followed by a comparison of model predictions with measured data. Finally, the ability of the MIT-S1 model to predict cyclic behaviour is examined.

8.2 Description of the MIT-S1 Model

The MIT-S1 model, which is based on incrementally linearised elasto-plasticity, was developed by Pestana and Whittle (1999) at the Massachusetts Institute of Technology (MIT). This model is able to predict the rate independent, effective stress-strain-strength monotonic behaviour of uncemented clays and sands over a wide range of confining pressures and densities. The model and model-input parameters are described briefly in the following.

8.2.1 Description of the MIT-S1 Model

The MIT-S1 model consists of a general framework of elasto-plasticity, including yield surface, hardening laws, flow rules, and consistency conditions. The framework of incrementally linearised elasto-plasticity is described fully in Appendix 1. The MIT-S1 model, as well as many other constitutive models, can fit within this framework. What distinguishes one model from another is the specification of the components such as yield surface, plastic potential surface and hardening of the yield surface. The details of the mathematical formulation and evaluation of the MIT-S1 model can be found in
several references (Pestana 1994, Pestana & Whittle 1999a, b and c), the specific features of which are briefly described in the following.

(a) Concept of Limiting Compression Curve and Unification of the behaviour of Clays and Sands

It is well known that the compression behaviour of clays and sands are significantly different. The compression curve of a normally consolidated clay in e-log $p'$ space is represented by a unique straight line, the Normal Compression Line (NCL), in the stress range of engineering interest, while the compression behaviour of a freshly deposited sand depends on its formation density (or void ratio) and fabric. The compression behaviour of sands is modelled by Pestana and Whittle (1995) through a four parameter compression model in a double logarithmic space of void ratio and mean effective stress ($\log e - \log p'$). The conceptual compression curves of a freshly deposited sand are depicted in Figure 8-1. Specimens of a given sand, which are compressed from different initial formation void ratios, converge to a unique void ratio-effective stress curve which is referred to as a Limiting Compression Curve (LCC). The LCC is linear in log $e$-log $p'$ space. The NCL of a reconstituted clay specimen is linear in e-log $p'$ and can also be well characterised by a linear relationship in log $e$-log $p'$ space. Thus, the NCL is qualitatively similar to the LCC.

After being normalised by effective confining pressure, shear stiffness and strength properties of sands in LCC regime are similar, i.e. normalised stress paths and normalised stress versus strain curves will come together. The behaviour of sands in the LCC regime is qualitatively similar to the well-known behaviour of normally consolidated clays, which show normalised (shearing) behaviour along the NCL. This is the basic principle of the MIT-S1 model used in unifying the behaviour of clays and sands. At lower confining pressures for sands, the model characterises the effects of formation density on the shear behaviour of sands through a number of key features included in the formation of the constitutive equations (e.g. the yield surface function, and the hardening of the yield surface). These features enable the MIT-S1 model to describe the characteristic transition from dilative to contractive shear response of sands as the confining pressure increases.
(b) Elastic Components and Hysteretic Equations

The MIT-S1 model does not have a region of true elastic behaviour. The response immediately after a load reversal is controlled by the small strain elastic bulk modulus $K_{\text{max}}$ and shear modulus $G_{\text{max}}$. The non-linear hysteretic response is modelled by relating the isotropic tangent moduli to the most recent stress reversal state.

(c) Yield Surface and Failure Condition

The model assumes that the plastic response during shearing is controlled by a separate yield function, while the strength at large strain is described by a critical state failure criterion. Conditions of maximum shear stress and peak friction angle are controlled by the size, shape and orientation of the yield surface, while large strain (critical state) conditions are considered to be independent of previous stress history and density. The inclusion of void ratio as a state variable in the description of the yield surface is the main conceptual difference between the MIT-S1 model and other effective stress models. Critical state failure conditions are represented by an isotropic function of the form proposed by Matsuoka and Nakai (1974) defining a conical surface in effective stress space.

(d) Hardening Laws

The MIT-S1 model assumes kinematic hardening laws including density and rotational hardening, which are described by changes in the size and orientation of the yield surface. The hardening laws describe the two important characteristics of clays and sands: (1) the normalised behaviour of clays and sands in the LCC regime, and (2) the transition from dilative to contractive behaviour as a function of confining pressure (non-normalised behaviour). The evolution of anisotropic properties is controlled by the rotational hardening, while the compression behaviour is accounted for by the density hardening.

(e) Flow Rules (Plastic Potential Surface)

Plastic strains on the yield surface are described by a non-associated flow rule that satisfies two constraints: (1) shearing at critical state causes no further change in volume, and (2) one dimensional consolidation of normally consolidated soils in the LCC regime is characterised by the measured stress ratio $K_{\text{ONC}}$. The directions of plastic flow are controlled by a second order tensor. Instead of an explicit plastic
potential function in effective stress space, the volumetric and deviatoric components of the plastic potential are directly defined.

(f) Bounding Surface Formulation

The concepts of bounding surface plasticity (Dafalias & Herrmann 1982) are incorporated in the MIT-S1 model to define plastic strains for over-consolidated soils. The yield function acts as the bounding surface, while the plastic strains of the over-consolidated soil are linked to the behaviour at an image point on this surface through a radial-mapping rule with a loading surface of the same shape as this surface. The plastic strains for the over-consolidated material are related to the plastic behaviour experienced by the normally consolidated material. Both the flow direction and the plastic modulus at the stress point are related to those on the bounding surface through specific mapping functions, which have been developed based on the observed behaviour of over-consolidated clays. The bounding surface formulation used in the MIT-S1 model is outlined in Appendix 2.

8.2.2 Input Parameters of the Model

The MIT-S1 model requires a total of 16 input parameters, with 13 of these being for clays and 14 for sands. Table 8-1 lists the parameters, the physical meaning of each parameter and the laboratory tests from which the values of these parameters are determined. It can be seen in Table 8-1 that there are two parameters (D, r) used only in predicting the behaviour of clays, while three parameters (σ', θ, p) are used only for sands.

Procedures for determining the input parameters have been recommended by Pestana & Whittle (1999a). Four types of tests are required, (1) isotropic (or one-dimensional) compression and swelling tests, (2) one-dimensional swelling tests, (3) triaxial shearing tests, and (4) methods for measuring the maximum shear modulus G\text{max} (e.g. wave velocity, bender element, small strain measurement in a shearing test, etc.). Some of the parameters can be measured directly, while others have to be determined by parametric study. The selection of model input parameters for the three calcareous sediments is explained in detail in the following sections.
8.3 Evaluation of the MIT-S1 Model for the Calcareous Muddy Silt

In this section, the MIT-S1 model is evaluated for predicting the monotonic behaviour of the calcareous muddy silt, which has been discussed in Chapter 5. The input parameters are first determined, and the model predictions are then compared with the measured behaviour for the soil in both triaxial and simple shear tests.

8.3.1 Determination of Input Parameters for the Calcareous Muddy Silt

The parameters for the calcareous muddy silt were determined using the methods described by Pestana and Whittle (1999a). Due to the fact that the compression and shear behaviour of the calcareous muddy silt are similar to those of soft clays, the required parameters are those in the clay category in Table 8-1. Table 8-2 summarises the 13 input parameters for the calcareous muddy silt, and their determination procedures are described as follows.

1. The slope of the Limiting Compression Curve (LCC) in log e-log p' space, \( p_c \), can be determined through compression curves. As explained in Section 8.2.1, for soil in the clay category, the LCC is equivalent to the Normal Compression Line (NCL). Several one-dimensional compression tests were performed using oedometer equipment for both undisturbed and floc samples of the calcareous muddy silt. From the compression curve, \( p_c \) was found to be 0.17.

2. The coefficient of lateral earth pressure at rest for soils in the NCL regime, \( K_{ONC} \), can be measured directly during \( K_0 \)-consolidation in a triaxial test on floc samples. No lateral strain was allowed in this test by adjusting the cell pressure to maintain axial strain equal to volumetric strain (i.e. to maintain zero average lateral strain). The value of \( K_{ONC} \) was found to be 0.4. Following initial consolidation, the swelling part of the test was used to obtain Poisson’s ratio \( \mu'_0 \) and parameter \( \omega \) (describing non-linear variation of the elastic Poisson’s ratio accompanying changes in the stress ratio) by matching the predicted and measured \( K_0 \)-OCR curve as shown in Figure 8-2. The parameters \( \mu'_0 \) and \( \omega \) were determined as 0.2 and 0.5 respectively.

3. Parameters \( D \) and \( r \), which describe the non-linearity in the volumetric response for clays, were determined as 0.024 and 0.33 by analysing the non-linear volumetric
response for OCR increasing from 1 to 10 in a one-dimensional swelling test (Figure 8-3).

4. Parameter $h$ accounts for the amount of residual plastic strain observed in hydrostatic and one-dimensional unload-reload cycles. A hydrostatic swelling test was performed on a floc sample. By comparing the calculated and measured irrecoverable volumetric strains using three different magnitudes of unload-reload cycles, the parameter $h$ was found to be 2.

5. Parameter $C_b$ defines the elastic bulk modulus at small strain levels (i.e. $K_{\text{max}}$ at stress reversal). The determination of $C_b$ requires measuring the maximum shear modulus $G_{\text{max}}$ under different consolidation stress levels. Since there was no measurement of $G_{\text{max}}$ on floc samples of the calcareous muddy silt, the values of $G_{\text{max}}$ that were measured on the undisturbed samples of the calcareous muddy silt, in a project for the Gorgon site investigation, were used for floc samples. $C_b$ is estimated as 500 from the measurement of the maximum shear modulus $G_{\text{max}}$ and the value of Poisson’s ratio $\mu'_0$.

6. The friction angle at critical state $\phi'_{cs}$ was determined as $42^\circ$ from the results of undrained triaxial compression tests on both undisturbed and floc samples of the calcareous muddy silt.

7. The parameters $\phi'_m$, $m$, $\omega_s$ and $\psi$ could not be directly measured but needed parametric study of an undrained triaxial compression and an extension test on normally consolidated soil. By comparing the undrained stress paths of a $K_0$-consolidated soil in triaxial compression and extension modes of shearing, the parameters $\phi'_m$ and $m$ were estimated as 80 and 0.8 respectively (Figure 8-4a). Parameter $\omega_s$ was determined by matching the predicted stress-strain curves for triaxial tests at small strains ($\varepsilon_a < 0.1\%$), with the value of $\omega_s$ of 6 obtained using this process (Figure 8-4b). The parameter $\psi$ was evaluated by simulating the shear induced pore pressures or stress-strain curves at $\varepsilon_a = 5$ to $10\%$, and a value of $\psi$ of 5 gave the predicted pore pressure responses that are plotted together with the measured responses in Figure 8-4c.
8.3.2 Predictions of the Undrained Shear Behaviour of the Calcareous Muddy Silt

(a) Undrained CAU Triaxial Behaviour

Figures 8-5 compares the predicted and measured effective stress paths, shear stress-strain response and pore pressure responses of the anisotropically consolidated calcareous muddy silt in undrained triaxial compression and extension tests with overconsolidation ratios OCR of 1, 2, 4 and 7 (OCR > 1 for compression tests only). The model predictions are represented by thick lines, while the measured data are represented by thin lines. Good agreement is expected between the predicted and measured effective stress paths of both the compression test with OCR = 1, and the extension test, since these two tests are part of the database from which the input parameters were determined. For the compression tests with OCR = 2, 4, and 7, the predicted effective stress paths are also in good agreement with the measured data (Figure 8-5a). For OCR ≤ 2, the predicted stress-strain responses agree very well with the measured data (Figure 8-5b). For OCR = 4 and 7, it seems that the model underestimates the mobilised deviator stress when the axial strain is greater than 0.8%. The shear induced pore pressure responses (Figure 8-5c) are under-estimated for OCR = 2, but over-estimated for OCR = 4 and 7.

(b) Undrained CIU Triaxial Behaviour

Undrained CIU triaxial tests were conducted on the floc samples of the calcareous muddy silt prepared by using the one-dimensional compression method (K₀-consolidation) as described in Chapter 4. This method inevitably leads to a fabric structure that is controlled by the K₀-consolidation. In MIT-S1 model, this fabric is described by a yield surface that is oriented in the K₀ direction. However, when samples experience one-dimensional swelling, followed by re-consolidation along the hydrostatic axis (to a larger p'), the model does not “remember” the anisotropic fabric formed during the initial one-dimensional compression. Therefore, it is assumed here that the orientation of the initial yield surface for a CIU test prior shearing is along the hydrostatic axis.

Figure 8-6 compares the model predictions for undrained triaxial compression tests on isotropically consolidated calcareous muddy silt with an initial OCR of 1 and 2 with measured data. Initially, predictions of effective stress paths are in good agreement
with measured data (Figure 8-6a), however, the model does not predict the dilative behaviour observed in the measured data. Figure 8-6b shows that the predicted stress-strain responses for the two tests is not in good agreement with measured results, which is probably due to the assumption in this calculation that the orientation of initial yield surface is isotropic, whilst anisotropic fabric exists in the tested samples. The shear induced pore pressure responses are slightly over-estimated (Figure 8-6c). Overall, however, the general trends in the behaviour of the soil in CIU triaxial compression tests are captured by the model.

(c) Undrained Simple Shear Behaviour

The predicted and measured behaviour of the calcareous muddy silt in undrained simple shear tests are plotted in Figure 8-7. It can be seen that the model over-estimates the mobilised shear stress at shear strain less than 1 %, and under-estimates it at shear strain greater than 1 % (Figure 8-7a). The predicted effective stress path lies below that measured (Figure 8-7b). In general the model predicts the basic trends for the simple shear tests reasonably well.

Overall, the model captures the general trend of undrained triaxial and simple shear behaviour of calcareous muddy silt well.

8.4 Evaluation of the MIT-S1 Model for the Calcareous Sand

This section evaluates the MIT-S1 model for predicting the undrained monotonic behaviour of the calcareous sand discussed in Chapter 5. The input parameters are first determined, and the model predictions are then compared with the measured behaviour for the calcareous sand in both triaxial and simple shear tests.

8.4.1 Determination of Input Parameters for the Calcareous Sand

As listed in Table 8-1, the MIT-S1 model uses 14 parameters to characterise the behaviour of a freshly deposited sand. These parameters can be subdivided into compression and shear input parameters according to their physical description. Table 8-3 summarises the 14 input parameters for the calcareous sand. The determination procedures are described as follows.
(a) Compression Input Parameters: $C_b$, $\rho_c$, $p'_r$, and $\theta$

The four compression parameters $C_b$, $\rho_c$, $p'_r$, and $\theta$ are used by the MIT-S1 model to characterise the hydrostatic (and one-dimensional) compression behaviour of freshly deposited cohesionless soils. Apart from the elastic stiffness $C_b$, the parameters, $\rho_c$, $p'_r$ and $\theta$ can be determined by isotropic (or one-dimensional) compression curves in log e-log $p'$ space.

1. Parameter $C_b$ defines the elastic stiffness, which controls the elastic bulk modulus at stress reversal (i.e. $K_{max}$). Pestana & Whittle (1995) reported that the value of $C_b$ is in the range 700-1000 for most sands. A value of 900 was used for $C_b$ in evaluating the compression behaviour of a calcareous sand (Dog's Bay sand) by Pestana and Whittle (1995). Owing to the lack of appropriate test data for the calcareous sand studied in this thesis, $C_b$ is chosen as 900.

2. The compression behaviour of the sand was investigated through a one-dimensional compression test performed with Constant Strain Rate (CSR). A small loading rate of 0.2 mm/min was used in this test in order to eliminate the effect of strain rate on the compression behaviour. Figure 8-8 presents the test result in log e-log $\sigma'_v$ space. According to Pestana and Whittle (1995), the slope of the LCC, $\rho_c$, and the reference stress in one-dimensional compression, $\sigma'_{vr}$, are determined respectively as 0.43 and 4800 kPa from the data shown in this figure.

Pestana (1994) suggests that isotropic and one-dimensional compressions are qualitatively similar although with some quantitative differences in the measured compression behaviour. The only compression parameter affected by one-dimensional compression is the reference stress ($p'_r$). Based on compression results of Dog's Bay Sand (Coop 1990; quoted by Pestana 1994), Pestana found that the relationship between $p'_r$ and $\sigma'_{vr}$ is as follow:

$$p'_r = \frac{3}{(1 + 2K_{onC})}\sigma'_{vr}$$

Pestana (1994) also pointed out that there is insufficient high quality data to establish the generality and limitations of this equation. However, in this case, this equation has been used to determine $p'_r$. The lateral stress ratio at rest for normally
3. The parameter $\theta$, which characterises the elasto-plastic transition, was estimated by matching measured data with predicted data in Figure 8-8 using the compression model proposed by Pestana & Whittle (1995). The parameter $\theta$ is determined as 0.5.

(b) Shear Input Parameters

Excepting the four compression parameters, the remaining parameters governing the stress-strain response during shearing are determined as follows.

1. The friction angle at critical state $\phi'_{cs}$ is determined from the results of three undrained triaxial compression tests. The critical state (or steady state) condition of sand mass is defined by Poulos (1981) as 'continuously deforming at constant volume, constant normal effective stress, constant shear stress, and constant velocity'. It is difficult to achieve true critical state for sands in undrained triaxial and simple shear tests. However, the value of $\phi'_{cs}$ can be estimated satisfactorily (within 1°) from the mobilised friction angle at large strain (> 20%) in drained and undrained tests. In this case the value of $\phi'_{cs}$ is determined as 39.4°.

2. The lateral earth pressure ratio at rest $K_{ONC}$ in the LCC regime can be determined from a one-dimensional compression test in the LCC regime in either triaxial or oedometer apparatus with measurement of lateral stress. These tests were not performed for the calcareous sand due to limitations of the equipment. The values of $K_{ONC}$ for a calcareous sand (Dog's Bay Sand) and a silica sand (Ham River Sand) at elevated stress level (i.e. in LCC regime) are reported by Coop (1993) as 0.51 and 0.57 respectively. In the absence of data, it is assumed that $K_{ONC} = 0.49$ for this calcareous sand (also see Section 5.6.2).

3. Parameter $\mu'_0$ represents the Poisson's ratio immediately after load reversal and controls the ratio of elastic shear modulus to bulk modulus at small strains (i.e. $G_{max}/K_{max}$). Pestana (1994) reported that the expected range of $\mu'_0$ is narrow with
typical values, $\mu'_b = 0.2-0.25$, for both freshly deposited sands and clays. In view of the lack of data for the calcareous sand it is assumed that $\mu'_b = 0.2$.

4. The parameter $\omega$ describes the variation of the Poisson’s ratio with changes in the stress ratio $\eta (= q/p')$. Common values of $\omega$ are found to be $\omega = 0.5-1.5$ for both clays and sands (Pestana 1994). In view of the lack of data, the value of $\omega$ is chosen as the average value of 1.

5. The parameter $\omega_s$ describes the small strain non-linearity in shearing and is evaluated through the degradation of the secant shear modulus with strain level (for $\gamma < 0.1 \%$).

Ideally, high-precision internal strain measurement is required to define this behaviour accurately. However, no tests of this type were carried out on this soil. In the case of three CIU triaxial tests on this soil (ref. Figure 5-16a), external strain measurement was used, and there is some evidence of “end effects” in these measurement. This can be observed in the stress-strain curves in Figure 8-9, in which the initial flat part (in the range of $\varepsilon_a = 0-0.1 \%$) in each test indicates “end-effects” in the three CIU triaxial tests.

The stress-strain curves of two CAU simple shear tests (no CAU triaxial data are available) are plotted in Figures 8-10a and b, together with the model predictions using an $\omega_s$ value of 2. The result is a slight under-prediction for the test with $\sigma'_{vc} = 75$ kPa and a slight over-prediction for the test with $\sigma'_{vc} = 300$ kPa. Therefore, the parameter $\omega_s$ was chosen as 2 for the calcareous sand. Attention must be drawn to the fact that in the process of evaluating this parameter, the values of four unknown parameters $m, \phi'_m, p$ and $\psi$ (describing plastic strain response) had to be assumed. However, it has been found that changes in the four parameters had negligible effect on the stress-strain curves at small strain level.

6. Parameter $m$ describes the slenderness of the yield surface and controls the shape of the undrained effective stress paths. By matching measured and model predicted effective stress paths, the parameter $m$ is determined as 0.6.

7. Parameter $\psi$ is a dimensionless constant that controls the rate of rotation of the yield surface, and hence the evolution of the anisotropic properties. It controls the
transition part from contractive to dilative behaviour in effective stress paths and has no effect on the initial and end parts of effective stress paths. By matching the predicted and measured effective stress paths of undrained CIU triaxial tests, the parameter \( \psi \) was determined as 50.

8. Parameters \( \phi'_{mr} \) and \( p \) describing the maximum friction angle as a function of the formation void ratio can be estimated through two undrained triaxial compression tests with two different void ratios. By use of the calculation procedures reported by Pestana (1994), \( \phi'_{mr} \) and \( p \) are determined as 47.7° and 0.61 respectively.

9. The parameter \( h \) controls the amount of residual plastic strain observed in isotropic and one-dimensional unload-reload cycles. It is not required for simulating the behaviour of the freshly deposited sand.

8.4.2 Prediction of the Behaviour of Calcareous Sand

(a) Undrained CIU Triaxial Behaviour

Figure 8-11a compares the predicted and measured effective stress paths, stress-strain behaviour and pore pressure response of the calcareous sand in undrained monotonic triaxial tests with three different consolidation pressures \( p'_c = 75, 500 \) and 1500 kPa. The effective stress paths of the test with \( p'_c = 1500 \) kPa were used in determining parameters \( m \) and \( \psi \), while the results of two tests \( p'_c = 75 \) and 1500 kPa were used in determining \( \phi'_{mr} \) and \( p \). The prediction for the test with \( p'_c = 500 \) kPa is a true prediction. The model describes the effect of confining pressures very well. Strongly dilative behaviour is simulated at low confining pressure. As the confining pressure increases, the stress paths of contractive behaviour followed by dilative behaviour is simulated. The predicted stress-strain curves are also in agreement with measured data for the three tests (Figure 8-11b). The model tends to slightly over-predict deviator stresses at low strain levels (\( \varepsilon_a < 1 \) %) and slightly under-predict for intermediate strain levels. Predicted pore pressure behaviour for the three tests is compared with measured data in Figure 8-11c, in which very good agreement is observed.

(b) Undrained Simple Shear Behaviour

Figure 8-12 compares the predicted and measured behaviour of the calcareous sand in CIU simple shear tests conducted under initial confining pressures of 75 and 300 kPa.
The predicted behaviour in CAU simple shear tests under two consolidation vertical pressures ($\sigma'_v$) of 75 and 300 kPa with consolidation stress ratio $\sigma_{hc}/\sigma'_v = 0.4$ are compared with the measured data in Figure 8-13. Overall, the predicted effective stress paths and stress-strain behaviour are qualitatively in good agreement with measured data. The model describes very well the transformation phase from contractive to dilative behaviour. The effects of confining pressures and anisotropic consolidation conditions (CIU or CAU) on the stress paths are also well simulated by the model (compare Figures 8-12a and 8-13a). Predicted effective stress paths of both CIU and CAU tests with higher confining pressures ($\sigma'_{vc} = 300$ kPa) exhibit less dilative behaviour than observed in the tests. The trend of stress-shear strain behaviour is in agreement with the measured results. The effects of consolidation pressures on the shear stress-strain behaviour are reflected in the plots, although the model tends to under-predict the effects (compare stress-strain curves in Figure 8-12b and 8-13b).

Overall, good agreement is observed in the comparison of predicted and measured behaviour of the calcareous sand in undrained monotonic triaxial and simple shear tests.

8.5 Evaluation of the MIT-S1 Model for the Calcareous Silt

The monotonic behaviour of the calcareous silt shows a dilative response in both undrained triaxial and simple shear tests (Figures 5-7 and 5-13a), indicating that the shear behaviour of the silt is in the sand category. The compression curves of the silt, discussed in Section 5.6.2, show evidence of the effect of formation void ratios on compression behaviour (Figure 5-27). However, the compression curves of the silt in log e-log p' space (Figure 8-14) indicate that there is no clear LCC regime. However, the compression curves of the calcareous silt with different formation void ratio converge together at a vertical effective pressure of 100 MPa. As mentioned in Section 8.2.1, the LCC regime plays an important part in the MIT-S1 model for simulating behaviour from contraction to dilation. In order to apply this model to the calcareous silt, the line tangent to the converging point (see Figure 8-14) is assumed to be a pseudo-LCC for the silt.

The 14 input parameters for the calcareous silt are listed in Table 8-4. The following section explains the procedures for determining these parameters.
8.5.1 Determination of the Input Parameters for the Calcareous Silt

(a) Compression Input Parameters: $C_b$, $p_c$, $p'_c$, and $\theta$

1. For the parameter $C_b$, Pestana & Whittle (1995) reported a range of 700-1000 for most sands and a range of 400-500 for most clays. In the absence of test data, $C_b$ is selected as 600, a value that falls between those recommended for clays and sands.

2. From the compression results shown in Figure 8-14, the slope of the pseudo-LCC, $p_c$, is determined as 0.4, and the reference stress for one-dimensional compression ($\sigma'_{vr}$) is determined as 7000. In determining the reference stress in isotropic compression ($p'_r$), the coefficient of lateral earth pressure $K_{0NC}$ of 0.45 (see below for the determination) is used. The parameter $p'_r$ can then be calculated from Equation 8-1 and is determined as 4430.

3. The elasto-plastic transition region in the compression curves of the silt (Figure 8-14) almost covers the whole stress range being tested. This feature cannot be simulated by use of the four-parameter compression model (Pestana & Whittle 1995), from which the parameter $\theta$ is determined. Thus, $\theta$ could not be determined through matching predicted and measured compression behaviour. In this evaluation, $\theta$ is estimated through parametric study. By matching predicted and measured shearing behaviour in a triaxial compression test, this parameter was found to be 0.15.

(b) Shear Input Parameters

1. The friction angle at critical state $\phi'_{cs}$ is estimated as 41° from the mobilised friction angle at large strain ($\varepsilon_a > 20 \%$) in undrained triaxial monotonic tests shown in Figure 5-7.

2. The coefficient of lateral earth pressure $K_{0NC}$ in the LCC regime can be determined from a one-dimensional compression test conducted in a triaxial apparatus with measurement of lateral stress. However, it is impractical to conduct triaxial tests in the LCC regime for this soil since this regime is not reached even at effective vertical stress of 100 MPa. In the low stress level ($p'_c = 150$ kPa), $K_{0NC}$ is determined as 0.45. In the absence of test data, $K_{0NC}$ is chosen as 0.45.
3. The typical values $u_0 = 0.2-0.25$ and $\omega = 0.5-1.5$ for both clays and sands are reported by Pestana (1994). In the absence of data for the calcareous silt, it is assumed that $u_0 = 0.2$ and $\omega = 1$.

4. The parameter $\omega_s$ is evaluated as 6 by matching predicted and measured stress-strain curves at small axial strain level in an undrained monotonic triaxial compression test.

5. By matching the predicted and measured effective stress paths of an undrained triaxial test, the parameters $m$ and $\psi$ are determined to be 0.4 and 20 respectively.

6. Parameters $\phi_{mr}$ and $p$ are determined through two undrained triaxial compression tests with two different void ratios. By use of the calculation procedures proposed by Pestana (1994), $\phi_{mr}$ and $p$ are determined to be $48.7^\circ$ and 0.02 respectively.

7. The parameter $h$ is not required for simulating the behaviour of normally consolidated silt.

8.5.2 Prediction of the Behaviour of the Calcareous Silt

Figure 8-15 compares the predicted and measured stress paths, stress-strain behaviour and excess pore pressure of the calcareous silt in a CIU triaxial compression test. The model simulations capture the basic trend of the stress path (Figure 8-15a) though some discrepancies exist in the contraction region. The basic trend of the stress-strain behaviour is well predicted, with initial peak strength being over-predicted (Figure 8-15b). Very good agreement is observed between predicted and measured excess pore pressure shown in Figure 8-15c.

Figures 8-16 compares the predicted and measured data for the calcareous silt in CAU triaxial compression and extension tests. The initial peak strength in the compression test is not captured by the model. However, the basic trend of the compression stress path is simulated. Some discrepancies exist in the simulated and measured stress paths of extension tests, especially in the area of transition from contractive to dilative behaviour. For the compression test (Figure 8-16b), the model tends to under-predict the deviator stress of small strain and over-predict in the large strain region. For the extension test, the model tends to over-estimate the strength. Although the overall
trends of excess pore pressure are simulated by the model, discrepancies exist between the predicted and the measured data, especially in the extension test.

Figure 8-17 compares the predicted and measured stress paths, shear stress-strain and excess pore pressure-strain behaviour of the calcareous silt in a CAU simple shear test. Good agreement is observed in the stress paths in the contraction region, with a discrepancy in the dilation region. The model tends to over-predict deviator stresses in the range of small to medium shear strain ($\gamma < 5\%$) and under-estimate them in the large shear strain region ($\gamma > 5\%$). The overall excess pore pressure behaviour simulated by the model is good, with slight over-predictions in the whole region.

8.6 Examination of the MIT-S1 Model Capability for Predicting Cyclic Behaviour

The ability of the MIT-S1 model to simulate the undrained monotonic behaviour of the three calcareous soils has been demonstrated in the previous sections. This section investigates the ability of the model to simulate undrained cyclic behaviour. Section 8.2.1 described how the model incorporates the concept of a bounding surface plasticity and the hysteretic formulation for calculation of the bulk and shear moduli. The bounding surface plasticity (see Appendix 2) and the hysteretic formulation together provide a theoretical basis for simulating behaviour in reverse loading. In order to examine how the model behaves in simulating undrained cyclic loading, the model has been implemented into a cyclic loading program, and applied to simulating undrained cyclic behaviour.

The mapping functions (see Equations A2-3 and A2-4) require the sizes of three surfaces: bounding surface ($\alpha$), initial loading surface ($\alpha_{oi}$), and current loading surface ($\alpha_0$). In order to satisfying the conditions: $0 \leq g_1 \leq 1$ and $0 \leq g_2$, required by the mapping rules (Equations A2-1 and A2-2), the size of current loading surface ($\alpha$) should always be greater than that of the initial loading surface ($\alpha_{oi}$). This is satisfied for over-consolidated behaviour. However, it was found that, in simulating undrained cyclic behaviour of normally consolidated soil, at some stage after unloading, $\alpha_0$ is found to be greater than $\alpha_{oi}$. Therefore, in the simulation of cyclic behaviour by using the original mapping rules, $\alpha_{oi}$ is set to equal to $\alpha_0$ arbitrarily.
Figure 8-18 presents the stress path, stress-strain behaviour and pore pressure generation predicted by the model in a symmetric undrained cyclic triaxial test. It can be seen that the pore pressure stabilises after about 10 cycles (Figure 8-18b) and hence the mean effective stress also stabilises the stress path plot (Figure 8-18a). Different cyclic stress amplitudes have been tried and a similar behaviour is predicted. The observed stabilisation is caused by the mapping rules, which are developed based on the observed behaviour of over-consolidated clays. After several cycles, elasto-plastic modulus $H$ becomes significantly large by using these mapping rules. Therefore the original mapping rules are not suitable for simulating cyclic behaviour.

In order to simulate cyclic behaviour, the mapping rules used in the MIT-S1 model have been modified in the following two aspects: (1) The flow direction, $P$, of the stress point is set equal to that of the image point on the bounding surface (i.e. in Appendix 2, Equations A2-1 and A2-5 are no longer necessary in the new mapping rules); (2) The elasto-plastic modulus, $H$, of the stress point is related to that of the image point on the bounding surface by the following equations:

$$H = \left( H^I \right) + H^0 g_1 \left( 1 - \frac{\eta \cdot \eta}{c^2} \right)^{1/2}$$  \hspace{1cm} 8-2

where $H^I$ is the elasto-plastic modulus at image point; $H^0$ defines the transition to LCC; $\eta$ is the stress ratio vector defined as the ratio of deviator stress vector $q$ to mean effective stress $p'$; $c$ is a scalar parameter accounting for the effect of maximum friction angle on the yield surface; $<>$ are Macaulay brackets; $g_1$ is a mapping function that is expressed as:

$$g_1 = \frac{\alpha' - \alpha'_0}{\alpha'}$$  \hspace{1cm} 8-3

where $\alpha'$, $\alpha'_0$ are the sizes of the bounding surface and the current loading surface respectively (see Figure A2-1 in Appendix 2). Except for the above two equations, all other terms in the original equations (as presented in Appendix 2) remain unchanged.

Figure 8-19 presents a predicted undrained symmetric cyclic CIU simple shear test with cyclic stress $\tau_{\text{cyc}}$ of 10 kPa and initial consolidation condition of $\sigma'_{\text{vc}} = 90$ kPa. It may be seen that the overall behaviour, such as decreasing vertical effective stress, degradation of shear stress-shear strain behaviour, accumulation of shear strain, and
generation of pore pressure, are well simulated in a qualitative sense. Figure 8-20 illustrates the predicted non-symmetric cyclic simple shear test cycling in the range of 0-10 kPa. Again the general behaviour of non-symmetric undrained cyclic simple shear tests is qualitatively well reproduced in this simulation. The model is also applied to simulate cyclic behaviour in undrained cyclic triaxial tests. Figures 8-21 and 8-22 represent respectively the simulated results for symmetric and non-symmetric cyclic CIU triaxial tests ($\sigma'_vc = 90$ kPa). It can be seen that the model performs qualitatively well in these simulations.

However, the above simulated stress paths for both triaxial and simple shear tests do not reproduce the observed 'butterfly'-shaped stress path and strain softening behaviour, which have been observed in the stress paths and stress-strain of the three calcareous soils (see Chapter 6) and other soils (Hyodo et al. 1998). This is due to an intrinsic drawback of operating the bounding surface plasticity in the deviator stress - mean effective stress plane (or $q$-$p'$ plane for triaxial loading) as discussed in Chapter 2.

### 8.7 Conclusions

This chapter has evaluated the MIT-S1 model with respect to its capability of simulating the behaviour of the three calcareous sediments. The following conclusions are drawn from the above discussions.

1. The model describes well the stress-strain-strength properties of both normally consolidated and over-consolidated (OCR $\leq 7$) samples of the calcareous muddy silt in undrained monotonic triaxial tests. The model also gives reasonable predictions for undrained monotonic simple shear tests.

2. The model gives good predictions for the undrained triaxial behaviour of the calcareous sand isotropically consolidated under a wide range of confining pressures. The undrained simple shear responses predicted by the model also show good agreement with measured results.

3. Application of this model to the calcareous silt encounters difficulties due to no clear LCC regime being observed in compression curves. A pseudo-LCC regime is assumed to be the line tangent to the compression curves at the convergence point. Based on this assumption, the MIT-S1 model has been applied to simulation of the monotonic behaviour of the calcareous silt. Comparisons between predicted and measured triaxial and simple shear behaviour show that the model tends to capture the basic trend of the behaviour in both triaxial and simple shear tests.
4. The model predictive capability for cyclic loading has been investigated. The model shows poor prediction of cyclic behaviour when the original mapping functions are used. These were developed on the basis of the observed behaviour of over-consolidated clays. By using modified mapping functions, the model predicts the behaviour of the calcareous muddy silt qualitatively well for both undrained cyclic triaxial and simple shear tests with either symmetric or non-symmetric loading. However, the model fails to predict the 'butterfly'-shaped stress paths in both undrained triaxial and simple shear tests.
9
Summary, Conclusions and Recommendations

9.1 Summary

This thesis has investigated the monotonic and cyclic behaviour of three un cemented calcareous soils, namely calcareous muddy silt, silt and sand. The cyclic behaviour has then been characterised using empirical methods. A constitutive model has been evaluated for simulation of the monotonic behaviour of the three soils. The main components of this thesis include the following:

1. Review of methods for characterising the behaviour of soils under undrained monotonic and cyclic loading (Chapter 2).

2. Comprehensive testing programmes for the three calcareous soils, which include a series of undrained triaxial and simple shear tests with the aim of investigating the fundamental behaviour of these three calcareous sediments (Chapter 3).

3. A method of preparation of a reconstituted aragonite soil using a new technique, which produces reconstituted samples with the same void ratio as natural undisturbed samples and with the same behaviour in monotonic triaxial and simple shear tests as well as cyclic simple shear tests as the natural undisturbed samples (Chapter 4).

4. Presentation of the behaviour of the three individual calcareous soils in undrained monotonic triaxial and simple shear tests and unification of the different responses of the three soils in the light of critical state soil mechanics (Chapter 5).

5. Presentation of the cyclic responses of the three calcareous soils in undrained simple shear tests, which show the similarity of the cyclic responses of the three soils even though their monotonic responses are quite different (Chapter 6).

6. Characterisation of the undrained cyclic responses of the three calcareous soils through a framework consisting of empirical models describing cyclic strength, effect of mean stress level, pore pressure generation, and shear strain hardening behaviour (Chapter 7).
7. Evaluation of a constitutive soil model (MIT-S1) for the monotonic responses of the three calcareous sediments and some efforts toward application of this model in predicting cyclic behaviour (Chapter 8).

9.2 Method of Preparation Reconstituted Aragonite Soils

A method of preparing reconstituted aragonite soil (the calcareous muddy silt) samples has been developed in such a way that the reconstituted samples possess similar void ratios to those found at considerable depths in undisturbed samples of this soil. The method consists of three major steps: mixing soil slurry with flocculant solution, consolidation in a consolidometer and curing in hot water. The mechanical behaviour (i.e. compression and shear behaviour) of the soil reconstituted using this process, has been found to be almost identical to that of undisturbed samples of the soil. The microstructures of the two samples are also very similar. Thus, the method can be used for this particular soil to produce reconstituted samples with consistent properties that are practically identical to those of the natural structured material.

9.3 Undrained Behaviour of Three Calcareous Sediments

9.3.1 Undrained Monotonic Behaviour

- The monotonic responses of the three soils are significantly different with respect to the effective stress paths and stress-strain responses. The behaviour of the calcareous muddy silt is much like that of soft clays in that a strain softening response is observed in CAU tests and when stresses are normalised by consolidation stress the stress paths and stress-strain curves are very similar. The behaviour of calcareous sand, depending on both void ratio and confining pressure, reveals a similarity to that of non-calcareous sand. The behaviour of the calcareous silt is similar to that of non-calcareous silt and it depends on both void ratios and confining pressures.

- A unique critical state line is found in q-p' space regardless of testing conditions and shear modes for the calcareous muddy silt. The phase transformation (PT) state line is also found to be unique for the calcareous silt and sand regardless of soil state, testing conditions and shear modes (i.e. triaxial or simple shear tests).
• Although the three calcareous soils present different stress-strain responses, similar stress ratio-shear strain responses are observed, which show strain hardening responses and can be represented by a simple relationship (e.g. hyperbolic-type relationship).

• The behaviour of the three calcareous soils has been analysed through the framework of critical state soil mechanics. Like non-calcareous soils, the undrained shearing behaviour of the three calcareous soils could be explained by use of the state parameter $\psi$ and initial stress anisotropy (or consolidation stress ratio $\sigma'_{vd}/\sigma'_{vc}$).

9.3.2 Undrained Cyclic Behaviour

• Similar undrained cyclic responses are observed for the three soils with respect to effective stress path, stress-strain behaviour, stress ratio-shear strain behaviour, pore pressure generation and shear strain development.

• Failure in undrained cyclic simple shear tests is defined as when a certain level of total shear strain is reached. In symmetric cyclic tests, shear strains develop symmetrically. The critical state (for contractive soil) or phase transformation state (for dilative soil) marks a transition point in shear strain development at which the rate of shear strain development changes dramatically. In non-symmetric cyclic tests, the total shear strain can be divided into mean shear strain and cyclic shear strain. As mean shear stress increases, shear strains are dominated by mean shear strain.

• In symmetric cyclic tests, the shapes of pore pressure generation curves corresponding to different cyclic shear stress levels are similar. In non-symmetric cyclic tests, as observed in the tests on the calcareous muddy silt, the curvature of the pore pressure generation curve increases with the increase of mean shear stress.

• The general cyclic responses in CIU and CAU simple shear tests are very similar. The only difference is that at the same consolidation vertical stress and same cyclic stress amplitude, the number of cycles to failure in a CIU test is greater than that in a CAU test.

• The cyclic responses (normalised by $\sigma'_{vc}$) of the calcareous sand subjected to undrained simple shear tests under two consolidation effective vertical stresses of 75 and 300 kPa are very similar.
• The PT\text{cyc} state surface separates stress space into contractive and dilative phases for the undrained cyclic loading test. The PT\text{cyc} state surface is unique for a particular soil in spite of the soil states, consolidation stress ratios, loading conditions (symmetric and non-symmetric), shearing modes (triaxial or simple shear). The PT\text{cyc} state surface is a function of the intrinsic properties of a soil.

• Strain hardening is found in the relationships of shear stress ratio-shear strain ($\tau/\sigma' - \gamma$) for the three soils subjected to undrained cyclic simple shear tests. The $\tau/\sigma' - \gamma$ responses of the three soils can be described using a backbone curve, which is a hyperbolic function, and two original and the first extended Masing rules which describe the hysteretic loops. The second extended Massing rule cannot be proved due to lack of irregular cyclic loading test data.

9.4 Empirical Models for Characterising Cyclic Responses of the Calcareous Sediments

• Cyclic strengths of soils can be expressed as the plot of cyclic shear stress normalised by consolidation stress level versus number of cycles to a certain level of strain. In order to compare cyclic strengths of different soils, a reference monotonic strength is used to normalise cyclic stress. This reference strength is the peak strength for contractive soils and the phase transformation strength for dilative soils. In this way, cyclic strengths of different calcareous sediments are compared and a band of cyclic strength curves is obtained.

• The modified Gerber model gives good performance in equalising symmetric and non-symmetric cyclic simple shear tests on the calcareous muddy silt. Application of the modified Gerber model to the other two soils shows good prediction. However, this model is found to be unsuitable for small levels of shear strains.

• The new empirical pore pressure generation model developed for both symmetric and non-symmetric cyclic simple shear tests on the calcareous muddy silt shows good performance in applications to two other calcareous soils.

• A framework for simulating cyclic simple shear behaviour has been proposed, which consists of several empirical relationships for characterising cyclic strength, equal damage contours, pore pressure generation and hysteretic stress ratio-shear strain behaviour. Application of this framework in simulating the behaviour of both
the calcareous muddy silt and sand show that the basic behaviour of the two soils in symmetric and non-symmetric cyclic simple shear tests is replicated qualitatively well before initial liquefaction is reached. After initial liquefaction, the framework fails to predict the ‘butterfly’-shaped stress paths and strain softening behaviour observed in the stress-strain curves.

9.5 Evaluation of a Constitutive Model for the Three Calcareous Sediments

A constitutive model (MIT-S1) has been evaluated for simulating the behaviour of the three calcareous sediments studied in this thesis. The following conclusions are drawn.

- The model describes well the stress-strain-strength properties of both normally consolidated and over-consolidated (OCR \(\leq 7\)) samples of the calcareous muddy silt in undrained monotonic triaxial tests. The model also gives reasonable predictions for undrained simple shear tests.

- The model gives good prediction for undrained triaxial behaviour of the calcareous sand isotropically consolidated under a wide range of confining pressures. The predicted undrained simple shear responses also show good agreement with measured results.

- Application of this model to the calcareous silt encounters difficulties due to no clear LCC regime being observed in compression curves of this soil. A pseudo-LCC line is assumed to be the line tangent to the compression curves at the point of convergence. Based on this assumption, the MIT-S1 model is applied to simulating the monotonic behaviour of the calcareous silt. Comparisons between predicted and measured triaxial and simple shear behaviour show that the model tends to capture the basic trend of the behaviour in both triaxial and simple shear tests.

- The model predictive capability for cyclic loading has been investigated. The model shows poor prediction of cyclic behaviour when the original mapping functions are used; these were developed on the basis of the observed behaviour of over-consolidated clays. With the aid of modified mapping functions, the model predicts the behaviour of the calcareous muddy silt qualitatively well for both undrained cyclic triaxial and simple shear tests with either symmetric or non-symmetric loading. However, the model fails to predict the ‘butterfly’-shaped stress paths in both undrained triaxial and simple shear tests.
9.6 Comparison with Conventional Soils

The behaviour of the three calcareous soils has been compared with that of conventional soils in Chapters 5 and 6. The friction angles for the calcareous muddy silt, silt and sand are 42°, 41° and 39.4° respectively, which are greater than those for conventional soils (21.8°-26° for clays (Schofield and Wroth 1968) and 30°-37° for sands (Bolton 1986). Higher initial void ratio and compressibility are also observed for the calcareous muddy silt comparing to conventional silt. However, Chapter 5 has shown that the undrained monotonic responses of the calcareous muddy silt, silt and sand are respectively similar to those of soft clays, conventional silts and sands. The undrained cyclic simple shear responses of the three calcareous soils discussed in Chapter 6 are also qualitatively similar to those of conventional soils. In addition, the framework of Critical State Soil Mechanics performs well in explaining the behaviour of the three calcareous soils, and a constitutive model, which has been successfully applied to conventional soils (clays and sands), can be applied successfully in prediction of the responses of the three calcareous soils. Therefore, the behaviour of the three calcareous soils is qualitatively similar to that of conventional soils.

9.7 Recommendations for Future Research

9.7.1 Behaviour of Calcareous Soils

(a) Monotonic Behaviour

Undrained monotonic behaviour of the three calcareous soils has been investigated through a series of monotonic triaxial and simple shear tests in Chapter 5. These three soils, from fine-grained muddy silt to coarse sand, show quite different shearing behaviour in their undrained monotonic loading. The framework of critical state soil mechanics is applicable to these soils. However, the drained monotonic behaviour is not included in this thesis and is scarcely investigated in the past for the calcareous soils. In order to fully understand the monotonic shearing behaviour of calcareous soils, the following aspects need to be investigated. Drained monotonic shearing (triaxial and simple shear) tests are required, the effects of density and sample preparation method should be considered, and the behaviour of over-consolidated samples (especially for silts and sands) should also be investigated.
(b) Cyclic Behaviour

Undrained cyclic behaviour of the three soils has been investigated through a series of cyclic simple shear tests in Chapter 6. From undrained cyclic tests, $PT_{cyc}$ state is found to be a soil intrinsic property and located below the $PT$ state associated with monotonic loading. It is worthwhile to investigate the $PT_{cyc}$ state in drained cyclic loading tests. In order to fully characterise the cyclic behaviour of the calcareous soils, the following aspects are needed. The cyclic behaviour in undrained triaxial tests is required to be examined. The behaviour of these soils in drained cyclic loading should be linked with their behaviour in undrained cyclic loading; i.e. volumetric response in drained tests linked with the pore pressure generation behaviour in undrained tests. The effects of density and sample preparation method on the cyclic behaviour should also be considered.

9.7.2 Modelling Cyclic Behaviour of Soils

(a) Empirical models

Chapter 7 presents a series of empirical models for characterisation of cyclic behaviour of calcareous soils. The modified Gerber model for equalising non-symmetric and symmetric cyclic loading works well at high total shear strain levels, but has been found to be less applicable as the total shear strain decreases. In order to apply this method to all total shear strain levels, the parameter $a$ in Equation 7-3 should be modified and made a function of total shear strain.

The stress ratio-shear strain response, which is a key aspect for simulating cyclic behaviour, is characterised on the basis of two assumptions: (1) the backbone curve is unaffected by loading rate, and (2) hysteretic loops obey the two original and two extended Masing rules. However, Chapter 6 has given evidence that the backbone curve is affected by loading rate. In order to accurately simulate cyclic behaviour, the backbone curve should be considered to be a function of loading rate.

The empirical methods for simulating undrained cyclic behaviour are based on the undrained cyclic simple shear tests, but not for cyclic triaxial tests where the response curves are quite different. Extension of the empirical approach to cyclic triaxial tests is also an area for future work.
(b) Constitutive models

After modification of the mapping functions in the bounding surface plasticity in the MIT-S1 model, the model predicts the behaviour in undrained cyclic loading qualitatively well as shown in Section 8.6. However, it has been found that the model failed to simulate the 'butterfly'-shaped stress paths and strain softening behaviour in stress-strain curves. This is mainly attributed to the manipulation of bounding surface plasticity in s-p' space (here s is deviator stress tensor) and the apices of the loading and bounding surfaces fixed to the origin of stress space.

A bounding-surface hypoplasticity model for sand proposed by Wang et al. (1990) shows the capability of simulating 'butterfly'-shaped stress path. Later, Li et al. (1999) modified this model to incorporate the basic concepts of critical state soil mechanics and make this model cover both loose and dense sand behaviour. The modified bounding-surface hypoplasticity model presents the capability of simulating both the 'butterfly'-shaped stress paths and strain softening in stress-strain response for sand in three-dimensional stress space. In the future, these models are worthwhile to be applied to simulation of cyclic behaviour especially after initial liquefaction occurs. However, the PT line is used in these models for simulating the dilative behaviour in both monotonic and cyclic loading. Therefore these models are only suitable for the behaviour of sands. In order to simulate the 'butterfly'-shaped stress path for clays (or muddy silt, which shows behaviour similar to clays), the PT_{cyc} line should be incorporated into this model and this should be an area for future research.
References


References


References


References

conference on engineering for calcareous sediments, Bahrain, K.A. Al-Shafei (ed), A.A. Balkema, Rotterdam, Brookfield, 1, 87-100.


References


for soils subject to cyclic loading. Géotechnique 31, No. 4, 451-469.

(eds), John Wiley and Sons, Chichester, U.K., 1984, 415-449.


'tress-reversal surface'. Geomechanical Modelling in Engineering Practice. R.
Dunger & J.A. Studer (eds), Balkema, Rotterdam, 351-367.

Chapter 1, Cyclic Loading of Soil, M.P. O'Reilly and S.F. Brown (eds), Blackie,
Glasgow, 1-18.

Geomechanical Modelling in Engineering Practice. R. Dunger & J.A. Studer (eds),
Balkema, Rotterdam, 369-395.

shear conditions. Journal of the Soil Mechanics and Foundations Division, ASCE.

Department of Civil & Environmental Engineering, Massachusetts Institute of

Géotechnique 45, No. 4, 611-631.

and sands. International Journal for Numerical and Analytical Methods in
Geomechanics, 23, 1215-1243.

clays and sands: I - sand behaviour. International Journal for Numerical and
Analytical Methods in Geomechanics, submitted.

clays and sands: II - clay behaviour. International Journal for Numerical and
Analytical Methods in Geomechanics, submitted.
References


Table 3-1a Summary of undrained monotonic triaxial tests on the calcareous muddy silt

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test Type</th>
<th>OCR</th>
<th>Void Ratio After Consolidation</th>
<th>$\sigma'<em>{hc}/\sigma'</em>{vc}$</th>
<th>$\sigma'_{vc}$ (kPa)</th>
<th>$\sigma'_{hc}$ (kPa)</th>
<th>$p'_c$ (kPa)</th>
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<td>true $K_0$ (2)</td>
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1 $p'_c = (\sigma'_{vc}+2\sigma'_{hc})/3$ is mean effective consolidation stress
2 true $K_0$ means one-dimensional consolidation condition
3 Representing measured value

Table 3-1b Summary of undrained monotonic simple shear tests on the calcareous muddy silt

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<th>Test No.</th>
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<th>$\sigma'<em>{hc}/\sigma'</em>{vc}$</th>
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<th>$p'_c$ (kPa)</th>
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<td>150</td>
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Table 3-1c Summary of undrained cyclic simple shear tests on the calcareous muddy silt

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<th>$\sigma_{hc}$ (kPa)</th>
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Table 3-2a Summary of undrained monotonic triaxial tests on the calcareous silt

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<tr>
<th>Test No.</th>
<th>Test Type</th>
<th>Void Ratio After Consolidation</th>
<th>$\sigma'<em>{hc}/\sigma'</em>{vc}$</th>
<th>$\sigma'_{vc}$ (kPa)</th>
<th>$\sigma'_{hc}$ (kPa)</th>
<th>$p'_c$ (kPa)</th>
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$^1$ Extension test

Table 3-2b Summary of undrained monotonic and cyclic simple shear tests on the calcareous silt

<table>
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<th>Test Type</th>
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<th>$\sigma'_{vc}$ (kPa)</th>
<th>$\sigma'_{hc}$ (kPa)</th>
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<th>$\tau_m/\tau_{cyc}$</th>
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</thead>
<tbody>
<tr>
<td>OsiltSS01</td>
<td>M</td>
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<td>150</td>
<td>60</td>
<td>0</td>
<td>18</td>
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<tr>
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<td>21</td>
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</tr>
<tr>
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<td>150</td>
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<td>24</td>
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<td>27</td>
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<td>33</td>
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<tr>
<td>OsiltSS09</td>
<td>C</td>
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<td>150</td>
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<td>10</td>
<td>21</td>
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<td>20</td>
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<td>OsiltSS11</td>
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<td>15</td>
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<td>20</td>
<td>1</td>
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<td>20</td>
<td>18</td>
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<tr>
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</table>
Table 3-3a Summary of undrained monotonic triaxial tests on the calcareous sand

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test Type</th>
<th>Void Ratio After Consolidation</th>
<th>$\sigma_{hc}/\sigma_{vc}$</th>
<th>$\sigma'_vc$ (kPa)</th>
<th>$\sigma'_hc$ (kPa)</th>
<th>$p'_c$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SandTX01</td>
<td>M</td>
<td>1.05</td>
<td>1</td>
<td>75</td>
<td>75</td>
<td>75</td>
</tr>
<tr>
<td>SandTX02</td>
<td>M</td>
<td>0.9</td>
<td>1</td>
<td>500</td>
<td>500</td>
<td>500</td>
</tr>
<tr>
<td>SandTX03</td>
<td>M</td>
<td>0.76</td>
<td>1</td>
<td>1500</td>
<td>1500</td>
<td>1500</td>
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Table 3-3b Summary of undrained monotonic simple shear tests on the calcareous sand

<table>
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<tr>
<th>Test No.</th>
<th>Test Type</th>
<th>Void Ratio After Consolidation</th>
<th>$\sigma_{hc}/\sigma_{vc}$</th>
<th>$\sigma'_vc$ (kPa)</th>
<th>$\sigma'_hc$ (kPa)</th>
<th>$p'_c$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SandSS01</td>
<td>M</td>
<td>1.10</td>
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<td>75</td>
<td>75</td>
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<td>SandSS02</td>
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<td>300</td>
<td>300</td>
</tr>
<tr>
<td>SandSS03</td>
<td>M</td>
<td>1.15</td>
<td>0.4</td>
<td>75</td>
<td>30</td>
<td>45</td>
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<tr>
<td>SandSS04</td>
<td>M</td>
<td>1.05</td>
<td>0.4</td>
<td>300</td>
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<td>180</td>
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</table>
Table 3-3c Summary of undrained cyclic simple shear tests on the calcareous sand

<table>
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<tr>
<th>Test No.</th>
<th>Test Type</th>
<th>$\sigma'<em>{hc}/\sigma'</em>{vc}$</th>
<th>$\sigma_{vc}$ (kPa)</th>
<th>$\sigma_{hc}$ (kPa)</th>
<th>$\tau_m$ (kPa)</th>
<th>$\tau_{cyc}$ (kPa)</th>
<th>$\tau_{cyc}/\sigma'_{vc}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>SandSS05</td>
<td>C</td>
<td>0.4</td>
<td>75</td>
<td>30</td>
<td>0</td>
<td>11.25</td>
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<tr>
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<td>C</td>
<td>0.4</td>
<td>75</td>
<td>30</td>
<td>0</td>
<td>15</td>
<td>0.20</td>
</tr>
<tr>
<td>SandSS07</td>
<td>C</td>
<td>0.4</td>
<td>75</td>
<td>30</td>
<td>0</td>
<td>20</td>
<td>0.27</td>
</tr>
<tr>
<td>SandSS08</td>
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<td>75</td>
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<td>0</td>
<td>30</td>
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<tr>
<td>SandSS09</td>
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<td>75</td>
<td>75</td>
<td>0</td>
<td>12.5</td>
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<td>75</td>
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<td>17.5</td>
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<td>75</td>
<td>0</td>
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</tr>
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<td>300</td>
<td>120</td>
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<td>0.10</td>
</tr>
<tr>
<td>SandSS13</td>
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<td>0.20</td>
</tr>
<tr>
<td>SandSS15</td>
<td>C</td>
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<td>120</td>
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<td>80</td>
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<td>0</td>
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<td>0.17</td>
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<td>SandSS17</td>
<td>C</td>
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<td>300</td>
<td>0</td>
<td>70</td>
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<tr>
<td>SandSS18</td>
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<td>300</td>
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Table 4-1 Details of simple shear tests on floe soil

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test Type</th>
<th>Initial Void Ratio</th>
<th>Void Ratio After Consolidation</th>
<th>$\sigma'_v_c$ (kPa)</th>
<th>$\sigma'_h_c$ (kPa)</th>
<th>$\tau_{peak}$ (kPa)</th>
<th>$\tau_{cyc}$ (kPa)</th>
<th>$N_{15}^{(1)}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C</td>
<td>1.75</td>
<td>1.49</td>
<td>150</td>
<td>60</td>
<td>20</td>
<td>6157</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>C</td>
<td>1.77</td>
<td>1.57</td>
<td>150</td>
<td>60</td>
<td>26</td>
<td>660</td>
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</tr>
<tr>
<td>3</td>
<td>M</td>
<td>1.61</td>
<td>1.45</td>
<td>150</td>
<td>60</td>
<td>57</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>C</td>
<td>1.60</td>
<td>1.49</td>
<td>150</td>
<td>60</td>
<td>33</td>
<td>42</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>M</td>
<td>1.61</td>
<td>1.46</td>
<td>150</td>
<td>60</td>
<td>54</td>
<td>1</td>
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</tbody>
</table>

$^{(1)}$ Number of cycles to total shear strain of 15%

Table 4-2 Compression Parameters of four samples

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<th></th>
<th>$\lambda$</th>
<th>$\kappa$</th>
<th>$\lambda / \kappa$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undisturbed, sample 1</td>
<td>0.17</td>
<td>0.016</td>
<td>10.6</td>
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<tr>
<td>Undisturbed, sample 2</td>
<td>0.22</td>
<td>0.016</td>
<td>13.8</td>
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<tr>
<td>Floc soil</td>
<td>0.21</td>
<td>0.022</td>
<td>9.5</td>
</tr>
<tr>
<td>Cured floc soil</td>
<td>0.23</td>
<td>0.033</td>
<td>7.0</td>
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</table>
### Table 4-3 Details of simple shear tests on cured floc soil

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test Type</th>
<th>Initial Void Ratio</th>
<th>Void Ratio After Consolidation</th>
<th>$\sigma_{vc}^*(kPa)$</th>
<th>$\sigma_{hc}^*(kPa)$</th>
<th>$\tau_{peak}(kPa)$</th>
<th>$\tau_{cyc}(kPa)$</th>
<th>$N_{15}^{(1)}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>C</td>
<td>1.65</td>
<td>NA</td>
<td>150</td>
<td>60</td>
<td>26</td>
<td>53</td>
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<tr>
<td>2</td>
<td>C</td>
<td>1.91</td>
<td>1.68</td>
<td>150</td>
<td>60</td>
<td>21</td>
<td>94</td>
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</tr>
<tr>
<td>3</td>
<td>C</td>
<td>1.97</td>
<td>1.68</td>
<td>150</td>
<td>60</td>
<td>16</td>
<td>1197</td>
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</tr>
<tr>
<td>4</td>
<td>M</td>
<td>1.97</td>
<td>1.68</td>
<td>150</td>
<td>60</td>
<td>44</td>
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<tr>
<td>5</td>
<td>M</td>
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<td>1.55</td>
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<td>78</td>
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<td>6</td>
<td>C</td>
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<td>100</td>
<td>30</td>
<td>4082</td>
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<tr>
<td>7</td>
<td>C</td>
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<td>1.63</td>
<td>150</td>
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<td>30</td>
<td>60</td>
<td></td>
</tr>
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<td>8</td>
<td>C</td>
<td>2.07</td>
<td>1.42</td>
<td>150</td>
<td>60</td>
<td>20</td>
<td>75</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>M</td>
<td>1.73</td>
<td>1.61</td>
<td>100</td>
<td>40</td>
<td>36</td>
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<td>150</td>
<td>60</td>
<td>16</td>
<td>365</td>
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<td>11</td>
<td>M</td>
<td>1.94</td>
<td>1.44</td>
<td>175</td>
<td>70</td>
<td>60</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>C</td>
<td>1.73</td>
<td>1.45</td>
<td>150</td>
<td>60</td>
<td>15</td>
<td>5820</td>
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</tr>
<tr>
<td>13</td>
<td>C</td>
<td>NA</td>
<td>NA</td>
<td>250</td>
<td>100</td>
<td>40</td>
<td>50</td>
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</tr>
</tbody>
</table>

1 Number of cycles to total shear strain of 15 %

### Table 4-4 Details of triaxial tests on cured floc soil

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test Type</th>
<th>Initial Void Ratio</th>
<th>Void Ratio After Consolidation</th>
<th>$\sigma_{vc}^*(kPa)$</th>
<th>$\sigma_{hc}^*(kPa)$</th>
<th>$\tau_{peak}(kPa)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>M</td>
<td>NA</td>
<td>1.77</td>
<td>150</td>
<td>60</td>
<td>110</td>
</tr>
<tr>
<td>2</td>
<td>M</td>
<td>NA</td>
<td>1.69</td>
<td>150</td>
<td>60</td>
<td>120</td>
</tr>
</tbody>
</table>
Table 5-1 Summary of undrained monotonic triaxial tests on undisturbed samples of the calcareous muddy silt

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test Type</th>
<th>Recovered Depth (m)</th>
<th>Initial Void Ratio</th>
<th>$\frac{\sigma'<em>{vc}}{\sigma'</em>{hc}}$</th>
<th>$\sigma'_{vc}$ (kPa)</th>
<th>$\sigma'_{hc}$ (kPa)</th>
<th>$p'_c$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GoTX01</td>
<td>M ext$^{(1)}$</td>
<td>9</td>
<td>1.56</td>
<td>0.4</td>
<td>80</td>
<td>32</td>
<td>48</td>
</tr>
<tr>
<td>GoTX02</td>
<td>M ext$^{(1)}$</td>
<td>22</td>
<td>1.68</td>
<td>0.4</td>
<td>150</td>
<td>60</td>
<td>90</td>
</tr>
<tr>
<td>GoTX03</td>
<td>M</td>
<td>21</td>
<td>1.64</td>
<td>0.4</td>
<td>150</td>
<td>60</td>
<td>90</td>
</tr>
<tr>
<td>GoTX04</td>
<td>M</td>
<td>10</td>
<td>1.59</td>
<td>0.4</td>
<td>180</td>
<td>72</td>
<td>108</td>
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<td>M</td>
<td>29</td>
<td>1.82</td>
<td>0.4</td>
<td>210</td>
<td>84</td>
<td>126</td>
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<tr>
<td>GoTX06</td>
<td>M</td>
<td>24</td>
<td>1.61</td>
<td>0.4</td>
<td>250</td>
<td>100</td>
<td>150</td>
</tr>
<tr>
<td>GoTX07</td>
<td>M</td>
<td>30</td>
<td>1.47</td>
<td>0.4</td>
<td>310</td>
<td>124</td>
<td>186</td>
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<tr>
<td>GoTX08</td>
<td>M</td>
<td>55</td>
<td>1.59</td>
<td>0.4</td>
<td>385</td>
<td>154</td>
<td>231</td>
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</tbody>
</table>

1 Extension test
2 $p'_c = \frac{\sigma'_{vc} + 2\sigma'_{hc}}{3}$ is mean effective consolidation stress

Table 7-1 Summary of the eight calcareous soils

<table>
<thead>
<tr>
<th>Soil No.</th>
<th>Origin of soil</th>
<th>Descriptions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil #1</td>
<td>18-30 m below seabed</td>
<td>Undisturbed muddy silt (the muddy silt from previous chapters)</td>
</tr>
<tr>
<td>Soil #2</td>
<td>20-30 m below seabed</td>
<td>Undisturbed muddy silt</td>
</tr>
<tr>
<td>Soil #3</td>
<td>At seabed</td>
<td>Undisturbed slightly-cemented silty sand</td>
</tr>
<tr>
<td>Soil #4</td>
<td>5-30 m below seabed</td>
<td>Undisturbed silt</td>
</tr>
<tr>
<td>Soil #5</td>
<td>At seabed</td>
<td>Mixture of a silt, a carbonate powder and silicon oil (the silt from previous chapter)</td>
</tr>
<tr>
<td>Soil #6</td>
<td>At seabed</td>
<td>Sand</td>
</tr>
<tr>
<td>Soil #7</td>
<td>At seabed</td>
<td>Sand</td>
</tr>
<tr>
<td>Soil #8</td>
<td>At seabed</td>
<td>Coarse sand (the sand from previous chapters)</td>
</tr>
</tbody>
</table>
Table 7-2 Constants required for the cyclic response framework

<table>
<thead>
<tr>
<th>Equation No.</th>
<th>Original Equation</th>
<th>Parameter</th>
<th>Meaning of parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eq. 7-2</td>
<td>( \frac{\tau_{\text{cyc}}}{\tau_{\text{ref}}} = N_f^{-r} )</td>
<td>( r )</td>
<td>Declining rate of S-N curve</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \tau_{\text{ref}} )</td>
<td>Peak strength for contractive soils or phase transformation strength for dilative soils</td>
</tr>
<tr>
<td>Eq. 7-3</td>
<td>( \tau_{\text{eq}} = \frac{\tau_{\text{cyc}}}{1 - \left( \frac{\tau_m}{\tau_{\text{ref}}} \right)^a} ) ( a = \min \left[ \frac{\tau_{\text{ref}}}{\tau_{\text{eq}}}, 2 \right] )</td>
<td>( a )</td>
<td>( 1 \leq a \leq 2 )</td>
</tr>
<tr>
<td>Eq. 7-4</td>
<td>( \frac{\Delta u}{\Delta u_f} = \left[ 1 - \left( 1 - \frac{N}{N_f} \right)^m \right]^{\frac{1}{\theta}} ) ( \theta = \theta_0 + \frac{\tau_m}{\tau_{\text{cyc}}} )</td>
<td>( \theta_0 )</td>
<td>Initial curvature in normalised pore pressure generation curve in a symmetric test</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( p )</td>
<td>Rate of increment of the curvature of normalised pore pressure curve with the ratio of ( \tau_m ) to ( \tau_{\text{cyc}} )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( m )</td>
<td>Final curvature of the normalised pore pressure generation curves</td>
</tr>
<tr>
<td>Eq. 7-5</td>
<td>( \chi - \chi_0 = \frac{G_i'(\gamma - \gamma_0)}{1 + G_i'(\gamma - \gamma_0) / a \chi_f} )</td>
<td>( G_i^* )</td>
<td>Initial gradient of the backbone curve</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \chi_f )</td>
<td>Asymptotic value of shear stress ratio ( \chi )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \alpha = 1 ) for initial loading curve; ( \alpha = 2 ) for unloading-reloading curves</td>
<td></td>
</tr>
<tr>
<td>Eq. 7-6</td>
<td>( \chi_{0A} = \frac{(\tau_m + \tau_{\text{cyc}})}{\sigma'<em>{ve} - \Delta u</em>{iA}} ) ( \chi_{0B} = \frac{(\tau_m - \tau_{\text{cyc}})}{\sigma'<em>{ve} - \Delta u</em>{iB}} )</td>
<td>( \chi_{0A} )</td>
<td>Slope of CSL (or monotonic PT line) in ( \tau - \sigma' ) space</td>
</tr>
<tr>
<td>Eq. 7-7</td>
<td>( \Delta u_f = \sigma'<em>{vc} - \left( \frac{(\tau_m + \tau</em>{\text{cyc}})}{\chi_{cs}} \right) )</td>
<td>( \chi_{cs} )</td>
<td>Slope of CSL (or monotonic PT line) in ( \tau - \sigma' ) space</td>
</tr>
</tbody>
</table>
Table 7-3 Constants required for the framework for the calcareous muddy silt

<table>
<thead>
<tr>
<th>Parameter</th>
<th>The calcareous muddy silt</th>
</tr>
</thead>
<tbody>
<tr>
<td>$r$</td>
<td>0.17</td>
</tr>
<tr>
<td>$\tau_{ref}$</td>
<td>43 kPa (at $\sigma'_{w} = 150$ kPa)</td>
</tr>
<tr>
<td>$\theta_0$</td>
<td>1.5</td>
</tr>
<tr>
<td>$p$</td>
<td>1.4</td>
</tr>
<tr>
<td>$m$</td>
<td>1.5</td>
</tr>
<tr>
<td>$G_i^*$</td>
<td>30</td>
</tr>
<tr>
<td>$\chi_f$</td>
<td>0.7</td>
</tr>
<tr>
<td>$\chi_{cs}$</td>
<td>0.61</td>
</tr>
</tbody>
</table>

Table 7-4 Constants required for the framework for the calcareous sand

<table>
<thead>
<tr>
<th>Parameter</th>
<th>The calcareous muddy silt</th>
</tr>
</thead>
<tbody>
<tr>
<td>$r$</td>
<td>0.12</td>
</tr>
<tr>
<td>$\tau_{ref}$</td>
<td>30 kPa (at $\sigma'_{w} = 75$ kPa)</td>
</tr>
<tr>
<td>$\theta_0$</td>
<td>2.5</td>
</tr>
<tr>
<td>$p$</td>
<td>-</td>
</tr>
<tr>
<td>$m$</td>
<td>0.6</td>
</tr>
<tr>
<td>$G_i^*$</td>
<td>300</td>
</tr>
<tr>
<td>$\chi_f$</td>
<td>0.73</td>
</tr>
<tr>
<td>$\chi_{cs}$</td>
<td>0.75</td>
</tr>
<tr>
<td>-----------</td>
<td>------------------</td>
</tr>
<tr>
<td>$\rho_c$</td>
<td>Compressibility of sand in the LCC regime (NCL for resedimented clay)</td>
</tr>
<tr>
<td>$p'_r$</td>
<td>Reference stress - defines LCC at void ratio, $e=1$</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Transitional compression behaviour</td>
</tr>
<tr>
<td>$D_r$</td>
<td>Non-linear volumetric swelling &amp; volumetric hysteresis response</td>
</tr>
<tr>
<td>$h$</td>
<td>Irrecoverable plastic strain for unload-reload cycle</td>
</tr>
<tr>
<td>$K_{ONC}$</td>
<td>Lateral earth pressure ratio in the LCC regime</td>
</tr>
<tr>
<td>$C_b$</td>
<td>Small strain elastic compressibility (at load reversal)</td>
</tr>
<tr>
<td>$\mu'_0$</td>
<td>Poisson's ratio at load reversal</td>
</tr>
<tr>
<td>$\omega$</td>
<td>Non-linearity in Poisson's ratio</td>
</tr>
<tr>
<td>$\phi'_{cs}$</td>
<td>Large strain (critical state) friction angle</td>
</tr>
<tr>
<td>$\phi'<em>m$ or $\phi'</em>{mr}$</td>
<td>Apex angle of bounding surface</td>
</tr>
<tr>
<td>$p$</td>
<td>Transition from contractive to dilative behaviour</td>
</tr>
<tr>
<td>$m$</td>
<td>Shape of the bounding surface</td>
</tr>
<tr>
<td>$\omega_s$</td>
<td>Small strain non-linearity in shear</td>
</tr>
<tr>
<td>$\psi$</td>
<td>Rate of evolution of anisotropy due to stress history</td>
</tr>
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Table 8-2 Input parameters for the calcareous muddy silt

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Parameter</th>
<th>Physical Meaning</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydrostatic or 1-D compression (Triaxial, Oedometer or CSR apparatus)</td>
<td>$\rho_c$</td>
<td>Compressibility of sand in the LCC regime (NCL for resedimented clay)</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>$D$</td>
<td>Non-linear volumetric swelling &amp; volumetric hysteresis response</td>
<td>0.024</td>
</tr>
<tr>
<td></td>
<td>$r$</td>
<td></td>
<td>0.33</td>
</tr>
<tr>
<td></td>
<td>$h$</td>
<td>Irrecoverable plastic strain for unload-reload cycle</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>$K_{ONC}$</td>
<td>Lateral earth pressure ratio in the LCC regime</td>
<td>0.4</td>
</tr>
<tr>
<td></td>
<td>$\mu'_0$</td>
<td>Poisson’s ratio at load reversal</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>$\omega$</td>
<td>Nonlinearity in Poisson’s ratio</td>
<td>1</td>
</tr>
<tr>
<td>Undrained Triaxial Shear Tests: OCR=1; CAUC</td>
<td>$\phi'_c$ ($^\circ$)</td>
<td>Large strain (critical state) friction angle</td>
<td>42</td>
</tr>
<tr>
<td></td>
<td>$\phi'_m$ ($^\circ$)</td>
<td>Apex angle of bounding surface</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>$m$</td>
<td>Shape of the bounding surface</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>$\omega_s$</td>
<td>Small strain non-linearity in shear</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>$\psi$</td>
<td>Rate of evolution of anisotropy due to stress history</td>
<td>5</td>
</tr>
<tr>
<td>Shear wave velocity, Bender element, Small strain triaxial</td>
<td>$C_b$</td>
<td>Small strain elastic compressibility (at load reversal)</td>
<td>500</td>
</tr>
</tbody>
</table>
Table 8-3 Input parameters for the calcareous sand

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Parameter</th>
<th>Physical Meaning</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydrostatic or 1-D compression</td>
<td>$\rho_c$</td>
<td>Compressibility of sand in the LCC regime (NCL for resedimented clay)</td>
<td>0.43</td>
</tr>
<tr>
<td>(Triaxial, Oedometer or CSR apparatus)</td>
<td>$p'_{ref}/p_a$</td>
<td>Reference stress at unity void ratio for the H-LCC</td>
<td>31.68</td>
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<tr>
<td></td>
<td>$\theta$</td>
<td>Describes first loading curve in the transitional regime</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>$h$</td>
<td>Irrecoverable plastic strain for unload-reload cycle</td>
<td>-</td>
</tr>
<tr>
<td>K$_0$-oedometer or K$_0$-triaxial</td>
<td>$K_{0NC}$</td>
<td>Lateral earth pressure ratio in the LCC regime</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>$\mu'_0$</td>
<td>Poisson’s ratio at load reversal</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>$\omega$</td>
<td>Nonlinearity in Poisson’s ratio</td>
<td>1</td>
</tr>
<tr>
<td>Undrained Triaxial Shear Tests:</td>
<td>$\phi'_{cs}$ (°)</td>
<td>Large strain (critical state) friction angle</td>
<td>39.4</td>
</tr>
<tr>
<td>OCR=1; CAUC</td>
<td>$\phi'_{mr}$ (°)</td>
<td>Peak friction angle as a function of formation density at low stresses</td>
<td>47.7</td>
</tr>
<tr>
<td></td>
<td>$p$</td>
<td></td>
<td>0.61</td>
</tr>
<tr>
<td></td>
<td>$m$</td>
<td>Shape of the bounding surface</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>$\omega_s$</td>
<td>Small strain non-linearity in shear</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>$\psi$</td>
<td>Rate of evolution of anisotropy due to stress history</td>
<td>50</td>
</tr>
<tr>
<td>Shear wave velocity, Bender element,</td>
<td>$C_b$</td>
<td>Small strain elastic compressibility (at load reversal)</td>
<td>900</td>
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<tr>
<td>Small strain triaxial</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Test Type</td>
<td>Parameter</td>
<td>Physical Meaning</td>
<td>Values</td>
</tr>
<tr>
<td>-----------------------------------------------</td>
<td>-----------</td>
<td>----------------------------------------------------------------------------------</td>
<td>---------</td>
</tr>
<tr>
<td>Hydrostatic or 1-D compression (Triaxial, Oedometer or CSR apparatus)</td>
<td>$\rho_c$</td>
<td>Compressibility of sand in the LCC regime (NCL for resedimented clay)</td>
<td>0.4</td>
</tr>
<tr>
<td></td>
<td>$p'/p_a$</td>
<td>Reference stress at unity void ratio for the H-LCC</td>
<td>44.3</td>
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<td></td>
<td>$\theta$</td>
<td>Describes first loading curve in the transitional regime</td>
<td>0.15</td>
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<tr>
<td></td>
<td>$h$</td>
<td>Irrecoverable plastic strain for unload-reload cycle</td>
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<td></td>
<td>$K_{0NC}$</td>
<td>Lateral earth pressure ratio in the LCC regime</td>
<td>0.45</td>
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<tr>
<td></td>
<td>$\mu_0'$</td>
<td>Poisson's ratio at load reversal</td>
<td>0.2</td>
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<tr>
<td></td>
<td>$\omega$</td>
<td>Nonlinearity in Poisson's ratio</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>$\phi'_{cs}$ (°)</td>
<td>Large strain (critical state) friction angle</td>
<td>41</td>
</tr>
<tr>
<td></td>
<td>$\phi'_{mr}$ (°)</td>
<td>Peak friction angle as a function of formation density at low stresses</td>
<td>48.6</td>
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<tr>
<td></td>
<td>$m$</td>
<td>Shape of the bounding surface</td>
<td>0.4</td>
</tr>
<tr>
<td></td>
<td>$\omega_s$</td>
<td>Small strain non-linearity in shear</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>$\psi$</td>
<td>Rate of evolution of anisotropy due to stress history</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>$C_b$</td>
<td>Small strain elastic compressibility (at load reversal)</td>
<td>600</td>
</tr>
<tr>
<td>Undrained Triaxial Shear Tests: OCR=1; CAUC</td>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 2-1 Definition of mean stress $\tau_m$ and cyclic stress $\tau_{cyc}$
Figure 2-2 Four patterns of constant stress amplitude cyclic loading
Figure 2-3 Definitions of total strain ($\gamma_t$), mean strain ($\gamma_m$) and cyclic strain ($\gamma_{cyc}$)

Figure 2-4 Typical stress-strain relationship
Figure 2-5 Degrading behaviour in stress-strain relationship

Figure 2-6 Diagrams for Goodman and Gerber models
Figure 2-7 Effective stress path for symmetric constant cyclic stress amplitude undrained triaxial test on a soil.
Photo 3-1 Triaxial apparatus
Photo 3-3 “New” simple shear apparatus
Figure 3-1 Schematic diagram of the triaxial apparatus
Figure 3-2 Schematic diagrams of the simple shear apparatuses
Figure 3-3 Microscopic view of the calcareous muddy silt particles

Figure 3-4 Grading curves for samples of the calcareous muddy silt from various depths at the site
Figure 3-5 Microscopic view of the seabed calcareous silt particles

Figure 3-6 Grading curves for the seabed calcareous silt and OMYA CARB powder
Figure 3-7 Microscopic view of the calcareous sand particles

Figure 3-8 Grading curves for the calcareous sand
Figure 3-9 Combinations of cyclic shear stress $\tau_{cyc}$ and mean shear stress $\tau_m$ used in the testing programme for the calcareous muddy silt.
Figure 4-1 Concept of bridging flocculation

Figure 4-2 Compression curves for soil slurries prepared with various flocculants, compared with slurry without flocculant
Figure 4-3 Monotonic simple shear test results for "floc soil", compared with undisturbed samples

Figure 4-4 Cyclic simple shear test results for "floc soil", compared with undisturbed samples
Figure 4-5 Compression curves for floc soil and cured floc soil, compared with two undisturbed samples.
Figure 4-6 Monotonic simple shear test results for "cured floe soil", compared with undisturbed samples

Figure 4-7 Cyclic simple shear test results for "cured floe soil", compared with undisturbed samples
Figure 4-8 Stress paths for four samples in undrained monotonic simple shear tests

Figure 4-9 Shear stress-strain curves for four samples in undrained monotonic simple shear tests
Figure 4-10 Normalised pore pressure-strain ($\Delta u/\sigma'_{vc}-\gamma$) curves for four samples in monotonic simple shear tests

Figure 4-11 Pore pressure response in cyclic simple shear tests on cured floc soil and undisturbed soil
Figure 4-12a  Cyclic shear strain development in cyclic simple shear test on cured floc soil sample

Figure 4-12b  Cyclic shear strain development in cyclic simple shear test on undisturbed soil sample
Figure 4-13 Comparison of effective stress paths of cured floe samples with undisturbed sample in undrained triaxial tests.

Figure 4-14 Comparison of stress-strain curves of cured floe samples with undisturbed sample in undrained triaxial tests.
Figure 4-15 ESEM micrographs of aragonite soil reconstituted without flocculant
Figure 4-16 ESEM micrographs of undisturbed piston-tube sample of aragonite from a depth of 29m below the seabed
Figure 4-17 ESEM micrographs of uncured “floc soil” sample of aragonite
Figure 4-18 ESEM micrographs of "cured floc soil" sample of aragonite
Figure 4-19 ESEM micrograph of “cured floc soil” sample of aragonite at 500 magnification

Figure 4-20 Coefficient of consolidation ($c_v$) obtained from oedometer tests on undisturbed samples and “cured floc soil” sample
Dilative behaviour

Initial peak strength

Contractive behaviour after initial peak strength

Mean effective stress  $p'$

(a) Contractive and dilative behaviour defined by Chern (1985)

Dilative behaviour as $p'$ increases

Contractive behaviour as $p'$ decreases

Mean effective stress  $p'$

(b) Contractive and dilative behaviour defined by Hyodo et al. (1998)

**Figure 5-1** Stress paths for undrained shearing tests and definitions of contractive and dilative behaviour used by different authors.
Figure 5-2(a) Effective stress paths of CAU triaxial tests on undisturbed samples of the calcareous muddy silt.
Peak strengths occur at $\varepsilon_a \leq 0.1\%$

Figure 5-2(b) Stress-strain curves of CAU triaxial tests on undisturbed samples of the calcareous muddy silt

Stress ratio becomes stable at $\varepsilon_a = 4-5\%$

Figure 5-2(c) Stress ratio-strain curves of CAU triaxial tests on undisturbed samples of the calcareous muddy silt.
Figure 5-3 Normalised results of CAU triaxial tests on undisturbed samples of the calcareous muddy silt: (a) Normalised stress paths; (b) $q/p'_c - \varepsilon_a$ curves.
Figure 5-4 Results of CAU triaxial tests on normally and over consolidated floe samples of the calcareous muddy silt.
Figure 5-5 Results of CIU triaxial tests on normally and over consolidated floc samples of the calcareous muddy silt.
Figure 5-6 Results of CAU simple shear tests on floc samples of the calcareous muddy silt
Figure 5-6 (cont.) Results of CAU simple shear tests on floc samples of the calcareous muddy silt
Figure 5-7 Normalised results of CAU simple shear tests on floc samples of the calcareous muddy silt
Figure 5-8 Comparison of effective stress paths of CAU triaxial and CAU simple shear tests on the calcareous muddy silt.
Figure 5.9 Effective stress paths of undrained triaxial tests on the calcareous silt.
Figure 5-10 Responses of stress-strain and stress ratio-strain in CIU triaxial tests on the calcareous silt.
Figure 5-11 Responses of stress-strain and stress ratio-strain in CAU triaxial tests on the calcareous silt.
Figure 5-12 Normalised results of CIU triaxial tests on the calcareous silt.
Figure 5-13 Normalised results of CAU triaxial tests on the calcareous silt.
Figure 5-14 Results of a CAU simple shear test on the calcareous silt
Figure 5-15 Comparison of effective stress paths of triaxial and simple shear tests on the calcareous silt.
Figure 5-16(a) Effective stress paths of three CIU triaxial tests on the calcareous sand.
Figure 5-16 (cont.) Responses of stress-strain and stress ratio-strain in CIU triaxial tests on the calcareous sand.
Figure 5-17 Normalised results of CIU triaxial tests on the calcareous sand.
Figure 5-18 Effective stress paths of four undrained simple shear tests on the calcareous sand.
Figure 5-19 Responses of stress-strain and stress ratio-strain in CIU simple shear tests on the calcareous sand.
Figure 5-20 Responses of stress-strain and stress ratio-strain in CAU simple shear tests on the calcareous sand.
Figure 5-21 Normalised CIU simple shear tests on the calcareous sand.
Figure 5-22 Normalised results of CAU simple shear tests on the calcareous sand
Figure 5-23 Comparison of effective stress paths of triaxial and simple shear tests on the calcareous sand
Figure 5-24 Concepts of state parameter $\psi$ and over consolidation ratio (OCR)
Figure 5-25 Compression curves of the calcareous muddy silt and critical state obtained from shear tests
Figure 5-26 Compression curves of the calcareous sand and critical state obtained from shear tests
Figure 5-27 Compression curves of the calcareous silt and critical state obtained from shear tests
Figure 6-1  Typical cyclic responses of the calcareous muddy silt in a symmetric CAU SS test (FlocSS12): $\frac{\tau_{cye}}{\sigma'_{ve}} = 0.17$
Figure 6-2  Typical cyclic responses of the calcareous muddy silt in a symmetric CIU SS test (FlocSS18): $\tau_{cy}/\sigma'_{vc} = 0.20$
Figure 6-3 Comparison of CIU and CAU symmetric cyclic simple shear results for the calcarous muddy silt: (a) Cycling to total shear strain of 5%; (b) Cycling to total shear strain of 15%.
Figure 6-4 Stress paths of symmetric CAU SS tests on the calcareous muddy silt: (a) Stress level ($\tau_{cyd}/\sigma'_{vc} = 0.253$); (b) Stress level = 0.167; (c) Stress level = 0.107.
Figure 6-5 Minor principal effective stress curves of symmetric CAU SS tests on the calcareous muddy silt: (a) Stress level ($\tau_{cy}/\sigma'_{vc}$) = 0.253; (b) Stress level = 0.167; (c) Stress level = 0.107.
Figure 6-6 Pore pressure generation curves of symmetric CAU SS tests on the calcareous muddy silt: (a) Stress level \( \tau_{cyd}/\sigma'_{vc} = 0.253 \); (b) Stress level = 0.167; (c) Stress level = 0.107.
Figure 6-7  Shear strain development curves of symmetric CAU SS tests on the calcareous muddy silt: (a) Stress level ($\tau_{cy}/\sigma'_{vc} = 0.253$); (b) Stress level = 0.167; (c) Stress level = 0.107.
Figure 6-8 Stress paths of non-symmetric cyclic simple shear tests on the calcareous muddy silt: (a) $\tau_m < \tau_{cyc}$; (b) $\tau_m = \tau_{cyc}$; (c) $\tau_m > \tau_{cyc}$.
Figure 6-9 Plots of minor principal effective stress $\sigma'_3$ vs. cycle number in non-symmetric cyclic simple shear tests on the calcareous muddy silt: (a) $\tau_m < \tau_{cyc}$; (b) $\tau_m = \tau_{cyc}$; (c) $\tau_m > \tau_{cyc}$.
Figure 6.10 Pore pressure generation curves of non-symmetric cyclic simple shear tests on the calcareous muddy silt: (a) $\tau_m < \tau_{cyc}$; (b) $\tau_m = \tau_{cyc}$; (c) $\tau_m > \tau_{cyc}$. 
Figure 6-11  Shear strain development curves of non-symmetric cyclic simple shear tests on the calcareous muddy silt: (a) $\tau_m < \tau_{cyc}$; (b) $\tau_m = \tau_{cyc}$; (c) $\tau_m > \tau_{cyc}$. 
**Figure 6-12** Typical cyclic responses of calcareous silt (oil silt) in a symmetric CAU simple shear test (OsiltSS04)
Figure 6-13  Effective stress paths of symmetric SS tests on calcareous silt: (a) Stress level ($\tau_{\text{cyg}}/\sigma'_{\text{ve}} = 0.2$); (b) Stress level = 0.14; (c) Stress level = 0.12.
Figure 6-14 Plots of minor principal stress vs. cycle number for symmetric SS tests on calcareous silt: (a) Stress level ($\tau_{cy} / \sigma'_{vc}$) = 0.2; (b) Stress level = 0.14; (c) Stress level = 0.12.
Figure 6-15 Pore pressure generation curves of symmetric SS tests on calcareous silt: (a) Stress level \( \tau_{cy} / \sigma'_{vc} = 0.2 \); (b) Stress level = 0.14; (c) Stress level = 0.12.
Figure 6-16  Shear strain development curves of symmetric SS tests on calcareous silt: (a) Stress level (τ_{cy}/σ'_{ve}) = 0.2; (b) Stress level = 0.14; (c) Stress level = 0.12.
Figure 6-17 Effective stress paths of non-symmetric cyclic simple shear tests on the calcareous silt: (a) $\tau_m < \tau_{cyc}$; (b) $\tau_m = \tau_{cyc}$. 

**Diagram Explanation:**
- **(a)** $\tau_m < \tau_{cyc}$: Shows a monotonic test with a PT state line featured.
- **(b)** $\tau_m = \tau_{cyc}$: Depicts a monotonic test with a PT state line, indicating specific conditions for the cyclic shear tests.
Figure 6-18 Pore pressure generation curves of non-symmetric cyclic simple shear tests on the calcareous silt: (a) $\tau_m < \tau_{cyc}$; (b) $\tau_m = \tau_{cyc}$. 
Figure 6-19  Shear strain development curves of non-symmetric cyclic simple shear tests on the calcareous silt: (a) \( \tau_m < \tau_{cyc} \); (b) \( \tau_m = \tau_{cyc} \).
Figure 6-20 Typical cyclic responses of the calcareous sand in a symmetric CAU cyclic simple shear test (SandSS06)
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- (c) Stress ratio-shear strain curve
- (d) Shear strain development
- (e) Pore pressure generation

**Input Constants**

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Input Constants

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Appendix 1
General Model Framework of Incremental Elasto-plasticity

The general framework of incremental elasto-plasticity includes yield surface, hardening laws, flow rules, and consistency conditions. The incrementally linearised elasto-plasticity used in the MIT-S1 model is described as follows:

The incremental strain (or strain rate) can be subdivided into elastic and plastic components:

\[
\{ \delta \varepsilon \} = \{ \delta \varepsilon^e \} + \{ \delta \varepsilon^p \} \tag{A1-1}
\]

The elastic strain increments are assumed to be isotropically related to the effective stress increments by

\[
\{ \delta \sigma^e \} = [D^e] \{ \delta \varepsilon^e \} \tag{A1-2}
\]

where \( \{ \delta \sigma^e \} \) is the incremental effective stress vector, and \([D^e]\) is the elastic stiffness matrix.

The loading direction \( L \), which defines the occurrence of plastic strains, is given by

\[
L = \frac{\partial f}{\partial \sigma^e} \{ \delta \sigma^e \} \tag{A1-3}
\]

\[
\begin{cases} \geq 0 & \text{Loading} \\ < 0 & \text{Unloading} \end{cases}
\]

where \( f = 0 \) is the yield function.

The plastic strain increments are defined by a general flow rule

\[
\{ \delta \varepsilon^p \} = \Lambda \left( \frac{\partial g}{\partial \sigma^e} \right) \tag{A1-4}
\]

where \( g = 0 \) is the plastic potential function, and \( \Lambda \) is a scalar that controls the magnitude of the plastic strains, and defined as:
where \( H \) is the elasto-plastic modulus.

Combining Equations A1-1 to A1-5, the general incremental effective stress-strain relationship can be written

\[
\{d\sigma'\} = [D^{ep}] \{d\varepsilon\} 
\]

where

\[
[D^{ep}] = [D^e] \frac{T}{H + \{\frac{\partial f}{\partial \sigma'} \}^T [D^e] \{\frac{\partial g}{\partial \sigma'} \}}
\]

where \([D^{ep}]\) is the elasto-plastic constitutive matrix, \( \{\frac{\partial f}{\partial \sigma'} \} \) and \( \{\frac{\partial g}{\partial \sigma'} \} \) are respectively the gradient vectors of the yield function \( f = 0 \) and plastic potential function \( g = 0 \). The elasto-plastic modulus \( H \) can be obtained from the consistency condition \( df = 0 \).

Many models can fit within the foregoing general framework. What will distinguish one model from another is the specification of the components such as yield surface, plastic potential surface, hardening of the yield surface, etc.
Appendix 2
Bounding surface formulation used in the MIT-S1 Model

The concepts of bounding surface plasticity (Dafalias & Herman 1982) are used in the MIT-S1 model to relate the plastic strains of overconsolidated soil to the plastic behaviour experienced by normally consolidated soil. The radial mapping rules used in the model are illustrated in Figure A2-1. The yield function acts as the bounding surface, while the plastic strains of the over-consolidated soil with a same-shaped loading surface are linked to the behaviour at an image point on this surface.

For a stress point A (see Figure A2-1) within the bounding surface, a loading surface can be determined as a same-shaped loading surface passing through point A. Plastic strains at stress point A are defined by specifying the elasto-plastic modulus (H) and flow direction (P) for loading. The model uses separate mapping rules for P and H:

\[ p = p^1 (1 - g_1) + p^0 g_1; \quad 0 \leq g_1 \leq 1 \]  
\[ H = \left( H^1 + H^0 g_2 \left( 1 - \frac{\eta \cdot \eta}{c^2} \right)^{1/2} \right); \quad g_2 \geq 0 \]

where \( p^1 \) and \( H^1 \) are respectively flow direction vector and elasto-plastic modulus at image point \( A^1 \) (see Figure A2-1), \( p^0 \) is the value of \( p \) at first yield, \( H^0 \) defines the transition to LCC; \( \eta \) is the stress ratio vector defined as the ratio of deviator stress vector \( q \) to the mean effective stress \( p' \); \( c \) is a scalar parameter accounting for the effect of maximum friction angle on yield surface, \( < > \) are Macaulay brackets, and \( g_1, g_2 \) are mapping functions describing the relative position of stress point A and image point \( A^1 \). The mapping functions, \( g_1 \) and \( g_2 \), are expressed as the following:

\[ g_1 = \frac{\alpha' - \alpha'_0}{\alpha' - \alpha'_{0i}} \]  
\[ g_2 = \frac{\alpha' - \alpha'_0}{\alpha'_0 - \alpha'_{0i}} \equiv \frac{g_1}{1 - g_1} \]

where \( \alpha' \) is the size of the bounding surface, and \( \alpha'_0 \) and \( \alpha'_{0i} \) are the sizes of the current loading surface and loading surface at first yield respectively. The mapping function \( g_1 \),
and \( g_2 \) describe two important observations of soil behaviour: (1) at first yielding, \( \alpha'_0 = \alpha'_0 \) and \( H' \rightarrow \infty \) so that there is a smooth matching of the perfectly hysteretic and bounding surface models; and (2) as the stress state approaches the bounding surfaces, \( \alpha'_0 \rightarrow \alpha' \), both \( H \) and \( P \) tend to the value at the image point, and hence describe a smooth transition in behaviour to that of the normally consolidated state.

The following equations for \( P_0 \) and \( H_0 \) are used in the model:

\[
P^0_p = -2[h - b]\frac{s}{\alpha'}; \quad p^0_i = p^1_s
\]

\[
H^0 = \left( \frac{\rho_r}{\rho_c - \rho_r} \right) \frac{h}{1 - \delta_p^0} K^l_{\text{max}} \| Q^l \| P^l
\]

where \( h \) is dimensionless material constant, \( \rho_r \) is the current (tangential) slope of the swelling curve in log e-log \( p' \) space, \( q \) is the deviator stress vector, \( \rho_c \) is the tangent compressibility in the range of the LCC on log e-log \( p' \) space, \( \delta_p^0 \) is the normalised distance of the current bounding surface to the limiting compression curve, \( K^l_{\text{max}} \) is the maximum tangent bulk modulus at the image point, and \( Q^l \) is the loading direction vector.
Figure A2-1 Bounding Surface Plasticity used in MIT-S1 model.