8.1 Introduction

This chapter presents data on the variation of $G_{\text{max}}$ with stress level and density for uncemented and cemented sands. The results of all the experiments on Toyoura Sand and Calcareous soil (both cemented and uncemented) for various combinations of dry unit weight and cement content have been presented in previous chapters. This chapter aims to develop a robust empirical formulation for the prediction of small-strain shear modulus ($G_{\text{max}}$) and for the degradation of cement content from this large database. Predictions of the engineering behaviour of cemented soils require appropriate constitutive models, and a number of such models have been proposed in the last few years (e.g. Shambhu 2003, Salvati 2002, Fernandez & Santamarina 2001, Chang & Wood 1992, Dobry et al 1988, Saxena et al. 1988, Acar & El-Tahir 1986, Chiang & Chae 1972). Most of these models contain parameters that describe the effect of the cementation and control its breakdown with volumetric strain. (e.g. Gens & Nova, 1993, Shambhu, 2003, Gajo and Muir Wood, 1999). In these models the breakdown is handled by assuming an exponential decay of the effects of the cementation with some measure of plastic strain. Direct measurement of the amount of cementation is extremely difficult and the validity of these models cannot be easily determined. If the $G_{\text{max}}$ data are used to provide an indication of the cementation it can be seen that the breakdown of cementation is more gradual and does not appear to be well predicted by the common assumptions. This chapter extends the $G_{\text{max}}$ relationship for uncemented calcareous soil to artificially cemented soil, which in turn gives the $G_{\text{max}}$ relationship for naturally cemented calcareous soil.

This chapter shows the influence of cementation and density on $G_{\text{max}}$ for sand. It also compares the numerically predicted shear responses of artificially cemented sand proposed by many previous researchers with the current data. Only the data in the very small ($\varepsilon_a<0.1\%$) to small strain ($\varepsilon_a<1\%$) range were used for this empirical formulation.
This chapter also presents further studies on the artificially cemented soil behavior. This includes more detailed examination of the creep behavior and cyclic behavior of artificially cemented sand with respect to density and cement content.

**8.2 Influence of Cementation and Density on $G_{\text{max}}$**

There has been increasing interest in the behaviour of soil at very small strains in recent years. One aspect of this is the variation of $G_{\text{max}}$ with stress level and density. This section presents data on actual and predicted $G_{\text{max}}$ variation for uncemented and cemented sands with different densities and different degrees of cementation during isotropic compression and shearing. This has been obtained using the ‘bender element’ technique developed by the author for the continuous monitoring of $G_{\text{max}}$ during consolidation and shearing in triaxial tests described in Chapter 5. This technique has been used to investigate the variation of $G_{\text{max}}$ for Toyoura sand and a carbonate sand subject to a range of stress paths. The results for these uncemented sands are broadly consistent with other published data but suggest that the effects of mean effective stress, $p'$, are not being correctly predicted by existing empirical relations at higher stress levels.

The same methodology has then been used to investigate the influence of cement content and density on $G_{\text{max}}$ for artificially cemented carbonate sands. The study has covered a wide range of cement contents, densities and stress levels as discussed in Chapter 7.

The following sections presents briefly the methodology, some typical results from uncemented and cemented sands, and some comparisons with existing empirical approaches for estimating $G_{\text{max}}$.

**8.2.1 $G_{\text{max}}$ Relationship for Uncemented Sand**

The automation of dynamic $G_{\text{max}}$ measurements from Bender Elements enabled the continuous monitoring of $G_{\text{max}}$ during isotropic compression and shearing in triaxial tests. This technique was used to investigate the variation of $G_{\text{max}}$ for Toyoura Sand and carbonate sand subject to a range of stress paths.

**8.2.1.1 Toyoura Sand**

Ttoyoura sand was used in this research because it has been investigated by many researchers (i.e. Yamashita et al. 2001, Iwasaki et al. 1978 etc.) and data on $G_{\text{max}}$ have been reported from a variety of test types. Figure 6.17 shows some results obtained during isotropic compression and shearing of specimens with a wide range of relative densities.
The data shown in Figures 6.17 and 6.18 are generally consistent with previously published data (e.g. Yamashita et al. 2001) as shown in Figure 5.53. However, it was noticed that the widely published empirical relations for $G_{\text{max}}$ underpredicted the data at higher effective stresses, and did not account for the influence of deviator stress. Using all the data from the tests performed in this research (Figures 6.17 and 6.18), a least squares best-fit procedure was used to determine a new relationship that could better describe the variation of $G_{\text{max}}$ as a function of void ratio, $e$, mean effective stress, $p'$ and deviator stress, $q$, given by

$$\frac{G_{\text{max}}}{p_r} = 4786 \left(\frac{p'}{p_r}\right)^{0.56} e^{-1.16} \left(\frac{q}{p'} + 1\right)^{0.12}$$

(8.1)

where $p_r$ is a reference pressure taken here as 1 kPa.

The above relationship derived for shear modulus covers a wide range of data from loose to dense i.e. initial void ratio of uncemented Toyoura sand varies from $e = 0.859$ to $e = 0.641$.

The predicted responses of $G_{\text{max}}$ from Equation 8.1 have been compared with the experimental data from the typical tests and they are presented in Figure 8.1. It can be seen from Figure 8.1 that Equation 8.1 gives an excellent prediction for all stages of the tests (i.e. isotropic compression, isotropic expansion and shearing) and in all ranges of densities (from loose to dense). The following common trends can be identified from the figure:

a) Toyoura sand gives identical moduli on isotropic loading and unloading.

b) During shearing the moduli of Toyoura sand increases with mean effective stress $p'$ until the peak strength is reached after that $G_{\text{max}}$ starts to decrease rapidly.

c) Post peak dilation leads to increase in void ratio and hence drop in $G_{\text{max}}$.

Interpretation is however complicated by non-uniform post peak response.

Iwasaki et al. (1978) proposed the following empirical equation for the small strain shear modulus from resonant column tests on Toyoura sand at small strain range.

$$G_{\text{max}} = 700 \left(\frac{2.17 - e^2}{1 + e}\right) (p')^{0.5}$$

(8.2)

Here $p'$ and $G_{\text{max}}$ are in kg/cm$^2$, (1kg/cm$^2 = 98.1$ kPa).

This equation was unable to cover a wide range of void ratio. Therefore, later Jamiolkowski et al. (1994) changed the void ratio function and proposed a power law
void ratio function, \( f(e) = e^{-\beta} \). The exponent \( \beta \) of 1.3 was found to be appropriate for a wide spectrum of geomaterials including Toyoura sand.

By adopting this power law for the void ratio function Shibuya et al. (1996) proposed the following empirical equation for uncemented Toyoura sand and showed that it agreed with data from seismic cone tests.

\[
G_{\text{max}} = 5000 \, e^{-1.3} \, (p')^{0.5} \quad (8.3)
\]

Here \( p' \) and \( G_{\text{max}} \) are in kPa.

Several widely published empirical relations for \( G_{\text{max}} \) of uncemented sands are summarized in Table 8.1. The predicted responses of \( G_{\text{max}} \) from Hardin's (1969) and Shibuya et al (1996) equations are compared with the experimental data in Figure 8.2. The data covers a range of relative densities. The figure shows that the widely published empirical relations for \( G_{\text{max}} \) under predict the data at higher mean effective stress. And some of the equations in Table 8.1 do not account for the influence of deviator stress \( q \), so that they cannot predict \( G_{\text{max}} \) appropriately during the shearing stage of the test. This reveals the limitations of previously published \( G_{\text{max}} \) relations, which were based on very limited amounts of small-strain, as well as large-strain, \( G_{\text{max}} \) data. This shows that there is room for improvement of existing empirical equations and for the development of a new robust empirical relationship for \( G_{\text{max}} \) based on a wide range of experimental data.

### 8.2.1.2. Carbonate Sand

The relationship for \( G_{\text{max}} \) of uncemented and uncompressed cast in-place calcareous soil has been derived in Chapter 7 utilizing various stress-path test results at different confining pressure and for samples with different relative densities. In this section only the uncemented and compressed calcareous soil samples have been considered in deriving the \( G_{\text{max}} \) relationship, because cemented samples were prepared to the same pre-determined densities, and in the same way as the uncemented and compressed carbonate sand.

The responses of \( G_{\text{max}} \) during isotropic compression, isotropic expansion and shearing at \( \sigma_c' = 300 \) kPa were considered to derive an empirical equation for uncemented calcareous soil. The variation of \( G_{\text{max}} \) during isotropic compression and shearing for the carbonate sand has been shown in Figures 7.14c and 7.20a. In addition some Toyoura sand specimens data, which were sheared at the same confining stresses as the carbonate sand have also been incorporated in this plot. Comparison of
uncemented carbonate sand $G_{\text{max}}$ responses with the Toyoura sand data are shown in Figure 8.3, where the legend ‘TS’ is for Toyoura sand samples. Figure 8.3a shows that the uncremented carbonate sand $G_{\text{max}}$ data generally lie above those for Toyoura sand. This can be explained in part by the high stress needed to produce the uncremented and compressed soil specimens. It can also be seen that the moduli of the carbonate sand on unloading are greater than during initial isotropic compression, unlike Toyoura sand, which gave nearly identical moduli on loading and unloading. This difference is believed to be a consequence of the much greater void ratio changes that occur during compression for the carbonate sand. The effect of the different initial unit weight is more evident in the variation of $G_{\text{max}}$ during drained shear test shown in Figure 8.3b. This shows $G_{\text{max}}$ is increasing with mean effective stress $(p^{'})$ until the peak strength is reached after which $G_{\text{max}}$ drops off more rapidly. A similar pattern is evident for the Toyoura sand shown in Figure 8.1. Using all the test data for the pre-compressed uncremented carbonate sand specimens a best fit relation was derived to describe the variation of $G_{\text{max}}$ as a function of void ratio $e$, mean effective stress $p'$ and deviator stress $q$, which is given as follows:

$$
\frac{G_{\text{max}}}{p_r} = 23404 \left( \frac{p'}{p_r} \right)^{0.38} e^{-1.33} \left( \frac{q}{p'} + 1 \right)^{-0.06} \quad (8.4)
$$

where, $p_r$ is a reference pressure taken here as 1 kPa.

A comparison of the predictions of Equation 8.4 with the experimental data from the uncremented carbonate sand is shown in Figure 8.4. The dry unit weight of the raw data in this plot vary from 13 to 17 kN/m$^3$. All results shown are for specimens subjected to isotropic compression followed by a drained shear test to failure at a mean effective stress of 300 kPa. When all the uncremented data with their wide range of dry unit weights (e.g. 10 to 17 kN/m$^3$) are combined i.e. compressed data shown in Figure 8.3 and uncompressed data shown in Figures 7.4 and 7.9 a best fit relation can be derived to describe the variation of $G_{\text{max}}$ as a function of void ratio $e$, mean effective stress $p'$ and deviator stress $q$, which is given as follows:

$$
\frac{G_{\text{max}}}{p_r} = 22330 \left( \frac{p'}{p_r} \right)^{0.39} e^{-1.37} \left( \frac{q}{p'} + 1 \right)^{-0.08} \quad (8.5)
$$

Equations 8.4 and 8.5 are very similar and the difference in $G_{\text{max}}$ prediction using these two equations are insignificant. Figure 8.5 shows the prediction of $G_{\text{max}}$ using Equation 8.5 and comparison of Figure 8.5 with Figure 8.4 shows the similarity.
Figure 8.4 shows that $G_{\text{max}}$ reduces from the isotropic response during shearing, whereas for Toyoura sand (at Figure 8.1) it increases and this is reflected in the different exponents on the stress ratio term in Equations 8.1 and 8.4. It is also apparent that the predictions for the uncedented carbonate sand are less capable of reproducing the observed behavior in isotropic compression and expansion. This is because the simple power law relation with void ratio, which can reproduce the large scale effects of differences in void ratio, is not sufficiently sensitive to the effect of over consolidation ratio (OCR). Alternatively this may be a consequence of the prior one- dimensional compression, and of changes in anisotropy following the isotropic load cycle. It may be noted that OCR is a function of $p'$ and void ratio, which have been used in $G_{\text{max}}$ derivation in this study. Therefore it cannot be said that the inclusion of another parameter, like OCR into Equation 8.4 will significantly improve the predictions, which are limited by the assumed simple power law relation. Coop (1990) and Lo Presti et al (1993) have used OCR in $G_{\text{max}}$ derivation for uncedented sand and have not got any significant improvement in $G_{\text{max}}$ predictions.

The relationship derived in this study for uncedented carbonate sand can be compared with Dobry et al.’s (1988) relation because they used similar material. Dobry et al. (1988) proposed the following relationship of $G_{\text{max}}$ for uncedented calcareous soil

$$\frac{G_{\text{max}}}{p_a} = 762 \left( \frac{p'}{p_a} \right)^{0.64}$$

(8.6)

where, $p_a = 100$ kPa.

Some of the studies reporting relations for $G_{\text{max}}$ of uncedented sands have been listed in Table 8.1. The predicted responses of $G_{\text{max}}$ from some of these equations are compared with the experimental data in Figure 8.6. It shows that the widely published empirical relations for $G_{\text{max}}$ under predict the data at higher mean effective stress and do not account for the influence of deviator stress $q$. The figure also shows that both Dobry et al.’s (1988) and Shibuya et al.’s (1996) empirical relations are unable to represent $G_{\text{max}}$ behaviour during shearing for uncedented calcareous soil. On the other hand Figure 8.4 shows that the relation proposed in this study gives a satisfactory prediction of $G_{\text{max}}$ for the range of densities and in all stages of the tests for the uncedented calcareous soil used in this study.
8.2.1.3 Discussion on Uncemented Sand

The results for uncemented sands (Toyoura sand and carbonate sand) are broadly consistent with other published data. However, the effects of mean effective stress $p'$ are not being correctly predicted by existing empirical relations at higher stress levels. It has also been shown that allowance for deviator stress ($q$) is required to predict $G_{\text{max}}$ in the shearing stage of the test.

From the investigation of uncemented Toyoura sand and the calcareous sand the following conclusions can be made.

- There is a distinct difference in $G_{\text{max}}$ behaviour during isotropic compression and isotropic expansion between uncemented Toyoura sand and the uncemented carbonate sand. For Toyoura sand $G_{\text{max}}$ follows the same path during isotropic expansion as for isotropic compression. But in case of carbonate sand $G_{\text{max}}$ path during isotropic expansion lies above that during isotropic compression, even though differences in void ratio are relatively small.
- Equation 8.1 for Toyoura sand is able to predict the $G_{\text{max}}$ behaviour during both isotropic compression and isotropic expansion accurately.
- Equation 8.4 for uncemented carbonate sand is able to predict the $G_{\text{max}}$ behaviour during isotropic compression but unable to reflect the $G_{\text{max}}$ increase during isotropic expansion proportionately because the change of void ratio during isotropic expansion is not high enough to cause a big increment in $G_{\text{max}}$ during isotropic expansion. Existing empirical relations for uncemented sand are not able to predict this $G_{\text{max}}$ increase accurately during isotropic expansion (i.e. Coop 1990).
- There is a noticeable difference in $G_{\text{max}}$ behaviour during shearing between uncemented Toyoura sand and the uncemented carbonate sand. For Toyoura sand $G_{\text{max}}$ increases during shearing and $G_{\text{max}}$ follows a path above the isotropic path before it starts to drops off at the time of yielding. Equation 8.1 for Toyoura sand is also able to reflect this phenomenon during shearing.
- For carbonate sand $G_{\text{max}}$ increases during shearing and follows the path along or below the isotropic path before it starts to drops off at the time of yielding. Equation 8.4 for uncemented carbonate sand is also able to predict this phenomenon satisfactorily during shearing. Existing empirical relations for
uncemented sand are generally unable to predict the $G_{\text{max}}$ behaviour during shearing.

- The results for the uncemented sands are broadly consistent with other published data but suggest that the effects of mean effective stress, $p'$, are not being correctly predicted by existing empirical relations at higher stress levels.

### 8.2.2. $G_{\text{max}}$ Relationship for Cemented Calcareous Soil

Figures 7.31c and 7.32c shows the response of $G_{\text{max}}$ for different percentage of gypsum content (by dry weight) and for different dry unit weight ($\gamma_d$) during isotropic compression and shear to failure. To derive an empirical $G_{\text{max}}$ relation the data of isotropic compression and shear before reaching the peak deviator stress were used from these plots.

Using the data from all the $G_{\text{max}}$ measurements covering the full range of gypsum contents and unit weights a best fit relation has been formulated for gypsum cemented carbonate sand. This relation has the following form

$$
\left( \frac{G_{\text{max}}}{p_r} \right)_{\text{cemented}} = \left( \frac{G_{\text{max}}}{p_r} \right)_{\text{uncemented}} + \left( \frac{G_{\text{max}}}{p_r} \right)^*_{\text{uncemented}}
$$

(8.7)

where the uncemented term is given by Equation 8.4. Equation 8.4 was used for the uncemented part, because the data shown in Figure 8.3, used to derive Equation 8.4, possess the same unit weights (i.e. 13, 15 and 17 kN/m$^3$), are manufactured similarly and both are over-consolidated samples.

$$
\left( \frac{G_{\text{max}}}{p_r} \right)^*_{\text{uncemented}} = 16047 (GC)^{1.08} \left( \frac{p'}{p_r} \right)^{0.01} e^{-2.42 \left( \frac{q}{p'} + 1 \right)}^{0.02}
$$

(8.8)

where $GC = \%$ of gypsum content by dry weight.

Comparisons between the values of $G_{\text{max}}$ predicted by Equation 8.7 and the experimental data are shown in Figure 8.7 for a wide range of cement content and unit weight. For all the curves shown here, the specimens were isotropically compressed to 300 kPa before being sheared drained at constant confining stress. The predicted and experimental data shown here agree until the specimens reach to their maximum deviator stress ($q_{\text{peak}}$) for the first cycle of loading. Considering the wide range of cement contents and unit weights and the simple power law relations used in Equation 8.4 and 8.8 the agreement is very encouraging. Only the initial parts of the shear test responses are shown in Figure 8.7. As the specimens approached failure the discrepancy between the predicted and measured data increased. This discrepancy can
be explained by the breakdown of the cementation bonds that occurs as the specimens are sheared. Indeed because of the breakdown of cementation the data used to derive Equation 8.8 were limited to the isotropic compression stages and the initial linear portion of the stress-strain responses. It can be noted here that Equations 8.7 and 8.8, coupled with the continuous recording of $G_{\text{max}}$ in these tests, enables the degradation of cementation to be estimated as the cemented specimens are subjected to a range of stress paths. This is explored in more detail in subsequent sections.

Figure 8.8 shows the comparison of measured and predicted Secant Young’s modulus $E_{\text{sec}}$ at $\varepsilon_a = 0.1\%$ and at different densities. The measured $E_{\text{sec}}$ was calculated from stress strain curve and the predicted $E_{\text{sec}}$ was estimated from shear modulus (i.e. from Equation 8.7) considering poison ratio of 0.3. It shows that $E_{\text{sec}}$ increases with the dry unit weight and the gypsum content. It is noticeable from the plot that there is a good agreement between estimated and predicted $E_{\text{sec}}$ values.

**8.2.2.1. Empirical Equations for $G_{\text{max}}$ of Cemented Soil**

The effects of various cementing agents on sandy soils have been investigated in several studies, eg. Sharma (2003), Salvati (2002), Fernandez & Santamarina (2001), Chang & Wood (1992), Saxena et al. (1998), Dobry et al. (1988), Acar & El-Tahir (1986), Chiang & Chae (1972). These earlier studies were usually limited to smaller range of density and cement content than in the current research, and generally followed a similar methodology to produce relations to describe the variation of $G_{\text{max}}$ with stress, void ratio and cement content. All these empirical relationships for $G_{\text{max}}$ are summarized in Table 8.2. It can be observed from Table 8.2 that these empirical equations are based on a wide range of soil type and cementing agents. The empirical relations proposed by these authors give widely different predictions for the effect of cementation on $G_{\text{max}}$. To allow for comparison an attempt has been made to determine the cement contents equivalent to gypsum cement contents of 10%, 20% and 30% respectively. This was based on the values of the unconfined compressive strength for different cement contents. The adopted strategy was to determine what cement contents would be required to get reasonable agreement with the tested $G_{\text{max}}$ results. Table 8.3 shows the typical value of different parameters adopted for different empirical equations to compare and determine the cement content equivalent to a gypsum cement content of 10%, 20% and 30% of this study. Huang (1994) and Clough et al (1981) have shown that the unconfined compressive strength (UCS) of
cemented soils is a function of cement content \( C \) and unit weight \( \gamma \), i.e. \( UCS = f(C, \gamma) \), but their relationships are only applicable for a narrow range of density. The actual relationship between UCS, \( C \) and \( \gamma \) for a wide range of density is not known in general. Nevertheless, use of UCS results to allow for cement content enables a consistent basis for comparison of different cemented materials.

Figures 8.9, 8.10, 8.11 show a comparison of the predictions of various equations referred to in Table 8.2 for different gypsum contents and different initial dry unit weights (i.e. 13, 15 and 17 kN/m\(^3\)). In all the figures the curve labeled “This Study” is the prediction from Equation 8.7 and the curve labeled “Uncemented” is the prediction from Equation 8.4. It can be seen from the figures that there is a wide range in the predicted responses for \( G_{\max} \). Figure 8.9 shows that for low and medium densities (i.e. \( \gamma_d = 13 \) to 15 kN/m\(^3\)) the empirical relations of Acar & El-Tahir, (1986) and Chiang & Chae (1972) give lower predictions of \( G_{\max} \) than the one measured for the uncemented soil, which is highly unrealistic. For moderately to well cemented samples (see Figures 8.10 and 8.11) some empirical relations give predictions of \( G_{\max} \) that are similar to the uncemented values regardless of density. This is not reasonable because increases of cementation and density increase the initial \( G_{\max} \) values. The theoretical relation for \( G_{\max} \) derived by Fernandez & Santamarina (2001) gives almost constant \( G_{\max} \) with the changing \( p' \) irrespective of cementation and density. This is not consistent with majority of the published data which shows \( G_{\max} \) increasing with \( p' \), and therefore this relation is of limited practical validity. Figures 8.9, 8.10, 8.11 show a trend for the various equations to, on average, over predict \( G_{\max} \) at low density and to under predict \( G_{\max} \) at high density. This may, in part reflect the different cementing agents used in other studies. As the effectiveness of the cement is likely to be a function of the sand characteristics some differences would be expected, however, the large range in the predictions is difficult to explain due to uncertainty in cement content estimation and there is a need for more data and a better understanding of how cementation affects the small-strain modulus.

8.2.2.2. Discussion

Automation has enabled the continuous recording of \( G_{\max} \) (mentioned in Chapter 5) during triaxial tests. The large data set has allowed investigation of \( G_{\max} \) with stress state, void ratio and cement content. This has enabled a new empirical relation for \( G_{\max} \) to be formulated. If the continuous trace of \( G_{\max} \) data were used in previously
Chapter 8 Empirical Modelling of Small-Strain Stiffness Response

published empirical equations then it could be seen that the breakdown of cementation was more gradual and was not well predicted by the common assumptions. This method (automation of $G_{\text{max}}$ measurement procedure) gives reasonable $G_{\text{max}}$ values for uncemented sands as well as cemented sands. Figures 8.9, 8.10, and 8.11 show that existing empirical relations for cemented sand, including the relationship proposed in this research give a wide range of predictions, and consequently they are of little general predictive value. The $G_{\text{max}}$ value depends on many factors and each of the existing empirical relations is based on a different cement type, different theoretical or experimental approach, and does not include all the influence factors or soil parameters. Therefore it appears to be impractical to derive a general empirical relations for $G_{\text{max}}$ for all cement types.

All the cemented samples tested in this research were compacted during their manufacturing procedure to reach specified densities. Equation 8.7 incorporates a whole range of parameters such as, gradation of the particles, cement type, cement effectiveness and density of the sample as well as being influenced by the stress history and cementation process. Further study is needed to quantify the factors responsible for the different relationships between $G_{\text{max}}$ and cement content.

An equation has been developed that relates $G_{\text{max}}$ of sandy soils to density, stress level and cement content. As well as its predictive capabilities this equation has the potential to allow the process of de-structuring to be observed under general states of stress, if measurements of $G_{\text{max}}$ are available.

8.3 Degradation of Cementation

In this section an attempt has been made to infer the degree of cementation from measurements of the small strain shear modulus, $G_{\text{max}}$. As noted in previous chapters the small strain modulus ($G_{\text{max}}$) is strongly dependent on the amount of cementation and this parameter can be measured non-destructively using wave velocity measurements. The degradation of cementation can occur in two ways either by shearing (cyclic or monotonic loading) or by isotropic compression. In this study degradation of cementation has been observed by shearing only. Previous studies by Baig et al. (1997) and many other authors have shown that the application of elevated confining stresses to cemented samples causes the breakdown of cementation and hence it causes the decrease in shear modulus ($G_{\text{max}}$). Due to the limitation in pressure capacity of the triaxial cell (i.e. 2 MPa) used in this study, degradation of cementation has not been observed by isotropic compression.
8.3.1 Introduction
Experimental investigation into the response of cemented carbonate sediments at small strain has involved a series of triaxial tests on carbonate sand artificially cemented with gypsum and prepared with a range of cement contents (0% to 30% by dry weight) and densities (13 kN/m³ to 17 kN/m³). In all tests the small strain shear modulus $G_{\text{max}}$ has been determined continuously throughout the tests (mentioned in Chapter 5), that is during isotropic compression and shearing, and internal Hall Effect Transducers (HET) have been used to monitor the sample deformations. These test data on carbonate sand have been used to explore the influence of cementation and density on the shear stiffness, $G_{\text{max}}$. By obtaining the best fit to those data two empirical relations for $G_{\text{max}}$ have been derived for uncemented and cemented carbonate sand in Equations 8.4 & 8.7 respectively. In some tests specimens were subjected to cycles of loading and degradation of shear stiffness ($G_{\text{max}}$) was observed. By using Equations 8.4 and 8.7 it was possible to estimate the reduction in cementation as percentage cement content, and qualitatively this approach was observed to give sensible results. This procedure has also been used to evaluate the cementation breakdown with strain and stress. The results show that for a wide range of cement contents and densities there appears to be a unique relation between loss of cementation and plastic shear strain ($\varepsilon_s^p$).

8.3.2 Response of Cement Degradation
The stress-strain and $G_{\text{max}}$ test data have been presented in the previous chapter (see Figures 7.42 and 7.46) for cemented samples. In this section cement degradation of these samples will be discussed in brief. These plots show that there is a significant increase in shear stiffness and peak deviator stress ($q_{\text{peak}}$) due to the increase of cement content. So there should be a close and proportional relationship between $q_{\text{peak}}$, $G_{\text{max}}$ and gypsum content (GC). As the cementation breaks down the strength reduces, and all cemented specimens approach towards the same ultimate state, which is the state of uncemented carbonate sand. Interpretation of the post-peak responses is complicated by the development of shear planes in dense and well cemented specimens, and from the responses in Figure 7.42 it is difficult to determine the extent of cement degradation. To show more clearly the significant effect of cement on the stiffness the initial parts of the responses from Figure 7.42, at very small strains, are shown in Figure 7.47. From the small strain responses it is evident that the linear-elastic portion is very limited and some degradation of cement can be expected from
the early stages of shearing. The increase of $G_{\text{max}}$ and $q_{\text{peak}}$ during shearing is linearly proportional to the cement content as shown in Figures 8.12 for the specimens with initial dry unit weight of 13 kN/m$^3$. In this plot ‘Best Fit’ lines are the linear regression line for $G_{\text{max}}$ and $q_{\text{peak}}$. The ‘Best Fit’ line indicates the slope of the line and for $G_{\text{max}}$ and $q_{\text{peak}}$ these slope are approximately 19 and 31.75 respectively. Figures 8.13 and 8.14 also show that the $G_{\text{max}}$ and $q_{\text{peak}}$ increase linearly with the increase of cement content for the specimens with initial dry unit weights of 15 and 17 kN/m$^3$. The slope of these lines increases due to increase of density.

Similarly $G_{\text{max}}$ decreases during shearing with the breakdown of cementation and approaches to the ultimate $G_{\text{max}}$ state as shown in Figures 7.46, which is similar to the uncemented carbonate sand. But the reduction of $G_{\text{max}}$ is not linear as of pre-yield state shown in Figures 8.12, 8.13, and 8.14, because the reduction of $G_{\text{max}}$ accelerates when the sample passes the $q_{\text{peak}}$ (see Figures 7.45).

The reduced value of $G_{\text{max}}$ during shearing reflects the breakdown of cement bonds or reduction of cement content. This reduction of cement content has been quantified by using Equation 8.7 and the measured $G_{\text{max}}$ values. The reduction of cement content will be shown and discussed in detail in the next section.

**8.3.2.1 Comparison between $G_{\text{max}}$ and $G_{\sec}$**

Figure 8.15 shows how the shear moduli $G_{\text{max}}$ and $G_{\sec}$ vary during shearing for cemented samples whose dry unit weight is 13 kN/m$^3$ and whose gypsum contents range from 0 to 30%. It shows that, as expected, $G_{\sec}$ drops off earlier than $G_{\text{max}}$ during shearing. The initial value of $G_{\sec}$ is similar to the initial value of $G_{\text{max}}$ as expected. It also shows that $G_{\sec}$ drops significantly at an axial strain around 0.01% whereas $G_{\text{max}}$ does not drop significantly until an axial strain greater than 0.2% for highly cemented samples (20 to 30% gypsum content), and gradually drops at an axial strain greater than 1% for weakly cemented samples (10% to 0% gypsum content).

Clearly the reducing $G_{\sec}$ is more rapid and occurs earlier than degradation of $G_{\text{max}}$ for cemented calcareous soil during shearing. This reveals a strong dependency of $G_{\text{max}}$ on $p'$ and $G_{\sec}$ on $q$. According to Equation 8.8 increase in $p'$ would be expected to lead to slight increase in $G_{\text{max}}$. But the fact that this did not occur suggests that some degradation of cement is occurring and thus $G_{\text{max}}$ or $G_{\sec}$ responses in the figure are consistent (i.e. both indicate loss of cementation). According to Figure 8.15 at the start of shearing $p'$ increases with $q$ and $G_{\text{max}}$ remains constant, possibly due to the equal effects of cement degradation and the $p'$ increases. From Figure 7.49a it is
evident that the linear-elastic portion is very limited and some degradation of cement can be expected from the early stages of shearing, but $G_{\text{max}}$ is apparently unaffected. It is believed that the effects of the minor cement degradation and the effect of increasing $p'$ cancel out one another. In a constant $p'$ test it was observed that even though $p'$ was constant the void ratio reduced at the start of shearing, and this could explain why $G_{\text{max}}$ remained unchanged, though $G_{\text{sec}}$ started to fall from the beginning. These observations suggest the accumulation of plastic strain is responsible for the reduction in $G_{\text{sec}}$, but their effect on $G_{\text{max}}$ at the start of shearing is small when soil remains cemented. At the early stages of shearing Figure 8.15 shows that $G_{\text{max}}$ remains unchanged with the increase of axial strain for a while, on the other hand $G_{\text{sec}}$ starts to decrease significantly due to the breakdown of cementation from the very beginning of shearing. This pattern was observed for cemented samples of other densities (i.e. $\gamma_d = 15$ kN/m$^3$ and 17 kN/m$^3$), as shown in Figures 8.16 and Figure 8.17. It is noticeable from Figures 8.16 and 8.17 that for some samples the initial value of $G_{\text{sec}}$ is less than the initial value of $G_{\text{max}}$.

Figures 8.18 and 8.19 show the $G_{\text{max}}$ and $G_{\text{sec}}$ responses with axial strain for constant $p'$ tests at different $p'$ values (i.e. at $p' = 1000$ and 1500 kPa) and different cement contents (i.e. 30% and 20% gypsum content) but with the same density (i.e. $\gamma_d = 15$ kN/m$^3$). Figures 8.18b and 8.18c show that $G_{\text{max}}$ increases more than $G_{\text{sec}}$ with the increase of $p'$ (i.e. $p' = 1000$ kPa to 1500 kPa). All the plots in Figure 8.18 show that, at constant $p'$, deviator stress increases with axial strain increase and $G_{\text{max}}$ remains unaffected, but $G_{\text{sec}}$ starts to decrease with axial strain increase for 30% cement content. According to Figure 8.19 the trend is similar for 20% gypsum content, only both $G_{\text{max}}$ and $G_{\text{sec}}$ value are less than that of Figure 8.18 due to the lesser degree of cementation. Both plots show that $G_{\text{sec}}$ is greater than $G_{\text{max}}$ at small strain. This may be a consequence of the assumed Poisson ratio of 0.5 and the unreliability of HETs at very small strain range.

### 8.3.2.2 Effect of Cementation on Strength and Stiffness

The $G_{\text{max}}$ data from Figures 8.12, 8.13 and 8.14 have been plotted together in Figure 8.20 for a particular $p'$ (= 300 kPa). Figure 8.20 shows that $G_{\text{max}}$ increases proportionately with the increase of cement content for a particular density of cemented calcareous soil. It also shows that the slope of the lines increases with dry unit weight ($\gamma_d$). That is the effectiveness of the cement increases as unit weight
increases (void ratio reduces). A similar trend was reported by Huang (1994) for the unconfined compressive strength. So the cement content has two effects on cemented calcareous soil:

a) It increases the peak deviator strength ($q_{\text{peak}}$).

b) It increases the initial shear modulus ($G_{\text{max}}$).

Huang (1994) has derived a relationship between unconfined compressive strength ($q_u$), dry unit weight ($\gamma_d$) and percentage of cement content ($C$) for the same soil as follows:

$$
\frac{q_u}{p_a} = 9.19 \frac{\gamma_d}{\gamma_w} C - 9.34 \frac{\gamma_d}{\gamma_w} - 13.22 C + 9.16 \tag{8.9}
$$

where $p_a$ is atmospheric pressure (= 98.1 kPa), and $\gamma_w$ is the unit weight of water (=9.81 kN/m$^3$).

Figure 8.21a shows the variation of estimated $G_{\text{max}}$ and $q_u$ value with different gypsum contents (GC) and dry unit weights ($\gamma_d$), calculated from Equation 8.7 and Equation 8.9 respectively. It shows that both $G_{\text{max}}$ and $q_u$ increase with the increase of dry unit weight and gypsum content except for one $q_u$ prediction at low cement content (e.g. 10%) for medium to dense cemented sample with $\gamma_d = 15$ kN/m$^3$. This figure also shows that cement is more effective at increasing $G_{\text{max}}$ and at increasing density. It is also noticeable that the pattern of $G_{\text{max}}$ and $q_u$ increase are consistent and parallel to each other, it indicates that the slopes of $q_u$ increase and $G_{\text{max}}$ increase are identical.

Figure 8.21b shows the comparison of UCS results from this study with Huang’s (1994) prediction by Equation 8.9 for different densities and cement contents. It shows that the Equation 8.9 can predict the $q_u$ value reasonably at increasing cement content and at higher density. It was found that Equation 8.9 cannot reliably predict the $q_u$ values at low density i.e. $\gamma_d = 13$ kN/m$^3$. Therefore $q_u$ prediction is not shown in Figure 8.21 for $\gamma_d = 13$ kN/m$^3$. Equation 8.9 is required to modify to reasonably predict $q_u$ value at low density.

Figure 8.21c shows the normalized $G_{\text{max}}$ variation with cement contents and densities and normalization has been performed by corresponding $q_u$ values in Figure 8.21a. It shows that the ratio of predicted $G_{\text{max}}$ from this study and $q_u$ from Huang’s (1994) study increases with the density but decreases with cement content. Shambhu(2003)
8.3.2.3 Breakdown of Cementation during Shearing

To explore the cementation degradation Equation 8.7 was used to continuously determine ‘apparent’ or ‘effective’ gypsum content, GC (which is changing during shearing), as the percentage of gypsum, from the measured values of $G_{\text{max}}$, $p'$, $q$, and $e$, for all tests on artificially cemented specimens. The variation of $G_{\text{max}}$ and the estimated gypsum content from Equation 8.7 are shown in Figures 8.22, 8.23 and 8.24 for specimens with dry unit weight of 13 kN/m$^3$ and initial cement contents of 10%, 20% and 30% respectively. The corresponding stress-strain behaviour has been shown in Figures 7.42a and in Figure 7.47a. Figure 8.22 shows that even though $G_{\text{max}}$ remains unaffected at the start of shearing, the cement content degrades from the start of loading. Because cemented materials are brittle, they therefore begin to degrade at much lower strain level than an uncemented material (Pestana and Salvati 2006). A similar trend can be observed for previously unsheared specimens from Figures 8.23 and 8.20.

The general patterns of behaviour shown in Figures 7.42a, 8.22 and 8.23, and in particular the degradation of $G_{\text{max}}$ with cyclic loading are consistent with several other studies (e.g. Sharma 2003, Yeoh 1996). The predicted degradation of cement content during the load cycles also appears reasonable. The cement content is inferred to be approximately constant during isotropic compression and then to gradually decrease as shearing occurs, the rate of decrease picking up as failure is approached. The decrease from the start of shearing is consistent with the small strain behaviour shown in Figure 7.47a. During unloading the cement content is shown to initially increase slightly and then remain constant. This could be a limitation of the derived equation (based on simple power law) and the indirect method of cement estimation. The second reason is that the equation for uncemented carbonate sand derived in the previous chapter does not predict uncemented $G_{\text{max}}$ with 100% accuracy, which introduces some inaccuracy into the predictions for cemented sand. It is believed that the increase in $G_{\text{max}}$ is physically reasonable and can be explained by the closure of microscopic cracks that develop during loading. This would increase the total contact area and result in increases in the shear wave velocity. But, this effect is small and overall the predicted trends in the cementation component appear reasonable with the
cementation decreasing continuously. One method of assessing whether the predicted cement degradation is reasonable or not, is by inspection of the stress-strain response. The variations of $G_{\text{max}}$ and hence cement content are different for the pre-cycled specimen (see Figure 8.23) and previously unsheared specimen shown in Figure 8.22. In Figure 8.23 the cycled specimen shows a practically constant cement content during the fourth loading i.e. at gypsum content of approximately 10%, whereas Figure 8.22 shows that for the unsheared specimen (whose initial gypsum content is 10%) the shear modulus and cement content decrease from the start of shearing. These observations indicate the limitation of estimating cement degradation from stiffness measurements as the void ratio, $p'$ and stress ratio are changing all the time during a triaxial test. A similar trend can be observed for the other unsheared specimens as well. For example Figure 8.24 shows the degradation of cementation for dry unit weight of 13 kN/m$^3$ and an initial 30% gypsum content. In this case (see Figure 8.24) the cement content is inferred to be approximately constant during isotropic compression but cement content starts to degrade gradually as shearing occurs with the rate of degradation picking up as failure is approached.

Another observation from Figure 8.23 is that by the end of the third loading-unloading cycle the cement content has dropped to 10% and $G_{\text{max}}$ has dropped as well to a value close to that of the sample in Figure 8.22 whose initial cement content was 10%. However the deviator stress mobilized by the sample with initial GC = 20% is considerably more than the specimen with the initial cement content of 10% (see Figure 7.42a, and samples C15G20%u13 and C8G10%u13). This can be explained from void ratio responses of two samples (see Figure 7.44a). Although the estimated GC has reduced to 10% after three cycles of loading in Figure 8.23, the void ratio of sample C15 at the end of third cycle is less than the sample with initial GC=10% according to Figure 7.44a. It can be calculated from Equation 8.9 that due to the decrease of cementation from 20% to 10% gypsum content, the likely strength ($q_0$) decrease is about 40%. From Figure 8.23 it can be observed that the likely stiffness ($G_{\text{max}}$) drop at the end of the third cycle of loading-unloading is about 40% as well.

The trend of cement degradation is consistent for all artificially cemented calcareous samples with different degrees of cementation (i.e. 10%, 20% and 30% gypsum content) at a particular dry unit weight (i.e. $\gamma_d = 13$ kN/m$^3$). These features of the cement degradation are observed for all artificially cemented calcareous soil. Similar
cement degradation can be observed for the denser (i.e. $\gamma_d = 15$ and 17 kN/m$^3$) cemented soil as shown in Figures 8.25 and 8.26.

Figures 8.25 and 8.26 shows the variation of $G_{\text{max}}$ and estimated Gypsum Content (GC) with $p'$ for artificially cemented calcareous specimens with initial dry unit weights of 15 and 17 kN/m$^3$. It shows the denser sample and the higher gypsum content the more drastic is the degradation of cement compared to the samples with lower density (i.e. $\gamma_d = 13$ kN/m$^3$). The figures also show that cement degradation occurs from the start of shearing with the exception of densest sample with 30% gypsum content (see Figure 8.26c). But in general both figures show a similar cement degradation behaviour to the specimens with $\gamma_d = 13$ kN/m$^3$.

8.3.3 Relation between Cement Content and Strain

Although it is difficult to verify, using $G_{\text{max}}$ gives sensible and consistent trends for the degradation of cement in all the tests. All specimens showed a reduction in cement content during shearing similar to that shown in Figures 8.22 to 8.24. The data were used to investigate relations between cement degradation and strain as appear in many constitutive models. When plotting cement content degradation against the plastic shear strain ($\varepsilon_{sp}$) it was noticed that all tests showed a similar trend as shown in Figure 8.27. In the key on this plot the value following “g” is the percentage of gypsum (by dry weight) and that following “u” the dry unit weight. It was observed from Figure 8.27 that all the cemented samples show an approximately exponential decay of cement content with plastic shear strain regardless of dry unit weight ($\gamma_d$). When normalized by the initial cement content ($G_{C_0}$) the cement content degradation gave a reasonably unique relationship between loss of cementation and plastic shear strain as shown in Figures 8.28 and 8.29. Figures 8.28a to 8.28c show the results from sets of tests at a particular initial dry unit weight ($\gamma_d$). The data indicate that when the cement content is normalized by the initial cement content at the start of shearing ($G_{C_0}$) a unique response is observed for the range of cement contents investigated regardless of cementation and density. For each unit weight the normalized cement degradation is practically identical for all initial cement contents. Comparison of Figures 8.28a to 8.28c shows that similar normalized cement content reductions occur irrespective of the dry unit weight of the cemented material. When the curves from Figure 8.28 are plotted together as shown in Figure 8.29 it can be seen that the normalized reduction of cement content is similar in all tests, i.e. for all the cement contents and dry unit
weights investigated. The data for the highest dry unit weight lie slightly above the other curves, but if the unusual, and physically unreasonable, increase in cement content at the start of shearing is removed, these data would also lie very close to the data from lower dry unit weights. A good fit to the data is given by the theoretical curve shown in Figure 8.29, which has the expression:

\[
\frac{GC}{GC_0} = e^{-0.712 \varepsilon_{sp}^p}
\]  
(8.10)

Where, \( \varepsilon_{sp}^p \) is the cumulative plastic shear strain from the start of shear. The data shown in Figures 8.28 and 8.29 were only obtained up to the stage, close to failure, at which the first unloading occurred. Beyond this stage the responses diverged from the unique relation given by Equation 8.10. This is believed to be related to the development of failure planes, and associated non-uniform deformations that have not been considered in estimating either the strains or small strain shear modulus.

In some models for cemented materials the cement degradation is solely linked to the plastic volumetric strain (\( \varepsilon_v^p \)). When the cement content degradation was plotted against plastic volumetric strain there was a much greater range in the responses, shown in Figure 8.30, than shown in Figures 8.28 and 8.29. Figure 8.30 shows the results from sets of tests at different initial dry unit weights (\( \gamma_d \)). In the key on this plot the value following “g” is the percentage of gypsum (by dry weight) and that following “u” the dry unit weight. The data indicate that when the cement content degradation is normalized by the initial cement content at the start of shearing (\( GC_0 \)) and plotted against plastic volumetric strain (\( \varepsilon_v^p \)) a range of responses are observed.

Figure 8.31 shows the relationship between plastic volume strain (\( \varepsilon_v^p \)), and plastic shear strain (\( \varepsilon_s^p \)) for the tests with dry unit weights (\( \gamma_d \)) of 13, 15 and 17 kN/m\(^3\) with different initial gypsum cement contents (\( GC_0 \)). It is clear from these data that there is not a unique relationship between plastic volume strain and plastic shear strain, and hence there cannot be a unique relationship between cement degradation and both plastic shear and plastic volume strain for cemented materials. Previous studies have assumed the cement degradation as an exponential decay related to some measure of plastic volumetric strain (e.g. Gens & Nova, 1993, Shambhu, 2003, Gajo and Muir Wood, 1999). It may be noted that in these tests the mean effective stresses were well below the stresses required to cause yield in isotropic compression, which varied,
depending on cement content and unit weight, from 2 MPa to 20 MPa (Huang & Airey, 1998).

8.3.4 Discussion

This research has been limited to one cementing agent, gypsum, and one soil, a carbonate sand. The degradation of the cementation has been inferred from the shear wave velocity measurements based on the behaviour of the uncemented sand and cemented specimens subjected to a range of cement contents and a limited range of confining pressures. In deriving the equations relating $G_{\text{max}}$ to the cement content it was necessary to limit the range of effective mean stress considered to avoid regions where significant degradation of cement occurs. Thus it may be expected that predictions of cement content will become less reliable as the mean effective stress increases, particularly as the isotropic yield stress is approached. Also the independence of the cement degradation from plastic volume strain must cease as the mean effective stress approaches the isotropic yield stress. Further tests at higher stress levels are needed to explore the range of applicability of the equations derived in this study.

The equations use the gypsum cement content to describe the cementation effects and degradation, and this may limit the applicability of this study. However, even though the effectiveness of different cementing materials varies significantly, Salvati (2002) has reported similar forms of the equations relating $G_{\text{max}}$ to cement content still apply for cemented silica.

A procedure has been presented here to infer the degree of cementation from measurements of the small strain shear modulus, $G_{\text{max}}$. As noted above the small strain modulus is strongly dependent on the amount of cementation and this parameter can be measured non-destructively using wave velocity measurements.

Shear wave velocity measurements, and hence the small strain shear moduli, have been obtained continuously during triaxial shear tests on carbonate sand specimens artificially cemented with gypsum. Relations have been proposed linking $G_{\text{max}}$ to cement content, stress state and density. These relations have enabled an estimation of the cement content at all stages of a test to be made.

A unique relation between the rate of cement degradation and plastic shear strain has been shown for gypsum cement contents between 10% and 30% and dry unit weights between 13 kN/m$^3$ and 17 kN/m$^3$. 

8-20
8.4 Creep Test

Previous studies of (gypsum) artificially cemented carbonate sand have shown that considerable strains can develop when the deviator stress exceeds 50% of its peak value (Yeoh, 1996). This has been assumed to be a consequence of cement degradation. A special test was conducted to determine the variation of $G_{\text{max}}$ and hence estimate the evolution of cement during a similar creep load test.

This test was performed on an artificially cemented sample (C16G20%u13) with initial dry unit weight of 13 kN/m$^3$ and initial cement content of 20%. The loading rates for isotropic compression and shearing were 1 kPa/min and 0.005 mm/min respectively. Creep stages were performed on this sample at four different deviator stress states during drained shearing. The sample was first isotropically compressed to 300 kPa and then the cyclic triaxial test (CID) was performed under 300 kPa confining pressure. Along the CID stress path the stresses were kept constant for certain period of time at stress-states of $q = 600, 900, 1200, 1400$ kPa and $p' = 500, 600, 700, 765$ kPa respectively, and the creep response as well as the degradation of shear modulus ($G_{\text{max}}$) were observed. The stress level ($S$) at these stress-states was calculated considering the peak deviator stress ($q_{\text{peak}}$) of 1654 kPa during drained shear to failure of the same specimen. Huang (1994) and Yeoh (1996) have reported slightly higher values of mean peak deviator stress (i.e. 1723 kPa) for the same density and cement content in CID tests. In this study creep has reduced the $q_{\text{peak}}$ value compare to Huang (1994) and Yeoh (1996) value. The stress level ($S$) is defined here as the ratio of deviator stress ($q$) to peak deviator stress ($q_{\text{peak}}$) of 1654 kPa. Therefore creep tests were performed at stress levels of 0.36, 0.54, 0.73 and 0.84 on the same specimen.

8.4.1 Stress-Strain Response

Figure 8.32 shows the stress-strain and $G_{\text{max}}$ curves during the first and second cycles of shearing. The loading rate was 0.005 mm/min. Figure 8.32a shows that the axial strain increases at constant stress ratio ($q/p'$) during creep. The amount of increase in axial strain also increases as the stress level increases. Similarly shear modulus starts to decrease, more at higher stress level (i.e. at $q = 1200$ and 1400 kPa) than at lower stress level i.e. $q = 900$ kPa. At the lowest stress level (i.e. $q = 600$ kPa) axial strain developed but this did not affect the average $G_{\text{max}}$ which remained essentially constant. The fact that straining took place indicates that some breakdown of the cementation must be occurring. Individual bonds must be breaking, but the lack of
effect on $G_{\text{max}}$ suggests this is balanced by a reduction in void ratio or the micro-
cracks that may have developed could be in a direction that has no effect on the wave 
propagation.

It is interesting to note that the reduction of $G_{\text{max}}$ still continues during unloading (see 
Figure 8.32b). It means that cement breakdown continues once it starts even though 
the stress level starts to decrease during unloading (i.e. until $q$ drops close to zero). At 
this point i.e. at the end of first cycle of shearing it is believed that microscopic 
cracks, which were created during shearing, have close their gaps during unloading 
and as a result particle contact increases. Moreover, particle rearrangement occurs 
following the removal of deviator stress ($q$). There was a slight decrease in void ratio 
at the end of first cycle of shearing (see Figure 8.33). The increase in soil particle 
contact at the end of unloading increases $G_{\text{max}}$ slightly even though the cement bonds 
have already been partially broken during the past loading cycle (see Figure 8.32b). 

Figure 8.32b also shows that the slope of initial part of the stress-strain curve before 
creep (i.e. slope A1) is more than that after creep i.e. slope A4 in Figure 8.32c, which 
is consistent with their corresponding $G_{\text{max}}$ values. In these plots slope A1 and A4 
were calculated as 1071MPa and 819MPa respectively. Di Benedetto et al (1999) 
have also reported similar results, that there is a decrease in the slope of the stress-
strain curve at the end of the creep period compared to the one without creep. As 
expected slope A3 (i.e. 1060 MPa) is also less than the slope A1 and A2 (i.e. 1206 
MPa) as the stress level increases.

Figure 8.32c shows that the shear modulus starts to decrease at the start of fourth 
creep cycle at stress level $q = 1400$ kPa in a similar fashion to the second (i.e. S=0.54) 
and third (i.e. S=0.73) creep cycles. But, the rate of decrease in $G_{\text{max}}$ is greater than 
for the previous two creep states (i.e. S = 0.54 and 0.7). The reduction in $G_{\text{max}}$ has 
further accelerated at the end of the fourth creep stage when the sample starts drained 
shearing and approaches the peak deviator stress. Even though the sample has 
experienced four creep periods previously it clearly shows a sharp peak deviator stress 
of 1654 kPa, which is a phenomenon expected for a well cemented sample. On the 
other hand uncemented specimen with same density (i.e. $\gamma_d = 13$kN/m$^3$) does not show 
such a clear peak deviator stress (see Figure 7.15a). Certainly creep has reduced the 
peak deviator stress value compare to the value reported by Huang (1994) and Yeoh 
(1996) for 20% cement content (i.e. mean peak deviator stress of 1723 kPa). The
sample shows a post peak strain softening behavior, and the shear modulus curve also shows an exponential decay after yielding.

Figure 8.33 shows the void ratio response with $p'$. As expected the void ratio decreases at constant stress during the creep periods. The slopes of unloading and reloading lines are the same. However there is significant void ratio reduction (see Figure 8.33) due to creep at $q = 0$ kPa i.e. sample gets denser and stiffer after the first cycle of shearing. This is consistent with the slight increase in $G_{\text{max}}$ shown in Figure 8.32. This fact matches very well with the stress-strain curves as shown in Figures 8.32b and 8.32c. It has been observed that the slope of the loading curve in Figure 8.32c is higher than the slope of unloading curve in Figure 8.32b. According to Figure 8.33 after yielding, the void ratio starts to increase and the response is not uniform due the development of a rupture plane as a result of cementation breakdown. At this point changes in void ratio accelerate.

Figure 8.34a shows the volumetric strain response with axial strain of two samples, one was subjected to the creep test and the other one was subjected to a cyclic CID test with four cycles of loading and unloading. Due to sustained and repeated creep periods at different stress levels the sample shows sharp peak of volumetric strain at the time of yielding. The similar sample (C15G20%u15) without creep did not show such a sharp peak in the volumetric strain at the time of yielding. The sample continuously compresses until the peak volumetric strain after which it started to dilate due to the breakdown of cementation. Figure 8.34b shows that creep test leads to increases in peak deviator stress above those in conventional CID test for same degree of cementation but eventually both produce similar ultimate deviator stress.

In can be concluded from Figure 8.34 that:

a) Creep before unload leads to increases in $\varepsilon_v$ above those in conventional CID tests.

b) Unloading to $q = 0$ kPa produces significant volumetric creep.

c) Cycling and creep both produce similar strain accumulation and cause increases in $\varepsilon_a$, $\varepsilon_v$ and loss of cementation.

d) Cycling and creep both produce similar ultimate deviator stress

### 8.4.2 Volumetric and Axial Strain Response during Creep Period

Figure 8.35 shows the axial strain response with creep period. It shows that the initial axial strain increases with the increase of stress level. The rate of increase of axial
strain with creep time also increases with the increase of stress level in a positive exponential form.

Figure 8.36 shows the volumetric strain response during the creep period. It shows that the initial volumetric strain increases with the increase of stress level. Unlike the axial strain, the rate of increase of volumetric strain with creep period remains the same and parallel with the increase of stress level in a positive exponential form. Figure 8.37 shows the volumetric strain response with $p'$ during the creep period. It shows that the volumetric strain increases at constant $p'$ and constant stress ratio. The amount of volumetric strain increase is more at constant stress ratio than the increase of $p'$ and the increase in volumetric strain depends on duration of creep period.

### 8.4.3 $G_{\text{max}}$ and Cement Content Response during Creep Period

Figure 8.38 shows the shear modulus ($G_{\text{max}}$) and gypsum content (GC) response with creep period at different stress level (S). The gypsum content was calculated from the Equation 8.7 using the continuous shear wave velocity measurements from bender elements. It shows that the initial $G_{\text{max}}$ and cement content decreases as the stress level increases. It also shows that $G_{\text{max}}$ and GC remain constant at the start of creep until approximately 1000 sec has elapsed. At low stress level i.e. $q = 600$ kPa the changes in $G_{\text{max}}$ and GC are very small and both increase slightly with creep time. But at higher stress level (i.e. $S = 0.54$ to $0.84$) $G_{\text{max}}$ and GC start to decrease with the increase of creep period. This reduction of $G_{\text{max}}$ and cement content accelerates as the stress level increases at a negative exponential form with time, the sample is much stiffer at lower stress level than at higher stress level. Similar findings were also reported by Yeoh (1996).

### 8.4.4 Degradation of Cementation during Creep

Figure 8.39 shows the gypsum content and $G_{\text{max}}$ with $p'$ during the creep period. Figure 8.39 shows that from the beginning of shearing the cement content starts to degrade, and during the creep period the cement content also continues to degrade. According to the figure the cement content continues to degrade from the start of the second cycle of shearing. The cement content also continues to degrade during the second and third creep period at stress level $S = 0.54$ and $S = 0.73$ respectively. At the end of third creep period the cement content is 17% and the sample was unloaded to $q = 0$. It can be observed from the figure that there is a small increase in cement content before reloading. This is a common feature for pre-cast cemented and uncemented calcareous soil observed in this study. This can be explained by particles micro-
mechanical behaviour i.e. particles bond mechanism and particles contact mechanism. During shearing with increase of deviator stress \((q)\) cement bonds starts to break gradually and microscopic cracks or fissures develop inside the soil specimen. These microscopic cracks create gaps between the soil particles and reduce the particle contact area inside the soil specimen that interrupt the shortest and direct travel of shear wave through the sample, hence shear modulus \((G_{\text{max}})\) starts to decrease. But when the deviator stress is taken off from the sample during unloading these microscopic cracks start to close their gaps and increase the particle contact area to some extent, hence \(G_{\text{max}}\) increases slightly at the end of unloading. It is very hard to quantify this phenomenon, Fernandez & Santamarina (2001) tried but with little success to represent the actual \(G_{\text{max}}\) response as of Figure 8.9, 8.10 and 8.11.

At the end of the last creep period at stress level \(S=0.85\) the sample was allowed for drained shear to failure and the sample showed a clear peak in deviator stress (see Figure 8.32c) before \(q\) starts to fall down. This was expected because at the end of last creep period the cementing bond did not degrade completely and the estimated gypsum content at that stress level is around 13%. Therefore sample showed a sharp peak in stress-strain curve similar to a cemented sample of 13% cement content. This peak deviator stress is much higher than that of sample with 10% cement content as shown in Figure 7.42. This can be explained by decreasing nature in void ratio during creep period as shown in Figure 8.33. It is also evident that after the yielding the breakdown of cementation accelerates rapidly and degrade to 6% gypsum content at the end of shearing. The effect of this degradation of cementation reflects in the values of shear modulus, \(G_{\text{max}}\). The shear modulus decreases in a similar fashion to the cement degradation.

### 8.4.5 Comparison and Prediction of Creep Data

The prediction of creep data are based on the parameters from the drained triaxial test and on a semi-empirical model developed by Mitchell (1993), which incorporates the interrelationship between creep rate, stress level \((S)\) and time. This model is widely applicable for both undrained and drained creep behavior. Yeoh (1996) has modified this model to suit the drained creep behavior of the artificially cemented soil. Further the creep deformation of ground, displacement of structures and constitutive modelling of time dependent stress-strain properties of geomaterials have been classically presented by Tatsuoka et al (2001), and Kuwano & Jardine (2002).
Figure 8.40 shows the comparison of strain data with previous study by Yeoh (1996) for the same unit weight (13 kN/m$^3$) and cement content (20%). The stress level of two samples is very close but not the same. Because Yeoh (1996) took the average peak deviator stress of 1723 kPa from four CID tests with unit weight of 13 kN/m$^3$ and cement content of 20% where $q_{\text{peak}}$ ranging from 1587 kPa to 1850 kPa. In this study $q_{\text{peak}}$ was found to be 1654 kPa, which is within the range of Yeoh’s (1996) data.

Figure 8.40a shows the similar initial axial strain for both study and the trend of axial strain increase with time is also similar for both tests. At higher creep time the axial strain from previous study is more than the present study. This was expected because present study deployed internal instrumentation for strain measurement compare to previous study. For other stress level the axial strain response can be expected similar with creep time.

Figure 8.40b shows the volumetric strain comparison with creep time. Previous study shows little bit higher volumetric strain than this study. The reason behind it is the GDS volume measurement for volumetric strain calculation for previous study, which is not very accurate in small-strain range compare to the internal Hall Effect Transducer (HET) in present study, otherwise data agreement and pattern are quite consistent. Therefore it can be expected that the data for other stress level would also be similar with the previous study.

An attempt has been made to compare the predicted and measured time-dependent axial strain rate and axial strain for sustained stress level from this study. Yeoh’s (1996) modified semi-empirical model has been used here to predict the time-dependent axial strain rate and axial strain for different stress level. The assumption of this model is that the stress-strain response is assumed to be linear.

It has been observed from the following plots that there is a good agreement between the predicted and measured time-dependent axial strain rate and axial strain of this research at different stress level during the creep test.

According to Figure 8.41a the predicted and actual strain rate decreases non-linearly with respect to creep time at each stress level. The stress level has been represented by the symbol ‘S’ in the plot, which is the ratio of deviator stress ($q$) over peak deviator stress ($q_{\text{peak}}$). The slope of each curve is influenced by the stress level and the slope decreases gradually as the creep time increases. Also this slope decreases more as the
stress level increases and may cause a reversal to the slope leading to creep rupture if the creep period prolongs for long time.

Figure 8.41b shows that the predicted and actual axial strain increases non-linearly with respect to the logarithm of time at each stress level. The predicted initial axial strain is below the actual one at high stress level because of the test procedure. The strain at stress level 0.36 has been accumulated in next two stress level. It also shows that the predicted axial strains are above the measured data at higher stress level, this was expected because the empirical formula assumed only linear small-strain response of stress-strain curve, which is not true for higher stress level and at large strain. The second reason was that only one sample was used for creep at different stress level, therefore creep at higher stress level might have influence by the creep at previous stress level.

The difference in the predicted and actual axial strain response shown in Figure 8.41b is related to the way creep test was conducted. That is the earlier creep stages influence the following two parameters-

a) Initial strain – which is bigger because of prior creep.

b) Rate of strain - which is reduced because of prior creep.

Nevertheless the predicted trend is similar to the actual response.

**8.4.6 Summary and Conclusion**

- Empirical $G_{\text{max}}$ relationships have been derived for uncemented and cemented soil, which are capable to predict the shear stiffness value and cement degradation with reasonable agreement with the tested data.

- The derived empirical relations have been compared with previously published relations. It has been shown that existing empirical relations either under predict or over predict the actual $G_{\text{max}}$ response.

- Degradation of cementation occurs from an early stage of shearing but it is not visible/evident at the start of shearing. For the highly cemented sample this degradation of cement is sudden/rapid but for low cemented soil it is slow and gradual.

- Cement degradation is not solely linked to the plastic volumetric strain ($\varepsilon_v^p$) or plastic shear strain ($\varepsilon_s^p$), and there is not a unique relationship
between cement degradation and both plastic shear and plastic volume strain for cemented materials.

- Creep causes increase in axial and volumetric strain and loss of cementation.
- The shear modulus ($G_{\text{max}}$) decreases with the increase of stress level (S) during the creep period.
## Table 8.1: Empirical Relations for $G_{\text{max}}$ for Uncemented Sandy Soil

<table>
<thead>
<tr>
<th>Maximum Shear Stiffness for Uncemented Soil</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$G_{\text{max}} = C \cdot e^{(-x)} \cdot (p'_{a})^n$</td>
<td>Lo Presti et al (1997)</td>
</tr>
<tr>
<td>Here, $C$ is material constant and $p_{a}$ is the atmospheric pressure. $C = 724$, $n = 0.45$ and $x = -1.3$ for Toyoura sand</td>
<td></td>
</tr>
<tr>
<td>$G_{\text{max}} = 5000 \cdot e^{(-1.5)} \cdot (p')^{0.5}$</td>
<td>Shibuya et al (1996)</td>
</tr>
<tr>
<td>Here, $p'$ and $G_{\text{max}}$ are in kPa</td>
<td></td>
</tr>
<tr>
<td>$G_{\text{max}} = 81.4 \cdot (4.16-e^2/(1+e)) \cdot (p')^{0.62}$</td>
<td>Fioravante et al (1994)</td>
</tr>
<tr>
<td>Here, $p'$ and $G_{\text{max}}$ are in MPa</td>
<td></td>
</tr>
<tr>
<td>$G_{\text{max}}/ p_{a} = 63.5 \cdot K_{2\text{max}} \cdot (\sigma'<em>{a}/p</em>{a})^{0.64}$</td>
<td>Dobry et al (1988)</td>
</tr>
<tr>
<td>$K_{2\text{max}} = 12$ for Carbonate sand, $p_{a} = 100$ kPa</td>
<td></td>
</tr>
<tr>
<td>$G_{\text{max}} = 700 \cdot [(2.17-e)^2/(1+e)] \cdot (p')^{0.5}$</td>
<td>Iwasaki et al (1978)</td>
</tr>
<tr>
<td>Here, $p'$ and $G_{\text{max}}$ are in Kg/cm$^2$</td>
<td></td>
</tr>
<tr>
<td>$G_{\text{max}} = 2630 \cdot [(2.17-e)^2/(1+e)] \cdot (\sigma')^{(0.5)}$</td>
<td>Chiang &amp; Chae (1972)</td>
</tr>
<tr>
<td>Here, $\sigma'$ (effective mean stress) and $G_{\text{max}}$ are in psi</td>
<td></td>
</tr>
<tr>
<td>$G_{\text{max}} = 3270 \cdot [(2.17-e)^2/(1+e)] \cdot (\sigma')^{(0.5)}$</td>
<td>Hardin &amp; Black (1969)</td>
</tr>
<tr>
<td>Here, $\sigma'$ (effective mean stress) and $G_{\text{max}}$ are in kPa</td>
<td></td>
</tr>
<tr>
<td>$G_{\text{max}} = F(e) \cdot C_{G} \cdot (p_{a})^{(1-2na-nb)} \cdot (\sigma'<em>{a})^{2na} \cdot (\sigma'</em>{b})^{2nb}$</td>
<td>Jamiolkowski et al (1991)</td>
</tr>
<tr>
<td>Here, $F(e)$ is void ratio function, $C_{G}$ is the material constant depending on $F(e)$ and PI, $\sigma'<em>{a}$ and $\sigma'</em>{b}$ are principal effective stress in the direction of shearwave propagation and particle motion respectively, $na$ and $nb$ are modulus exponents, $p_{a}$ is the atmospheric pressure $= 98.1$ kPa $= 1$ bar.</td>
<td></td>
</tr>
</tbody>
</table>
### Table 8.2: Empirical Relations for $G_{\text{max}}$ for Cemented Soil

<table>
<thead>
<tr>
<th>Maximum Shear Stiffness ($G_{\text{max}}$) for Cemented Soil</th>
<th>Material Studied</th>
<th>Cement Type</th>
<th>Test Type or Apparatus</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$G_{\text{max}}/p_a = G_b \cdot e^{(-1.3)((p' + p_l)/p_a)^m}$</td>
<td>1. Fine grained offshore calcareous soil from Goodwyn 2. Coastal aeolian calcareous soil from Ledge Point</td>
<td>CIPS chemical test</td>
<td>Cyclic Triaxial test</td>
<td>Shambhu (2003)</td>
</tr>
<tr>
<td>$G_{\text{max}}/p_a = G_b \cdot F_{cc} \cdot e^{(-1.3)} {(p' + a_{cc} CC^2)/p_a} \cdot \left[1 + (1/3)*(1+\nu)/(1-2\nu)\right]^{n/2}$</td>
<td>Sacramento river sand &amp; Monterey No.0 sand</td>
<td>Portland cement</td>
<td>Bender element test &amp; Resonant column test</td>
<td>Salvati (2002)</td>
</tr>
<tr>
<td>$E_T/G_m = {(1-(1-\nu))}/[(CC+1)^{2/3} - 1 + {(3/2)<em>(1-\nu)</em>(p'/G_m)}^{2/3}]^{0.5}$ $E_T = 2<em>G_{\text{max}}</em>(1+\nu)$</td>
<td>Fine sub-angular siliceous sand</td>
<td>Portland cement No.2</td>
<td>Stokoe torsional resonant column test</td>
<td>Fernandez &amp; Santamarina (2001)</td>
</tr>
</tbody>
</table>
Table 8.2: Empirical Relation of $G_{\text{max}}$ for Cemented Soil

<table>
<thead>
<tr>
<th>Maximum Shear Stiffness ($G_{\text{max}}$) for Cemented Soil</th>
<th>Material Studied</th>
<th>Cement Type</th>
<th>Test Type or Apparatus</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$G_{\text{max}} = M_g G_o$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$M_g = [M_{\text{th}} + (C - C_{\text{th}}) \tan {(C_u 0.3/(1+D_{10})) \beta \sigma_o^{(-n)}}]$</td>
<td>$\sigma_o$ in psi &amp; $D_{10}$ in mm; $C_u = D_{60}/D_{10} = \text{uniformity coefficient}$, $M_g$ is modulus ratio of cemented sand, $C_{\text{th}}$ is threshold cementation level, $\beta$ &amp; $n$ are constants depend on cement type</td>
<td>$M_{\text{th}} = M_g$ corresponding to $C_{\text{th}}$ and CC is cement content</td>
<td>1. Sodium silicate cement</td>
<td>Chang &amp; Wood (1992)</td>
</tr>
<tr>
<td>$G_0 = \rho V_s^2$ and $V_s = (159-53.5*e) \sigma_0^{1/4}$</td>
<td>$\sigma_0$ is in psf and $V_s$ in ft/sec</td>
<td>2. Medium grained sand</td>
<td>Hall type resonant column test</td>
<td></td>
</tr>
<tr>
<td>$G_{\text{max}} / p_a = [428.2/(0.3+0.7<em>e^2)] \sigma_o^{0.574} + (773/e) * CC^{1.2} * [(\sigma_o / p_a)]^{0.698</em>e - 0.04*CC - 0.2}$</td>
<td>Monterey No.0 sand</td>
<td>Portland cement Type 1</td>
<td>Drnevich type long torsional resonant column test</td>
<td>Saxena et al (1988)</td>
</tr>
<tr>
<td>$G_{\text{max}} / p_a = 63.5*K_{2\text{max}} *(\sigma_o / p_a)^{0.64}$</td>
<td>Calcareous soil from North Rankin “A” site at 9 to 34m depth</td>
<td>Naturally cemented</td>
<td>1. Strain controlled undrained cyclic triaxial test 2. Consolidated resonant column test</td>
<td>Dobry et al (1988)</td>
</tr>
</tbody>
</table>

$K_{2\text{max}} = 23$ to $27$ for Carbonate sand, and $p_a = 100$ kPa
Table 8.2: Empirical Relation of $G_{\text{max}}$ for Cemented Soil

<table>
<thead>
<tr>
<th>Material Studied</th>
<th>Cement Type</th>
<th>Test Type or Apparatus</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Monterey No.0 sand and 2. Monterey No.30 sand</td>
<td>Portland cement</td>
<td>Torsional resonant column test</td>
<td>Acar &amp; El-Tahir (1986)</td>
</tr>
<tr>
<td>1. Uniform sand and 2. Fine silty clay</td>
<td>Type 1 Portland cement</td>
<td>Torsional resonant column test</td>
<td>Chiang &amp; Chae (1972)</td>
</tr>
</tbody>
</table>

Maximum Shear Stiffness ($G_{\text{max}}$) for Cemented Soil

\[
G_{\text{max}} / p_a = (1 + CC^{0.49} - 2*CC^{0.1}*e^{0.46})*(G_o / p_a)
\]
\[
G_o / p_a = \left[ \frac{631}{(0.3 + 0.7*e^2)} \right] \times (\sigma_o' / p_a)^{0.43}
\]

Here, $p_a$ is the atmospheric pressure, CC is cement content.

\[
G_{\text{max}} = \{G_o - 0.343*CC*(\sigma_o')^{0.5}]*((\sigma_o')^{0.06*CC})
\]
\[
G_o = 2630*[(2.17 - e)^2/(1 + e)]*(\sigma_o')^{0.5}
\]

Here, $G_{\text{max}}$, $G_o$ and $\sigma_o'$ are in psi.
Table 8.3: Comparison of Parameters Used for Different Empirical Equation Study for Cemented Carbonate Sand Samples.

<table>
<thead>
<tr>
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<tbody>
<tr>
<td>$p_a = 1$ kPa</td>
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<tr>
<td>GC = 10%</td>
<td>$m = 0.55$</td>
<td>$v = 0.18$</td>
<td>$n = 0.4$</td>
<td>$n = 0.5$</td>
<td>$n = 0.5$</td>
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<tr>
<td>$\gamma_d = 13$ kN/m$^3$</td>
<td>$G_b = 858$</td>
<td>$G_b = 700$</td>
<td>$F_{cc} = 1$</td>
<td>$\beta = 0.233$</td>
<td>$\beta = 0.233$</td>
<td>$\beta = 0.233$</td>
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<td>$\beta = 0.233$</td>
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<tr>
<td>$p_l = 1750$ kPa</td>
<td>$a_{cc} = 0.7$</td>
<td>$CC = 10%$</td>
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<tr>
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<td>$v = 0.3$</td>
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<tr>
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<td>$a_{cc} = 0.7$</td>
<td>$CC = 20%$</td>
<td>$G_m = 40$ MPa</td>
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Note: (1) – Refer to Equation 8.8  
(2) – Refer to Table 8.2
Table 8.3: Comparison of Parameters Used for Different Empirical Equation Study for Cemented Carbonate Sand Samples.

<table>
<thead>
<tr>
<th></th>
<th>This Study (1)</th>
<th>Shambhu (2) 2003</th>
<th>Salvati (2) 2002</th>
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<th>Chang et (2) al 1992</th>
<th>Saxena et (2) el 1988</th>
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Table 8.3: Comparison of Parameters Used for Different Empirical Equation Study for Cemented Carbonate Sand Samples.

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<td>CC = 6%</td>
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<tr>
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<td>G_b = 700</td>
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<td>CC = 8%</td>
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<tr>
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<td>G_b = 700</td>
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Note: (1) – Refer to Equation 8.8  
(2) – Refer to Table 8.2
Figure 8.1: Variation of actual and predicted response of $G_{\text{max}}$ for Toyoura Sand with $p'$.
Figure 8.2: Variation of actual and predicted response of $G_{\text{max}}$ from other studies for Toyoura Sand with $p'$
Figure 8.3: Variation of $G_{\text{max}}$ for uncedmented and compressed pre-cast carbonate sand with $p'$ (a) During Isotropic Compression, and (b) Drained Shearing at $\sigma_c' = 300$ kPa
Figure 8.4: Comparison of $G_{\text{max}}$ data and Prediction from Equation 8.4 for Uncemented Carbonate Sand (a) $\gamma_d=17\text{kN/m}^3$, (b) $\gamma_d=15\text{kN/m}^3$ and (c) $\gamma_d=13\text{kN/m}^3$.
Figure 8.5: Comparison of $G_{\text{max}}$ data and Prediction from Equation 8.5 for Uncemented Carbonate Sand (a) $\gamma_d=17\text{kN/m}^3$, (b) $\gamma_d=15\text{kN/m}^3$ and (c) $\gamma_d=13\text{kN/m}^3$
Figure 8.6: Comparison of $G_{\text{max}}$ data and Predictions from Other Studies for Uncemented Carbonate Sand (a) $\gamma_d=17\,\text{kN/m}^3$, (b) $\gamma_d=15\,\text{kN/m}^3$ and (c) $\gamma_d=13\,\text{kN/m}^3$.
Figure 8.7: Comparison of $G_{\text{max}}$ data and Prediction from Equation 8.7 for Cemented Carbonate Sand (a) $\gamma_d=17\text{kN/m}^3$, (b) $\gamma_d=15\text{kN/m}^3$ and (c) $\gamma_d=13\text{kN/m}^3$, [GC = Gypsum Content].
Figure 8.8: Variation of $E_{sec}$ at $\varepsilon_a = 0.01\%$ of artificially cemented soil samples with (a) void ratio, (b) gypsum content and (c) predicted $E_{sec}$ at different dry densities.
Figure 8.9: Comparison of $G_{\text{max}}$ Predictions from Other Studies Cemented Carbonate Sand (a) $\gamma_d=13\text{kN/m}^3$, (b) $\gamma_d=15\text{kN/m}^3$ and (c) $\gamma_d=17\text{kN/m}^3$ for 10% Gypsum Content.
Figure 8.10: Comparison of $G_{\text{max}}$ Predictions from Other Studies Cemented Carbonate Sand (a) $\gamma_d=13\text{kN/m}^3$, (b) $\gamma_d=15\text{kN/m}^3$ and (c) $\gamma_d=17\text{kN/m}^3$ for 20% Gypsum Content.
Figure 8.11: Comparison of $G_{\text{max}}$ Predictions from Other Studies Cemented Carbonate Sand (a) $\gamma_d=13\text{kN/m}^3$, (b) $\gamma_d=15\text{kN/m}^3$ and (c) $\gamma_d=17\text{kN/m}^3$ for 30% Gypsum Content.
Figure 8.12: Small strain response $G_{\text{max}}$ with degree of cementation for cemented carbonate sand samples and dry unit weight of 13kN/m$^3$.

Figure 8.13: Small strain response $G_{\text{max}}$ with degree of cementation for cemented carbonate sand samples and dry unit weight of 15kN/m$^3$. 
Figure 8.14: Small strain response $G_{\text{max}}$ with degree of cementation for cemented carbonate sand samples and dry unit weight of 17kN/m$^3$.

Figure 8.15: Comparison between $G_{\text{max}}$ and $G_{\text{sec}}$ during shearing for Cemented Carbonate Sand Samples of $\gamma_d=13$ kN/m$^3$ and $\sigma'_c=300$ kPa.
Figure 8.16: Comparison between $G_{\text{max}}$ and $G_{\text{sec}}$ during shearing for Cemented Carbonate Sand Samples of $\gamma_d=15 \, \text{kN/m}^3$ and $\sigma_c' = 300 \, \text{kPa}$.

Figure 8.17: Comparison between $G_{\text{max}}$ and $G_{\text{sec}}$ during shearing for Cemented Carbonate Sand Samples of $\gamma_d=17 \, \text{kN/m}^3$ and $\sigma_c' = 300 \, \text{kPa}$. 
Figure 8.18: Comparison between $G_{\text{max}}$ and $G_{\text{sec}}$ during shearing at constant $p'$ for Cemented Carbonate Sand Samples of $\gamma_d = 15 \text{kN/m}^3$ and GC = 30%. 
Figure 8.19: Comparison between $G_{\text{max}}$ and $G_{\text{sec}}$ during shearing at constant $p'$ for Cemented Carbonate Sand Samples of $\gamma_d=15$ kN/m$^3$ and GC = 20%.

Figure 8.20: Response of $G_{\text{max}}$ to Cement Content for Cemented Carbonate Sand of different initial unit weight i.e. $\gamma_d=13$, 15 and 17 kN/m$^3$ (at $p'=300$ kPa and $q=0$).
Figure 8.21: (a) Prediction of $G_{\text{max}}$ from Equation 8.7 (at $p'=300\text{kPa}$) and $q_u$ from Equation 8.9, (b) Comparison of $q_u$ from this study anf from Equation 8.9, and (c) ratios of predicted $G_{\text{max}}$ and $q_u$ with cement content.
Figure 8.22: Variation of $G_{\text{max}}$ and estimated Gypsum Content (GC) with $p'$ for specimen with initial dry unit weight of 13 kN/m$^3$ and initial cement content of 10%.

Figure 8.23: Variation of $G_{\text{max}}$ and estimated Gypsum Content (GC) with $p'$ for specimen with initial dry unit weight of 13 kN/m$^3$ and initial cement content of 20%.

Figure 8.24: Variation of $G_{\text{max}}$ and estimated Gypsum Content (GC) with $p'$ for specimen with initial dry unit weight of 13 kN/m$^3$ and initial cement content of 30%.
Figure 8.25: Variation of $G_{\text{max}}$ and estimated Gypsum Content (GC) with $p'$ for specimen with initial dry unit weight of 15 kN/m$^3$ and initial cement content of (a) 10%, (b) 20% and (c) 30%.
Figure 8.26: Variation of $G_{\text{max}}$ and estimated Gypsum Content (GC) with $p'$ for specimen with initial dry unit weight of 17 kN/m$^3$ and initial cement content of (a) 10%, (b) 20% and (c) 30%.
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Figure 8.27: Variation of estimated Gypsum Content (GC) with Plastic Shear Strain for specimen with initial dry unit weight of a) $\gamma_d=13$ kN/m$^3$, b) $\gamma_d=15$ kN/m$^3$ and c) $\gamma_d=13$ kN/m$^3$.
Figure 8.28: Variation of Normalized Gypsum Content with Plastic Shear Strain for specimen with initial dry unit weight of a) $\gamma_d=13\,\text{kN/m}^3$, b) $\gamma_d=15\,\text{kN/m}^3$ and c) $\gamma_d=17\,\text{kN/m}^3$. 
Figure 8.29: Variation of Normalized Gypsum Content with Plastic Shear Strain for specimen with initial dry unit weight of 13, 15 and 17 kN/m$^3$ and initial cement content of 10%, 20% and 30%.
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Figure 8.30: Variation of Normalized Gypsum Content with Plastic Volumetric Strain for specimen with initial dry unit weight of a) $\gamma_d = 13\, \text{kN/m}^3$, b) $\gamma_d = 15\, \text{kN/m}^3$ and c) $\gamma_d = 17\, \text{kN/m}^3$. 
Figure 8.31: Variation of Plastic Shearing Strain with Plastic Volumetric Strain for specimen with initial cement content of 10%, 20% and 30% and initial dry unit weight of (a) $\gamma_d=13$ kN/m$^3$, (b) $\gamma_d=15$ kN/m$^3$, and (c) $\gamma_d=17$ kN/m$^3$. 
Figure 8.32: Variation of $q$ and $G_{\text{max}}$ during a creep test for cemented carbonate sand with initial $\gamma_d = 13 \text{ kN/m}^3$ and GC = 20%, a) complete shearing, b) 1$^{\text{st}}$ cycle of shearing and c) 2$^{\text{nd}}$ cycle of shearing at $\sigma_c' = 300 \text{ kPa}$
Figure 8.33: Response of void ratio and $G_{\text{max}}$ for cemented carbonate sand with initial unit wt. of 13 kN/m$^3$ and gypsum content of 20%.
Figure 8.34: Comparison of creep test and normal CID test for the response of (a) volumetric strain and (b) deviatoric stress with axial strain for cemented carbonate sand with initial dry unit weight of 13 kN/m$^3$ and gypsum content of 20%.
Figure 8.35: Response of axial strain with creep time for cemented carbonate sand with initial unit wt. of 13 kN/m$^3$ and gypsum content of 20%.
Figure 8.36: Response of volumetric strain with creep time for cemented carbonate sand with initial unit wt. of 13 kN/m$^3$ and gypsum content of 20%.

Figure 8.37: Response of volumetric strain with $p'$ for cemented carbonate sand with initial unit wt. of 13 kN/m$^3$ and gypsum content of 20%.
Figure 8.38: Response of $G_{\text{max}}$ with creep time for cemented carbonate sand with initial unit wt. of 13 kN/m$^3$ and gypsum content of 20%.

Figure 8.39: Degradation of cementation and $G_{\text{max}}$ during creep for cemented carbonate sand with initial unit wt. of 13 kN/m$^3$ and gypsum content of 20%. ($S =$ stress level)
Figure 8.40: Comparison of Strain with creep time for cemented carbonate sand with initial unit wt. of 13 kN/m$^3$ and gypsum content of 20% a) for axial strain, b) for volumetric strain.
Figure 8.41: Effect of stress level on a) axial strain rate and b) axial strain with creep time for cemented carbonate sand with initial unit wt. of 13 kN/m$^3$ and gypsum content of 20%.