CHAPTER 7

BEHAVIOUR OF CEMENTED AND UNCEMENTED CALCAREOUS SOIL

7.1. Introduction
A series of drained triaxial tests was carried out to study the behavior of artificially cemented soil. All the tests were performed on samples of 55 mm. diameter and 110 mm height. Effective confining stresses ranging from 100 kPa to 1500 kPa were used in monotonic and cyclic load tests to investigate the stiffness behaviour of artificially cemented calcareous soil. The data of samples sheared at pressures in the range of 100 kPa to 1500 kPa effective confining stress are presented in this chapter.

As discussed in Chapter 2 the cost of obtaining undisturbed samples, the variability within the samples, and possible sample disturbance may hinder the derivation of correct stress-strain relationships for naturally cemented specimens. Therefore it was necessary to use artificially cemented samples for the triaxial test program using gypsum, as the cementing agent. The advantage of artificially cemented samples is their uniformity, homogeneity and reproducibility. This cemented soil has been widely used in the University of Sydney to study the behavior of footings (e.g. Yeoh, 1996; Pan, 1999) and various soil parameters (e.g. Huang, 1994).

The general aim of this chapter is to present and analyze experimental data concerning the small strain stiffness of carbonate sediments both cemented and uncemented, using Bender Elements and internal displacement measurements. This chapter mainly focuses on the stress-strain behaviour (at small and large strain) of cemented and uncemented carbonate sand. It reports how the shear modulus \( G_{\text{max}} \) varies due to different levels of cementation and dry density \( \gamma_d \) during isotropic compression and shearing, and investigates the effects of cement content and density on the stress-strain behaviour.

7.2 Materials Used, Sample Preparation and Testing Procedure
The basic ingredients used for the investigation of cemented and uncemented soil samples consist of calcareous soil, gypsum cementing agent and water. The material
properties, sample preparation, testing procedure and mix design ‘recipe’ have been explained in detail in Chapter 4. This section describes the engineering characteristics of the materials used.

7.3 Behavior of Uncemented Calcareous Soil Samples

The fundamental approach employed to investigate the small-strain response of uncemented calcareous soil involved a series of triaxial tests from which relationships could be drawn linking $G_{\text{max}}$ with void ratio, $e$, mean effective stress, $p'$, and stress ratio ($q/p'$). A similar test series has been performed for the cemented calcareous soil. In order to increase the validity of the results, tests were carried out with different stress path to observe the variation of $G_{\text{max}}$. Two types of uncemented calcareous soil samples were tested. One type was cast in-place, this was a conventional sand preparation technique prepared by pluviation and tamping at relative densities from 0 to 1. The other type was compressed one dimensionally to produce over-consolidated specimens with higher densities.

This section describes in detail the preparation of the samples and procedures used to execute the tests. This section also gives a summary of the key findings from the experiments and the relationships found between $G_{\text{max}}$ and other soil parameters.

This project extends previous research (Huang, 1994) that investigated the large strain behavior of uncemented and Gypsum cemented calcareous soil. Huang, 1994 reported that cementation shifts the Critical State Line (CSL) as well as the Normally Consolidated Line (NCL) to some extent as shown in Figure 7.1. In this figure ‘GC’ refers to the ‘Gypsum Content’. In the following sections all the data for uncemented and cemented calcareous soil samples from this research will be compared with Huang’s data, and in particular with the CSL and INCL.

7.3.1 Behavior of Uncemented Cast In-Place Calcareous Soil Samples

Eight uncemented cast in-place samples were tested to investigate the stiffness ($G_{\text{max}}$) responses along different stress paths with densities varying from loose to dense. The same grading was used for these samples as for sand in the cemented samples. The sample manufacturing technique was the same as for the Toyoura samples. Different types of test procedure were followed for these samples such as CID, CIU, constant $p'$, constant $q$, constant $\sigma_1'$ etc. The test conditions of all the uncemented cast in place calcareous soil samples have been summarized in Table 3.3.
Information on conditions at failure and other salient findings for the uncemented cast in-place calcareous samples are summarized in Table 7.1. The various stress paths followed during shearing for these tests are shown in Figure 7.2. A detailed discussion and presentation of the test data is given in the following section.

7.3.1.1 Effect of Isotropic Compression on Void Ratio and $G_{\text{max}}$

Figure 7.3 shows the isotropic compression response i.e. void ratio and $G_{\text{max}}$ with $p'$ for the uncemented cast in place calcareous samples. Figure 7.3a shows that the void ratio decreases with the increase of $p'$. There is a significant amount of void ratio reduction at the end of isotropic compression, indicating that this material is highly compressible. All the samples reached the INCL during Isotropic compression except samples N8 and N9 due to their low $p'$. One sample (N2) has been isotropically expanded after compression to 1MPa stress, and Figure 7.3a shows that some permanent volume change has occurred and the sample has become over-consolidated.

Figure 7.3b shows the variation of $G_{\text{max}}$ with void ratio during the isotropic compression. It shows that initially dense samples possess higher $G_{\text{max}}$ values compared to the initially loose samples with higher void ratio. It also shows that $G_{\text{max}}$ increases with the reduction of void ratio during isotropic compression. Figure 7.3b also shows that there is an increase in $G_{\text{max}}$ at the end of isotropic expansion.

Figure 7.3c shows the variation of $G_{\text{max}}$ with $p'$. $G_{\text{max}}$ increases with the increase of $p'$. It is noticeable from the plot that as the stress level increases $G_{\text{max}}$ of all the samples converges towards one single ‘normally consolidated’ path. But during isotropic expansion, when the stress is reduced it follows a different path which is above the ‘normally consolidated path’. This behaviour is in contrast to the almost unique $G_{\text{max}}$ - $p'$ response shown by Toyoura sand in compression and expansion. The main difference between the two sands is the greater compressibility of the calcareous sand and the significant reduction in void ratio, which occurs during compression. It is believed that during recompression $G_{\text{max}}$ will follow the previously traveled isotropic expansion path and eventually meet with the ‘normally consolidated’ path. Therefore for uncemented insitu calcareous soil a range of recompression paths may be expected but these should approach a unique ‘normally consolidated path’ for $G_{\text{max}}$ with increasing $p'$. Shibuya et al (2001) have presented a similar relationship for reconstituted clay and termed the ‘normally consolidated path’ a state boundary curve (SB) similar to $e$-$\text{ln}(p')$ relationship. Figure 7.3c also shows that $G_{\text{max}}$ increases at
constant $p'$ due to decrease in initial void ratio similar to Toyoura sand. This effect is small possibly due to the proportionally smaller change in void ratio between loose and dense states for this sand.

7.3.1.2 Small-Strain $G_{\text{max}}$ Comparison

Figure 7.4 shows the comparison of $G_{\text{max}}$ measurements between bender elements and estimated from stress-strain curves. It shows that conventional $G_{\text{max}}$ measurements from the stress-strain curve tend to under-estimate the $G_{\text{max}}$ values. The reason for the under estimation of $G_{\text{max}}$ values in these tests may lie in inappropriate alignment of axial HETs during gluing of the HET onto the membrane. The accuracy of HETs, comparison with $G_{\text{max}}$, and with external measurements has been discussed in detail in Chapter 3. Therefore $G_{\text{max}}$ measurements using wave velocity method are considered more consistent and reliable over internal or external measurements.

7.3.1.3 Large-Strain Response of Uncemented Cast in-Place Samples

Figure 7.5a shows the stress strain curves of all the uncemented cast in place calcareous samples. The samples with high confining pressure (e.g. $\sigma_c' \geq 870$ kPa) show strain hardening behaviour during shearing. On the other hand the samples with low ($\sigma_c' \leq 500$ kPa) confining pressure show strain softening behaviour. It also shows the initial bedding problem for one sample (constant $p'$ test). Due to the instrument limitation one sample (N7) could not reach failure, but it shows a distinct yield point, due to the high confining pressure ($\sigma_c' = 1500$ kPa).

Figure 7.5b shows the stress ratio ($q/p'$) response of all uncemented and uncompressed cast in place calcareous soil samples with axial strain. It shows that as the axial strain increases all the samples merge towards an ultimate stress ratio of 1.5 during shearing. Huang (1994) has reported an ultimate stress ratio of 1.6 for uncemented calcareous soil in undrained tests. Normally for drained tests the ultimate stress ratio appears to be less than for undrained tests because of the samples developing non-uniformity and Huang (1994) has also reported a similar tendency.

Figure 7.6 shows the volumetric strain responses of uncemented cast in place calcareous samples with axial strain. For the drained shear tests volumetric strain changes with the increase of axial strain. It also shows that some samples expanded and some samples compressed during shearing depending on their void ratio ($e$), and confining pressure ($p_c'$) at the start of shear.
Figure 7.7a shows the relationship between void ratio and $p'$ during shearing for all uncemented cast in place calcareous samples. It shows that one sample initially resides on the dense side of the CSL but the rest of the samples reside on the loose side of the CSL. Therefore during shearing the dense sample dilates, void ratio increases and moves to meet the CSL upward. On the other hand void ratio of the samples residing on the loose side of the CSL decreases during shearing, and samples try to reach the CSL downward. The loose state of the samples is not the only reason for such behavior, but the different stress paths of the samples are also a contributory factor. It is noticeable that all the samples reside below the INCL for uncemented carbonate sand as expected. Figure 7.7 also shows a common trend for the uncemented cast in place calcareous samples to try to reach to the CSL during shearing, whether it is drained, or undrained test, or constant $p'$ test. Figure 7.7a also shows that most of the samples did not reach Huang’s (1994) CSL for uncemented carbonate sand due to insufficient axial strain. Normally for uncemented carbonate sand to reach the CSL axial strains more than 25% are required (Ismail 2000), but in this study tests were stopped at axial strains less than 15%, because the aim of this research was to explore the small-strain behaviour.

Figure 7.7b shows the variation of $G_{\text{max}}$ with $p'$ during shearing for uncemented cast in-place calcareous samples. It shows that for undrained shear tests, constant $p'$ tests and constant $\sigma_1'$ tests $G_{\text{max}}$ decreases from the beginning of shearing until failure, but for the drained (CID) tests $G_{\text{max}}$ increases at the beginning of shearing with the increase of $p'$ and when the sample passes some yield point, $G_{\text{max}}$ starts to decrease with further increase of $p'$. It also shows that eventually the path of $G_{\text{max}}$ approaches towards an ultimate state $G_{\text{max}}$ line with the increase of $p'$. All the $G_{\text{max}}$ data for uncemented cast in-place calcareous soil falls in a narrow band during shearing associated with the change of $p'$.

Figure 7.8 shows the complete variation of $G_{\text{max}}$ along with the stress path during isotropic compression and shearing for sample, N9. It shows that $G_{\text{max}}$ drops rapidly after the sample crosses the ‘yield’ point. It also shows that $G_{\text{max}}$ increases during constant $q$ stages and constant $p'$ stages where the state of stress is well below the yield envelope. This suggests that void ratio is more important than $q$ for $G_{\text{max}}$ variation.
During the constant $p'$ stage $G_{\text{max}}$ increases with the increase of deviator stress ($q$) and with the decrease of void ratio (see Figure 7.8a and 7.8b). From the available data of this research it is not possible to quantify how much increase of $G_{\text{max}}$ has occurred due to increase of $q$ and how much increase of $G_{\text{max}}$ has occurred due to the decrease of void ratio. But, it can be said that the $G_{\text{max}}$ increase is not independent of either $q$ or void ratio, rather it is a combined effect of $q$ and $e$.

7.3.1.4 Relationship Derived for $G_{\text{max}}$

All the $G_{\text{max}}$ data in Figure 7.3 and Figure 7.7 were used to derive a simple empirical relationship for $G_{\text{max}}$. The data after sample failure have not been included in this derivation because when a failure plane develops in samples it is difficult to interpret $G_{\text{max}}$ behaviour. In this study $G_{\text{max}}$ was considered as a function of void ratio ($e$), mean effective stress ($p'$) and stress ratio ($q/p'$) although some researchers previously considered $G_{\text{max}}$ as a function of $e$ and $p'$ only. According to Figure 7.3 and 7.8 it is noticeable that changes in void ratio and $p'$ are clearly very important in influencing the $G_{\text{max}}$ values. But tests at constant $p'$ involve changes in void ratio and deviator stress ($q$). So void ratio alone is not sufficient to explain the change of $G_{\text{max}}$ value and parameters like ‘$q$’ needs to be incorporated into the $G_{\text{max}}$ relationship. There is an advantage in using stress ratio ($q/p'$) instead of ‘$q$’ alone. It not only makes the relationship dimensionless but also avoids the difficult situation when deviator stress is zero. Salvati (2002) also followed a similar approach.

Applying a least square best fit method to all the uncemented carbonate sand data a predicted relationship was formulated as

$$
\frac{G_{\text{max}}}{p_r} = 13435 \left( \frac{p'}{p_r} \right)^{0.46} e^{-1.35 \left( 1 + \frac{q}{p'} \right)^{-0.18}}
$$

(7.1)

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

Equation 7.1 can describe the variation of $G_{\text{max}}$ with satisfactory degree of accuracy. Figure 7.9 Shows the comparison of predicted $G_{\text{max}}$ from Equation 7.1 with the raw data of a typical test, where the sample was isotropically compressed to 1134 kPa, isotropically expanded to 100 kPa and then sheared (CID) at a constant confining stress of 100 kPa. It shows that the Equation 7.1 gives a good prediction for all stages of the test. Equation 7.1 also can predict $G_{\text{max}}$ with satisfactory degree of accuracy for some special triaxial test such as constant $p'$ test (see Figure 7.10), ICU test (see Figure 7.11), and stress path test (see Figure 7.12). In all types of tests reasonable
agreement between predicted $G_{\text{max}}$ and the actual values during compression and shearing can be observed.

### 7.3.1.5 Mode of Large Strain Failure

Figure 7.13 shows the mode of large strain failure during shearing for uncemented cast in place calcareous soil samples. The picture shows the oven dry samples after shearing. The common modes of failure for all the samples are bulging and punching. Two samples (N2 and N7) disintegrated during removal from the triaxial cell and the oven. Sample N1 shows a relatively clear failure plane compared to sample N6 or N8, but other samples do not show any visible failure plane except bulging and punching around the top and bottom platens. From these observations it was found that only dense samples show a definite plane of failure during shearing.

### 7.3.2 Behavior of Uncemented and Compressed Pre-cast Calcareous Soil Sample

Four uncemented and compressed samples with dry unit weights $\gamma_d = 13, 15$ and $17$ kN/m$^3$ were tested to investigate their mechanical behavior. All the samples used the same material as in the tests described above and only triaxial drained tests (CID) were performed on those samples. The test conditions of all the samples have been summarized in Table 3.4 and the key findings of uncemented and compressed samples are summarized in Table 7.2. Discussion and presentation of uncemented and compressed specimens test data will be given in the following section.

#### 7.3.2.1 Compression Behaviour of Uncemented and Compressed Samples

The samples were isotropically compressed to high confining pressure and then isotropically expanded to 100 kPa confining pressure ($\sigma_c'$) except the one with $\gamma_d = 13$ kN/m$^3$ which was only compressed to 300 kPa and then sheared. Figure 7.14 shows the isotropic compression behaviour of the uncemented and compressed samples. The key on this figure consists of ‘N’ representing ‘Uncemented Carbonate Sand’ specimen number, the letter ‘u’ followed by the dry unit weight ($\gamma_d$) of that specimen. All the results shown in this plot are for the specimens subjected to isotropic compression followed by drained shear tests to failure at an effective confining pressure ($\sigma_c'$) of 300 kPa. In one case a specimen with $\gamma_d = 13$ kN/m$^3$ experienced a maximum isotropic effective stress of 1500 kPa before isotropic expansion to the effective mean stress of 300 kPa. As all the samples were pre-cast with some degree of over-consolidation and the cell pressure was limited to 2 MPa it was not possible to reach the normal consolidation line (NCL) for samples with $\gamma_d > 13$ kN/m$^3$. Figure
7.14a shows that as the dry unit weight increases so the initial void ratio of the samples decreases. It is noticeable from the figure that the compression lines and the swelling lines are almost parallel for all the samples i.e. density has no effect on compression index and swelling index of uncemented and compressed soil. The slope of the compression and swelling line of uncemented and compressed samples has been calculated from this plot as 0.01563 and 0.00517 respectively. Figure 7.14b shows that the initial shear modulus (G_max) increases with the decrease of initial void ratio. From this figure it is noticeable that all the curves are parallel i.e. all of them possess the same slope. In comparison with uncemented and uncompressed cast in-place samples shown in Figure 7.3 these samples are less compressible, because initially these samples (uncemented and compressed) have been compressed more during sample manufacturing i.e. these are heavily over-consolidated samples.

Figure 7.14c shows that the relationship between shear modulus (G_max) and p’ for uncemented and compressed pre-cast calcareous soil samples and comparison of with the data for uncemented and uncompressed cast in-place calcareous soil samples presented previously. The relationship is linear in log-log plot during isotropic compression and the slope of the line is approximately 1. Another important feature that is noticeable from this plot is that during isotropic expansion G_max follows a path, which is slightly higher than the isotropic compression path. The variation of G_max is similar to uncompressed cast in-place sample N2 during unloading shown in Figure 7.3. From the Toyoura sand data it might be expected that there would be a unique G_max – p’ response for these pre-compressed specimens. A possible explanation is that the carbonate sands were pre-compressed one-dimensionally and then subjected to isotropic stresses in the triaxial tests, whereas the Toyoura sand specimens were only loaded isotropically. As discussed previously the pre-compressed specimens experienced high vertical stresses, the maximum one dimensional compression pressure applied during sample manufacturing has been summarized in Table 3.5. It suggests that to some extent there occurs some permanent densification or reorientation of particles during the isotropic compression process, and the elastic deformation is also very small compared with the non-recoverable plastic strain during isotropic compression and expansion. The low value of swelling line slope of 0.00517 also supports this fact. Figure 7.14c also shows that G_max lies above the cast in-place samples in Figure 7.3 during compression because of low initial void ratio and heads towards the uncompressed cast in-place samples at high mean effective
stress, and it is expected that at high mean effective stress the behaviour of compressed and uncompressed uncemented carbonate soil will be similar.

7.3.2.2 Shearing of Uncemented Compressed Samples

7.3.2.2.1 Large Strain Behavior

Figure 7.15 shows the stress–strain, volumetric strain and void ratio responses of uncemented compressed soil during drained shearing (CID) at confining pressure $\sigma_c' = 300$ kPa. From Figure 7.15 it is clear that, as expected, increasing density leads to an increase in the peak deviator stress ($q$). As the density increases the yield state becomes more clearly identifiable. The large strain behaviour is similar to that reported by Huang (1994). It should be noted here that failure has coincided with the maximum stress ratio. Only for the densest sample (i.e. $\gamma_d = 17$ kN/m$^3$) was strain–softening behaviour observed. Figure 7.15b suggests all the samples are headed towards the same ultimate stress ratio of 1.5 as the axial strain increases, similar to that reported by Huang (1994) in drained tests. When the tests were stopped non-uniform deformations were evident and the failure mode was always associated with significant bulging of the samples, axial cracks and punching of the top cap into the sample. The failure modes are shown in Figure 7.16. The bulging of the samples limits the accuracy of the deduced values of stress and strain and prevents a critical state from being reached.

Figure 7.15c shows that the volumetric strains during shearing are as expected and are consistent with the deviator stress-strain responses. The medium to dense samples (i.e. $\gamma_d = 15$ and 17 kN/m$^3$) shows dilation at large strain with the amount of dilation increasing as the density increases. On the other hand as the density decreases compressive behaviour is shown for all axial strains. It may also be noted that at the start of shearing all the samples get compressed to some extent, irrespective of the density.

Figure 7.17 shows the variation of Secant Young’s Modulus ($E_{sec}$) with void ratio at axial strain $\varepsilon_a = 0.01\%$ for all the uncemented samples during the shearing. The void ratios in this plot are the initial values at the end of sample saturation. It shows that $E_{sec}$ decreases with the increase of void ratio for the same mean stress. It also shows that $E_{sec}$ increases with mean stress at a particular dry unit weight ($\gamma_d$). Applying a least square best fit method to the uncemented carbonate sand data at $p' = 100$ kPa, a predicted relationship for $E_{sec}$ was formulated given by
\[
\frac{E_{sec}}{p_r} = A^* e^{-1.8 \left( \frac{p'}{p_r} \right)^{0.5}}
\]

where \(p_r\) is a reference pressure taken as 1 kPa. and \(A\) is a constant.

Equation 7.2 demonstrates that the uncemented Secant Young’s Modulus (\(E_{sec}\)) data are consistent with \(G_{max}\) Equation 7.1 i.e. \(E_{sec}\) is proportional to the product of void ratio and \(p'\). The exponent of the void ratio function is different in Equations 7.1 and 7.2, but both are within the range of −1.3 to −1.8 reported for sands by Jamiolkowski et al (1991) and Hardin and Black (1969).

Figure 7.18 shows that two samples reside on the dense side of the CSL and one sample resides on the loose side of the CSL at the time of shearing, and there is not much change in void ratio until sample failure. After the failure occurs, the void ratio of the dense samples (\(\gamma_d = 15\) and 17 kN/m\(^3\)) starts to increase and heads upward towards the CSL. On the other hand the void ratio of loose sample (\(\gamma_d = 13\) kN/m\(^3\)) starts to decrease and heads downward towards the CSL. None of the samples appears to reach the critical state line (CSL), which is typical of over-consolidated samples, and because tests were stopped at axial strains less than 8%, the deformations were not homogeneous, and the volumetric strains measured were the average strains not truly representative.

**7.3.2.2 Mode of Large Strain Failure**

Figure 7.19 shows the effect of density on the large strain failure mode during shearing for the uncemented compressed samples. It shows that for the samples with low density (\(\gamma_d = 13\) kN/m\(^3\)) to medium density (\(\gamma_d = 15\) kN/m\(^3\)) the failure mode is predominantly bulging, and there are no clear cracks or failure planes. But for the sample with high density (\(\gamma_d = 17\) kN/m\(^3\)) the failure mode involves punching and bulging associated with some clear shear planes. The deformation of the samples is not uniform and mostly concentrated in the upper half of the samples irrespective of the dry unit weight. Huang (1994) also showed similar failure modes i.e. failure at one end of the sample for his samples that had been prepared similarly. It is reasonable to suppose that failure has occurred at the weaker end of the samples, but Huang (1994) reported that there were no significant variations in density along the length of uncemented samples.
7.3.2.2.3 Shear Stiffness ($G_{\text{max}}$) Behaviour

Figure 7.20 shows the variation of $G_{\text{max}}$ for uncedmented compressed samples. Figures 7.20a and 7.20b show that the stiffness ($G_{\text{max}}$) increases with the increase of dry unit weight ($\gamma_d$). The increase of $G_{\text{max}}$ with the dry unit weight ($\gamma_d$) at the start of shearing is linear as seen from the inset of Figure 7.20a. During shearing all the samples show a common trend for $G_{\text{max}}$ to increase with the increase of $q$ and $p'$ until reaching the peak deviator stress ($q_{\text{peak}}$), then $G_{\text{max}}$ starts to decrease as the stresses reduce as the sample proceeds towards an ultimate state. Figure 7.20b shows the variation of $G_{\text{max}}$ with axial strain. At very small strain $G_{\text{max}}$ is constant until $\varepsilon_a \approx 0.02\%$ after which $G_{\text{max}}$ increases along with $q$ and $p'$. When the samples approach the peak deviator stress $G_{\text{max}}$ starts to decrease. The inset of Figure 7.20b shows the amount of axial strain required to mobilize the reduction of $G_{\text{max}}$, and indicates that the mobilized axial strain increases with the decrease of density and the relationship seems to be linear.

Figure 7.21 shows the measured and predicted $G_{\text{max}}$ values from Equation 7.1 of all the uncedmented samples at very small strain, at the start of shearing plotted against void ratio (Figure 7.21a) and $p'$ (Figure 7.21b). The void ratios in Figure 7.21a are the initial values at the end of sample saturation. Figure 7.21a shows that $G_{\text{max}}$ decreases with the increase of void ratio at a particular cell pressure, and for the same density $G_{\text{max}}$ increases with the increase of mean stress ($p'$). The predicted $G_{\text{max}}$ value from Equation 7.1 also decreases with the increase of void ratio at a particular $p'$. It can be observed from the plot that the tested $G_{\text{max}}$ data coincide with the predicted $G_{\text{max}}$ value from Equation 7.1 at a particular $p'$. Figure 7.21b shows that measured and predicted $G_{\text{max}}$ from Equation 7.1 increase with the increase of $p'$ for the same density samples, and for the same $p'$, $G_{\text{max}}$ increases with the increase of dry unit weight ($\gamma_d$).

7.3.2.2.4 Small Strain Behavior

Figure 7.22 shows the small strain data of the uncedmented compressed samples during shearing. To shows the trend with density more clearly, data of a lower density uncompressed cast in place sample (N1) have also been presented in Figure 7.22a. The stiffness of the stress strain response increases as the density increases. The $G_{\text{max}}$ values estimated from the stress-strain curves in Figure 7.22a, and from the bender element measurements are presented in Figure 7.23. Comparison of Figure 7.23 and Figure 7.4 also suggests that compacted uncedmented samples give better agreement.
between $G_{\text{max}}$ from bender and from stress-strain curve than uncompacted samples. As expected the conventional method of $G_{\text{max}}$ measurement tends to underestimate the shear stiffness. The inset of Figure 7.22a also shows that at very small strains the true nature of the small strain stiffness behaviour cannot be reliably interpreted from the stress-strain responses.

Figures 7.22b and 7.22c show the expected patterns of behaviour with increasing density. The volumetric compression reduces and the radial strain becomes more expansive as shearing occurs as density increases. The responses at small strains are approximately linear except the loosest sample, with the departure from linearity occurring earlier as density increases. The radial strain data for the loosest sample shows scatter at small strains, probably due to the radial strain measurements only being taken at the mid height of sample, instead of being an average radial strain. This is a consequence of the effect of end restraint and experimental procedure. As shown in Figure 7.15 the denser samples tend to dilate at large strains and significant differences with density are apparent. However, at small strains particle arrangement and dilation are insignificant and samples compress due to the increase in $p'$ that accompanies shearing in CID tests. The small strain Poisson’s ratio increases from 0.0171 to 0.165 as density increases.

### 7.3.3 Summary

Uncemented calcareous sand has been prepared at initial void ratios from 1.5 to 0.6 (i.e. $10.2 \text{ kN/m}^3 < \gamma_d < 17 \text{ kN/m}^3$) and subjected to a range of stress paths. The large strain behaviour has been shown to be consistent with critical state framework proposed by Coop and Atkinson (1993) and demonstrated previously for this sand by Huang (1994).

The techniques developed in this research, and described in earlier chapters have been used to investigate how $G_{\text{max}}$ varies with stress and void ratio. A relationship has been derived that gives reasonable prediction of modulus over this wide range of stress and density, and for both compression and shearing. Comparisons with estimates of $G_{\text{max}}$ from internal Hall effect transducers show that the HETs cannot provide reliable data at very small strains for uncemented sand. However, they do provide valuable data on the response of the soil at small strain.
7.4 Behavior of Artificially Cemented Calcareous Soil Samples

Huang (1994) has investigated the large strain behaviour of artificially cemented calcareous soil. In practice however, strains are usually limited to the small-strain range and the pre-failure response is required. For cemented soils the interpretation of the response is complicated by the possibility of cement degradation. A series of tests have been designed to explore the pre-failure response of the cemented soil in more detail.

The tests and discussion focus on the variation of $G_{\text{max}}$ as this is assumed to be a good indicator of the degree of cementation. The samples were subjected to at least one cycle of shear before failure, and a couple of stress path tests were also performed to investigate the cementation breakdown phenomenon and its effect on $G_{\text{max}}$.

The preparation of the artificially cemented samples and procedure of the triaxial tests have been explained in Chapter 4. The material and the cementing agent used in this study have been described in Chapter 3. This section describes the testing program and gives a summary of the key findings from the experiments and the relationships found between $G_{\text{max}}$ and other soil parameters.

7.4.1 Data Integrity

The key findings of the testing program on cemented calcareous soil samples are summarized in Table 7.3. Figure 7.24 shows the consistency of the test data, where 95% of $G_{\text{max}}$ data from the bender elements and the stress-strain curve lies close to 45° line. In less than 5% of cases $G_{\text{max}}$ from stress-strain curves underestimates the BE $G_{\text{max}}$ values. This problem is thought to be associated with improper alignment of the axial HETs when they were glued on the membrane. If the alignment of an axial HET is not correct then there is a chance that the arms may lift up from the corresponding sliding base (see Figure 3.13), and lead to an underestimation of $G_{\text{max}}$ value from the stress-strain curve. With few exceptions Figure 7.24 shows that there is a better agreement between $G_{\text{max}}$ from bender elements and from stress-strain curves for cemented samples than for uncemented samples shown in Figure 7.4. This is thought to be associated with the cemented sample ends. The cemented sample ends were more level, and the samples were homogeneous, due to the method of preparation and large stress application.

Figures 7.25 and 7.26 show the $G_{\text{max}}$ variation, from the bender elements, with void ratio and Gypsum content (GC) when specimens are subjected to isotropic stresses of
\[ \sigma_c' = 100 \text{ and } 300 \text{ kPa respectively. Both figures also show best fit empirical } G_{max} \text{ relationships with void ratio and gypsum content. Figure 7.25 shows that } G_{max} \text{ increases with the decrease of void ratio (i.e. with the increase of } \gamma_d) \text{ and with the increase of Gypsum content (GC) for } \sigma_c' = 100 \text{ kPa. Similar trends are shown for the confining pressure } \sigma_c' = 300 \text{ kPa in Figure 7.26, but with increased } G_{max} \text{ value for } \sigma_c' = 300 \text{ kPa compared to } \sigma_c' = 100 \text{ kPa. The } G_{max} \text{ measurements from the bender elements are very consistent, an indicator of the repeatability and reliability of specimen manufacturing procedure. The lines in Figure 7.25 and 7.26 are parallel, and the empirical relationship of } G_{max} \text{ with void ratio in Figure 7.25a at } \sigma_c' = 100 \text{ kPa is}
\]

\[
\text{Log}(G_{max}) = C - 1.15e
\]

(7.3)

where C is a constant that depends on cement content and varies from 3.61 to 4.1 for gypsum contents from 10% to 30%.

The empirical relationship of \( G_{max} \) with gypsum content (GC) in Figure 7.25b at \( \sigma_c' = 100 \) kPa is

\[
\text{Log}(G_{max}) = D + 0.023GC
\]

(7.4)

where D is a constant that depends on dry unit weight and varies from 2.33 to 2.77 for dry unit weights from 13 kN/m\(^3\) to 17 kN/m\(^3\).

The empirical relationship of \( G_{max} \) with gypsum content in Figure 7.26a at \( \sigma_c' = 300 \) kPa is

\[
\text{Log}(G_{max}) = E - 1.23e
\]

(7.5)

where E is a constant that depends on cement content and varies from 3.73 to 4.02 for gypsum contents from 10% to 30%.

The empirical relationship of \( G_{max} \) with gypsum content (GC) in Figure 7.26b at \( \sigma_c' = 300 \) kPa is

\[
\text{Log}(G_{max}) = F + 0.016GC
\]

(7.6)

where F is a constant that depends on dry unit weight and varies from 2.33 to 2.77 for dry unit weights from 13 kN/m\(^3\) to 17 kN/m\(^3\).

Equations 7.3 to 7.6 show that the slopes of \( G_{max} \) versus e, \( p' \) decrease with an increase of confining pressure.

Figure 7.27 shows the variation with Gypsum content of the Secant Young’s modulus \( (E_{sec}) \) at axial strain \( \varepsilon_a = 0.01\% \) for the artificially cemented samples. All the samples were sheared at the confining pressure \( \sigma_c' = 300 \) kPa. The \( E_{sec} \) data show consistent
trends with gypsum content and void ratio. Figure 7.27b shows that $E_{sec}$ increases with increase of Gypsum content as expected and decreases with the increase of void ratio (or decreasing dry unit weight).

Figure 7.28 shows the variation of peak deviator stress ($q_{peak}$) with void ratio and Gypsum content for all the artificially cemented samples in this study and from the previous study by Huang (1994), that were sheared at the confining pressure, $\sigma_c' = 300$ kPa. Figure 7.28 shows that $q_{peak}$ increases in two ways, with the increase of Gypsum content and with the increase of dry unit weight ($\gamma_d$) (i.e. decrease of void ratio), and that the data are consistent with the previous study. The overall trend of $q_{peak}$ shows consistent variations for all the samples in terms of void ratio, Gypsum content, and dry unit weight.

Figure 7.29 shows the variation of volumetric strain ($\varepsilon_{vf}$) at peak deviator stress ($q_{peak}$) with void ratio and Gypsum content (GC) for all the artificially cemented samples. The plots show that there is no significant effect of GC on $\varepsilon_{vf}$ for densest sample (i.e. $\gamma_d = 17$ kN/m$^3$) but for loosest sample (i.e. $\gamma_d = 13$ kN/m$^3$) there is trend for $\varepsilon_{vf}$ to reduce with GC. But in general the plots show that $\varepsilon_{vf}$ increases with increase of void ratio as expected. But with the Gypsum content the relationship is opposite i.e. $\varepsilon_{vf}$ decreases with the increase of Gypsum content. This indicates that as the Gypsum content and dry unit weight increase volumetric expansion is occurring, which is a common phenomenon for dense sand. Again there is a consistency for $\varepsilon_{vf}$ data for all the samples.

Figure 7.30 shows the variation of angle of shear resistance ($\phi_d$) at peak deviator stress ($q_{peak}$) with void ratio and Gypsum content for all the artificially cemented samples. It shows that $\phi_d$ decreases with the increase of void ratio, but with the Gypsum content the relationship is opposite, i.e. $\phi_d$ increases with the increase of Gypsum content as expected. It also reveals that the $\phi_d$ for dense samples lies above the medium dense samples. This indicates that as the Gypsum content and dry unit weight increase so the samples peak stress ratio ($\eta$) also increases, which is expected. So there is a consistency for $\phi_d$ data for all the samples.

The results of each test will be discussed in detail in the following sections. For convenience of discussion this will focus on trends with respect to density and with respect to degree of cementation.
7.4.2 Isotropic Compression

The cemented samples were prepared by pressing the soil cement mixture to produce the desired sample dry unit weights ($\gamma_d$) of 13 or 15 or 17 kN/m$^3$. Therefore all the cemented samples were over consolidated. Due to the limitations of the equipment, it was not possible to bring these samples to a normally consolidated state. Therefore it was decided to compress the samples up to $p' = 100$ kPa initially, then $p' = 300$ kPa at which point they were sheared to failure.

Figures 7.31a and 7.31b shows the response of void ratio with $p'$ and $G_{\text{max}}$ during isotropic compression for different percentage of Gypsum content (GC) and for different dry unit weight ($\gamma_d$). In the key on these figures ‘C or T’ represents ‘cemented carbonate sand’ specimen number, the value following ‘g’ is the percentage of Gypsum content and the value following ‘u’ is the dry unit weight ($\gamma_d$) of that specimen. Both plots show that the void ratio of cemented carbonate sand decreases with the increase of GC and density. It can be noticed from Figure 7.31a that void ratio does not change significantly during the isotropic compression with the increase of $p'$. Figure 7.31b shows that $G_{\text{max}}$ increases with the reduction of void ratio and remains relatively unchanged during the isotropic compression. These typical features coincide with the response of most naturally cemented samples, which are remote from yield.

Figure 7.31c shows the response of $G_{\text{max}}$ with $p'$ during isotropic compression for different GC and $\gamma_d$. It can be seen from this plot that increasing the gypsum content leads to very significant increases in $G_{\text{max}}$. The uncemented carbonate sand data given in Figure 7.14c shows a range of $G_{\text{max}}$ from 100 MPa to 300 MPa at a mean effective stress of 100 kPa, whereas the cemented specimens have given $G_{\text{max}}$ values from 200 MPa to 2500 MPa at the same stress level. Figure 7.31c further shows that the effectiveness of a given amount of cementation increases dramatically with the increase of dry unit weight. It also shows that the three curves for gypsum content of 10%, 20% and 30% for $\gamma_d = 17$ kN/m$^3$ lie above the 3 curves for $\gamma_d = 15$ kN/m$^3$ which, in turn lies above the 3 curves for $\gamma_d = 13$ kN/m$^3$.

Another feature of the results shown in Figure 7.31c is the independence of $G_{\text{max}}$ from effective confining stress ($p'$). This is in contrast to the strong dependence of $G_{\text{max}}$ on mean effective stress for uncemented carbonate sands. The general pattern of behavior shown in Figure 7.31 has been reported in several other studies (e.g. Fernandez &
Santamarina, 2001). Figure 7.31a and Figure 7.31c show that the cemented samples reside far away from the NCL and the ultimate state of $G_{\text{max}}$ for uncemented samples.

### 7.4.3 Shearing

All test results presented here are from the confining pressure $\sigma'_c = 300$ kPa with three different dry unit weights (13, 15 and 17 kN/m$^3$) and three different cement contents (10%, 20% and 30%). Results of this research have shown that the qualitative response of artificially cemented calcareous soil is similar to that of other cemented sand and naturally cemented calcarenite (e.g. Airey, 1993). Figure 7.32 shows the shearing response of all cemented samples. In general Figure 7.32a shows that peak deviator stress ($q_{\text{peak}}$) increases with the increase of gypsum content and density. Figure 7.32b shows that there is a tendency for cemented soil to reach towards the CSL either upward for denser ssamples or downward for looser samples during shearing.

Figure 7.32c shows that $G_{\text{max}}$ remains unchanged with mean effective stress at the beginning of shearing but starts to decrease as the cementing bonds start to break down. It is evident from Figure 7.32c that the breakdown of bonds starts before the specimen reaches its peak strength. It further shows, from the reduction of $G_{\text{max}}$, that the degradation of cementation as well as $G_{\text{max}}$ does occur during cyclic loading. Similar behaviour of cemented soil during cyclic loading was reported by Yeoh (1996) and Shambhu (2003). Figure 7.32c also shows the post failure behaviour of cemented calcareous soil. After the large strain failure of the cemented specimens $G_{\text{max}}$ for all the samples move towards a point on the ultimate $G_{\text{max}}$ state observed for uncemented calcareous soil during shearing. These behaviours of the cemented soil will be thoroughly investigated in respect of density and gypsum content in the following sections.

### 7.4.4 Effect of Density

The influence of density on shearing behaviour has been investigated for three categories of cementation i.e. cement contents of 10%, 20% and 30%. The behaviour of $G_{\text{max}}$, the stress-strain, void ratio and small-strain responses have been assembled for the above three categories. In general the increase of density is associated with an increase of the peak deviator stress, increasing $G_{\text{max}}$ and an expansion of the yield surface for the same degree of cementation. These will be discussed in more detail in the following section.
7.4.4.1 Large Strain Behavior

The stress-strain responses for the different degrees of cementation are shown in Figure 7.33. To make the general trend of stress-strain curves more apparent the data of uncemented samples have also been presented in these plots. It is clear from the curves that for a particular cement content, as the density increases so does the peak deviator stress ($q_{peak}$) and the stress-strain curves for cemented samples lie above the uncemented responses. After the peak the deviator stress drops off and approaches to the uncemented soil values due to the breakdown of cementation. This was expected because specimens were subjected to CID tests and the ultimate stress ratio ($M$) has been shown to be independent of gypsum content (GC) or dry unit weight ($\gamma_d$). Huang (1994) has also reported similar trends. For lightly cemented soil (10% cement content) the drop of $q$ is gradual, but for highly cemented soil (30% GC) or high-density samples (i.e. $\gamma_d = 17$ kN/m$^3$) this drop of deviator stress can be rapid. Analysis of all stress strain responses shown in Figure 7.33 suggests that all samples show a stiff approximately linear response up to a well-defined yield point. The data show a clear trend from strain hardening to strain softening as the density increases, which is very typical and similar to any naturally cemented carbonate soil.

7.4.4.2 Volumetric Strain Response

The volumetric strain data of all nine samples reported in Figure 7.33 are shown in Figure 7.34. All the graphs reveal one common characteristic, that the samples with high density (i.e. $\gamma_d = 17 \& 15$ kN/m$^3$) tend to dilate during shearing irrespective of their degree of cementation. On the other hand samples with low density (i.e. $\gamma_d = 13$ kN/m$^3$) even with high cement content (i.e. 30% cc) always compressed during shearing. The void ratio vs. $p'$ plot also support this tendency, which is shown in Figure 7.35. To make the general trend of volumetric strain and void ratio more apparent the data of uncemented samples have also been presented in these plots. The initial void ratio decreases with the increase of density for a particular degree of cementation. The changes in void ratio for the dense samples (i.e. $\gamma_d = 17$ and 15 kN/m$^3$) are very small during shearing compared to lightly dense sample (i.e. $\gamma_d = 13$ kN/m$^3$). Consistent with Huang’s (1994) data all the samples show the expected behaviour with all samples appearing to head towards the critical state line (CSL) by either void ratio going upward or downward. It should be noted here that all cemented...
samples with $\gamma_d = 13$ kN/m$^3$ compressed more than uncemented specimens with the same density.

**7.4.4.3 $G_{\text{max}}$ Response**

The responses of $G_{\text{max}}$ with axial strain and $p'$ during shearing are presented in Figure 7.36 and Figure 7.37. To make the general trend of $G_{\text{max}}$ response more apparent the data of uncemented samples have also been presented in these plots. These plots show that $G_{\text{max}}$ increases with the increase of density for the same cementation. Normally $G_{\text{max}}$ remains unchanged at the beginning of shearing but eventually starts to decrease as the cementation starts to breakdown. After the samples fail $G_{\text{max}}$ for all the cemented samples proceeds towards a line, which is the ultimate state for uncemented samples or in other words samples behave increasingly as if uncemented. Figure 7.36 shows that during the break down of cementation (or after failure) $G_{\text{max}}$ of all cemented samples starts to converge towards the values of the uncemented soil sample, i.e. the ultimate state of $G_{\text{max}}$ for uncemented soil. It should be mentioned here that the post-peak $G_{\text{max}}$ response is very difficult to interpret for the cemented samples. This is because pronounced shear planes develop during the post-peak state for the cemented samples, and the deformation and the stress concentrate on these shear planes. On the other hand the shear waves travel through the intact materials. Therefore measurement of $G_{\text{max}}$, $\epsilon_v$, $\epsilon_a$ would not be expected to be very accurate for the post-peak state. The post-peak responses for all samples suggest that cementation breakdown occurs throughout the specimens. A few samples in Figure 7.36, especially the densest ones, show an increase in $G_{\text{max}}$ with $p'$, which is not expected for cemented sample. It is thought that this may be caused by difficulty in fitting the bender elements snugly into the sample.

It is evident from Figures 7.37b and 7.37c that the reduction of $p'$ is much more drastic for dense (i.e. $\gamma_d = 17$ kN/m$^3$) samples than for less dense samples after the peak deviator stress ($q_{\text{peak}}$). It is noticeable from Figure 7.37 that at the end of each unloading cycle there is a little increase in shear stiffness ($G_{\text{max}}$). This suggests that the microscopic cracks, which are created during shearing close during unloading due to particle rearrangement that follows the removal of deviator stress ($q$). This is suggested by the slight decrease in void ratio that is observed at the end of unloading. The increase in soil particle contact at the end of unloading increases $G_{\text{max}}$ slightly even though the cement bonds have already partially broken during the past loading
cycle. This phenomenon has been observed for all artificially cemented calcareous soil samples regardless of density and degree of cementation. In Figure 7.37c the densest sample (i.e. $\gamma_d = 17 \text{ kN/m}^3$) with high cement content does not appear to be strongly cemented. This is also evident in stress-strain response shown in Figures 7.38. This sample was first tested under a different stress path and could not be sheared to failure due to load cell limitations. The data shown are from when it was tested a second time with a higher capacity load cell and this time it failed during shearing. It appears that this sample has been disturbed due to its first testing.

### 7.4.4.4 Small-Stain Response

Data showing the pre-failure, small strain, responses of deviator stress ($q$), volumetric strain ($\varepsilon_v$), and radial strain ($\varepsilon_r$) with axial strain ($\varepsilon_a$) are presented in Figures 7.38, 7.39 and 7.40. To make the general trends of the stress-strain curves more apparent the data of uncemented samples have also been presented in these plots. Figure 7.38 shows that as the density increases for a given degree of cementation the stiffness of the sample also increases, as expected. The shear stiffness ($G_{max}$) from the stress-strain curves and from the bender element measurements for each sample are shown in Figure 7.24. These show that the $G_{max}$ values from the dynamic measurements are slightly higher than those estimated from the stress–strain curve. There is a pronounced trend of non-linearity in the stress-strain responses in Figure 7.38. It was expected that a more linear response would be observed as cement content increases, but this is not evident from the plots.

Figure 7.39 shows that volumetric strain ($\varepsilon_v$) decreases as the density increases and increased cementation reduces compression. Uncemented samples show more compression than the cemented samples. It appears from the figure that particle rearrangement and dilation occurs from an early stage of shearing, even for well cemented samples. Figure 7.39 also shows that even though the dense samples dilate (irrespective of cementation) during shearing they initially compressed with the amount of volumetric compressive strain ($\varepsilon_v$) decreasing as the density increases for a given cementation. The figure also shows that the change of $\varepsilon_v$ is apparently linear with $\varepsilon_a$ at the beginning of shearing, which is expected for the cemented samples.

Figure 7.40 shows that the radial strain ($\varepsilon_r$) increases with the increase of density for the same cementation. It indicates that dense samples have got higher Poisson’s ratio ($\nu$) than the less dense samples of the same cementation. Increased cementation also increases the radial strain. Uncemented samples show less radial strain than the
cemented samples irrespective of density or degree of cementation. In Figure 7.40 the radial strain ($\varepsilon_r$), also shows an apparent linear relationship with axial strain ($\varepsilon_a$) at the start of shearing.

### 7.4.4.5 Effect of Density on Mode of Large Strain Failure

Figure 7.41 shows the effect of density on the large strain failure mode during shearing for the different levels of cementation i.e. GC = 10% 20% and 30%. It shows that the artificially cemented samples become more brittle with the increase of dry unit weight and with the increase of degree of cementation. For the 10% gypsum content sample with low density ($\gamma_d = 13 \text{ kN/m}^3$) the failure mode is predominantly bulging, whereas for same cementation sample with high density ($\gamma_d = 17 \text{ kN/m}^3$) the failure mode is predominantly wedge like, or blocky associated with punching at the top and there is a development of a couple of shear planes, at least one of them propagating from top to bottom crushing the wedges one against another. Similar features can also be observed for the 20% gypsum content samples. But for the 30% gypsum content samples bulging is less dominant and failures are totally brittle in nature. Therefore samples with low to high density show the development of cracks, punching and predominantly wedge failure. The number of cracks and intensity of punching are greater for the high density ($\gamma_d = 17 \text{ kN/m}^3$) samples than for the low density ($\gamma_d = 13 \text{ kN/m}^3$) samples.

### 7.4.5 Effect of Degree of Cementation

The previous discussion has shown that the cement content has a significant effect on the response. To show this more clearly the previous test data are replotted to show the effects of varying GC for a given dry unit weight ($\gamma_d$).

#### 7.4.5.1 Stress-Strain Response

Figure 7.42 shows the stress–strain responses from a series of drained triaxial tests on specimens that had different initial gypsum content ranging from zero to 30% and three different initial dry unit weights ranging from 13 kN/m$^3$ to 17 kN/m$^3$. The obvious effect of cementation is seen as the significant increase of peak deviator stress ($q$) as well as shear stiffness (E) with the addition of cement. The only exception in Figure 7.42c is the dense sample with 30% cement content which did not fit the trend for reasons mentioned above. Another important observation is that samples with high levels of cementation (20% and 30%) show brittle failure with a clear yielding followed by progressive strain softening. On the other hand samples with low levels
of cementation (10% CC) show strain hardening behavior with no distinct yield, which is similar to the uncemented calcareous soil behavior. Clearly the influence of the cementation results in a transition from ductile to brittle behavior as well as in a sharp increase of the yield stress. The increasing degree of cementation also moves the state of stress outside the failure envelope of the uncemented soil. At higher levels of cementation, the strain softening continues towards the residual strength of uncemented soil sample as the cement bonds gradually break down with increase of axial strain \((\varepsilon_a)\). Table 7.3 shows the measured peak deviator stress for cemented and uncemented specimens with similar density. The uncemented specimens failed with a relatively uniform deformation whereas distinctive rupture planes were observed for cemented specimens.

### 7.4.5.2 Effect of Volumetric Strain and Void Ratio.

Figure 7.43 shows the volumetric-strain \((\varepsilon_v)\) responses with axial strain for different degree of cementation at the same density. Figure 7.43a shows that the volumetric-strain, and axial strain corresponding to peak deviator stress \((q_{\text{peak}})\) are reduced notably with the increase of cementation. For these low density (i.e. \(\gamma_d = 13\ \text{kN/m}^3\)) samples, volumetric responses are always compressive. The cemented samples compress more than the uncemented sample. For dense samples (i.e. \(\gamma_d = 15\ \text{kN/m}^3\) and \(17\ \text{kN/m}^3\)) the volumetric strain response is dominated by the dry density, all the samples try to dilate after a certain axial strain increase (e.g. Figures 7.43b and 7.43c). Another important observation from Figure 7.43 is that addition of cement has made samples more compressible at small strain.

Figure 7.44 shows the void ratio responses during shearing. The figure shows that addition of cement increases slightly the void ratio of cemented samples compared to the uncemented sample. It indicates that cementation makes samples more porous but stiff compared to the uncemented sample. Among the cemented samples there is a tendency that the void ratio decreases with the increase of cementation. During shearing the void ratio of low density samples try to decrease (i.e. compressed as of Figure 7.44a), on the other hand for dense samples void ratio try to increase (i.e. dilate as of Figure 7.44b and 7.44c) to meet the critical state line (CSL) and cementation appears not to have any effect to these trends.

### 7.4.5.3 Stiffness \((G_{\text{max}})\) Response

Figure 7.45 and 7.46 show the stiffness data with \(p'\) and axial strain. All the plots show that there is a significant increase in \(G_{\text{max}}\) with the addition of cement. Figure
7.45 shows that cementation causes an increase in initial $G_{\text{max}}$ for the same density samples. It is noticeable from the Figure 7.46a that at the start of shearing cement does not degrade therefore $G_{\text{max}}$ does not change but as the shearing progress cement bonds starts to breakdown and microscopic cracks starts to develop inside the sample so $G_{\text{max}}$ starts to decrease with the increase of axial strain and this reduction of $G_{\text{max}}$ accelerates when the sample passes $q_{\text{peak}}$. So there is a close connection between cement degradation to the $G_{\text{max}}$ reduction during shearing. Similar trends can be observed for the other cemented calcareous samples with different densities (i.e. $\gamma_d = 15$ kN/m$^3$ and 17 kN/m$^3$) as shown in Figures 7.46b and 7.46c.

In fact the effect of cementation on $G_{\text{max}}$ response is similar to the effect of density on $G_{\text{max}}$ as shown in Figure 7.37. Figures 7.46a, 7.46b, and 7.46c show that the small strain stiffness ($G_{\text{max}}$) of cemented samples (with 30% cement content) increases approximately 4, 5 and 7 times respectively compared to uncemented samples. This indicates that effectiveness of cement increases with the increase of density. At $\gamma_d = 17$ kN/m$^3$ (i.e. Figure 7.46c) there is a greater tendency for shear modulus ($G$) to increase from initial value, possibly due to more significant loss of cementation before reaching the peak.

**7.4.5.4 Small–Strain Response**

All the small-strain data of deviator stress ($q$), volumetric strain ($\varepsilon_v$), and radial strain ($\varepsilon_r$) with axial strain ($\varepsilon_a$) has been plotted in Figures 7.47, 7.48 and 7.49. To make the general trend of stress-strain curves more apparent the data of uncemented samples have also been presented in those plots. Figure 7.47 shows that there is an increase in initial shear stiffness as the degree of cementation increases for a particular density samples. Similarly according to Figure 7.27b the Secant Young’s modulus ($E_{\text{sec}}$) increases as the degree of cementation increases for a particular density samples. The medium dense sample (i.e. $\gamma_d = 15$ kN/m$^3$) with 20% GC in Figure 7.47b does not fit with this trend as expected. All the figures also show that non-linearity of stress-strain response exist from very early stage of straining regardless of cementation.

Figure 7.48 shows that the volumetric strain decreases with the increase of cementation for the same density samples. Medium dense samples (i.e. $\gamma_d = 15$ kN/m$^3$) show higher degree of linearity in volumetric strain response compared to medium to high dense samples and increase of cementation causes non-linear volumetric strain response among the cemented samples. It is evident from the plots
that addition of cement decreases the compressibility of the samples. It also reveals that all the samples compressed initially whether the samples eventually dilated or compressed and the relationship of $\varepsilon_v$ and $\varepsilon_a$ is linear at the early stage of axial strains. Figure 7.49 shows that the radial strain increases as the degree of cementation increases for the same density samples and the relationship is linear initially. It is noticeable from the figure that there is an increase in Poisson’s ratio ($\nu$) due to the increase in degree of cementation. Cemented samples possess higher Poisson’s ratio than uncemented samples.

### 7.4.6 Summary for Artificially Cemented Calcareous Soil

From the investigation of artificially cemented calcareous soil the following observations have been made:

- The behaviour of artificially cemented calcareous soil resembles the naturally cemented calcareous soil.
- The behaviour of cemented calcareous soil is similar to the behaviour of uncemented calcareous soil after the breakdown of cement bonds.
- During isotropic compression with the increase of mean stress ($p'$) shear stiffness ($G_{\text{max}}$) for cemented calcareous soil remains almost unchanged.
- During shearing with the increase of axial strain ($\varepsilon_a$) the ultimate shear stiffness ($G_{\text{max}}$) and ultimate deviator stress ($q$) for artificially cemented calcareous soil collapse to the corresponding uncemented calcareous soil.
- The small strain shear stiffness ($G_{\text{max}}$) increases with the increase of dry unit weight ($\gamma_d$) at the same cementation level for artificially cemented calcareous soil.
- The small strain shear stiffness ($G_{\text{max}}$) increases with the increase of cementation at the same level of dry unit weight ($\gamma_d$) for artificially cemented calcareous soil.
- At the end of each unloading cycle there is a little increase in shear stiffness ($G_{\text{max}}$), due to the removal of deviator stress ($q$) and the closer of gaps of microscopic cracks regardless of density and degree of cementation. for artificially cemented calcareous soil.
- With the increase of cementation in cemented calcareous soil the peak deviator stress ($q_{\text{peak}}$) as well.
• With the increase of dry unit weight (γ_d) also increases the shear stiffness (G_{max}) for artificially cemented calcareous soil.

• With the increase of dry unit weight (γ_d) also increases the peak deviator stress (q_{peak}) for artificially cemented calcareous soil.

• With the increase of cementation and dry unit weight (γ_d) artificially cemented calcareous soil shows strain hardening to strain softening behavior.

• During shearing G_{max} starts to decrease for artificially cemented calcareous soil with the increase of deviator stress (q), this reduction of G_{max} is more drastic for highly cemented calcareous soil than the lightly cemented calcareous soil specially after the q_{peak}

• With the increase of cementation and density the failure axial strain (ε_{af} at q_{peak}) starts to decrease for artificially cemented calcareous soil.

• For the low cementation the dense (γ_d = 17 kN/m^3) cemented calcareous soil always dilate during shearing on the other hand lightly (γ_d = 13 kN/m^3) to medium (γ_d = 15 kN/m^3) dense cemented calcareous soil only compressed.

• For the high cementation the dense (γ_d = 17 kN/m^3) to medium (γ_d = 15 kN/m^3) dense cemented calcareous soil always dilate during shearing on the other hand only lightly (γ_d = 13 kN/m^3) dense cemented calcareous soil compressed.

• For the low density (γ_d = 13 kN/m^3) cemented calcareous soil the volumetric strain response is always compressive regardless of degree of cementation during shearing on the other hand medium (γ_d = 15 kN/m^3) to high (γ_d = 17 kN/m^3) density cemented calcareous soil volumetric strain response is always dilatation regardless of degree of cementation.

• For the same level of cementation with the increase of dry unit weight (γ_d) artificially cemented calcareous soil shows an increase of radial strain (ε_r).

• For the same level of density with the increase of degree of cementation artificially cemented calcareous soil shows an increase of radial strain (ε_r).
Table 7.1 Test Result of Uncemented Cast in-place Calcareous Soil Sample

| Test No | CC % | \( \gamma_d \) kN/m\(^3\) | L/D | e | OCR 1 | Test Type | \( G_{\text{max}} \) from B.E. MPa | \( G_{\text{max}} \) from stress-strain MPa | \( q_{\text{peak}} \) kPa | \( p'/\sigma'_{\text{at q}} \) kPa | m = \( q/p' \) at \( q_{\text{peak}} \) | \( \phi_d \) deg. | \( \varepsilon_{af} \) at \( q_{\text{peak}} \) % | \( \varepsilon_{v'f} \) at \( q_{\text{peak}} \) % | \( E_{\text{sec}} \) at \( \varepsilon_a=0.01\% \) MPa |
|---------|------|----------------|-----|---|-----|----------|----------------|----------------|----------------|----------------|----------------|---------|----------------|----------------|----------------|----------------|
| N1      | 0    | 12.67          | 2.007 | 1.129 | 1   | 300 CID  | 120 | 87.71 | 879.77 | 591.5 | 1.487 | 36.58 | 17.9 | 4.53 | 71.3 |
| N2      | 0    | 11.4           | 2.123 | 1.364 | 1   | 100 CID  | 92.79 | 51.28 | 339.55 | 214.2 | 1.585 | 38.826 | 7.59 | -1.08 | 92.1 |
| N3      | 0    | 10.45          | 2.002 | 1.582 | 1   | 500 CIU  | 111.2 | 42.1  | 284.42 | 290.3 | 0.98  | 24.905 | 3.71 | -0.024 | 62.67 |
| N5      | 0    | 10.45          | 2.128 | 1.582 | 1   | 1500 CIU | 262.6 | 116.9 | 738.2  | 782.7 | 0.943 | 24.049 | 3.05 | -0.005 | 135  |
| N6      | 0    | 11.13          | 1.927 | 1.335 | 1   | 870 CPL  | 283.2 | 138.9 | 1291.5 | 888.9 | 1.453 | 35.792 | 15.81 | 7.74  | 171  |
| N7      | 0    | 11.33          | 1.927 | 1.38  | 1   | 1000 CPL | 316  | 272.7 |         |       |       |        |       |       | 904  |
|         |      |                | 1200 CPL | 367 | 360  |        |       |       |        |       |       |        |       |       | 1227 |
|         |      |                | 1000 CS1L | 362.3 | 357 |       |       |       |        |       |       |        |       |       | 1699 |
|         |      |                | 1000 CID | 359 | 358  |       |       |       |        |       |       |        |       |       | 869  |
| N8      | 0    | 11.35          | 1.996 | 1.377 | 1   | 100 CID  | 96.2 | 55.55 | 327.1  | 211.1 | 1.55  | 38.005 | 14.7 | 3.03  | 74.8 |
| N9      | 0    | 11.41          | 1.993 | 1.364 | 1   | 100 CPL  | 76.3 | 49.4  |         |       |       |        |       |       | 66.8 |
|         |      |                | 300 CPL  | 131.7 |     |       |       |       |        |       |       |        |       |       |
|         |      |                | 500 CPL  | 188.4 |     |       |       |       |        |       |       |        |       |       |
|         |      |                | 500 CS1L | 240.4 | 240.9 | 400.5 | 240.3 | 1.667 | 40.706 | 4.42  | -0.49 | 1879 |

**Legend:** e = Void ratio, Dr = Relative Density, L/D = Height and Diameter ratio of the Sample, \( q_{\text{peak}} \) = Maximum Deviator Stress, \( E_{\text{sec}} \) = Secant Young’s Modulus, m = Stress Ratio at failure
### Table 7.2 Test Result of Uncemented and Compressed Calcareous Soil Sample

<table>
<thead>
<tr>
<th>Test No</th>
<th>CC %</th>
<th>( \gamma_d ) kN/m³</th>
<th>L/D</th>
<th>e</th>
<th>OCR</th>
<th>( \sigma'_c ) kPa</th>
<th>( G_{\text{max}} ) from B.E. MPa</th>
<th>( G_{\text{max}} ) from stress-strain MPa</th>
<th>( q_{\text{peak}} ) kPa</th>
<th>( p'<em>{\text{at } q</em>{\text{peak}}} ) kPa</th>
<th>( \frac{m=q/p'}{q_{\text{peak}}} )</th>
<th>( \phi_d ) deg.</th>
<th>( \varepsilon_{\text{af}} ) at ( q_{\text{peak}} ) %</th>
<th>( \varepsilon_{\text{vf}} ) at ( q_{\text{peak}} ) %</th>
<th>( E_{\text{sec}} ) at ( \varepsilon_a=0.01% ) MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>N4</td>
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<td>13</td>
<td>2.031</td>
<td>1.105</td>
<td>4.71</td>
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<td>376</td>
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<tr>
<td>N10</td>
<td>0</td>
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<td>2.039</td>
<td>1.106</td>
<td>15.37</td>
<td>300</td>
<td>217.8</td>
<td>208</td>
<td>934</td>
<td>609.7</td>
<td>1.535</td>
<td>37.6</td>
<td>6.265</td>
<td>0.276</td>
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<td>0.832</td>
<td>37.04</td>
<td>300</td>
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<td>1086</td>
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<td>N12</td>
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<td>2</td>
<td>2</td>
<td>100</td>
<td>336</td>
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<td>1015.6</td>
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**Legend:**  
- \( e \) = Void ratio, \( Dr \) = Relative Density, \( L/D \) = Height and Diameter ratio of the Sample,  
- \( q_{\text{peak}} \) = Maximum Deviator Stress, \( E_{\text{sec}} \) = Secant Young’s Modulus, \( m = \) Stress Ratio at failure
Table 7.3 Test Results of Artificially Cemented Calcareous Soil Sample

<table>
<thead>
<tr>
<th>Test No</th>
<th>CC %</th>
<th>γ_d, kN/m³</th>
<th>L/D</th>
<th>e</th>
<th>OCR</th>
<th>σ_c′, kPa</th>
<th>Test Type</th>
<th>G_max from B.E., MPa</th>
<th>G_max from stress-strain MPa</th>
<th>q' at q_peak, kPa</th>
<th>m=q/p' at q_peak</th>
<th>φ_d at q_peak deg.</th>
<th>ε_af at q_peak %</th>
<th>ε_ef at q_peak %</th>
<th>E_sec at ε_a=0.01% MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>C8</td>
<td>10</td>
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<td>1.12</td>
<td>8.33</td>
<td>100</td>
<td>309.6</td>
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<td>624.4</td>
<td>1.563</td>
<td>38.32</td>
<td>10.4</td>
<td>1.68</td>
<td>434</td>
</tr>
<tr>
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<td></td>
<td></td>
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<td>377.5</td>
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<td>976</td>
<td>624.4</td>
<td>1.563</td>
<td>38.32</td>
<td>10.4</td>
<td>1.68</td>
<td>617.8</td>
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<td>38.75</td>
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<td>0.713</td>
<td>0.291</td>
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<td>CID</td>
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<td>730.2</td>
<td>1.769</td>
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<td>0.713</td>
<td>0.291</td>
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<td>1420.8</td>
<td>775.1</td>
<td>1.833</td>
<td>44.59</td>
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<td>0.583</td>
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<td>1.26</td>
<td>0.301</td>
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<td>655.8</td>
<td>CID</td>
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<td>1653.4</td>
<td>850.8</td>
<td>1.942</td>
<td>47.18</td>
<td>1.26</td>
<td>0.301</td>
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</table>

**Legend:** c = Void ratio, CC = Gypsum Content, L/D = Height and Diameter ratio of the Sample, q_peak = Maximum Deviator Stress, E_sec = Secant Young’s Modulus, CID = Isotropically consolidated drained shear test,
### Table 7.3 Test Results of Artificially Cemented Calcareous Soil Sample

<table>
<thead>
<tr>
<th>Test No</th>
<th>CC %</th>
<th>γ_d kN/m^3</th>
<th>L/D</th>
<th>e</th>
<th>OCR</th>
<th>σ'_c</th>
<th>Test Type</th>
<th>G_max from B.E. MPa</th>
<th>G_max from stress-strain MPa</th>
<th>q_peak at q_peak kPa</th>
<th>p'/q_peak at q_peak kPa</th>
<th>m = q/p' at q_peak</th>
<th>φ_at q_peak deg.</th>
<th>ε_{af} at q_peak %</th>
<th>ε_{vf} at q_peak %</th>
<th>E_{sec} at e_a=0.01% MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>C6 30 13</td>
<td>2.017</td>
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<td>12.5</td>
<td>100 CID</td>
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<td>740</td>
<td>2185.8</td>
<td>1029.2</td>
<td>2.124</td>
<td>51.65</td>
<td>0.619</td>
<td>0.217</td>
<td>2328</td>
</tr>
<tr>
<td>T7 30 13</td>
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<td>1.088</td>
<td>10.9</td>
<td>100 CID</td>
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<td>100 CID</td>
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</table>

**Legend:** e = Void ratio, CC = Gypsum Content, L/D = Height and Diameter ratio of the Sample, σ'_c = Maximum Deviator Stress, q_peak = Maximum Deviator Stress, E_{sec} = Secant Young’s Modulus, CID = Isotropically consolidated drained shear test, CPL = Constant p’ loading drained shear test, CQL = Constant q loading drained shear test
### Table 7.3 Test Results of Artificially Cemented Calcareous Soil Sample

<table>
<thead>
<tr>
<th>Test No</th>
<th>CC</th>
<th>γ_d</th>
<th>L/D</th>
<th>e</th>
<th>OCR</th>
<th>σ'_c</th>
<th>Test Type</th>
<th>G_max from B.E.</th>
<th>G_max from stress-strain</th>
<th>q_peak</th>
<th>p' at q_peak</th>
<th>m=q/p' at q_peak</th>
<th>φ_d at q_peak</th>
<th>ε_af at q_peak</th>
<th>ε_rf at q_peak</th>
<th>E.sec at ε_a=0.01%</th>
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<tbody>
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<td>-</td>
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<td>CPL</td>
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<td>1118</td>
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<td></td>
<td></td>
<td>1000</td>
<td>CPL</td>
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<td>CPL</td>
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<td>1875</td>
<td>1500</td>
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<td>1849</td>
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<td>Sample did not reach to failure</td>
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</table>

**Legend:** e = Void ratio, CC = Gypsum Content, L/D = Height and Diameter ratio of the Sample, q_peak = Maximum Deviator Stress, E_sec = Secant Young’s Modulus, CID = Isotropically consolidated drained shear test, CPL = Constant p’ loading drained shear test, Cσ_1L = Constant σ_1 loading drained shear test
Table 7.3 Test Results of Artificially Cemented Calcareous Soil Sample

<table>
<thead>
<tr>
<th>Test No</th>
<th>CC %</th>
<th>γₐ (kN/m³)</th>
<th>L/D</th>
<th>e</th>
<th>OCR</th>
<th>σₑ' (kPa)</th>
<th>G_max from B.E. (MPa)</th>
<th>G_max from stress-strain (MPa)</th>
<th>q_peak (kPa)</th>
<th>p' at q_peak (kPa)</th>
<th>m=q/p' at q_peak</th>
<th>φ_d at q_peak (deg.)</th>
<th>ε_af at q_peak (%)</th>
<th>ε_rf at q_peak (%)</th>
<th>E_sec at εₐ=0.01% (MPa)</th>
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<td>62.18</td>
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<td>-0.142</td>
<td>3095</td>
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<td></td>
<td>300 CID</td>
<td>2527</td>
<td>2500</td>
<td>185</td>
<td>100 CPL</td>
<td>2406</td>
<td>1667</td>
<td>4627.5</td>
<td>1844.8</td>
<td>2.508</td>
<td>62.18</td>
<td>0.836</td>
<td>-0.142</td>
<td>3095</td>
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**Legend:** e = Void ratio, CC = Gypsum Content, L/D = Height and Diameter ratio of the Sample, q_peak = Maximum Deviator Stress, E_sec = Secant Young’s Modulus, CID = Isotropically consolidated drained shear test, CPL = Constant p’ loading drained shear test,
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Huang'94
NCL, 0% GC
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\[ \gamma_d = 13 \text{kN/m}^3 \quad \gamma_d = 15 \text{kN/m}^3 \quad \gamma_d = 17 \text{kN/m}^3 \]
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