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(Finite element analysis in Structure Mechanics)

(Civil Engineering Research Preparation)   (Soil mechanics)   (Plates and shells)   (Railway Engineering)

(Traffic engineering)   (Civil Engineering Research Project Organisation and Design)   Stress-strain behaviour of cemented sand

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1.2.5　　　　: Stress-Strain behaviours of cemented sands

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Cell pressure control

Back pressure control

Pore pressure transducer

Computer

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Stress-strain behaviour of cemented sand

Department of Civil Engineering

Stress-strain behaviour of cemented sand

By   TSAI, MING-TA

Supervisor: Dr. Dariusz Wanatowaski

Date: August 2009

Abstract

Most prior studies have noted the importance of understanding behaviour of naturally and artificially cemented granular materials. However, artificially cemented which is an alternative to simulate the natural cemented sandy soil materials were tested mostly under conventional pressure (1 MPa) and most of the studies carried out at high pressures were focused on uncemented granular materials. In undrained conditions, pore pressure may increase significantly to lead to liquefaction, instability and failure which are hard to be avoided and cause causalities and heavy loss of properties easily. This study therefore aim to achieve better understanding of behaviour of cemented granular materials at high pressures under the undrained conditions

A total of 8 consolidated undrained triaxial tests including uncemented and cemented Portaway sand CIU triaxial tests of at high pressures and conventional pressure are carried out. Portland cement was used as cementation agent for preparing artificially cemented specimens. A high pressure triaxial cell which can provide up to 64MPa confining pressure in Nottingham Centre of Geomechanics was used to conduct the high confining pressure triaxial test.

It was observed that higher cement content, relative density, confining pressure, and drained condition cause higher maximum deviatoric stress and stress ratio. Furthermore, the higher cement content leads to higher maximum deviatoric stress and let the peak deviatoric stress show earlier. The excess pore water pressure is affected by relative density, confining pressure, and cement content.

KEYWORDS: High-pressure triaxial test, undrained test, excess pore pressure, cemented sand, stress-strain behaviour

Acknowledgements

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Chapter 2   Literature review

Chapter 3   Methodology

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Nomenclature

B	=B value
e	=Void ratio
e_{max}	=Maximum void ratio
e_{min}	=Minimum void ratio
L_o	=Original specimen length
M_S	=Mass of solids
p'	=Mean effective depth
p_f'	=Peak mean effective depth
q	=Deviator stress
q_c'	=Effective confining stress
q_f	=Peak deviatior stress
q/p'	=Effective stress ratio
V_T	=Total volume
V_o	=Original specimen volume
W_S	=Weight of solids
ε_a	=Axial strain
σ_1	=Axial stress
σ_3	=Confining pressure
σ_1'	=Effective axial stress
σ_3'	=Effective confining pressure
ρ_d	=Dry density
ΔL	=Change in specimen length
Δu	=Excess pore water pressure
ΔV	=Change in specimen volume
$\varnothing_p$	=Peak friction angle

Acronym

ADPVC   =Advanced digital pressure/volume controller

CID   =Isotropically consolidated drained test

CIU   =Isotropically consolidated undrained test

UC   =Unconfined compression test

GDS   =Global Digital Systems

Chapter 1 Introduction

1.1 Background

Adding cemented materials artificially to improve local soil has been used successfully in considerable areas, such as ground improvement to increase soil strength or prevent liquefaction, pavement base layers, and slope protection for earth dams. In spite of extensive applications, there are still no reliable criteria to evaluate their strength.

Naturally cemented granular soils often cause engineering problems. For instance, the cemented carbonate soils are generally composed of the skeletal remains of marine organisms, and have various particle sizes from sand to silt size. Cemented carbonate sediments are widely distributed on the continental shelves in temperate and tropical area of the world, and exist in many regions where are substantial petrochemical reserves (e.g., the Arabian Gulf and around the coastal of Australia and India). Therefore, the interest in the behaviour of carbonate soils has increased in recent years, owing to difficulties on these sediments (McLelland, 1988). The engineering behaviours of carbonate soils are quite different from the carbonate soils with similar grain sizes. These cemented carbonate soils are highly compressible and relative low densities. These features may cause very low skin friction to be mobilized on driven piles and cause much greater settlements under footings than expected from theoretical settlement analysis. For example, on the North Rankin Platform installed off North West Australia in 1982, unexpected low driving resistance indicated the piles might not provide the design shaft friction capacity (King & Lodge, 1988).

Several studies investigating the behaviour of carbonate soil were based on laboratory testing of nature soil specimens retrieved from the field. These approaches presented some difficulties resulting from the disturbance to the structure that can

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occur during the sampling process. A loss of stiffness was caused by the disturbance of interparticle contacts and breakage of cement bonds. The other principal problem associated with the testing of naturally cemented sands is its high variability, both in the degree of cementation and in the nature of the particle. Therefore, the variability and the difficulty of sampling without disturbance make it difficult to study the fundamental behaviour of natural cemented materials.

In order to avoid above difficulties, artificially cemented soils have been used to approach the naturally cemented soils. The appropriateness of the approach is concerned as follow. Although the behaviour of the naturally cemented soil is different from sandy soils, it has been demonstrated by Coop (1990) that uncemented carbonate sand exhibit all the features of conventional sands. The engineering responses of noncarbonated and carbonate sands at similar void ratios show similar behaviours (Semple, 1988). Clough et. al (1981) also indicated the general features shown by the artificially cemented specimens are identical to cemented noncarbonated sands. Thus, artificial cemented specimen is an alternative simulation of natural cemented soils in the laboratory to achieve qualitative understanding of the behaviour of cemented soils without excessive sample variability of sampling disturbance.

Soils needed to add cement artificially are generally important or in poor soil conditions. The behaviour of cemented or uncemented granular materials can normally be characterized for practical applications from the results of drained tests because in many situations the material can be considered as drained. However, if the loading rate is fast, such as those caused by explosions or earthquakes, or if the permeability of the soil is relative low, as in fine sands, silts, or carbonate sand, and undrained condition can exist. In undrained condition, pore pressure may increase significantly to lead to liquefaction, instability and failure. If these catastrophic failures occur in important infrastructures, causalities and heavy loss of properties cannot be

avoided. These illustrate the importance of understanding the undrained behaviour in cemented granular materials.

In recent studies, artificially cemented materials were tested mostly under low pressures and most of the studies carried out at high pressures were focused on uncemented granular materials. There is a lack of data on cemented granular materials tested at high pressures under undrained conditions. Thus, for future geotechnical applications, more experimental studies are required to improve understanding of undrained behaviour of cemented soils at high pressure.

1.2 Research objectives

The goal of this research is to achieve better understanding of behaviour of cemented granular materials at high pressures under the undrained conditions. This involves a detailed literature review and the laboratory testing.

Specific tasks of the project include:

(1) A literature review on the effects of cement type, cement content, effective confining pressure, and drainage on behaviours of the cemented granular materials.

(2) To conduct a series of undrained triaxial tests of uncemented Portaway sand at high pressures and conventional pressure.

(3) To conduct a series of undrained triaxial tests of cemented Portaway sand at high pressures and conventional pressure.

(4) To analyses and compare the results with previous studies.

The data obtained could be used to examine or improve existing constitutive models to simulate behaviour of cemented materials more accurately.

1.3 Organisation of the dissertation

This thesis is divided into five chapters. The brief outline of each chapter is given below.

Chapter 1 gives an introduction to this research work. In Chapter 2, a review of background knowledge and literature relevant to this work is presented. The literature review is divided into three parts: background, the effect of cement type, isotropic compression behaviour, and shear behaviour.

Chapter 3 describes methodology including the apparatus and experimental procedures. Initially, the properties of the Portaway sand and cementing agent are described. Secondly, the sample moulding and curing procedures are presented. Thirdly, the high pressure triaxial testing system is introduced. Then, the isotropically concentrated undrained triaxial testing procedures are illustrated. Finally, the testing program is given.

Chapter 4 consists of the results and interpretations of the undrained high pressure triaxial tests on cemented specimens. .

Chapter 5 presents the conclusions of the work and recommendations for further research. The effects of confining pressure and cement content on the behaviour of cemented Portaway sand are discussed. Comparison of drained and undrained behaviour of cemented sand at high pressures is also presented.

Appendix A is the short paper related to the work carried out for the project. Appendix B contains two risk assessments for experimental works carried out on specimen preparation and assembling the high pressure triaxial apparatus. Appendix C and D are the paper submitted to the winter and summer conferences.

Chapter 2   Literature review

2.1 Introduction

Cemented and bonded materials are often encountered in geotechnical field. Cementation plays an important role in stress–strain, stiffness, bearing capacity and mechanical parameters of cemented soils. However, from a mechanical point of view, cemented soils, weak rocks, and similar bonded materials constitute an intermediate class of geomaterials placed between classical soil mechanicals and rock mechanicals (Schnaid et al.2001).

This chapter presents a literature review, divided into five sections: (1) background, (2) cement type, (3) isotropic compression behaviour and (4) shear behaviour. The first part, section 2.2, presents a general review of behaviour of naturally and artificially cemented soils. Next, section 2.3, the effect of cement type is discussed briefly. Following this, isotropic compression behaviour in cemented soils is reviewed. Finally, in section 2.5, the shear behaviour is discussed.

2.2 Background

Cementation in naturally cemented sands can be attributed to several sources as follows: (i) existence of agents such as silica, hydrous silicates, hydrous iron oxides and carbonate deposits between grains, (ii) cold welding between the soil grains, (iii) presence of a matrix of silts and clays between sand and gravel particles (Haeri et al. 2005). Experimental research on naturally cemented soils is rare because acquiring undisturbed specimens from naturally cemented course grained soils is extremely difficult. The other problem associated with the testing of naturally cemented sands is that, depending on their geological origin, there can be high spatial variability, on the degree of cementation, the nature of particles and the density. An alternative is therefore to use artificially cemented specimens made up through the addition to the soil of a cementing agent, such as Portland cement, gypsum, or lime. This allow the

simulation of natural cemented soils in the laboratory and the qualitative understanding of the behaviour of cemented sands without excessive sample variability and any uncertainty due to sampling disturbance (Rotta et al. 2003).These supported by Coop (1990) demonstrated that uncemented carbonate sand exhibits all the features of conventional soil. In addition, the engineering response of noncarbonated and carbonate sand at similar void ratios shows similar behaviour (Semple 1988). Furthermore, Boey (1990) also demonstrated that artificially and naturally cemented soils show similar behaviour.

In last three decades, a number of researchers have been giving important experimental contributions in this area. Some of them are reviewed below to identify some significant characteristics of the cemented granular soils.

Clough et al. (1981) reported a set of triaxial and unconfined compression tests to examine the effects of cementation and density on the behaviour of naturally and artificially cemented sands. Test results showed that stiffness and peak strength increased with density and the amount of cementing agent. They suggested that in addition to cementation and density, grain size distribution and grain size arrangement (fabric) have an important role on the behaviour of cemented sands.

Leroueil and Vaughan (1990) reviewed studies that had been carried out by others on cemented and structured soils. They suggested that structure in soils such as stiff over consolidated clays, clay-shales, cemented sands, residual soils, artificially bonded soils and weak rocks may arise from many different sources but they have similar feathers. They have shown that the yield in structured soils can occur under compression, shear and swelling stresses and the stress-strain behaviour of naturally and artificially cemented soils to be basically dependent on their initial state, and its position in relation to the yield curve and the critical state line of the uncemented remoluded soil.

Airey (1993) carried out conventional and stress path triaxial tests to examine the

behaviour of natural calcarenite soils. He used tension testing for estimating of the degree of cementation of naturally cemented specimens. He reported the responses of the naturally cemented soils were found to be similar to those of other cemented soils. He suggested that the cementation increases the shear modulus and the size of the yield locus; however, its effects on the volumetric response are negligible.

Coop and Atkinson (1993) conducted tests on artificially cemented carbonate sands. Test results showed that the direction of the stress path, the drainage conditions and the confining pressure affect the peak strength. They showed that the stress–strain behaviour of cemented specimens depends on the position of the initial state of the stress of the soil with respect to the yield locus of the bonding. They also described the idealized behaviour of cemented soils, which is divided into three different classes, as illustrated in figure 2.1. The first class shown in line 1 of figure 2.1 occurs when the soil reaches its yield stress during isotropic compression; in this case, shearing will produce a similar behaviour to that observed for an equivalent uncemented soil. The second class shown in line 2 of figure 2.1 occurs for intermediate stress states, in which the bonds will be broken during shear; the strength is controlled by the frictional component of the equivalent uncemented soil and the stress-strain curve shows a well-defined yield point after an apparent linear behaviour. In the third class shown in line 3 of figure 2.1 the soil is sheared at low confining stresses, when compared to the bond strength; a peak in the stress-strain curve occurs at small strains and for stresses outside the limit state surface of the equivalent uncemented soil.

Das et al. (1995) carried out unconfined compression tests on the artificially cemented sands. They reported that unconfined compression strength increase with cement content and compressive stains at failure decrease with cement content.

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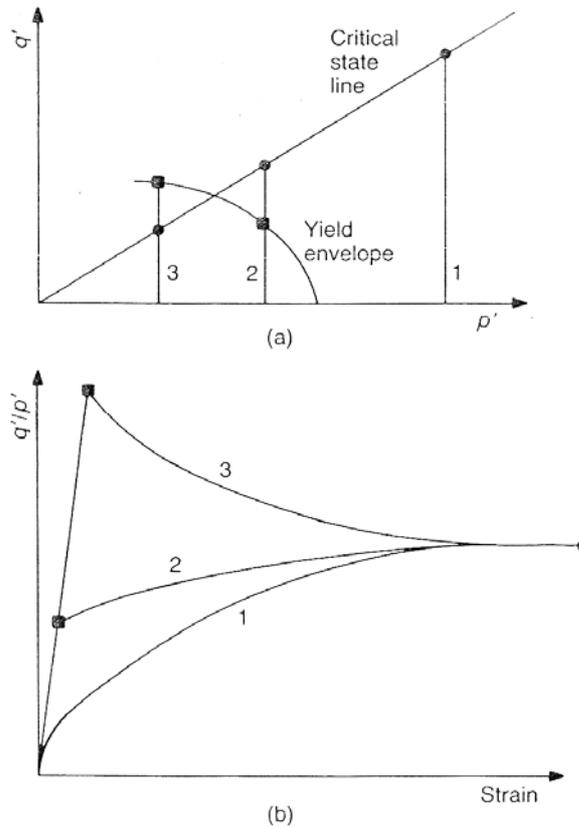
Figure 2.1 Idealized behaviour of cemented soils: (a) stress path; (b) stress-strain behaviour (after Coop & Atkinson, 1993)

2.3 Cement type

Ismail et al. (2002) studied the effect of cement type on the shear behaviour of cemented calcareous sand. They used three different types of cement agent (Portland cement, gypsum and calcite) for preparing artificially cemented specimens. Triaxial tests on specimens of different cementation type with similar unconfined compression strength and density, showed different effective stress paths and post yield responses. The Portland cemented specimens showed ductile behaviour while calcite and gypsum cemented specimens exhibited brittle yield. Their typical behaviours are shown in figure 2.2.

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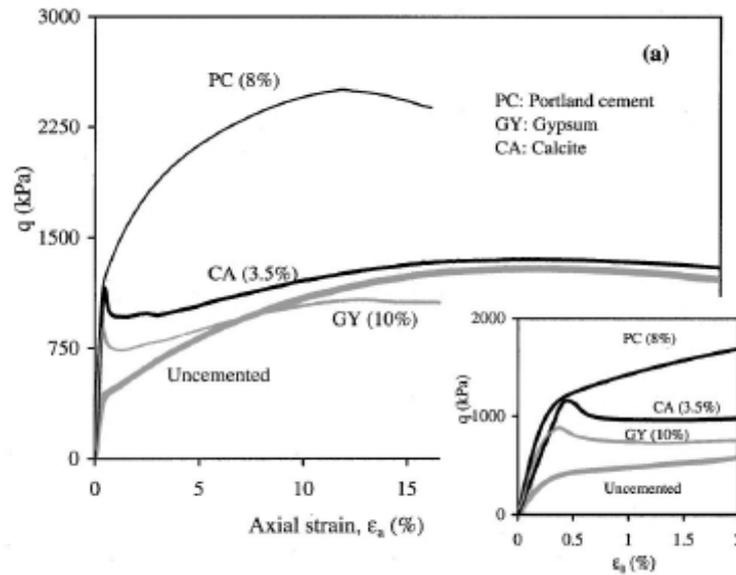
Figure 2.2 Typical stress-strain curve (Modified after Ismail et al. 2002)

2.4 Isotropic compression behaviour

The compression behaviour of the cemented sands is similar to that of other compressive soils. In isotropic consolidation, the specimens show a stiff response up to pre-consolidation pressure and then follow the normal compression lines (NCL). The normal compression line can be expressed as:

$$v = N - \lambda \ln p' \qquad (2.1)$$

Cuccovillo and Coop (1999) described the idealized isotropic compression behaviour of calcarenite of various degrees of bonding shown in figure 2.3.

Rotta et al. (2003) reported a series isotropical compression test shown in figure 2.4. It presents the results obtained from the isotropic compression tests carried out on specimens cured under different confining stresses and for cement contents of 1%, 2% and 3% respectively. The identification of the tests follows the general nomenclature ISO(x)y-z, where x is the cement content, y is the curing confining stress, and z is the maximum isotropic stress reached in the test. They found that, for the artificially cemented specimens, the primary yield stress in isotropic compression

is a function of the curing void ratio and cement content; it is also dependent on the curing stresses, but is independent of any increase in the OCR after curing. Indeed, the relative contribution of cementation to the soil behaviour in isotropic compression reduces with decreasing curing void ratio. At similar void ratios, the curing confining stress influences the primary yield stress, but does not affect the incremental yield stress.
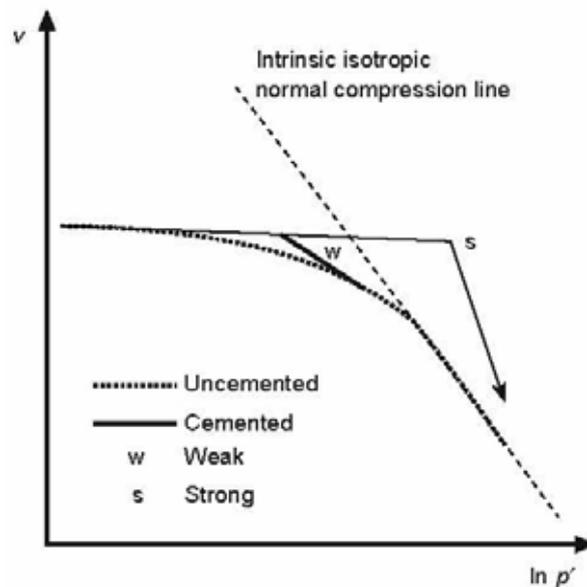
Figure 2.3 Schematic comparison of the isotropic compression of weakly and strongly cemented carbonate sand (after Coop and Atkinson 1993)

The results also demonstrates the coupled effect of density and cementation on the cemented soils. Huang and Airey (1998) reported that, the effects of the cementation are only significant for stresses below an apparent preconsolidation stress for the artificial soil. The strength and stiffness increase with increasing density and cement content, but the influence of the cementation decreases as the density increases.
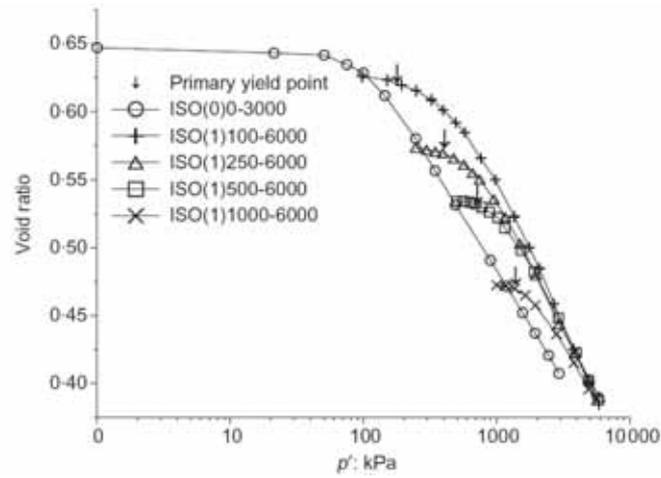
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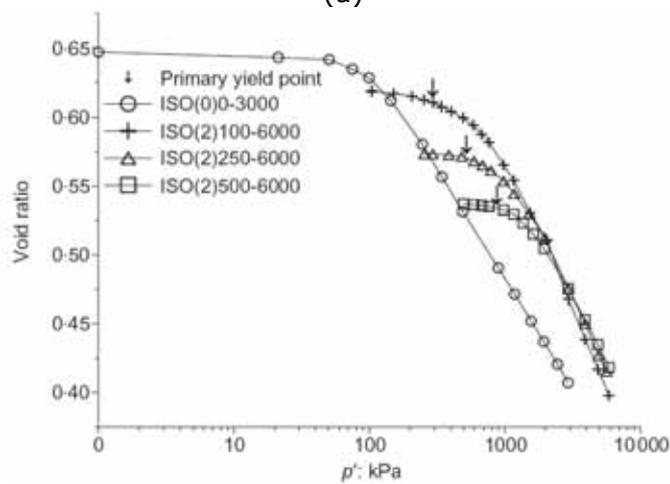
(b)
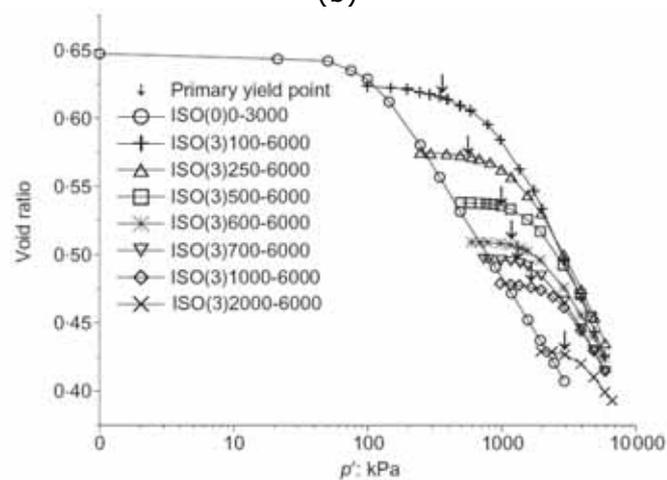
(c)

Figure 2.4 Isotropic compression responses for the 0% to 3% cement content specimens. (a) 0% and 1% cement content. (b) 0% and 2% cement content. (c) 0% and 3% cement content (Modified after Rotta et al. 2003).

2.5 Shear behaviour

Triaxial test is widely used to investigate the stress-strain behaviour of soils. Isotropically consolidated drained (CID) test and undrained (CIU) test are normally used to accurately characterize the behaviour of uncemented and cemented materials. In this section, both of them will be reviewed. All undrained behaviour of sand at high pressures is also discussed at the end of this section.

2.5.1 Drained behaviour

Schnaid et al. (2001) had carried out CID tests on artificially cemented sand. The specimens were prepared with cemented content of 1, 3, and 5% respectively. The soil specimens were derived from weathered sandstone and obtained from the region of Porto Alegre in southern Brazil, and the cement type was ordinary Portland cement. Their results, shown in figure 2.5, illustrate typical trend of deviatoric stress versus axial strain and volumetric strain versus axial strain for specimens with different cement content.

Figure 2.5 (a) shows the results on uncemented specimens and figures 2(b–d) shows the artificially cemented specimens with cemented content of 1, 3, and 5% respectively. Figure 2.6 illustrates the changes in soil behaviour, resulting from the addition of cement, by comparing the stress-strain curves obtained for different cement contents and a constant initial mean effective stress.

An initial examination of the curves presented in figures 2.5 and 2.6 clearly shows the soil behaviour to be strongly dependent on the cement content. As the cement content increases, both peak strength and initial stiffness increase. Unlike the uncemented soil, cemented specimens show a brittle behaviour at failure with well defined shear planes being formed. This brittle response increases with increasing cement content and decreases as the initial mean effective stress increases. The axial strain at failure decreases with increasing cement content and it also decreases

with increasing initial mean effective stresses. As for the volumetric response, the cemented specimens show an initial compression followed by a strong expansion with the maximum dilation rate taking place right after the peak strength. Subsequently, the dilation rate decreases as the soil approaches an ultimate stable condition. Therefore the existence of an ultimate state was suggested. In figure 2.6, the deviatoric stress approaches a constant value with increasing axial strain, which does not seem to be affected by cementation.
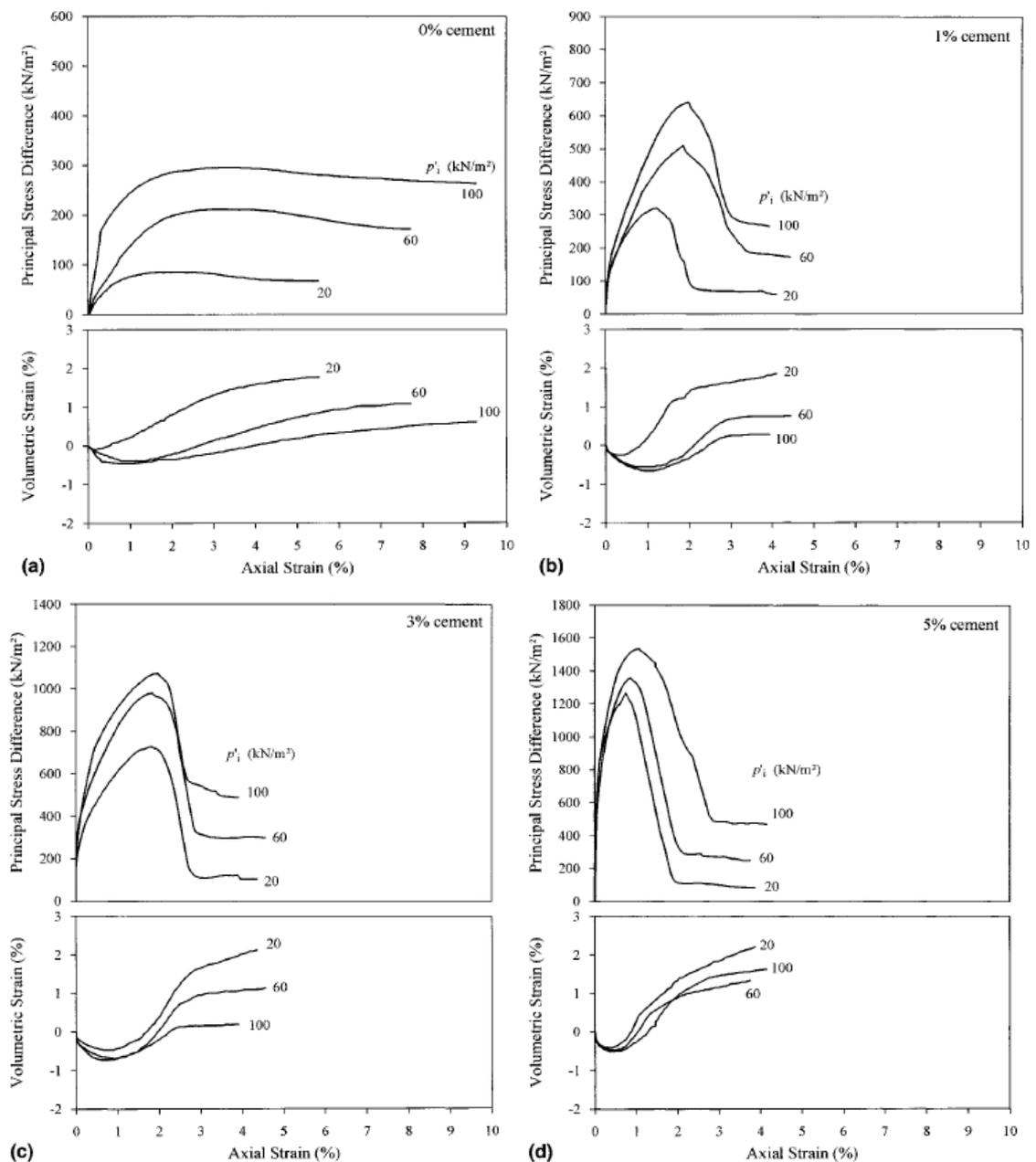
Figure 2.5 Stress-strain-volumetric response for: (a) 0% cement; (b) 1% cement; (c) 3% cement; (d) 5% cement (after Schnaid et al. 2001).

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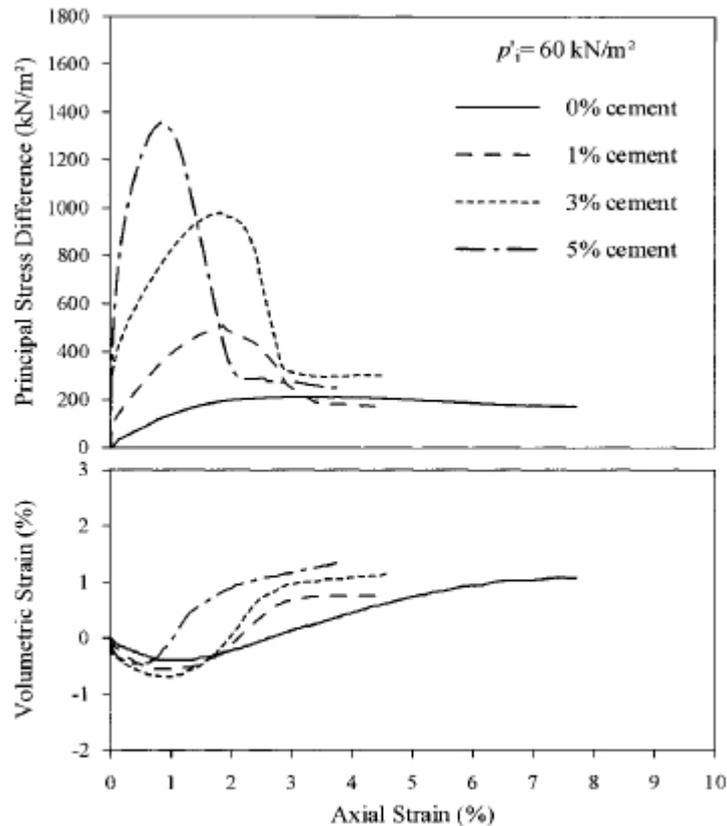
Figure 2.6 Stress-strain-volumetric response for a constant initial mean effective stress p'=60 kN/m² (after Schnaid et al. 2001).

2.5.2 Undrained behaviour

Haeri et al. (2005) conducted a series of isotropically consolidated undrained tests (CIU) on artificially cemented sandy gravel. The typical failure modes of uncemented and cemented specimens are shown in figure 2.7. They observed that the specimens tested at confining pressures higher than 100 kPa showed barreling failure modes during shear. However, shear zones formed in specimens tested at the low confining pressure (25 kPa). Uncemented specimens at confining pressures from 50 and 100 kPa and relative density of 70% showed barreling failure mode at low strain levels of up to 10% and then shear zones were formed after 10% strain. Lightly cemented specimens (1.5% cement) showed a brittle failure mode accompanied by shear zones at low confining pressures. The same specimens tested at higher pressures showed a barreling failure mode at small strain followed by shear zones at strains greater than about 8%. Other cemented specimens (3%, 4.5%, 6% and 9% cement) showed a

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brittle failure mode accompanied by shear zones. The brittle behaviour increased with increasing of cementation and density and decreased with an increase in confining pressure. They also found that the thickness of the shear zone in cemented specimens varied between 2 and 5 cm and the inclination angle of the shear zones with respect to horizontal varied between 60 and 70 degrees.
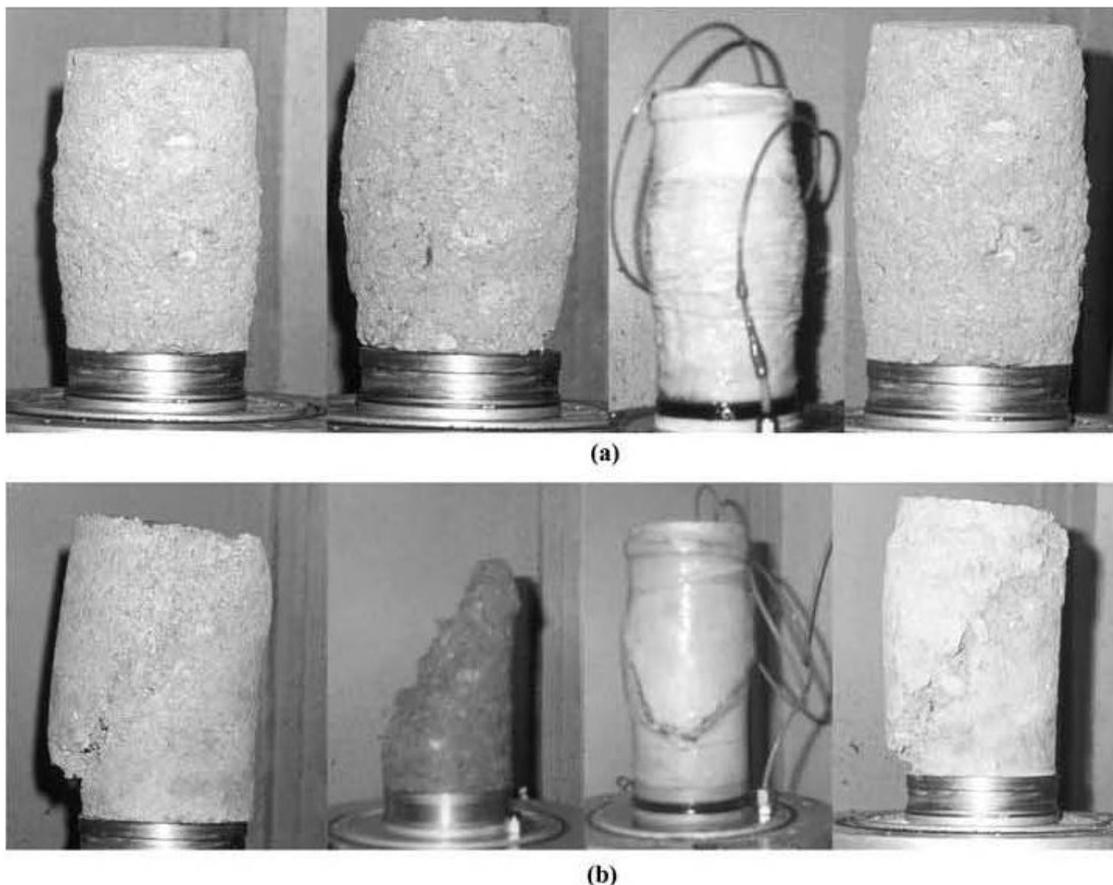
Figure 2.7 Prevailing failure modes of specimens: (a) barreling in uncemented specimens, (b) shear zone in cemented specimens (after Haeri et al. 2005)

Rsults form Haeri et al. (2005) carried out on dense specimen prepared with cemented content of 0, 1.5, 4.5, and 9% and performed at different confining pressures including 25, 50, 100, 300, and 400, or 500 kN are shown below. Response of stress-strain, excess pore pressure-strain and stress ratio-strain are shown in figures 2.8 and 2.9 and the stress paths are shown in figure 2.10.

(a)   (b)
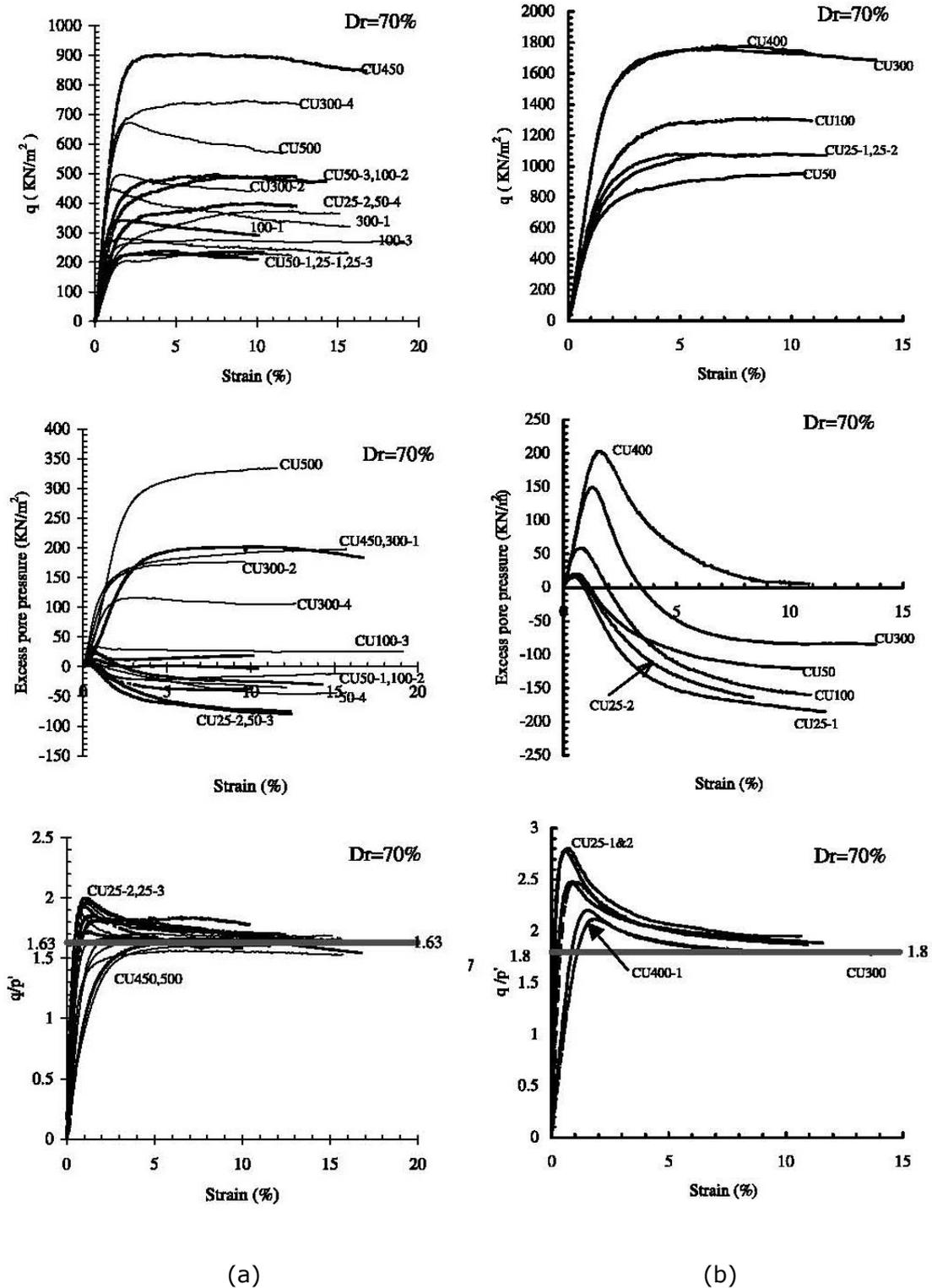
Figure 2.8 Responses of Stress-strain, excess pore pressure-strain and stress ratio-strain for: (a) uncemented, (b)1.5% cement   (modified after Haeri et al. 2005)
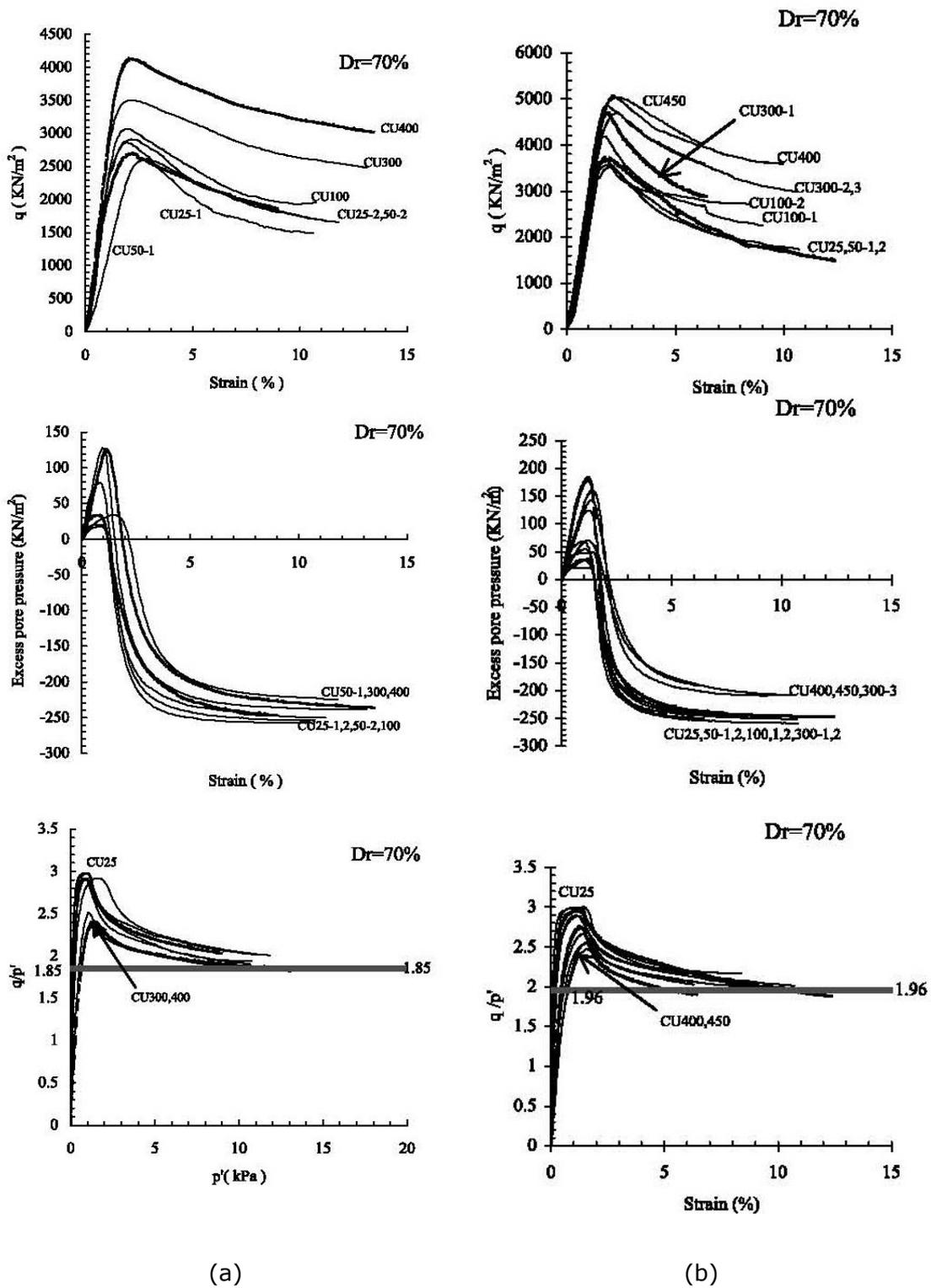
Figure 2.9 Responses of Stress-strain, excess pore pressure-strain and stress ratio-strain for: (a) 4.5% cement, (b) 9% cement (modified after Haeri et al. 2005)

(a)

(b)

(c)

(d)
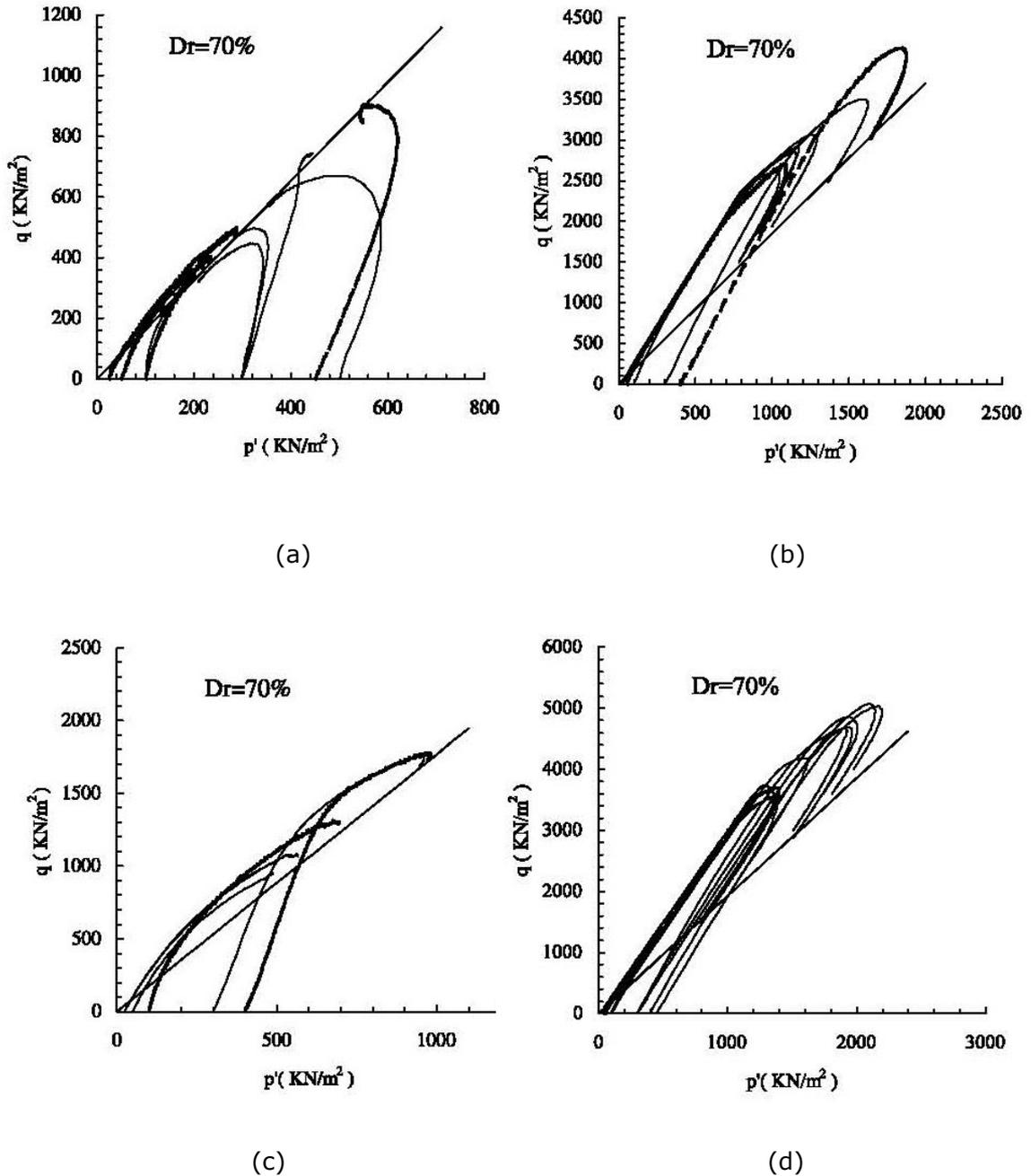
Figure 2.10 Stress paths: (a) uncemented, (b) 1.5% cement, (c) 4.5% cement, (d) 9% cement (modified after Haeri et al. 2005)

In these figures, the tests results are named based on test conditions. CU stands for consolidated undrained tests. A two or three digit number indicating confining pressure follows the CU caption. This number is followed by a dash and a one-digit number showing the number of repeated tests on a specific sample with specific cement content tested under the same confining pressure.

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Test results for uncemented and lightly cemented samples shown in figure 2.8 indicate that the stiffness increases with an increase in confining pressure for such soils. However, the effect of confining pressure on the stiffness is much less significant for strongly cemented specimens shown in figure 2.9. As can be observed from figures 2.8 and 2.9, stiffness increases with the increase in cement content. The stress–strain curves for uncemented and lightly cemented samples shown in figure 2.8 do not have any peak strength, while those for highly cemented samples present clear peak strength shown in figure 2.9. The maximum stress ratio, $(q/p')_{max}$, decreases with an increase in confining pressure are shown in figure 2.8 and 2.9. However, the value increases with relative density and cement content.

Consider the maximum value of $(q/p')_{max}$ which is associated with the lowest confining pressure. In triaxial tests, the stress ratio usually approaches a constant value at ultimate state. The ultimate stress ratio is the slope of critical state line in q–p' space. The variation of slope with respect to cement content is plotted in Figure 2.11. The value increases for cemented samples with increase in cement content and density (Haeri et al. 2005).
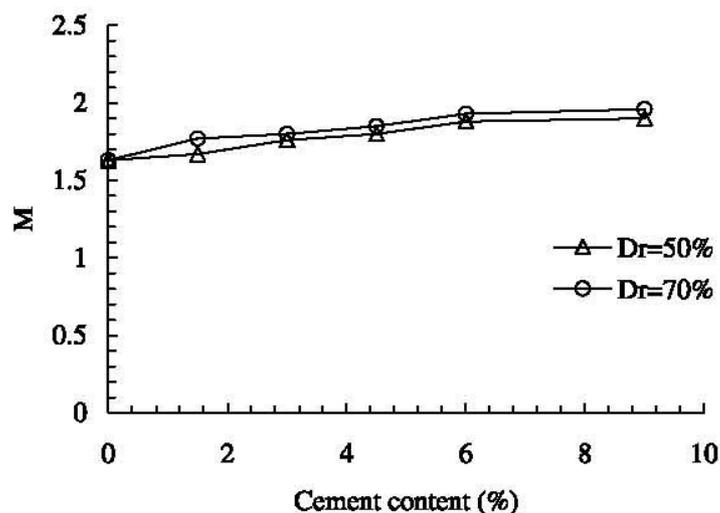
Figure 2.11 Variation of ultimate stress ratio (M) versus cement content (modified after Haeri et al. 2005)

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2.5.3 Undrained behaviour at high pressures

Lade and Yamamuro (1996) reported a series of isotropically consolidated undrained tests (CIU) on Cambria sand at high pressure. The initial effective confining pressure was between 6.4 MPa and 68.9 MPa. Their results shown in figure 2.12, illustrate the deviatoric stress versus axial strain and excess pore pressure versus axial strain. The results of effective stress paths are shown in figure 2.13.
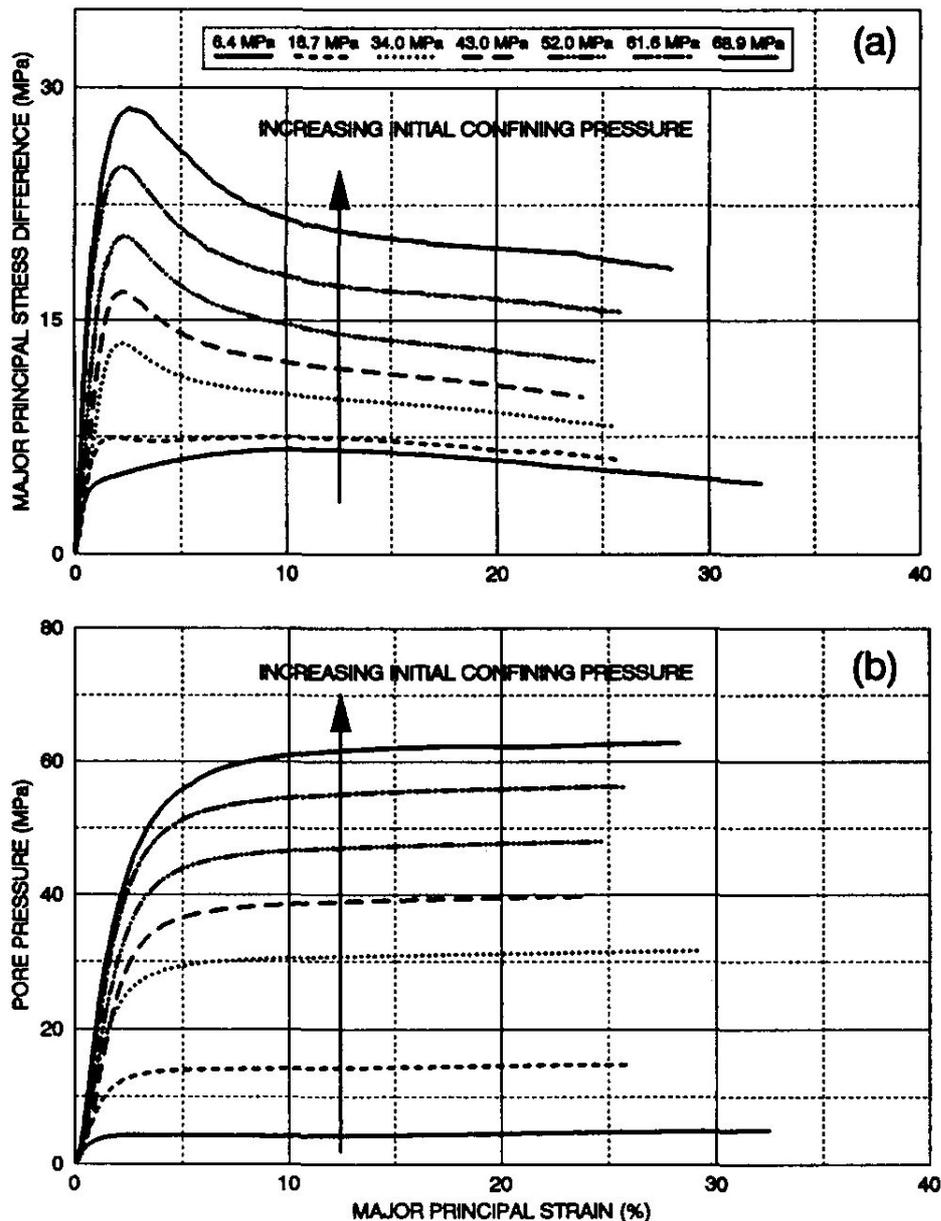
Figure 2.12 Undrained compression tests between 6.4 and 68.9 MPa initial confining pressure on dense Cambria sand: (a) deviatoric stress; and (b) excess pore pressure relationship. (after Lade and Yamamuro 1996)
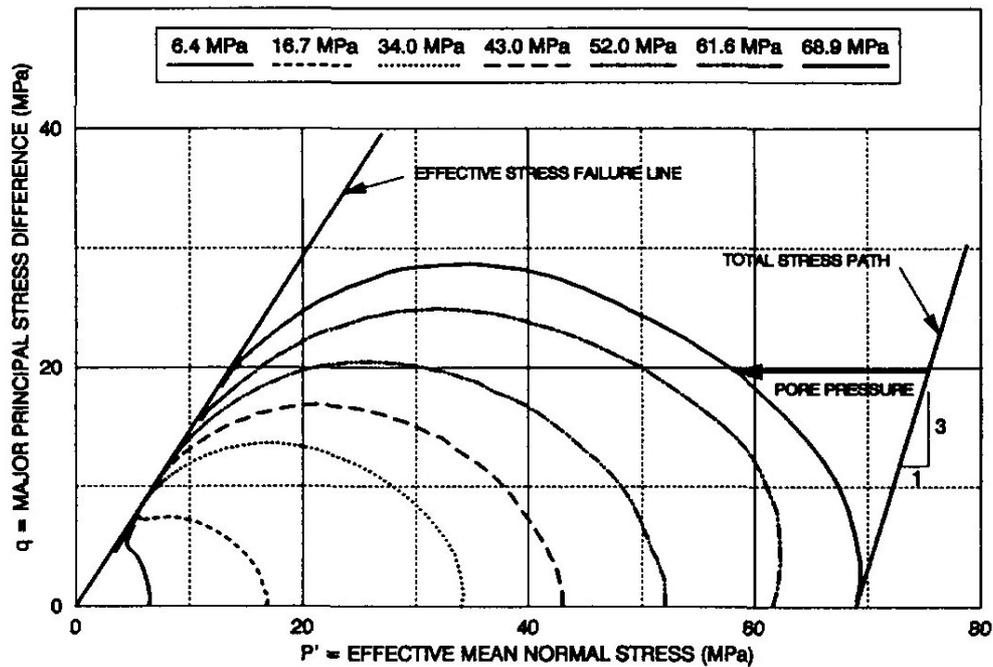
Figure 2.13 Effective stress paths of undrained compression tests on dense Cambria sand shown on p'-q diagram. (after Lade and Yamamuro 1996)

Lade and Yamamuro (1996) found that the initial slopes of stress-strain curves are steeper as the initial confining pressures increase.   At low initial confining pressures, such as 6.4 MPa, the maximum deviatoric stresses occurs at large strains. The test at 16.7 MPa confining pressure shows the different behaviour. It increases to a peak, following by a decrease, but then rise again to its maximum value at failure. The tests at confining pressures higher than 16.7 MPa increase to a peak, then following by a decrease to the end of test. At large strains the effective confining pressure has decreased to allow the specimen to present volumetrically dilatant tendencies, leading to the pore pressure to decrease, therefore, increasing the effective stress. This can be observed clearly in figure 2.13.

The excess pore pressures shown in figure 2.12 (b) indicate that all tests shows tendencies for overall net compressive volumetric behaviour, because the generated pore pressure are all positive. As the initial confining pressure is above 16.7 MPa, the pore pressure which induced quickly results in a rapid decrease in the effective confining pressure. Hence, the deviatoric stress reaches to a peak soon after shearing

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begins. After the peak, the magnitude of the deviatoric stress declines with increasing strain. This is caused by pore pressure which continues to increase with additional shearing. In figure 2.13, the tests of initial confining pressures above 16.7 MPa show in effective stress paths that rise up, and then move toward the origin due to increasing pore pressure. Then, the maximum deviatoric stress is reached, and it decreases with continuing pore pressure increase until it become tangential to the effective stress failure line. These effective stress paths exhibit contractive volumetric change behaviour. However, the lower initial confining pressure, such as 6.4 MPa, shows different behaviour. It increases upward, but it moves only slightly to the left, because the pore pressure generated is not as significant as that at higher confining pressure. This test shows a tendency toward volumetric dilation before the maximum deviatoric stress, and this result in decreasing pore pressure. The specimen continues to strengthen, the stress path moves upward with increasing deviatoric stress and effective mean stress. In the end, it reaches the maximum deviatoric stress and follows the effective stress failure line.

Lade and Yamamuro (1996) also discussed the differences between high pressure stress paths for drained and undrained tests which are shown as figure 2.14. The specimen in the undrained test, first undergoes large isotropic compression under drained conditions. Following this, it experiences undrained shearing, in which induced large positive pore pressures occurs rapidly. Therefore, the effective confining pressure in the specimen decreases. In figure 2.14, the drained test shows a smaller isotropic compression and the most of the particle crushing and corresponding densification occurs during the shearing.

2.6 Summary

In this chapter, the relationship between artificially and naturally cemented soils is reviewed. Because of their similar behaviours at similar void ratio and initial state, using artificially cemented specimens is an alternative to understand the behaviour of cemented sands without excess sample variability or sampling disturbance. It has
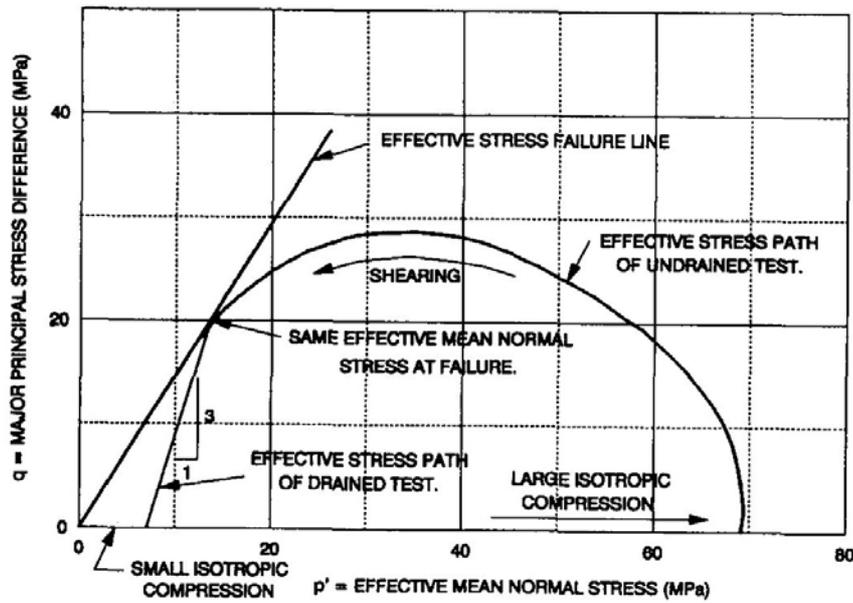
Figure 2.14 Effect of different high-pressure stress paths of drained and undrained tests that fall at common effective mean normal stress. (after Lade and Yamamuro 1996)

been shown that the stress-strain behaviours of cemented soils depend on the combined effects of dry density, effective confining pressure and percentage of cement content. However there is a lack of understanding on the stress-strain undrained behaviour of cemented granular materials at high pressure.

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Chapter 3   Methodology

3.1 Introduction

The focus of this chapter is to describe the materials, equipments and procedures that were used in this research work. Firstly, the properties of the Portaway sand and Portland cement are described. Secondly, the sample moulding and curing procedures are presented. Thirdly, the high pressure triaxial testing system is introduced. Then, the isotropically concentrated undrained triaxial testing procedures are illustrated. Finally, the testing program is presented.

3.2 Materials

The granular material used in this study is Portaway sand. Portland cement (CEM II /B-V 32, 5 N) used as cementing agent. Detailed description of both materials is given below.

3.2.1 Portaway sand

The sand tested was Portaway sand which was used in previous experimental projects at Nottingham Centre for Geomechanics (Wang 2005). It is well-graded, medium quartz sand from Sheffield, England. Before using the Portaway sand for specimen preparation, the sand was washed and oven dried, and then sieved passing through 2.0mm and retaining on 63μm. Portaway sand has a specific gravity G_s of 2.65, a mean grain size D_{50} of 0.4mm, a effective grain size D_{10} of 0.22mm, a coefficient of uniformity C_u of 2.05. The maximum void ratios e_{min} and e_{max} are 0.46 and 0.79, respectively (Wang 2005). The size gradation curve is shown in figure 3.1 and the index properties are listed in table 3.1.

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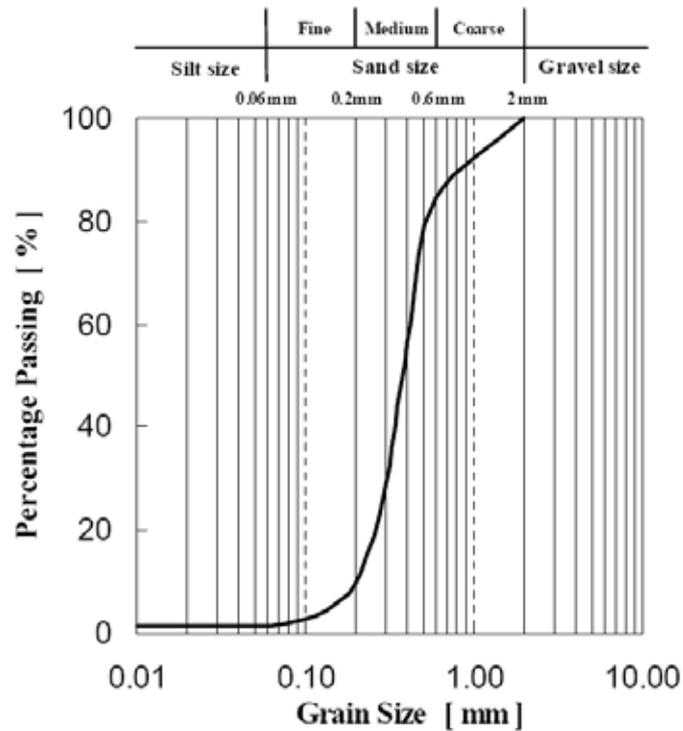
Figure 3.1 Grain size distribution curve of Portaway sand

Table 3.1 Index Properties of Portaway sand

Portaway sand index properties	
Effective grain size, D10: mm	0.22
Mean grain size, D50: mm	0.40
Uniformity coefficient: D60 / D10	2.05
Specific Gravity, Gs	2.65
Maximum void ratio e max	0.79
Minimum void ratio e min	0.46

3.2.2 Cementing agent

Portland cement (BS EN 197-1 CEM II /B-V 32, 5 N) is used for cementing agent in this study in order to compare the previous study. According to BS EN 197-1, CEM II is Portland-composite cement. It contains Portland cement and up 35% of other single constituents. It mainly contains 65-79% of Clinker and 21-35% of siliceous fly ash. Its composition and strength are shown in tables 3.2 and 3.3, respectively.

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Table 3.2 The composition of CEM II /B-V 32, 5 N Portland cement (Modified from BS EN 197-1)

Main types	Notation of the 27 products (types of	Clinker	Blast-furnace slag	Silica fume	Pozzolana natural	Pozzolana natural calcined	Fly ash siliceous	Fly ash calcareous	Burnt shale	Limestone	Limestone	Minor additional constituents
		K	S	D [b]	P	Q	V	W	T	L	LL	
CEM I	Portland cement	CEM I	95-100	–	–	–	–	–	–	–	–	–	0 to 5
CEM II	Portland-slag cement	CEM II/A-S	80 to 94	6 to 20	–	–	–	–	–	–	–	–	0 to 5
		CEM II/B-S	65 to 79	21 to 35	–	–	–	–	–	–	–	–	0 to 5
	Portland-silica fume cement	CEM II/A-D	90 to 94	–	6 to 10	–	–	–	–	–	–	–	0 to 5
	Portland-pozzolana cement	CEM II/A-P	80 to 94	–	–	6 to 20	–	–	–	–	–	–	0 to 5
		CEM II/B-P	65 to 79	–	–	21 to 35	–	–	–	–	–	–	0 to 5
		CEM II/A-Q	80 to 94	–	–	–	6 to 20	–	–	–	–	–	0 to 5
		CEM II/B-Q	65 to 79	–	–	–	21 to 35	–	–	–	–	–	0 to 5
	Portland-fly ash cement	CEM II/A-V	80 to 94	–	–	–	–	6 to 20	–	–	–	–	0 to 5
		CEM II/B-V	65 to 79	–	–	–	–	21 to 35	–	–	–	–	0 to 5
		CEM II/A-W	80 to 94	–	–	–	–	–	6 to 20	–	–	–	0 to 5
		CEM II/B-W	65 to 79	–	–	–	–	–	21 to 35	–	–	–	0 to 5
	Portland-burnt shale cement	CEM II/A-T	80 to 94	–	–	–	–	–	–	6 to 20	–	–	0 to 5
		CEM II/B-T	65 to 79	–	–	–	–	–	–	21 to 35	–	–	0 to 5
	Portland-limestone cement	CEM II/A-L	80 to 94	–	–	–	–	–	–	–	6 to 20	–	0 to 5
		CEM II/B-L	65 to 79	–	–	–	–	–	–	–	21 to 35	–	0 to 5
		CEM II/A-LL	80 to 94	–	–	–	–	–	–	–	–	6 to 20	0 to 5
		CEM II/B-LL	65 to 79	–	–	–	–	–	–	–	–	21 to 35	0 to 5

Table 3.3 The strength of CEM II /B-V 32, 5 N Portland cement (Modified from BS EN 197-1)

Strength class	Compressive strength MPa				Initial setting time	Sound-ness (expan-sion)
	Early strength		Standard strength			
	2 days	7 days	28 days		min	mm
32,5 N	–	≥ 16,0	≥ 32,5	≤ 52,5	≥ 75	
32,5 R	≥ 10,0	–				
42,5 N	≥ 10,0	–	≥ 42,5	≤ 62,5	≥ 60	≤ 10
42,5 R	≥ 20,0	–				
52,5 N	≥ 20,0	–	≥ 52,5	–	≥ 45	
52,5 R	≥ 30,0	–				

3.3 Moulding and curing of cemented specimens

Two types, A and B, of cemented samples were used in the isotropically consolidated undrained triaxial tests. Type A was 5cm in diameter and 10cm in height and type B was 3.8cm in diameter and 7.6cm in height. The dry density of all the cemented specimens was 17.4kN/m^3.

At the beginning of moulding, the soil, cement and water were weighted as shown in table 3.4 and table 3.5, and the soil and cement were mixed until the mixture acquired a uniform consistency. The water was then added continuously to the mixing process until a homogeneous paste was created. After mixing sufficient material for one specimen, the mixture was placed in the box to avoid moisture losses as shown in figure 3.2.

Table 3.4 The composition of type A

The composition of type A				
Dimension= 5cm(Diameter) x 10cm(Height) r_d=17.4 kN/m^3				
Cement content (%)	weight of sand (g)	Weight of cement (g)	Weight of water (g)	Total weight (g)
5%	334.25	16.71	35.10	386.06
10%	321.24	32.12	35.34	388.71
15%	309.24	46.39	35.56	391.18

Table 3.5 The composition of type B

The composition of type B				
Dimension= 3.8cm(Diameter) x 7.6cm(Height) r_d=17.4 kN/m^3				
Cement content (%)	weight of sand (g)	Weight of cement (g)	Weight of water (g)	Total weight (g)
5%	146.73	7.34	15.41	169.47
10%	141.02	14.10	15.51	170.63
15%	135.75	20.36	15.61	171.72

(a)

(b)

(c)

(d)
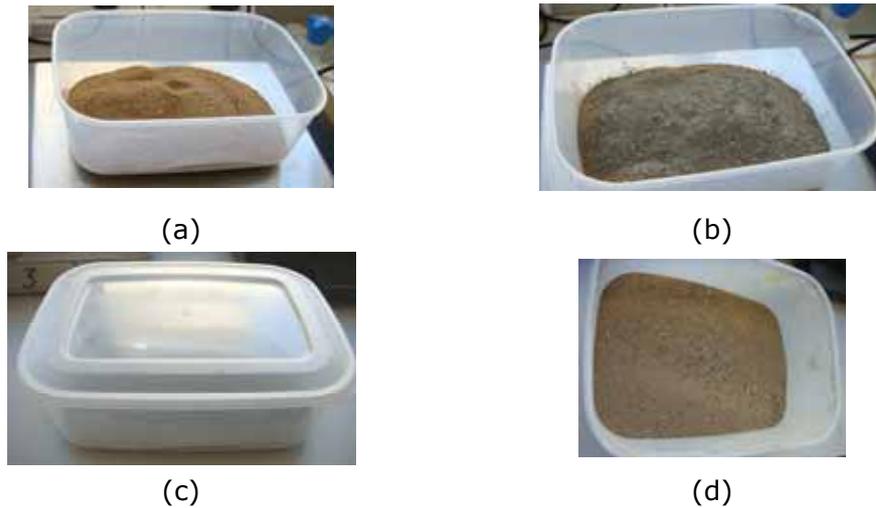
Figure 3.2 The process of mixing sample: (a) Adding and measuring sand. (b) Adding and measuring cement. (c) Shaking the box to mix. (d) Mixed cemented sand.

The specimen was then compacted in three layers into split cylindrical mould shown in figure 3.3, which was lubricated. For the purpose of decreasing unevenness of the specimen surface, two transparencies were placed into the mould. One is placed on the bottom of the mould, and the other is placed on the side. In order to increase the consistency of the specimen, the weight of mixture and the thickness of each layer were kept the same. Prior to place next layer, the top of each layer was slightly scarified to avoid discontinuity. After moulding process, the specimens were left in the mould for 24 hours for curing. Then, the specimens were taken out from the mould and were stored in relative humidity above 95% for 14 days.

(a)   (b)

Figure 3.3 Moulds: (a) The mould of 5cm in diameter and 10 cm in height. (b) The mould of 3.8cm in diameter and 7.6cm in height.

3.4 High pressure triaxial system

The high pressure triaxial testing is the main area in this study. However, the conventional triaxial tests were also conducted to compare the stress-strain behaviours of sand at conventional and high pressures. Hence, before introducing high pressure triaxial test system, conventional triaxial test system will be described. Detailed description of high pressure triaxial test system is given in following sections.

3.4.1 Conventional triaxial testing system

The conventional triaxial test system used in this project is shown in figure 3.4. The conventional apparatus and control system schematic diagram are also illustrated in figure 3.5 and figure 3.6 respectively. The system is based on the classic Bishop & Wesley-type stress path triaxial cell, and the GDS pressure/volume controllers. Three of these pressure controllers link the computer to the test cell as follows:

- one for axial stress and axial displacement control.

- one for cell pressure control.

- one for setting back pressure and measuring volume change.

Figure 3.4 The conventional triaxial test system

(a)
Proving ring, reacting against fixed beam

Piston

Top cap
O-ring seal
Rubber membrane
Soil sample

Perspex cell filled with water

Pedestal

Pressurized cell water

Cell pressure transducer

Pore water pressure transducer

Volume change transducer

Motor drive

Pore water into or out of sample

(b)
Electrical connections to load cell

Axial screw adjustment

Displacement transducer or dial guage

Load cell

Perspex cylinder

Sample

Cell pressure

Bellofram seal

Hollow ram linking Bellofram pistons
Linear motion bearing
Crosshead for displacement measurement
Drainage and pore-pressure lead

Bellofram seal

Pressure chamber

Loading pressure

Base

Figure 3.5 Conventional triaxial test apparatus: (a) schematic; (b) hydraulically operated cell (after Powrie 2004)

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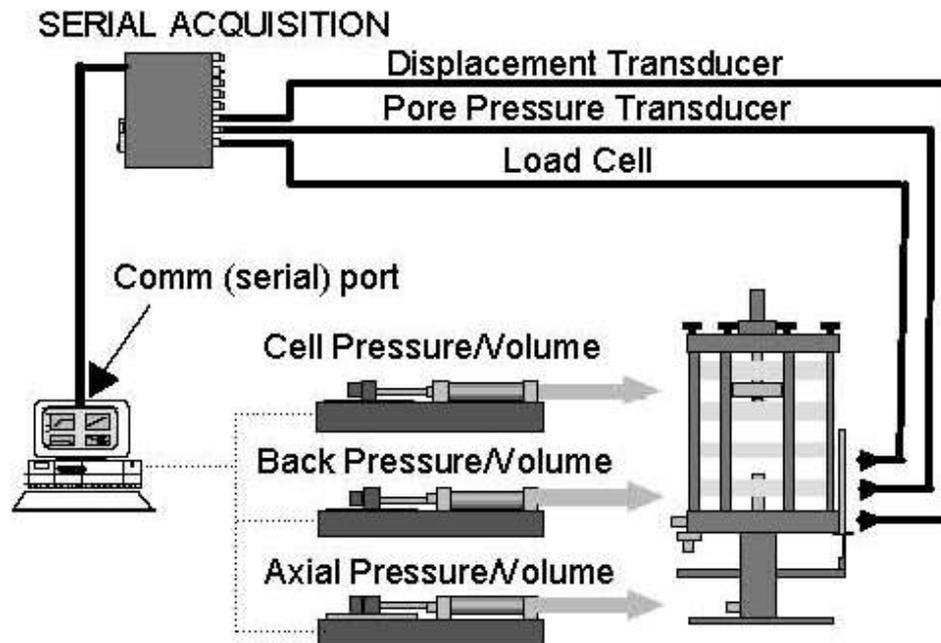
Figure 3.6 The complete triaxial test system (up to 1.7 MPa) (after Hooker 2002)

This conventional system can only provide confining pressure up to 1.7MPa and chamber pressure up to 3MPa. Its capacity is not enough to investigate the behaviour of cemented specimens or dense specimens which diameter are larger than 50mm. Hence the high pressure triaxial system is essential in this project.

3.4.2 High pressure triaxial testing system

In Nottingham Centre of Geomechanics (NCG), there is a GDS high pressure triaxial cell which is an advanced design for high quality high pressure triaxial testing. In order to apply high confining pressure and provide higher load range, the cell needs to be designed to resist high pressure and the axial load providing system also need to provide higher force. Controlling and measuring system have to be modified as well. The control system is shown in figure 3.7. The cell and back pressure/volume controllers are connected to the triaxial cell via steel pipes and linked to the computer by cables. The axial pressure/volume controller, load cell, pore pressure transducer and displacement transducer connected to the loading frame are connected to the serial acquisition. The serial acquisition is connected to computer via Comm port for

transferring data.

The overall setup of high pressure triaxial apparatus is shown in figure 3.8. Detailed description of the high-pressure triaxial cell, the loading frame, and the advanced digital pressure/volume controller is given in following section.
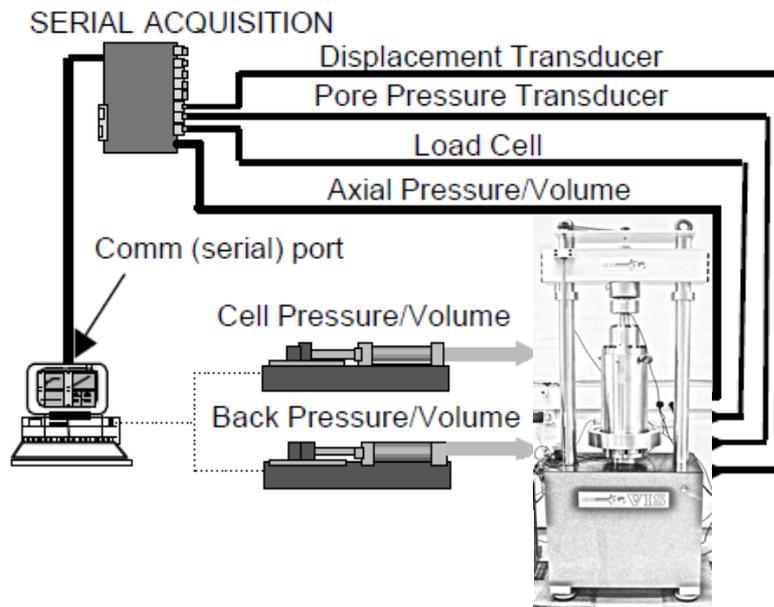
Figure 3.7 Schematic diagram of the high pressure apparatus (modified after GDS 2003)
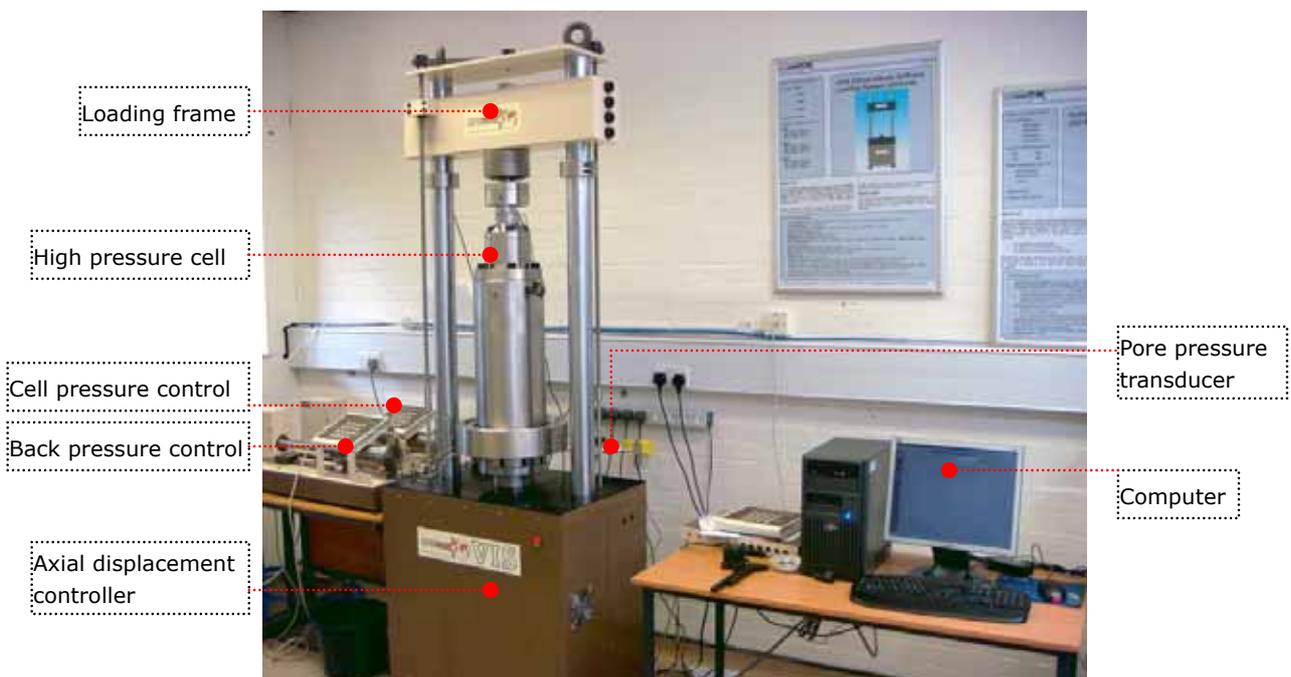
Figure 3.8 Overall setup of high pressure triaxial system.

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3.4.3. High Pressure Triaxial Cell

This GDS high pressure triaxial cell is different from conventional triaxial cell in two major aspects. One is that high pressure triaxial cell can provide confining pressure up to 64 MPa. The other is that high pressure triaxial cell is made by metal, so alteration of the specimen does not be observed during the test. The high-pressure triaxial cell and its cross section are shown in figure 3.9. The high-pressure triaxial cell is assembled with seven parts. These are cell top, cell base, pedestal, topcap, clamping ring, retaining ring and two load cell spacers. The cell top includes the main chamber, balanced ram assembly and the ram slip coupling and the clamp ring, which could be split into three parts and retaining ring are used to combine the cell top and cell base. The cell base and its connections are shown in figure 3.10, around the outside of the cell base, swagelok connectors are built for steel pressure pipe connections.
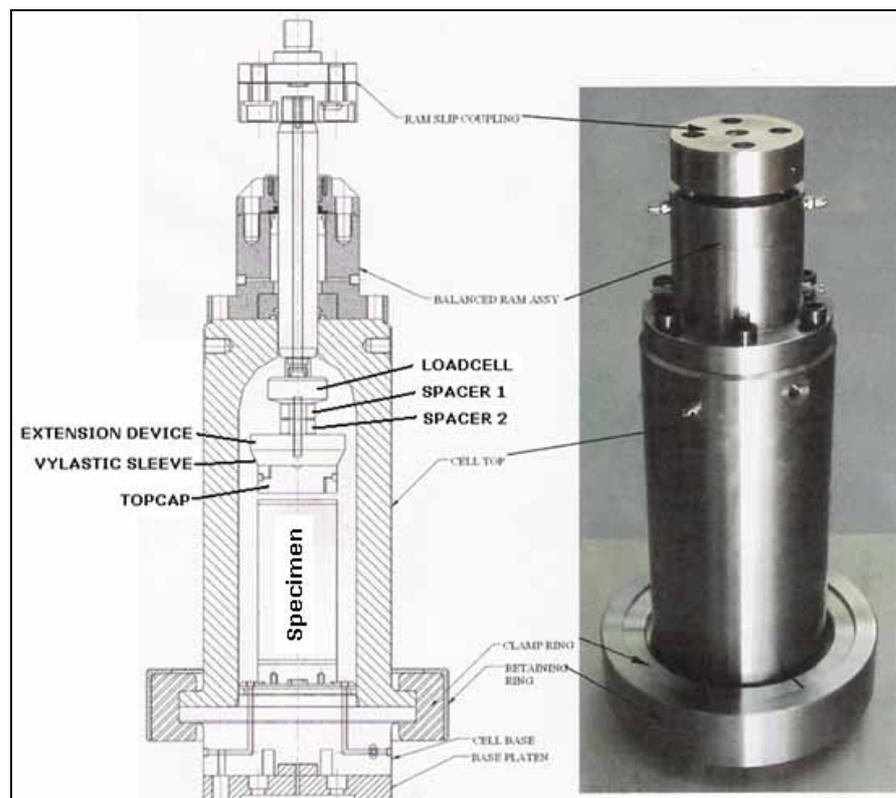
Figure 3.9 Cross-section of high pressure triaxial cell (after GDS 2003).
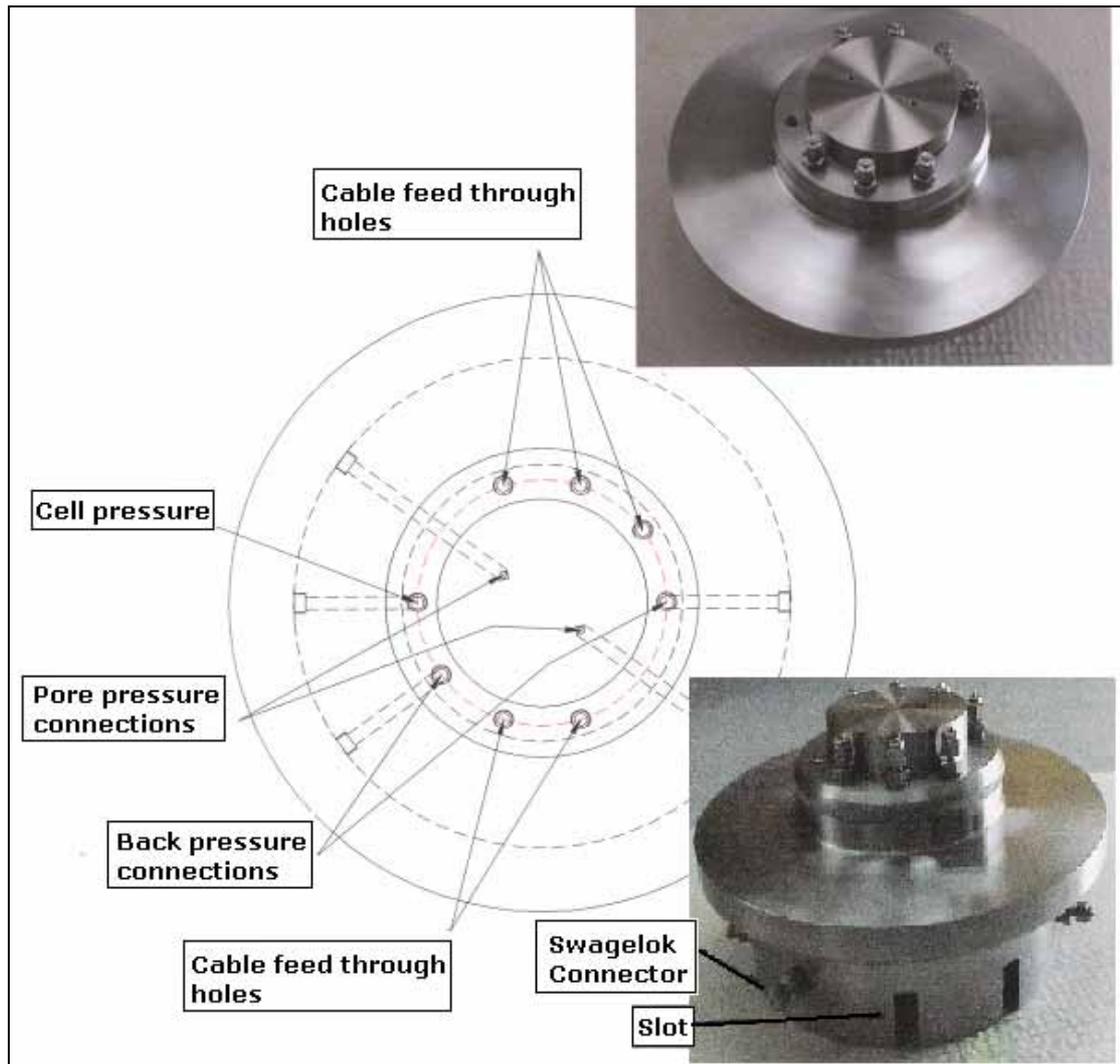
Figure 3.10 Cell base connections (after GDS 2003)

3.4.4 Loading frame

The GDS Virtual Infinite Stiffness Loading System (GDSVIS) is used as loading system to apply the axial loads onto the triaxial cell as shown in figure 3.11. It could apply the axial load up to 400kN. For the entire loading range, both the measurement and control of platen displacement is automatically corrected so that it corresponds to the deformation that occurs between the platen and the load button of the load cell. In this way, the platen displacement is corrected for strain in the load cell and side columns, bending flexure of the cross beams, and distortion within the motorised mechanical transmission. Therefore, in terms of the test specimen, it allows the axial

loading system to appear to have infinite stiffness.
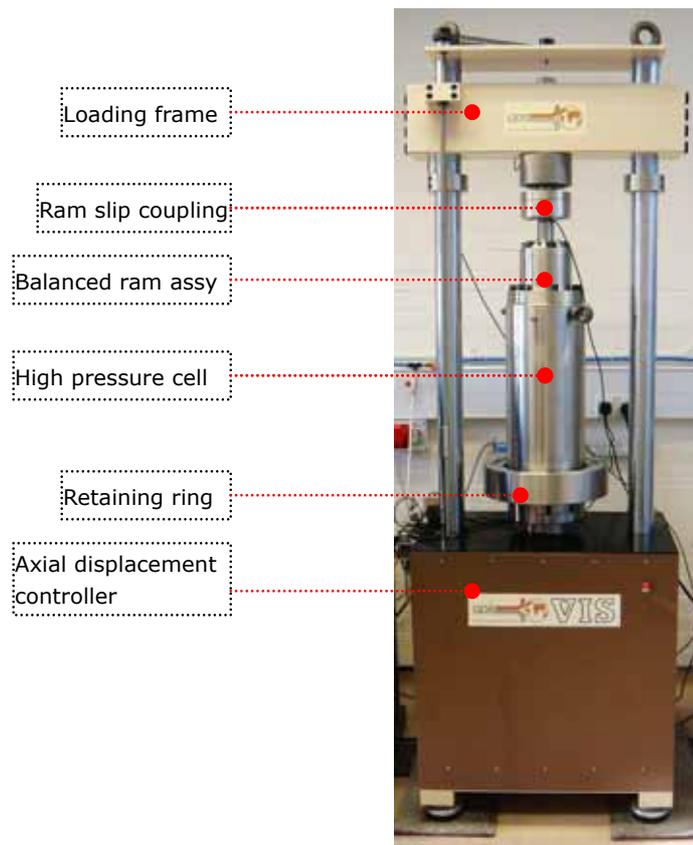
Figure 3.11 Overall setup of high pressure triaxial apparatus in the loading frame.

3.4.5 Advanced Digital Pressure/Volume Controller

Advanced Digital Pressure/Volume Controller (ADPVC) (see figure 3.12) generates, measures and logs both liquid pressure and volume change. The ADPVC has a pressure range up to 64MPa and volume ranges up to 1000cc. The device has its own computer interface and can be controlled directly from a computer. In stand alone mode it can be a constant pressure source, a volume change gauge, a pore pressure measuring system and it also can be programmed through its own control panel to ramp and cycle pressure and volume change linearly with respect to time. In computer control mode, it is a computer peripheral enabling computer-automated test control and data logging via computer interface.

Figure 3.12 Advanced Digital Pressure/Volume Controller

Liquid in a cylinder is pressurised and displaced by a piston moving in the cylinder. The piston is actuated by a ball screw turned in a captive ball nut by an electric motor and gearbox that move rectilinearly on a ball slide (see figure 3.13). Pressure is measured by an integral solid state transducer. Control algorithms are built into the onboard microprocessor to cause the controller to seek to a target pressure or step to a target volume change. Volume change is measured by counting the steps of the incremental motor (GDS 2003).
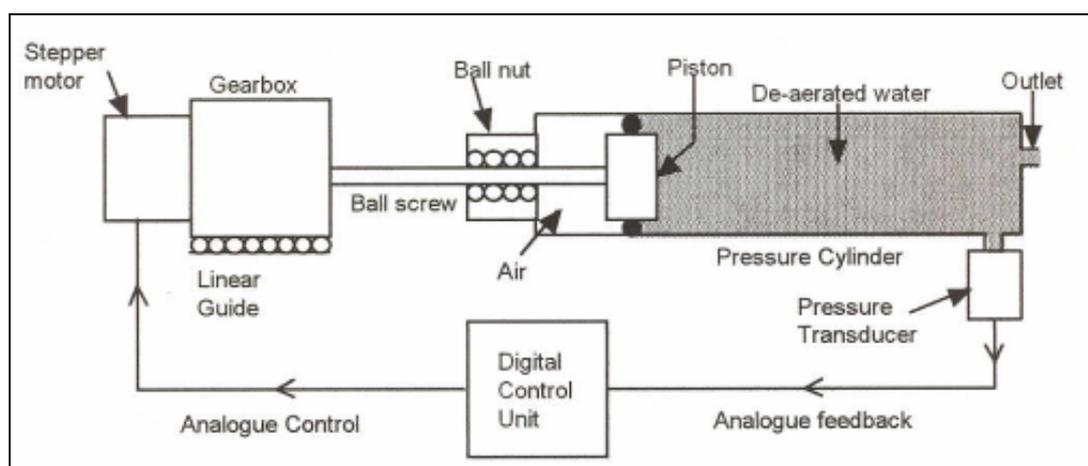
Figure 3.13 Control of Advanced Digital Pressure/Volume Controller (after GDS 2003)

3.5 Testing procedures

Isotropically consolidated undrained triaxial test procedures are similar in high pressure triaxial test system and conventional triaxial test system. Therefore, this section will mainly describe the procedures in high pressure test. Firstly, before the specimen is placed into high pressure cell, the specimen preparation will be divided into two parts. One is uncemented specimen preparation and the other is cemented specimen preparation. Secondly, the high pressure cell assembling processes are introduced. Finally, the stages of sample flushing, saturation, consolidation, and shearing stage are described.

3.5.1 Uncemented specimen preparation

Firstly, the membrane was placed on the pedestal which grease had applied on the surface, and then the O-ring stretcher was used to fix one end of membrane. The function of membrane is crucial, because it separate cell pressure and back pressure to create specific confining pressure. Therefore, careful check was taken to ensure there was no cut on its surface is indispensible before the membrane is employed. Following this, the porous disc which was saturated was placed onto the pedestal and then, the filter paper was placed on the porous stone. The filter paper and porous disc were used to prevent sand particles causing blockages inside the valves.

Secondly, the membrane was stretched in a split membrane stretcher. It has a pipe connection at its mid-height to enable a suction to be applied to the air gap between the membrane and the membrane stretcher. Next, the diameter and height of mould with membrane was measured and recorded to calculate the volume. Afterward, required Portaway sand was collected and separated into 3 layers. Then the specimen was dynamically compacted in three layers in side the membrane with hammer and the height of each layer was controlled to 1/3 of the required height of specimen. In order to increase the specimen consistency, the top of first and second layer was slightly scarified to avoid discontinuity.

Once the sand was placed into the membrane, the filter paper was placed on the sample and put the porous disc on the filter paper. Following this, the top part membrane was rolled up from the split mould and then the O-ring stretcher with O-ring seals stretched on the mould. Then, the top cap which grease applied on the surface, was carefully placed into the membrane and onto the top of the porous stone. O-ring seals were rolled from the mould to hold the top cap. The top cap was kept horizontally and its top drainage pipe tightened on the top drainage connection which was on the cell base. After the vylastic sleeve was applied onto the top cap which is shown in figure 3.16, the bottom drainage connection was closed, and then suction was applied from the top drainage valve to keep the specimen intact during the setting of high pressure cell. In the end, the mould was removed carefully to avoid disturbance of specimen.

3.5.3 Cemented specimen preparation

Before specimen preparation, cemented specimen was moulded and cured 13 days. After curing, the specimens were submerged in water for 24 hours for saturation, bringing the total curing time to 14 days. Immediately before the test, the specimen was taken out of the water and dried superficially with an absorbent cloth.

To begin with, the porous disc (bottom), filter paper (bottom), cemented specimen, filter paper (top), and porous disc (top) were placed on the pedestal in sequence. Next, after checking two membranes to ensure there was no cut on its surface, they were stretched in membrane stretcher, which was simply a tube with a pipe connection at its mid-height to enable a suction to be applied to the air gap between the membranes and the membrane stretcher. Afterward, the membranes were applied onto the specimen. Following this, 2 O-ring seals were used to tight the membranes on the pedestal. After the top cap was placed on the top of porous disc horizontally, its top drainage pipe was tightened on the top drainage connection which was on the cell base. Finally, the vylastic sleeve was applied onto the top cap, the bottom drainage

connection was closed, and then suction was applied from the top drainage connection to keep the specimen intact during the setting of high pressure cell. Figure 3.14 and figure 3.15 show the specimen before removing the split membrane stretcher and prepared cemented specimen on the cell base respectively.
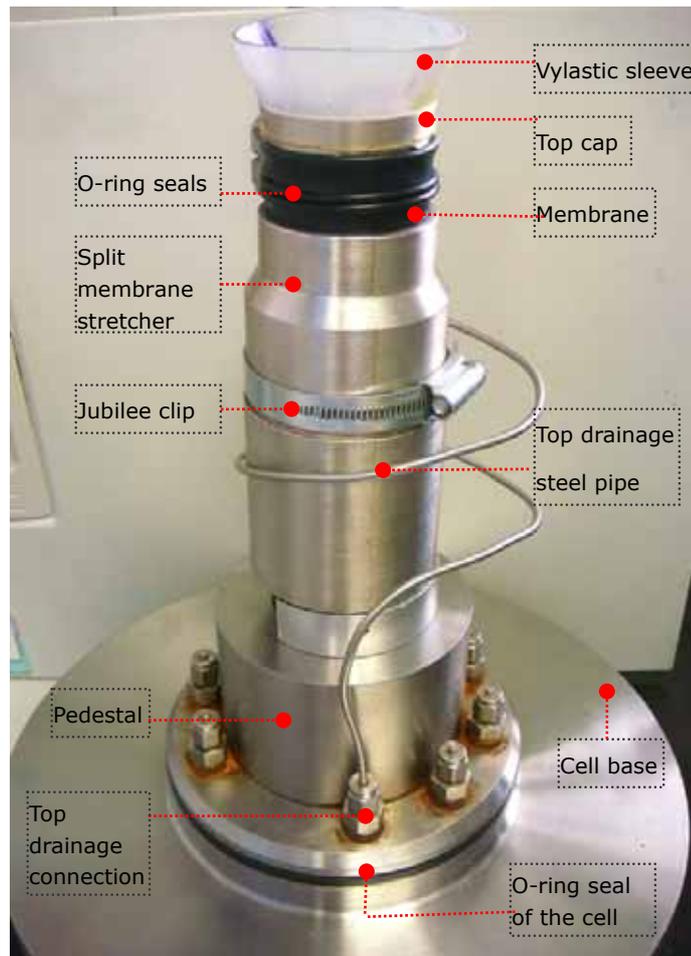
Figure 3.14 Uncemented specimen preparation before removing the split membrane stretcher

Figure 3.15 Prepared cemented specimen.

3.5.3 Setting up the high pressure cell

Before assembling the high pressure cell, there were several things which should be checked. First, base plate was cleaned and the grease applied on its O-ring seal. Next, the ram height was checked that it would not disturb the specimen during assembling the cell. Then, the direction of cell base was confirmed that the top drainage connection was in the front.

After the top cell was lifted, the cell base was moved to the proper place where was below the lifted top cell. Following this, the top cell was lowered slowly and vertically onto the cell base. Extreme care was taken to ensure no disturbance was applied on the specimen. Next, the three parts of the clamp ring were placed as shown in figure 3.16(a), and then the remaining rings as shown in figure 3.16(b) was placed over the assembled clamping ring.

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(a)   (b)

Figure 3.16 Assemble the Clamp Ring(a) and retaining ring(b) (after GDS 2003)

Due to a large mass of the cell, a trolley was designed and used to transport the high pressure cell onto the loading frame, as shown in figure 3.17 and figure 3.19. The trolley was made together with two rails on each side, so that, once the triaxial cell was loaded onto the trolley, the cell could be rolled on the rails. The cell was moved slowly and carefully to avoid disturbing the specimen or slipping out of the rails.

After placing the high pressure cell into the loading frame, two bolts were used to connect the ram to the loading frame diagonally. Then, the cable was connected to serial acquisition and the power of loading frame was turned on. Afterward, the GDS testing control software was executed to move the base plate of loading frame to lift cell from the rails of the trolley. Then, the trolley was removed from the bottom of the cell. The overall setup of high pressure triaxial apparatus in a loading frame is shown in figure 3.11. Before filling water in the cell, the steel pipes of providing cell pressure and linking between balance ram and cell were connected. When the cell was filled with water, the air valve was tightened. In the end, the confining pressure was increased to 150 kPa, then the suction applied on the specimen was released.
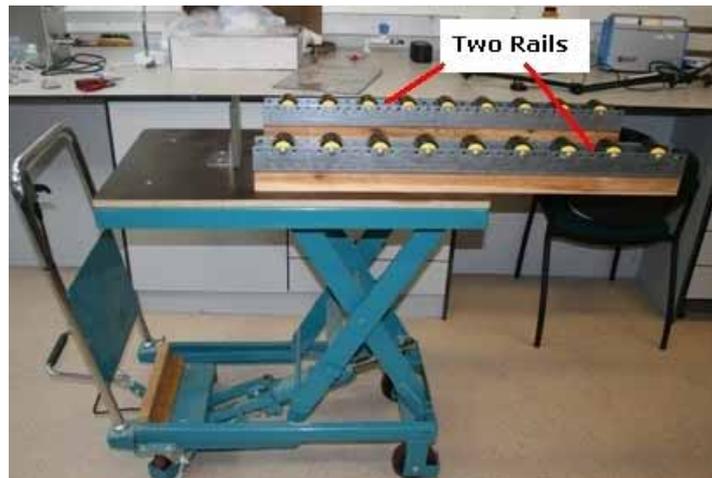
Figure 3.17 The Trolley

Figure 3.18 Fitting the triaxial cell into the loading frame

3.5.4 Procedures of triaxial test

After specimens had been prepared and placed into the triaxial cell, together with all the testing equipments were also set up, saturation and isotropical consolidation would be conducted before triaxial compression test. Detailed description of these procedures is given below.

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3.5.4.1 Saturation

All the specimens were saturated by flushing with de-aired water. For expelling the air from specimens, back pressure was applied from the bottom drainage to let the void inside filled with water. In the meanwhile, the top drainage was open to let air discharge with water. The back pressure was adjusted to 100 kPa which was lower then the cell pressure. It took at least an hour, and was longer in cemented specimen or dense specimen.

The process of saturation was followed by increasing back pressure until the B value was greater than 0.95. The B value, Skempton is pore water parameter, can be expressed by equation 3.1.

$$B = \frac{\Delta u}{\Delta \sigma_3}$$

(3.1)

Where the u = change in pore pressure and   3= change in cell pressure.

During the process, an effective confining pressure 100 kPa was maintained on the specimen and drainage was not allowed. This process was controlled by GDS software. In this project, the targets of saturation ramp were 2100 kPa in cell pressure and 2000 kPa in back pressure and took 12 hours to reach. Real time monitor was also provided by GDS software.

After completing the saturation ramp, B-check was carried out to ensure that the specimen was sufficiently saturated. Before starting the check, the current cell pressure and pore pressure were recorded. Then, the cell pressure was increased to the specified value, then at the same time, the back pressure remained the same.   In this project, the target cell pressure is set to 2200kPa. When the cell pressure reached the target, the pore pressure was recorded to calculate the B value according to equation (3.1). In this project, the pore pressure was increased greater than 2095kPa and the B values reached 0.95. During the process, the top drainage valve and back

pressure valve are closed. Following this, the cell pressure and back pressure were adjusted to 2100kPa and 2000kPa, respectively.

3.5.4.2 Isotropic consolidation

Isotropic consolidation was carried out to ensure the specimen was at the state of the required boundary stress conditions and had no excess pore pore pressure. Hence, the process was followed by increasing cell pressure to specific confining pressure. For example, if the backpressure was 2000kPa and specific confining pressure was 4000 kPa, so the target cell pressure was increased to 6000kPa. The consolidation was continued until there was no further volume change and all the excess pore pressure dissipated. During the procedure, the top drainage valve was open and the back pressure valve closed. This process was controlled and monitored by GDS software. The typical back volume against time during the process of consolidation is shown in figure 3.19.
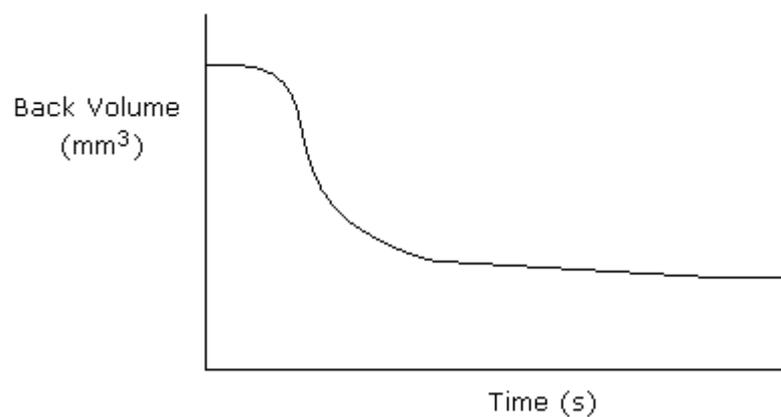
Figure 3.19 Typical back volume against time during the process of consolidation

3.5.4.3 Undrained Shearing

During the undrained shearing test, a compression took place by applying an axial load onto the specimen. The cell pressure was maintained at a specific value while the specimen was sheared at a constant rate of axial deformation until failure occurred. Drainage was not allowed through the entire test, so that the volume of the specimen

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was remained constant during compression. The top drainage valve and back pressure valve were closed. The measurement of changes in pore pressure was done at the base of the specimen by a pore pressure transducer. The rate of axial deformation was applied slowly to the specimen at the rate of 0.2 mm per minute or strain rate 0.2% per minute in specimen of 100mm in height or 0.26% per minute in specimen of 76 mm in height.

During all the tests, parameters such as axial strain (ε_a), pore pressure (u), axial stress (σ_1) and radial stress (σ_3) were recorded and used for plotting results.

3.6 Experimental Program

In order to understand the effect of cementation, effective confining pressure and drainage on the behaviour of the cemented granular material, a detailed experimental program was planed. Portland cement in amounts of 5%,10% and 15% by weight of dry soil was used to prepare artificially cemented specimens. All the cemented specimens were prepared at dry unit weights of 17.4kN/m^3. They moulded and cured as described in Section 3.3. The triaxial tests were carried out at three different effective confining pressures of 500,1000,4000 kPa. A summary of all the tests is given in table 3.6

Table 3.6 Summary of test conducted

NO	Cement Content	σ3'(Mpa)	Sepcimen size	Comment
1	0%	0.5	D=50mm, H=100mm	Dr=47%
2	0%	0.5	D=38mm, H=76mm	Dr=81%
3	0%	1	D=50mm, H=100mm	Dr=82%
4	0%	4	D=50mm, H=100mm	Dr=81%
5	5%	0.5	D=38mm, H=76mm	
6	5%	4	D=50mm, H=100mm	
7	10%	4	D=50mm, H=100mm	
8	15%	4	D=50mm, H=100mm	

3.7 Summary

In this chapter, the material properties, high pressure triaxial testing system, and detailed experimental procedures including moulding and curing of cemented specimens, and isotropically consolidated undrained triaxial test procedures have been introduced. The following chapters describe the results and the associated interpretations based on the outcome from experimental program.

Chapter 4 Experimental results and discussion

4.1 Introduction

A total of 8 consolidated undrained triaxial tests on uncemented and artificially cemented sand have been carried out. Test results are presented as including stress–strain curves, stress paths, excessive pore pressure versus strain and effects of density, cementation, and drainage effects are discussed. The test results from this research have been compared with the results of a parallel study, which used the same base soil but conducted in isotropically consolidated drained condition.

The test results have been analysed using σ_1', σ_3', ε_1, ε_v, u, q and p', where σ_1' and σ_3' are, respectively, the effective axial stress and effective confining stresses on a cylindrical sample, ε_1 and ε_v are respectively the axial and volumetric strains, u is excessive pore water pressure, q and p' are respectively deviatoric and mean effective stresses. q and p' are defined as:

$$q = \sigma_1' - \sigma_3' \qquad\qquad (4.1)$$

$$p' = (\sigma_1' + 2\sigma_3')/3 \qquad\qquad (4.2)$$

4.2 Undrained behaviour of uncemented sand

This section will focus on the undrained behaviour of uncemented sand. There are two parts: effect of density and effect of confining pressure. The detailed description is given below.

4.2.1 Effect of density

The results on two different relative densities of specimens in undrained triaxial test are shown in figure 4.1. Deviatoric stress (q), excess pore water pressure (Δu),
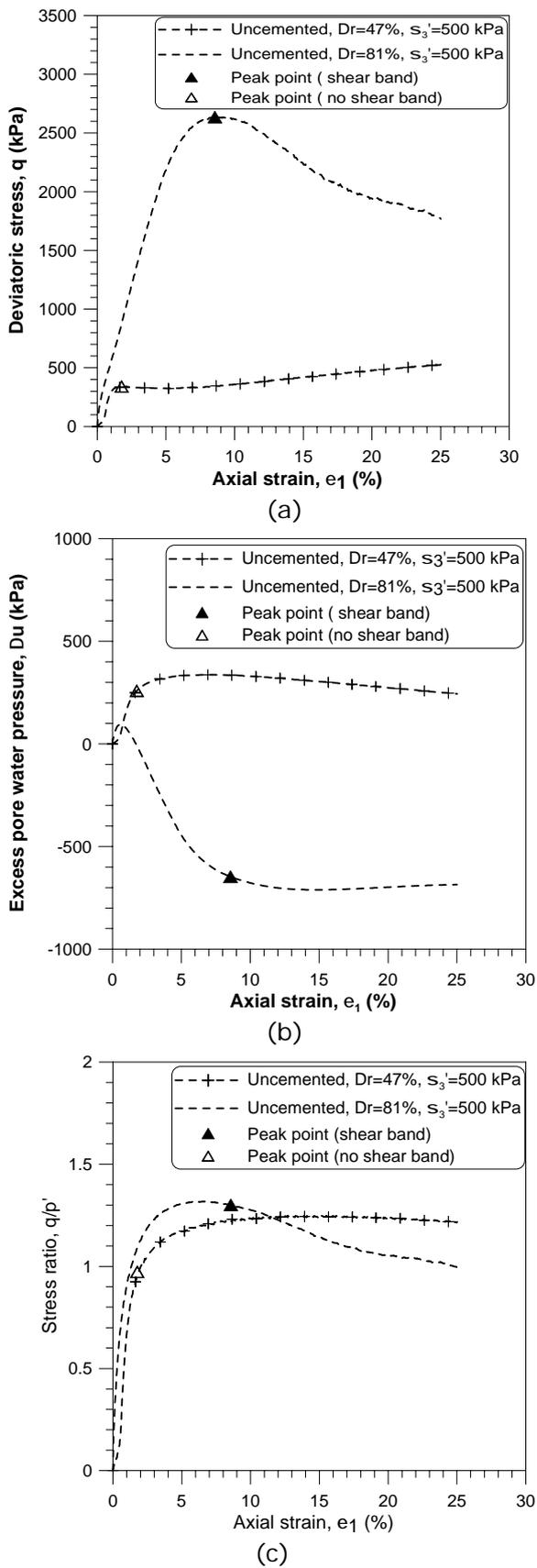
Figure 4.1 Results of CIU triaxial test on medium dense and dense sand: (a) stress-strain behaviour; (b) u versus ₁ curves; (c) q/p′ versus ₁ curves; (d) effective stress paths.
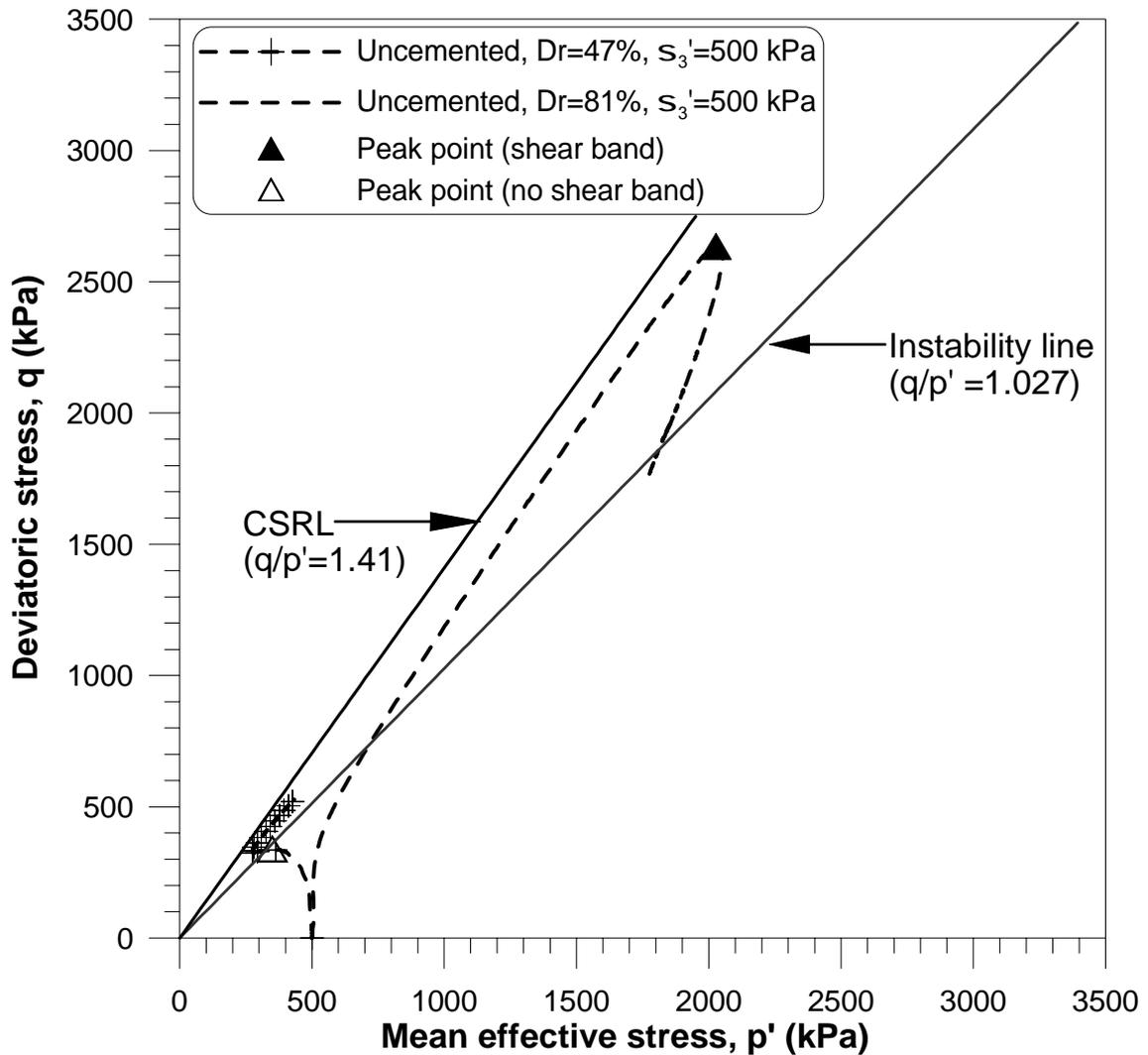
(d)

Figure 4.1 Continued.

and stress ratio (q/p') plotted against axial strain ($_1$), and the effective stress paths

are shown as well.

In figure 4.1 (a), the initial slopes of stress-strain curve of dense specimen is steeper

than medium dense as expected. In medium dense specimen, shows the deviatoric

stress increasing to a peak, followed by a decrease, but it rises again to its maximum

value at failure. After the first peak is reached, the drop in the deviatoric stress is

caused by the tendency of the specimen to compress volumetrically, causing the pore

pressure to rise, thus decreasing the effective stress. The test of dense sample shows

the different behaviour. It increases to a peak, then following by a decrease with increasing strain to the end of test.

The excess pore water pressures shown in figure 4.1 (b) indicate that dense and medium dense specimens show different tendencies. The generated pore water pressure of the medium dense specimen is positive, which displays tendency for overall net compressive volumetric behaviour. At larger strain, the effective confining pressure has declined sufficiently to allow the specimen to show volumetrically dilatant tendencies, causing the pore pressure to drop. However, the positive excess pore water pressure in dense specimen is found at beginning of shearing with small magnitude, followed by a rapid decrease to about relative large negative value at about 10% axial strain, and then keeps steady to the end.

In figure 4.1 (c), the stress ratios against axial strain are presented. The dense specimen has higher value of maximum stress ratio then medium dense specimen. However, the ultimate stress ratio of medium dense specimen is higher than dense specimen. Both of them may approach a constant value at high strain levels.

In figure 4.1 (d), the CIU triaxial test in medium dense specimen indicates an effective stress path that increases upward, followed moving slightly to the left. Then, the specimen continues to strengthen, and it causes the stress path to continue upward with increasing deviatoric stress and effective mean stress. In the end, it reaches the maximum deviatoric stress. Lade (1992) indicates that the stress state at instability would occur corresponds to the top of the current yield surface which is slightly before, but very close to the top of the undrained effective stress path. The location of instability line is shown in figure 4.2. Therefore, the instability line can be determined experimentally by a line connecting the peaks of a series of effective stress paths obtained from undrained test. In figure 4.1 (d), the instability line is plotted by a line connecting the peak and the origin. However, more results are needed to determine the instability line. The stress path of dense specimen, it increases upward at the beginning, and then it inclines to follow the constant stress ratio line (CSRL) with the

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slop of 1.323 until the maximum deviatoric stress reached. Following this, it turns to opposite direction to the end.
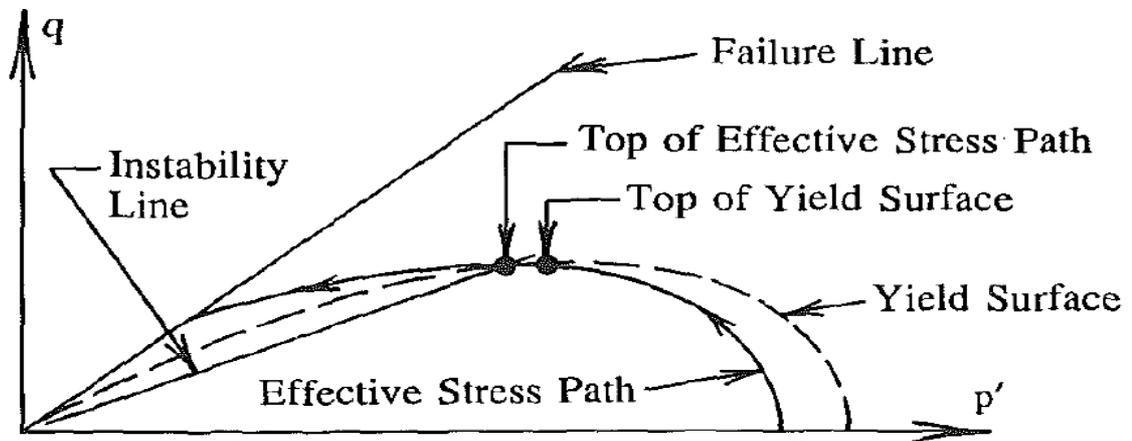
Figure 4.2 Location of instability line for loose sand (after Lade 1992)

4.2.2 Effect of confining pressure

The results on three different confining pressure specimens in undrained triaxial test are shown in figure 4.3. Deviatoric stress (q), excess pore water pressure (Δu), and stress ratio (q/p') plotted against axial strain (ϵ_1), and the effective stress paths are shown as well.

As shown in figure 4.3 (a), as the effective confining pressures increase the peak deviator stresses increase. Table 4.1 shows peak deviatoric stresses, peak effective mean stresses and peak stress ratios of three confining pressure. For the confining pressure of 500kPa the maximum deviatoric stress is 2635kPa, and for the confining pressures of 1000kPa and 4000kPa the maximum deviatoric stresses are 5529kPa and 6597kPa. The confining pressure increase twice from 500kPa to 1000kPa and the deviatoric stress increase about 2.1 times from 2635kPa to 5529kPa. However, the confining pressure increase four times from 1000kPa to 4000kPa and the deviatoric stress increase about 1.19 times from 5529kPa to 6597kPa. It appears that the effect

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(a)

(b)

(c)
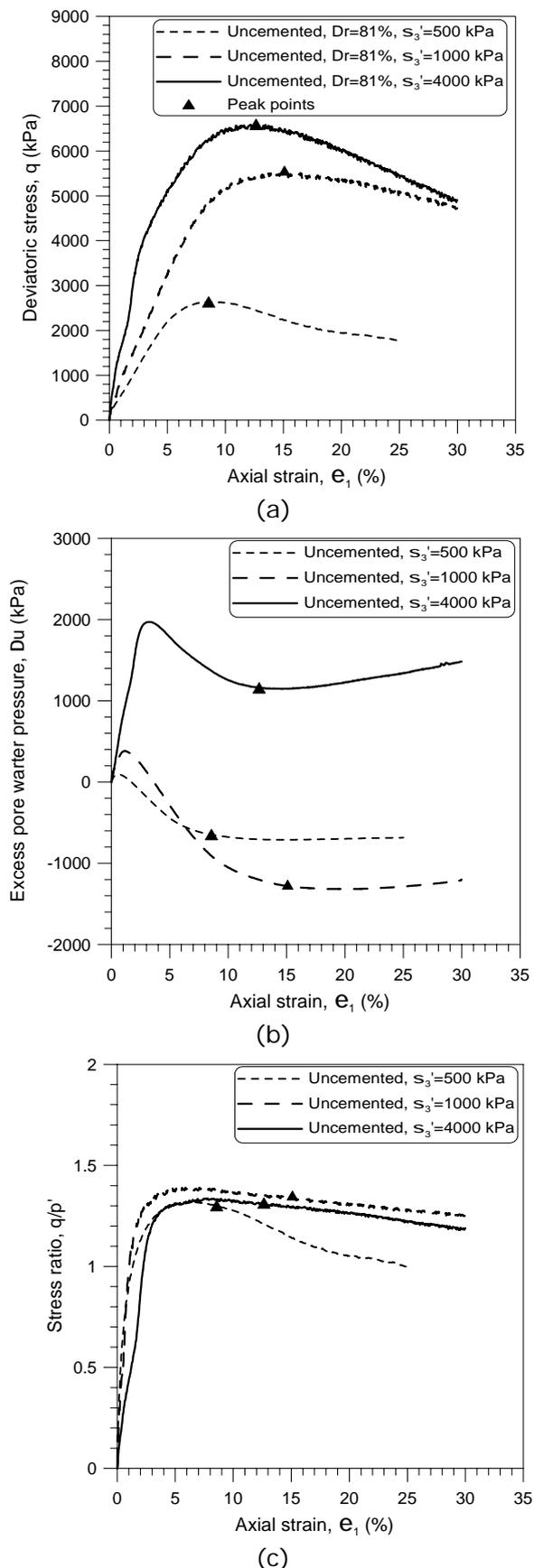
Figure 4.3 Results of CIU triaxial test in uncemented specimens at different confining pressures: (a) stress-strain behaviour; (b) u versus $_1$ curves; (c) q/p' versus $_1$ curves; (d) effective stress paths.
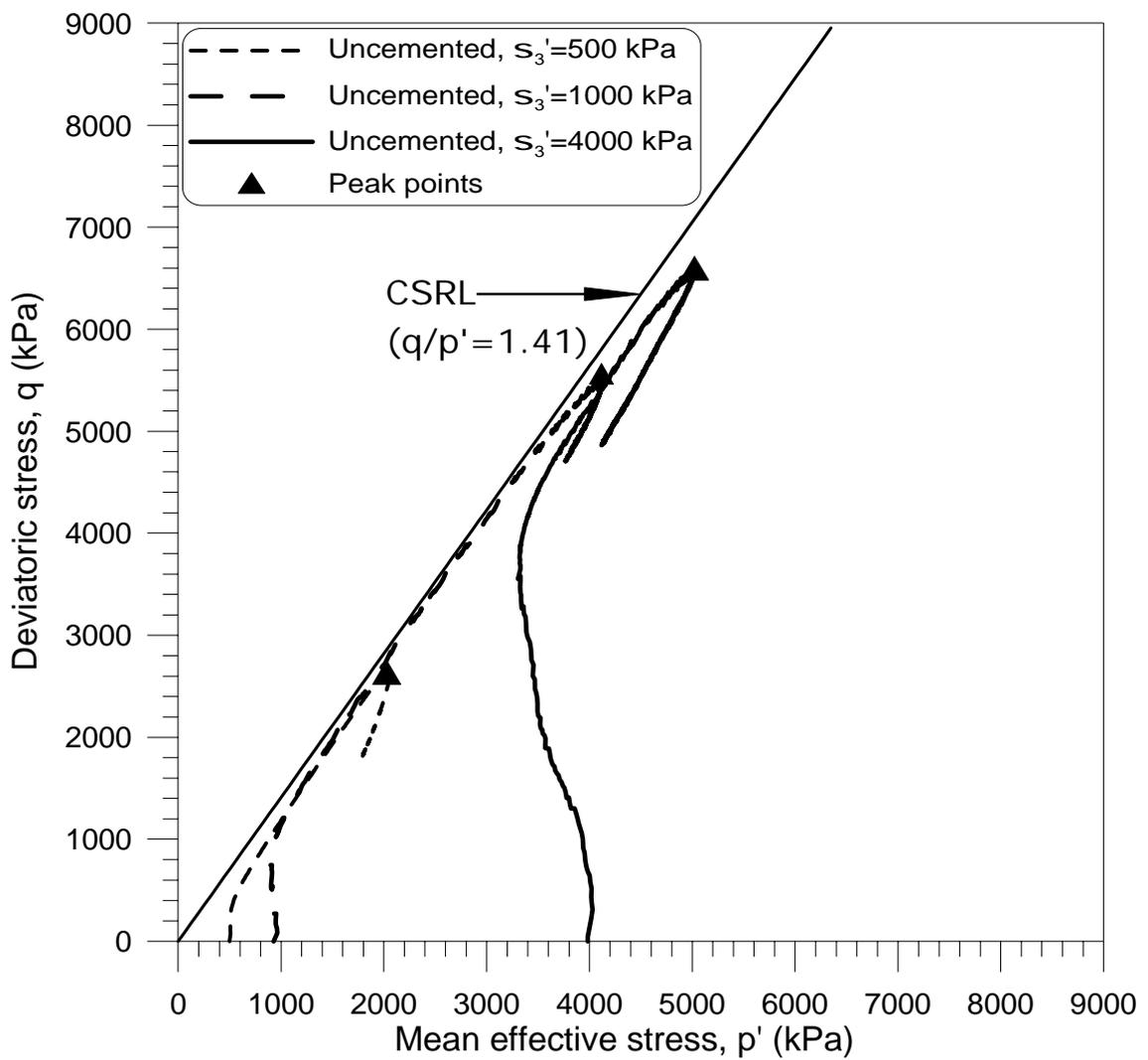
(d)

Figure 4.3 Continued.

Table 4.1 Peak deviatoric stresses, Peak effective mean stresses and peak stress ratios of three confining pressure.

Effective confining pressure (kPa)	Peak Deviator stress (kPa)	Peak effective mean stress (kPa)	Peak stress ratio
500	2635	2026	1.300
1000	5529	4117	1.343
4000	6600	5022	1.314

of confining pressure to deviatoric stress declines with the increase of confining pressure.

In figure 4.3 (b), the induced excess pore pressures of three different confining pressures show the similar tendencies. The positive excess pore water pressure is generated at the beginning, following with a decrease and then keeps steady to the end. However, higher confining pressure induces higher positive excess pore water pressure at beginning. The higher maximum positive excess pore pressure occurs at larger confining pressure.

The stress ratios against axial strains are plotted in figure 4.3 (c). The maximum stress ratios in three confining pressures of 500kPa, 1000kPa and 4000kPa are 1.3, 1.343 and 1.314 respectively. They seem close.

As figure 4.3 (d) shown, the stress paths in three confining pressure are similar. The stress paths increase upward at the beginning, and then they incline to follow the constant stress ratio line (CSRL) until the maximum deviatoric stress reached. Following this, it turns to opposite direction to the end. The CSRL can be determined by fit the three maximum deviatoric stresses from the origin.

4.3 Undrained behaviour of cemented sand

This section will focus on the undrained behaviour of cemented sand. There are two parts: effect of cement content in conventional and high conventional pressure and effect of confining pressure. The detailed description is given below.

4.3.1 Effect of cement content

The results of uncemented and 5% cement content under conventional confining pressure specimens in undrained triaxial test are shown in figure 4.4. Deviatoric stress (q), excess pore water pressure (u), and stress ratio (q/p') plotted against

axial strain ($_1$), and the effective stress paths are shown as well.

As shown in figure 4.4 (a), the deviatoric stress of the cemented specimen is higher than uncemented specimen in conventional confining pressure. The initial slope of stress-strain curve of cemented specimen is steeper than uncemented specimen and the maximum deviatoric stress of cemented specimen occurs earlier than uncemented specimen.

In figure 4.4 (b), the excess pore pressures of uncemented and cemented specimens show similar tendencies in CIU tests. The positive excess pore water pressure is induced at small axial strain (< 1 %) and small magnitude, then following with a significant decrease and then keeps steady to the end.

The stress ratios against axial strains are plotted in figure 4.4 (c). The maximum stress ratios in uncemented and cemented specimens are about 1.3, 1.7 respectively. The maximum stress ratio of cemented specimen occurs earlier than uncemented sample. It appears that the cementing agent shows obvious influence in stress ratio.

In figure 4.4 (d), the stress paths of 5% cement content specimen and uncemented specimen show different routes. The stress path of uncemented specimen is described in Section 4.2.2. The stress path of cemented specimen increase upward at the beginning, and then they incline to follow the constant stress ratio line (CSRL) until the maximum deviatoric stress reached. Following this, it turns to opposite direction to the end. If the CSRL of cemented specimen assumes that it has the same slop with the CSRL of uncemented specimen, the CSRL of cemented specimen can be determined by shifting the CSRL of uncemented specimen to fit the maximum deviatoric stress of cemented specimen. Therefore, the CSRL of 5% cement content specimen can be expressed as:

$$q = 1.41 p' + 600 \qquad\qquad (4.3)$$

(a)

(b)

(c)

Figure 4.4 Results of CIU triaxial test in of different cement content at conventional confining pressure(500 kPa): (a) stress-strain behaviour; (b) u versus $_1$ curves; (c) q/p′ versus $_1$ curves; (d) effective stress paths.
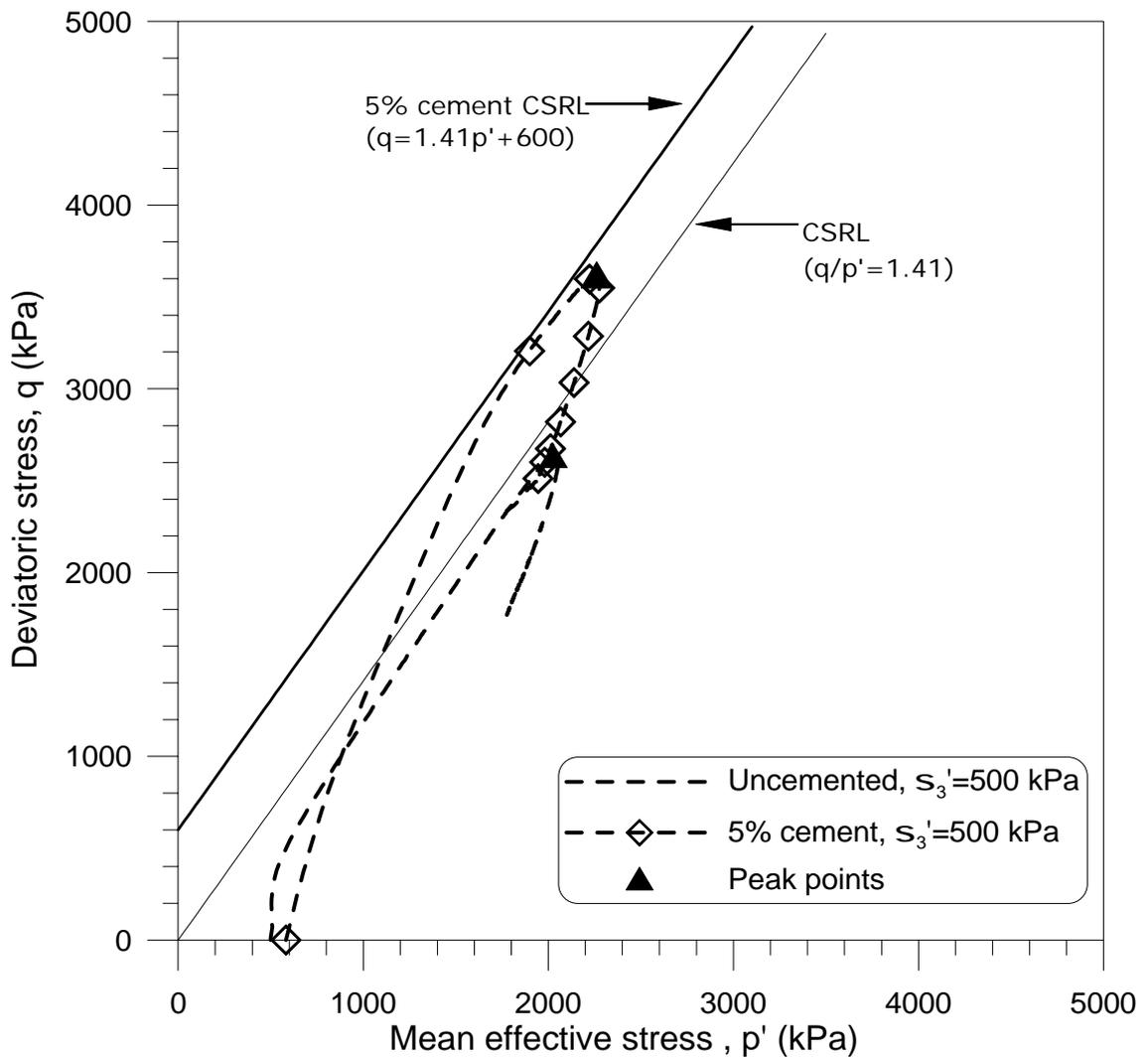
Figure 4.4 Continued.

The results of uncemented, 5%, 10%, and 15% cement content under high confining pressure (4MPa) specimens in undrained triaxial test are shown in figure 4.5. Deviatoric stress (q), excess pore water pressure (Δu), and stress ratio (q/p') plotted against axial strain (ε_1), and the effective stress paths are shown as well.

As shown in figure 4.5 (a), the higher cement content leads to higher maximum deviatoric stress. The peak deviatoric stresses of uncemented, 5%, 10%, and 15% occurs in 12.6%, 9.0%, 6.4%, and 4.6% respectively. It appears that higher cement content let the peak deviatoric stress show earlier and the initial slope of stress-strain

curve be steeper. The behaviours of uncemented and 5% cemented specimen are different from 10% and 15% cemented specimen. 10% and 15% cemented specimens show clear peaks and their maximum deviatoric stresses are 15975kPa and 17384kPa, respectively. However, the peak deviatoric stress of uncemented and 5% cemented specimens is not so obvious and their maximum deviatoric stresses are 6597kPa and 7661kPa, respectively. It appears that the deviatoric stress in high confining pressure may be sensitive when cement content between 5% and 10%. Anyway, it needs more results to confirm.

In figure 4.5 (b), the positive excess pore water pressure is induced at small axial strain which is about 1% in cemented specimen and about 3% in uncemented specimen. The generated maximum excess pore water pressure is larger in higher cement content. The excess pore pressures show similar tendencies to deviatoric stresses shown in figure 4.5 (a). The behaviours of uncemented and 5% cemented specimen are different from 10% and 15% cemented specimen. The excess pore water pressure of uncemented and 5% cemented keep positive during the test. However, the induced excess pore water pressure in 10% and 15% cemented specimen are positive at beginning, and then declined sharply to the negative values.

As figure 4.5 (c) shown, the specimen of higher cement content has higher maximum stress ratios. It appears that stress ratios of uncemented and 5% cemented specimens show different tendency from 10% and 15% cemented specimen. The stress ratios of 10% and 15% cemented specimen have noticeable peaks.

In figure 4.5 (d), the specimen of different cement contents show similar type of stress paths which increase upward at the beginning, and then they incline to follow the constant stress ratio line (CSRL) until the maximum deviatoric stress reached. Following this, it turns to opposite direction to the end. If the CSRL of cemented specimen assumes that it has the same slop with the CSRL of uncemented specimen, the CSRL of cemented specimen can be determined by shifting the CSRL of uncemented specimen to fit the maximum deviatoric stress of cemented specimen.

(a)

(b)
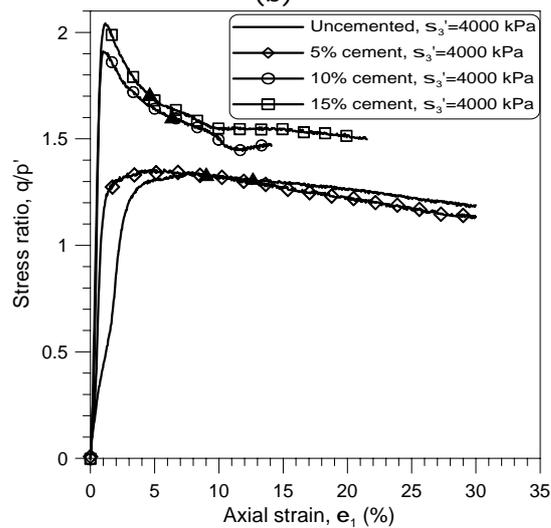
(c)
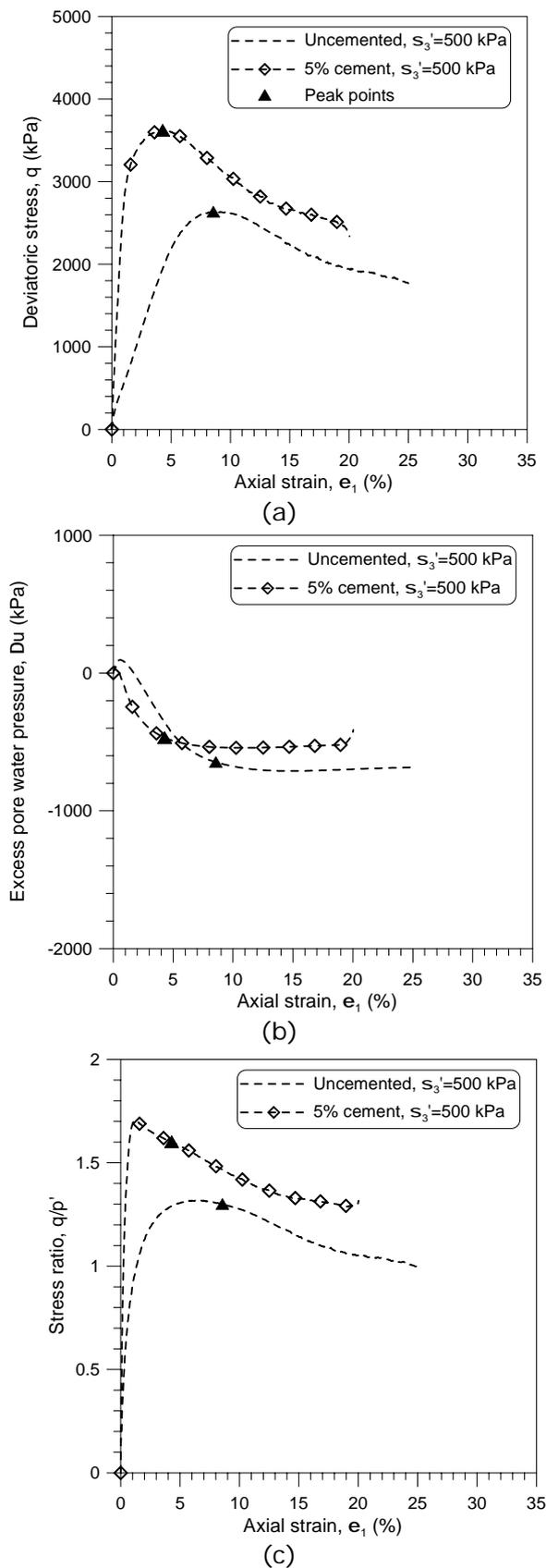
Figure 4.5 Results of CIU triaxial test in of different cement content at high confining pressure(4000 kPa): (a) stress-strain behaviour; (b) Δu versus ε_1 curves; (c) q/p' versus ε_1 curves; (d) effective stress paths.
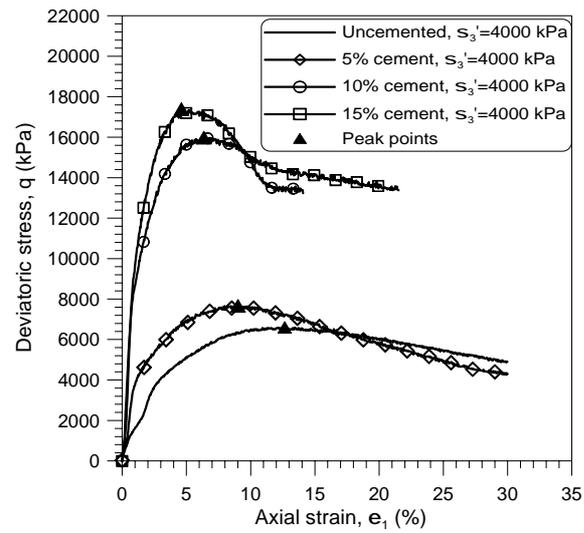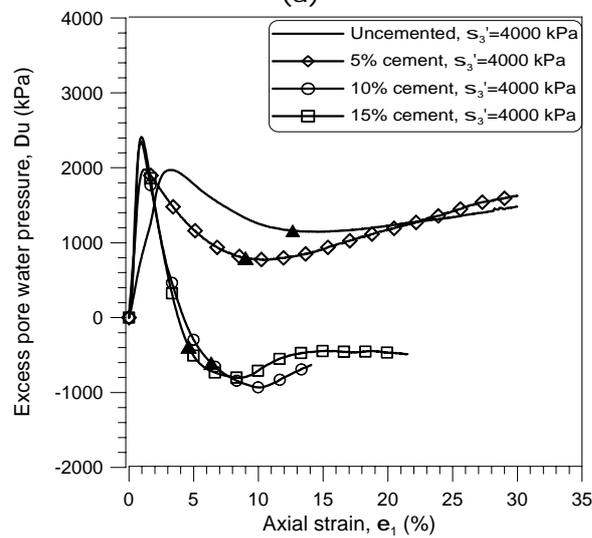
Figure 4.5 Continued.

Therefore, the CSRL of 10% and 15% cement content specimen can be expressed as:

$$q=1.41p'+2850 \qquad\qquad (4.4)$$

$$q=1.323p'+3950 \qquad\qquad (4.5)$$

In figure 4.6 the cement contents and cohesion intercepts of equation (4.3), (4.4) and (4.5) are plotted. It appears that cohesion intercepts increase to power 1.77 of cement content. It means that non-linear relationship between the cohesion intercept of CSRL.
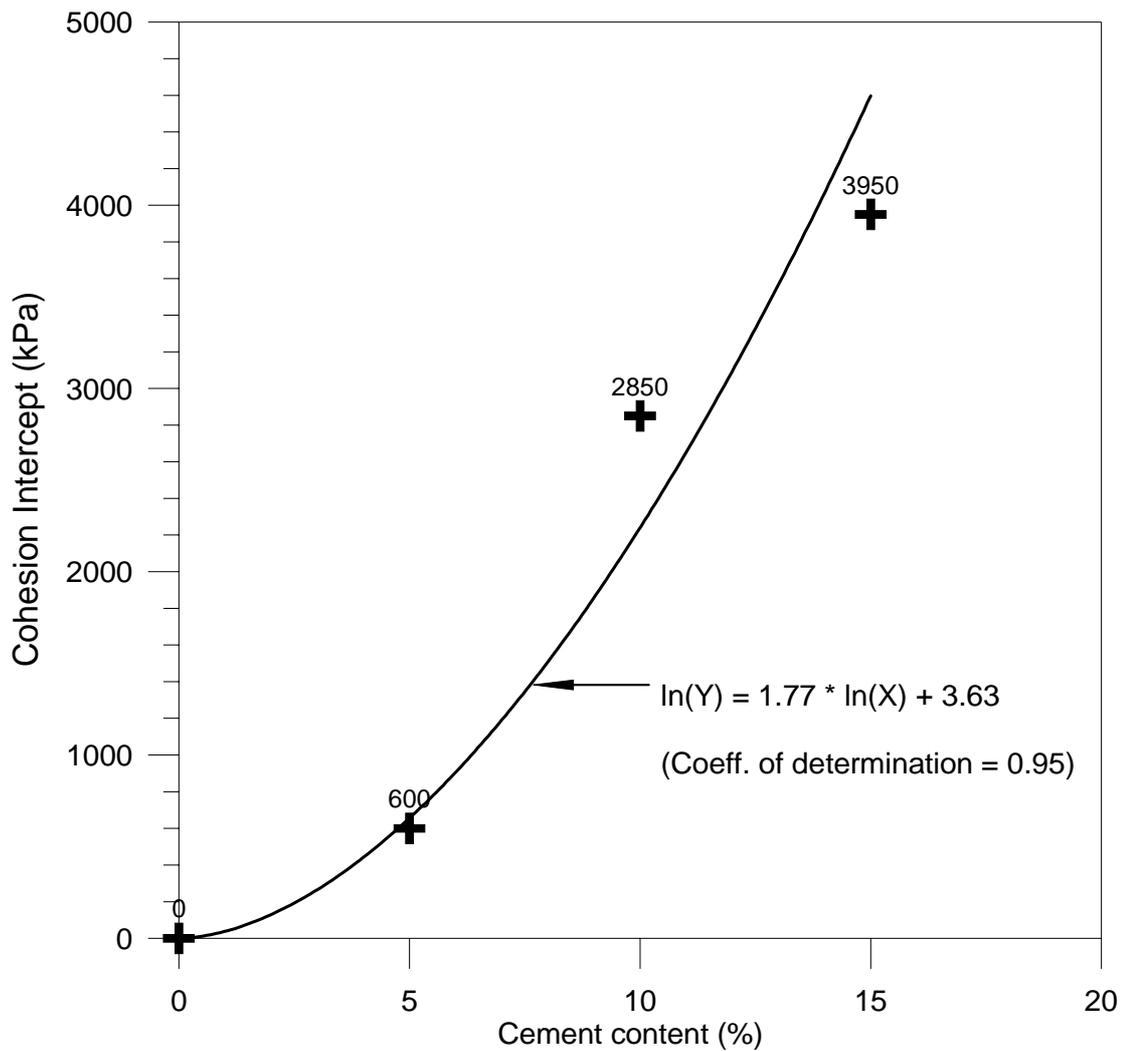
Figure 4.6 The relationship between cement content and intercept

4.3.2 Effect of confining pressure

The results of 5% cement content under conventional confining pressure (500kPa) and high confining pressure (4000kPa) specimens in CIU tests are shown in figure 4.7. Deviatoric stress (q), excess pore water pressure (Δu), and stress ratio (q/p') plotted against axial strain (ε_1), and the effective stress paths are shown as well.

In figure 4.7 (a), the peak deviatoric stress in high confining pressure and conventional confining pressure occurs in 9.00% and 4.29% axial strain respectively. It appears that high confining pressure let peak stress occurs later than conventional confining pressure. In other words, the high confining pressure seems to let behaviour

(a)

(b)

(c)
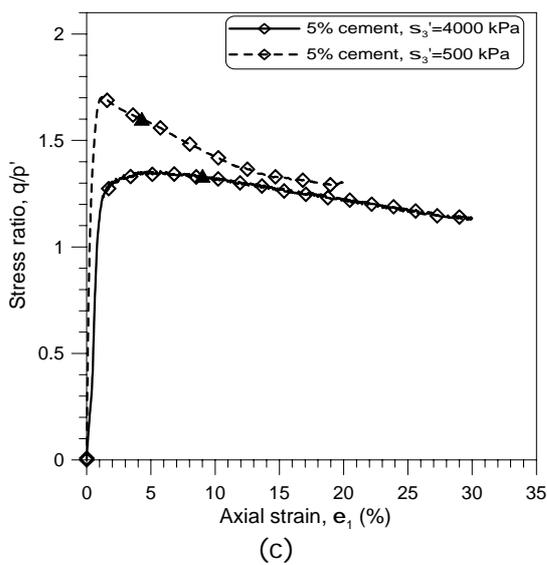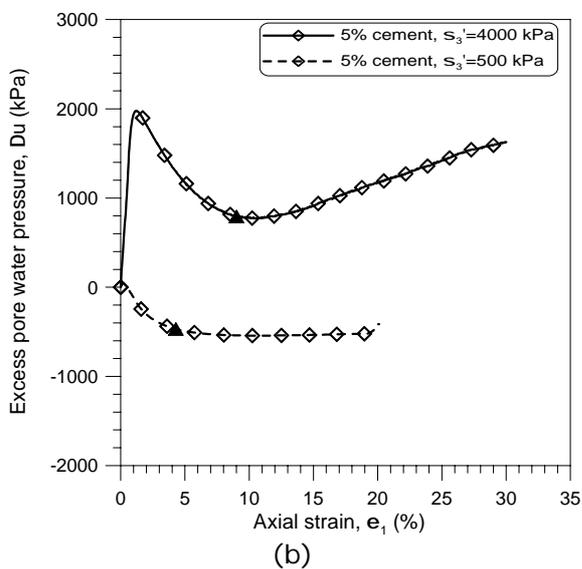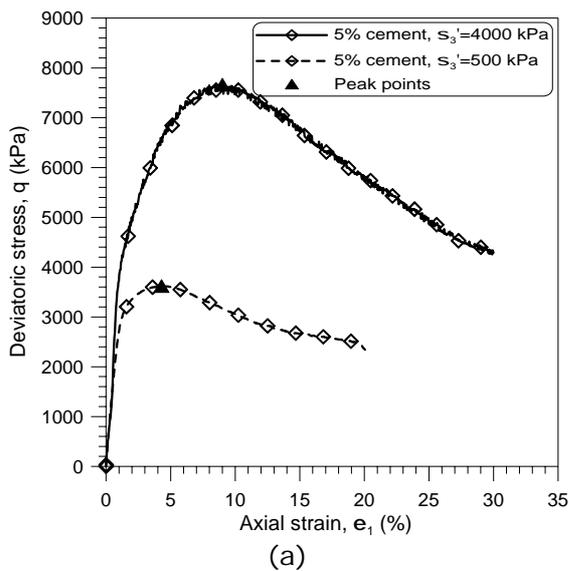
Figure 4.7 Results of CIU triaxial test in different confining pressure : (a) stress-strain behaviour; (b)   u versus $_1$ curves; (c) q/p′ versus $_1$ curves; (d) effective stress paths.
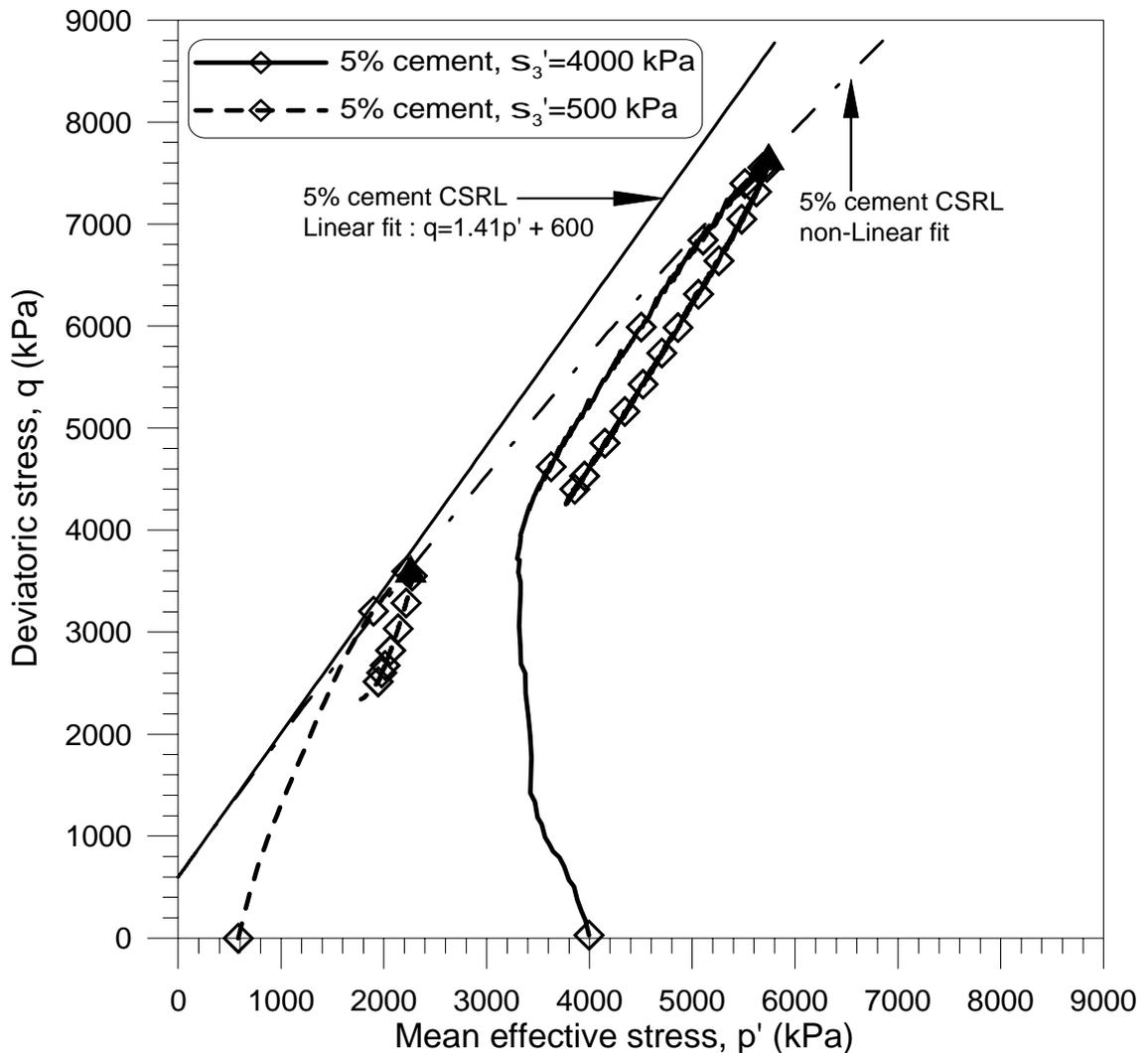
(d)

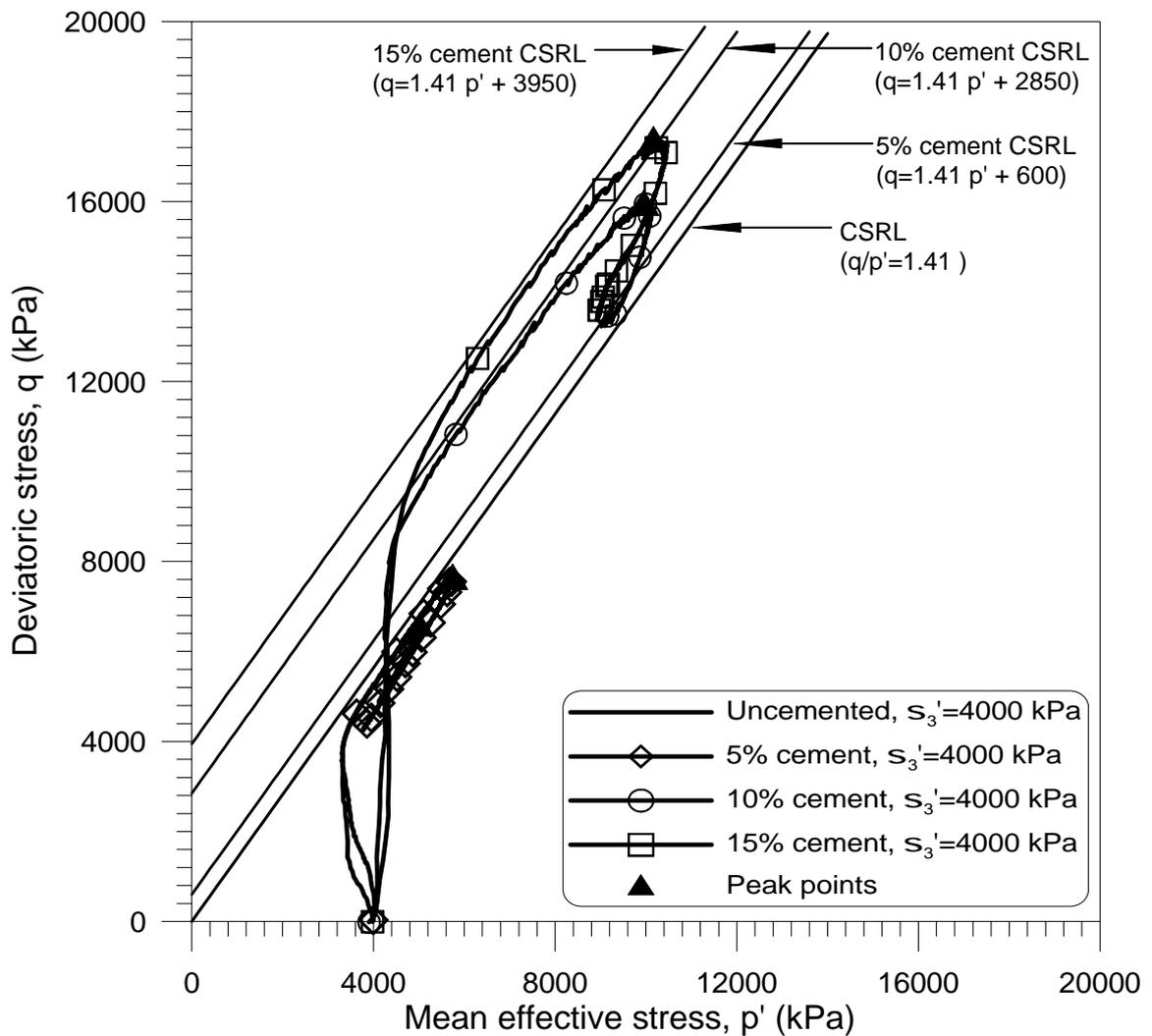Figure 4.7 continued

of cemented soils show similar characteristics of looser sand in stress-strain behaviour. However, the maximum deviatoric stress in high confining pressure is higher than conventional confining.

As shown in figure 4.7 (b) the excess pore water pressures in high pressure and conventional pressure both described in Section 4.3.1 show different behaviours. It appears that high confining pressure let the 5% cemented specimen has positive

excess pore water pressure during the test and induced excess pore water pressure has noticeable peak and higher value.

In figure 4.7 (c) the stress ratios against strain of 5% cemented specimen in high pressure and conventional pressure both described in Section 4.3.1 are presented. In high confining pressure, the maximum stress ratio shows obvious peak and high than conventional confining pressure. However, they seem to approach a constant value at ultimate state (high strain levels).

As shown in figure 4.7 (d), the stress paths in high and conventional confining pressure approach the same CSRL to reach their maximum deviatoric stress. It seems that CSRL can be a curve. However, more results are needed to clarify which type is rational.

4.4 Comparison between drained and undrained behaviour

This section will compare undrained behaviour with drained behaviour of uncemented and cemented sand at high pressure. The detailed description is given below.

4.4.1 Uncemented sand

The results of uncemented content under high confining pressure (4000kPa) specimens in CIU and CID tests are shown in figure 4.8. Deviatoric stress (q), excess pore water pressure (Δu), and stress ratio (q/p') plotted against axial strain (ε_1), and the effective stress paths are shown as well.

As shown in figure 4.8 (a), the deviatoric stresses against strain in CID and CIU tests shows different behaviour. In CID and CIU test, the peak deviatoric stresses which occur at 24.35% and 12.65% axial strain are 11479kPa and 6597kPa, respectively. The peak deviatoric stress of CID test is 1.74 times than CIU test, and occurs at very large strain. However, the deviatoric stress against strain in CIU test has noticeable peak. Furthermore, the initial slop of CID test is steeper than CIU test.

In figure 4.8 (b), the maximum stress ratio in CID test is higher than CIU test and it occurs large axial strain. As shown in figure 4.8 (c), the stress paths of CID and CIU tests show different routes. In CIU test, the stress path which increase upward at the beginning, and then they incline to follow the constant stress ratio line (CSRL) until the maximum deviatoric stress reached. Following this, it turns to opposite direction
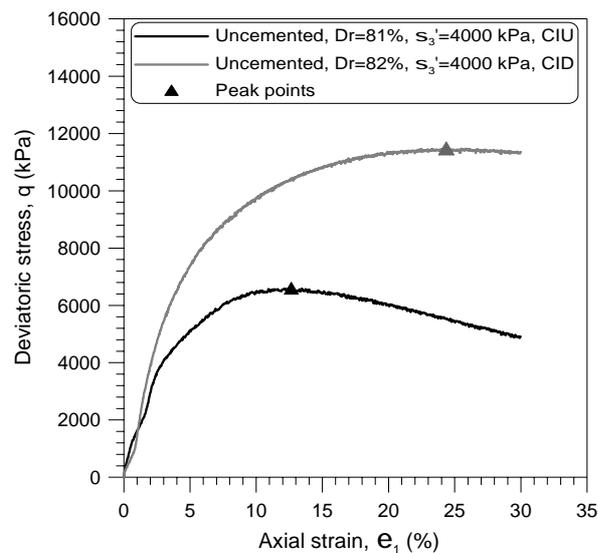
(a)
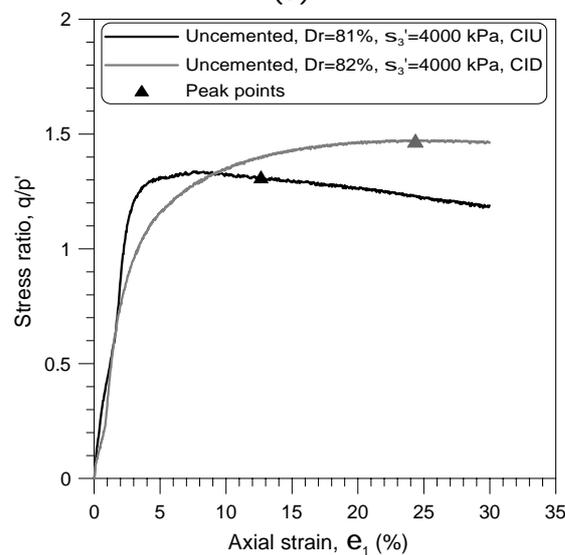
(b)

Figure 4.8 Results of uncemented specimens in CIU and CID triaxial test: (a) stress-strain behaviour; (b) q/p′ versus ε_1 curves; (c) effective stress paths.

to the end. In CID test, the total stress path which expect to increase in slop 1:3 to approach the failure line. However, the result seems not to follow the expectation and

the peak point is above the CSRL. This may cause by the difference of the relative density or the fit of CSRL. Nevertheless, It appears that more test should be conducted to clarify this. In CIU test, the stress path which increase upward at the beginning, and then they incline to follow the constant stress ratio line (CSRL) until the maximum deviatoric stress reached. Following this, it turns to opposite direction to the end. In CID test, the total stress path which expect to increase in slop 1:3 to approach the failure line. However, the result seems not to follow the expectation and the peak point is above the CSRL.
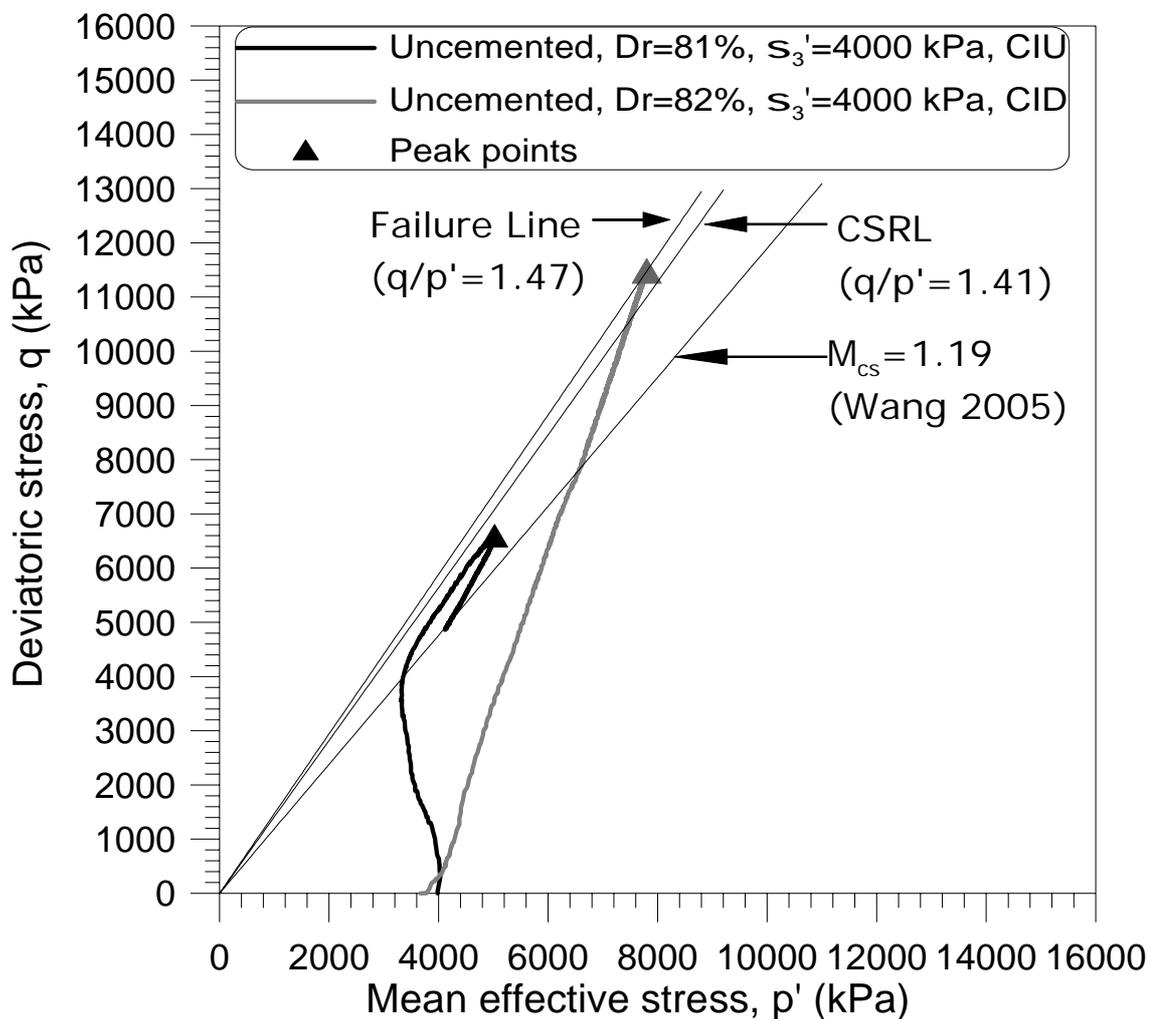

(c)

Figure 4.8 Continued

4.4.2 Cemented sand

The results of cemented content under high confining pressure (4000kPa) specimens

in CIU and CID tests are shown in figure 4.9. Deviatoric stress (q), excess pore water

pressure (u), and stress ratio (q/p') plotted against axial strain ($_1$), and the
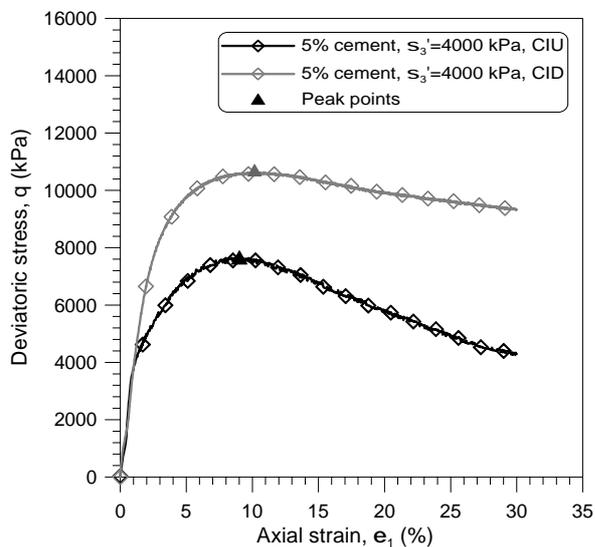
(a)
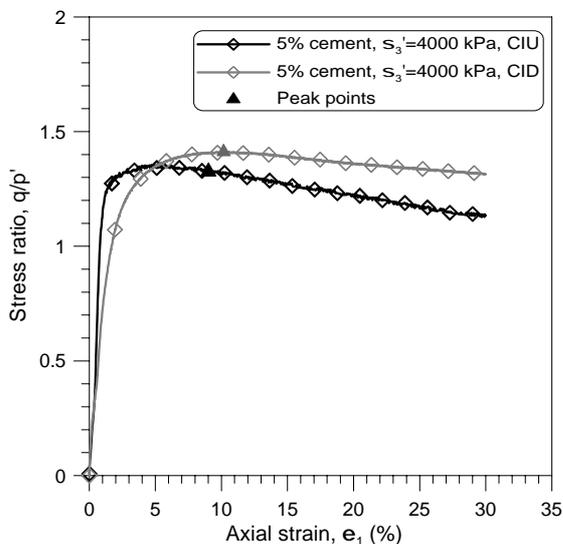
(b)

Figure 4.9 Results of cemented specimens in CIU and CID triaxial test: (a) stress-strain behaviour; (b) q/p' versus $_1$ curves; (c) effective stress paths.
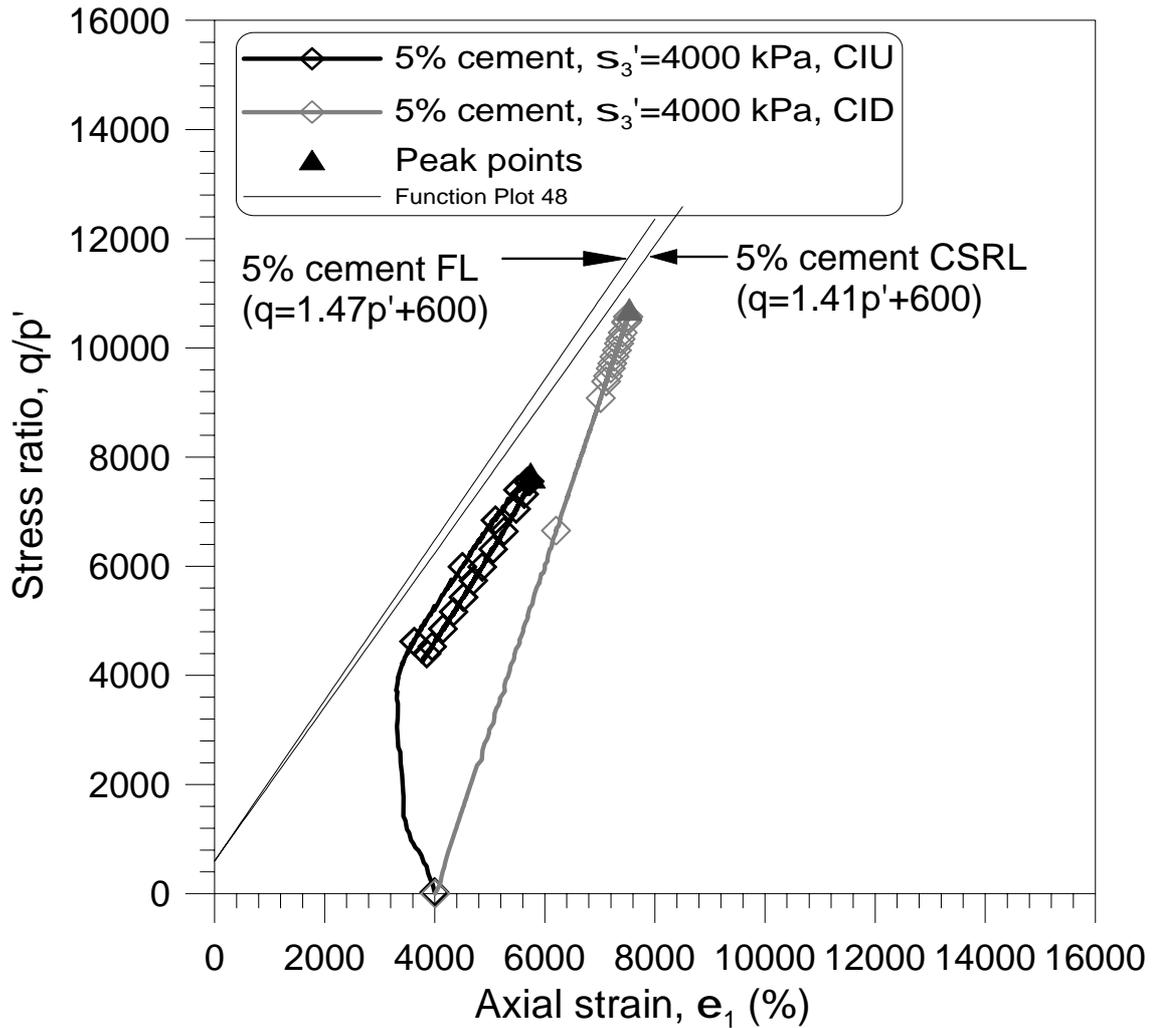
Figure 4.9 continued.

effective stress paths are shown as well.

As shown in figure 4.9 (a), the deviatoric stresses against strain in CID and CIU tests shows the same tendency as uncemented specimen. In CID and CIU test, the peak deviatoric stresses which occur at 10.17% and 9.01% axial strain are 10637kPa and 7661kPa, respectively. The peak deviatoric stress of CID test is higher than CIU test, and occurs at larger strain.

In figure 4.9 (b), the maximum stress ratio in CID test is higher than CIU test and it occurs large axial strain. As shown in figure 4.9 (c), the stress paths of CID and CIU tests show different routes. In CIU test, the stress path which increase upward at the

beginning, and then they incline to follow the constant stress ratio line (CSRL) until the maximum deviatoric stress reached. Following this, it turns to opposite direction to the end. In CID test, the total stress path which increase in slop 1:3 to approach the failure line. It appears that FL and CSRL can be curves. However, more results are needed to verify these.

Chapter 5   Conclusions and recommendations for further research

5.1 Conclusions

A series of laboratory tests including uncemented and cemented Portaway sand CIU triaxial tests of at high pressures and conventional pressure has been carried out to understand the behaviour of cemented sand. Portland cement was used as cementation agent for preparing artificially cemented specimens. The results provide the following main conclusion:

1. Stress-strain curves show that maximum deviatoric stresses and stiffness increases with an increase in cement content, density, and the confining pressure. The effect of confining pressure to peak deviatoric stress declines with the increase of confining pressure in uncemented specimens.

2. The drainage conditions influence maximum deviatoric stresses, stress ratios and stiffness in both cemented and uncemented specimens. The stress-strain curves in CID tests have higher maximum deviatoric stresses, stress ratios and stiffness than CIU ones. The Failure line and CSRL can be curves in p'-q space.

3. Stress ratio against strain curves show that peak stress ratios increase with an increase in cement content and density. Each curves approaches to a constant value at large strain level.

4. The excess pore water pressures for both uncemented specimen of medium dense density and 10% and 15% cement content specimens at high pressure are positive during the tests.

5. The excess pore water pressure for uncemented specimens of dense density, 5% cement content specimens at high confining pressure and cemented specimens at conventional confining pressures are positive at beginning of the test, and

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then the values change to negative as axial strains increase. The transition from positive excess pore water pressure to negative in uncemented and 5% cement content specimens are gradually, while it is extremely sharp in 10% and 15% cement content at high confining pressure.

6. The effective stress paths in dense and cemented specimens at conventional and high confining pressure are similar. They all increases upward at the beginning, and then they incline to approach the CSRL with the slop of 1.41 until the maximum deviatoric stress reached. It appears that cohesion intercepts increase to power 1.77 of cement content. It means that non-linear relationship between the cohesion intercept of CSRL.

7. The effective stress path of medium dense sand can be used to determine the instability line.

8. The effect of confining pressure to peak deviatoric stress declines with the increase of confining pressure in uncemented specimens.

9. The higher cement content leads to higher maximum deviatoric stress and let the peak deviatoric stress show earlier. The behaviours of uncemented and 5% cemented specimen are different from 10% and 15% cemented specimen. 10% and 15% cemented specimens show clear peaks and their maximum deviatoric stresses. However, the peak deviatoric stress of uncemented and 5% cemented specimens is not so obviously. The deviatoric stress in high confining pressure may be more sensitive when cement content between 5% and 10%. However, it needs more results to clarify.

5.2 Recommendations for future research

This project had provided some understanding of the stress-strain behaviour of cemented and uncemented sands at high pressures. However, more experimental studies are still essential for further understanding. Further experiments and ideals

are outlined below.

- CIU tests conducted in higher confining pressure, i.e. the confining pressure over 20MPa to investigate the influence of the high confining pressure.

- CIU tests for lower cement content, i.e. 1%, 3%, 7% to clarify the effect of cement content.

- Relative CID tests should be carried out to explore the effect of drainage.

- With more data obtained could be used to examine or improve existing constitutive models to simulate behaviour of cemented materials more accurately.

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