MECHANICAL BEHAVIOUR OF LIME-SLAG TREATED COODE ISLAND SILT

By

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DEDICATION

To
My Parents

Late Chitta Ranjan Chowdhury
&
Mrs. Ava Rani Chowdhury

STATEMENT

This thesis contains no material that has been previously submitted for any other degree or diploma in any university or other institution. To the best of my knowledge, this thesis contains no material previously published or written by others, except where due reference has been made in the text.

Bishwajit Chowdhury

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LIST OF PUBLICATIONS

1. **Chowdhury, B.**, Haque, A. and Muhunthan, B. (2013). "A new pressure-void ratio relationship for structured soils in the virgin compression range." *Journal Geotechnical and Geoenvironmental Engineering,* ASCE (revised and resubmitted after first review)

2. Islam, S., **Chowdhury, B.** and Haque, A. (2012). "On the sustainable ground improvement aspects of pyrite bearing clay using lime activated slag" *Grouting and Deep Mixing 2012,* Geotechnical Special Publication (GSP 228), ASCE, 1781-1788

3. Stanley, B., **Chowdhury, B.**, Haque, A. (2012). "A preliminary study of strength behaviour of lime-slag treated pyrite bearing soft Coode Island Silt." *Australian Geomechanics Journal,* 47(2), 17-24

4. Stanley, B., **Chowdhury, B.** and Haque, A. (2011). Discussion of "Addressing Sulfate-Induced Heave in Lime Treated Soils" by Dallas N. Little, Syam Nair; and Bruce Herbert. *Journal Geotechnical and Geoenvironmental Engineering,* ASCE, 137(8), 812-813.

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NOTATIONS AND ABBREVIATIONS

a and b = Hyperbolic fit parameters in transformed pressure-void ratio space

a' and b' = Hyperbolic fit parameters

c' = Effective cohesion

C_c = Compression index

C'_c = Modified compression index

C'_{cs}= Secant compression index

C^T_{cs} = Secant compression index in transformed pressure-void ratio space

C_e = Void ratio transformation constant

C_σ = Pressure transformation constant

e = Void ratio

e^T = Transformed void ratio

e_y = Void ratio at yield stress

e^T_y = Transformed void ratio at yield

E_i = Initial undrained modulus

φ' = Effective friction angle

σ' = Vertical effective stress

σ^T = Vertical effective stress in transformed pressure-void ratio space

σ'_y = Yield Stress

σ^T_y = Yield stress in transformed pressure-void ratio space

p' = Mean effective stress

q = Deviator stress

q_c= Cone tip resistance

s' = Average of major and minor principal stress

t' = Half of the deviator stress

CBD = Central Business District

GGBFS = Ground Granulated Blast Furnace Slag

CIS = Coode Island Silt

CIU = Isotropically Consolidated Undrained

UCS = Unconfined Compressive Strength

ABSTRACT

This research aims to provide an in-depth analysis and interpretation of the mechanical behaviour of lime-slag treated pyrite bearing Coode Island Silt (CIS), a widely occurring problematic soft soil in the Melbourne CBD area. This study analyses the time-dependent strength development characteristics, compression behaviour and undrained shear behaviour of CIS treated with different amounts of lime and slag. In the course of this study a new virgin compression model that can reproduce the compression behaviour of a wide range of structured soils including naturally structured soils and artificially cemented soils has been developed. The new model has been employed to interpret the compressibility behaviour of lime-slag treated CIS.

An investigation on the time-dependent strength development of the treated soils mass through unconfined compressive strength tests revealed that slag, in combination with a lime content higher than the lime saturation point (the minimum lime content at which the pH of the solution prepared with soil, lime, and water becomes approximately 12.4), can effectively improve the strength and stiffness of soft pyrite bearing CIS. Strength was found to be almost independent of lime content at different curing periods whereas slag was found to have considerable influence on the developed strength at all curing periods. The effect of curing was found to be most prominent at earlier phases of curing. However, its effect on the strength development was found to cease gradually due to gradual decline in the rate of pozzolanic reactions.

A comprehensive review of literature revealed that the virgin compression behaviour of structured soils immediately after yield is controlled by the progressive collapse of larger

inter-aggregate pores and when these macro pores are collapsed, compression behaviour is controlled by the mineralogy of the soil aggregates. A new virgin compression model based on two easily determinable parameters was developed and the developed model is able to reproduce the compression behaviour of a wide range of structured soils including naturally structured soils and artificially cemented soils. One of the parameters corresponds to the compression caused by the collapse of larger macro-pores while the other parameter corresponds to the compression behaviour at the stress range where the effect of soil mineralogy is the dominant factor.

Study on the compressibility behaviour of lime-slag treated CIS revealed that the influences of different experimental variables on the yield strength were very similar to their influences on the UCS values. It was found that compressibility generally increases with an increase in yield strength. The model parameter which characterizes the compression behaviour in the structure controlled zone was found to decrease with an increase in the degree of cementation. On the other hand the parameter that controls the compression behaviour in the de-structured state was found to vary within a much narrow range for all the cases investigated possibly due to comparatively less influence of different experimental variables on the mineralogical alteration of the treated material.

Investigation on the undrained shearing behaviour of treated CIS divulged that slag content increases both strength and stiffness of the treated CIS. Consolidation pressure was found to have significant influence on the strength, stiffness and overall stress-strain characteristics. The effect of a change in consolidation pressure on the stress-strain behaviour was found to be more prominent when the treated CIS was sheared from an elevated level of consolidation pressure. It was suggested that at low level of consolidation pressure, the behaviour up to the peak stress is controlled by the response of the cementitious bonds and the post-peak response is characterized by the inhomogeneous deformations of the samples. At elevated consolidation pressure, the brittleness at peak stress state was found to decrease and the stress-strain behaviour was found to be of progressively strain-softening type. The peak strength parameters derived from the result of triaxial tests indicated that peak friction angle does not change significantly with a change in the degree of cementation but the cohesion intercept was found to increase with the degree of cementation. Due to presence of strong discontinuities in the post-peak region, especially for the samples sheared from low level of consolidation pressures, it was not possible to derive a unique set of strength parameters corresponding to de-structured state of the lime-slag treated CIS.

The outcomes of this study will equip the geotechnical practitioners with the knowledge required to incorporate cementitious stabilization of pyrite bearing soft soils with lime and slag, which is an abundantly available industrial by-product in Australia.

CHAPTER 1: INTRODUCTION

1.1 Background

Increasing urbanisation and rapid population growth require rapid expansion of infrastructure facilities. As a result of that the demand for suitable land that offers economy in terms of foundation cost has been ever increasing. As the amount of suitable land is being gradually depleted, the need for the use of land previously deemed unsuitable due to presence of problematic subsoil conditions is on the rise. The conventional foundation solutions adopted in this type of soil condition is the relatively expensive deep foundations which transfer the structural loads to competent deep soil layers. Although deep foundation may be the only practical solution for heavily loaded structures and can be economically justified for large scale projects, the use of deep foundations for moderately or lightly loaded structures, such as embankments, low-rise buildings, can increase the foundation cost disproportionately in many cases. Therefore, for building moderately loaded structures on soft soils, the search for economical alternatives to deep foundations appears to be a logical endeavour. From this consideration, different ground improvement/modification techniques, such as prefabricated vertical drains, stone columns, chemical stabilization etc. came into existence.

Problems associated with soft soils are prevalent in many of the large cities of the world, for example, soft marine clay in Singapore, and soft clay underlying majority of the city of Bangkok, Thailand. In the city of Melbourne, an extensive deposit of a soft (undrained shear strength in the range of 5-30 kPa) and highly compressible clay, locally known as Coode Island Silt (CIS), underlies majority of the Central Business District (CBD). This particular soft soil layer has been found to extend up to a depth of 30m. Regardless of the magnitude of the applied loads, total and differential settlements in the range of 500-700mm can be expected for shallow foundation supported structures built on CIS (Ervin, 1992). To deal with the poor strength and compressibility properties of CIS, most of the structures built in the Melbourne CBD area are founded on piled foundations socketed into the Melbourne mudstone layer located at around 30m depth. The problem with CIS is further complicated by the presence of pyrite, a sulphate bearing phase, at depths greater than 10m from the ground surface (Stanley, 2010). In order to avoid any risk of potential environmental hazard, Environment Protection Authority (EPA) regulations require the excavated sulphate bearing CIS be appropriately treated before its disposal. Although, as mentioned earlier, use of deep foundations can be justified for large-scale projects, for moderately loaded structures there is a strong need for exploring different cost-effective and environment friendly alternatives to deep foundations for development of infrastructure on soft CIS deposits.

1.2 Statement of the problem

Presence of vast deposits of CIS, a sulphate bearing soft soil, in the prime locations of the Melbourne CBD area poses significant economic and environmental challenges in the development of infrastructure. In the development of new infrastructures and renewal/rehabilitation of existing ones, environmental sustainability, in addition to cost-effectiveness, is becoming an increasingly important issue (Jefferson et al., 2010). Spaulding et al. (2008) has shown how ground improvement techniques can enhance the sustainability of projects while reducing costs. Results of preliminary researches on the use of cementitious stabilization, one of the many available methods of ground improvement, for improving the shear strength of soft sulphate bearing CIS have been found to be very encouraging (Soon, 2003; Rex et al., 2008; Stanley, 2010). Previous laboratory investigations revealed that different industrial by-products such as Ground Granulated

Blast Furnace Slag (GGBFS) and fly-ash can be effectively used to improve the properties of this problematic soil by cementitious stabilization. While Australia produces enormous quantities of GGBFS, it is largely under-utilized (Gregory & Jones, 2005). Therefore, optimal utilization of GGBFS for improving the properties of problematic soft soils such as CIS has the potential of addressing both the economic and sustainability issues related to the development of infrastructure on this type of soft deposits.

For advancing the use of cementitious stabilization technique in projects involving CIS, an understanding of the different properties and behaviour of the stabilised soil under complex geo-environment is of vital importance. The past investigations on stabilized CIS have been mainly based on Unconfined Compressive Strength (UCS) tests and tests of other index properties (liquid limit, plastic limit). However, UCS test results have very limited utility in advanced analyses of geotechnical problems. In advanced geotechnical analyses, precise description of the mechanical behaviour (such as compression behaviour and shearing response at different levels of confinement) of the geomaterials plays a very important role in the realistic prediction of load-deformation responses. Therefore, there is a strong need for comprehensive investigation on the mechanical behaviour of GGBFS stabilized CIS through elaborate laboratory testing. In view of that a series of laboratory testing program aimed at mechanical characterization of the stabilized CIS is proposed in the current research.

1.3 Objectives

The specific objectives of the current research are

- To study the progressive strength development characteristics of stabilized CIS through Unconfined Compressive Strength (UCS) tests.

- To develop a simple virgin compression model for structured soils which can reproduce the compression curves of a wide range of structured soils ranging from highly sensitive soils to artificially cemented soils by using easily determinable parameters.
- To study the time-dependent compressibility behaviour of CIS treated with different amounts of additives.
- To demonstrate the generality of the proposed compression model by simulating the compression curves of artificially cemented soils including stabilized CIS.
- To critically analyse the effects of additive contents and curing time on the parameters of the proposed compression model.
- To investigate the effect of the degree of cementation and consolidation pressure on the undrained shearing response of lime-slag treated CIS.

1.4 Organization of the thesis

- In **Chapter 2**, literature relevant to the current project is critically reviewed and the relevance of the current research is clearly demonstrated by highlighting the limitations of the earlier researches.
- In **Chapter 3**, details of the experimental investigations are described.
- In **Chapter 4**, results from Unconfined Compressive Strength (UCS) testing program on stabilized CIS are discussed.
- In **Chapter 5**, a new virgin compression model for structured soil is developed based on the compression behaviour observed for a large number of naturally

structured soils. The proposed model is validated against the virgin compression data of a large number of naturally structured soils. Parametric studies are conducted and the factors affecting the model parameters are discussed in details.

- In **Chapter 6**, 1-D compression behaviour of CIS treated with different amounts of additives and cured for different periods is described. The compression data is then critically analyzed based on the compression model developed in Chapter 5.

- In **Chapter 7**, results from a comprehensive series of Isotropically Consolidated Undrained (CIU) triaxial shear testing program on the stabilized CIS is critically analyzed with special focus placed on the influence of degree of cementation and pre-shear consolidation pressure on the undrained shearing response.

- In **Chapter 8**, several important conclusions are drawn and recommendations for further researches are provided.

1.5 References

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CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

Geotechnical problems due to low shear strength and high compressibility of in-situ soils are often encountered in modern urban constructions. In such circumstances, stabilization of soft soils through introduction of cementitious additives has been found to be a very useful and effective technology and in many cases a cost-effective alternative to more expensive deep foundations. Chemical stabilization of soil originated in Japan and Sweden almost simultaneously in the 1970s. Since then soil stabilization technique has been improved continuously and it is currently a proven technology which is in wide use all over the world. Porbaha et al. (2000) reported that soil stabilization technique has been successfully employed in a wide range of applications such as follows:

- for the construction of quay walls, wharf structures, and breakwaters in marine applications
- as foundation of various structures such as tanks, towers, bridge abutments, embankments, underground facilities, retaining structures and high-rise buildings
- for excavation control, and also as cut-off wall for dams, dykes, and river banks

However, its applicability to improve the engineering properties of a widely occurring problematic Melbourne soft soil, locally known as Coode Island Silt (CIS), is still under-explored.

The focus of the current research is to carry out an in-depth investigation on the effectiveness of soil stabilization technique for the improvement of soft CIS and to explore the mechanical behaviour of the treated soil mass with the help of elaborate laboratory investigations. Research literatures relevant to the current project are reviewed in this chapter. The review commences with a brief review of the engineering properties of soft CIS and the problems that are typically encountered in the development of infrastructures on this particular soil. After that the fundamentals of soil stabilization are discussed in details and then a review of relevant literature on the stabilization of different soft soils is presented with particular attention given to strength, compressibility and shearing behaviour of these stabilized soils. Previous researches on the cementitious stabilization of CIS are then discussed and the relevance of the current research is clearly demonstrated by highlighting the limitations of the earlier researches.

2.2 Coode Island Silt

Coode Island Silt (CIS) is one of the four youngest formed sedimentary formations in the Melbourne geological region of the Yarra Delta. The extent of the Yarra Delta is shown in Figure 2.1. CIS is widespread throughout the lower Yarra Delta region and is present beneath much of South Melbourne, Port Melbourne and Footscray (Ervin, 1992).

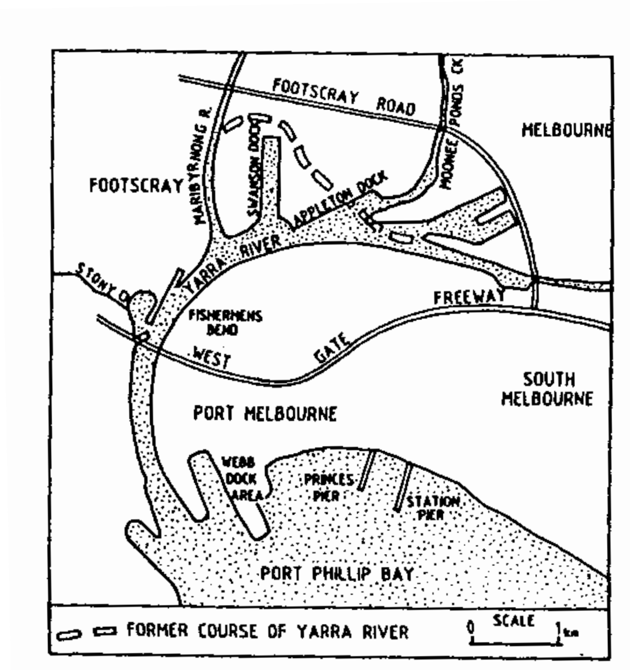


Figure 2.1. Distribution of Yarra Delta (from Ervin, 1992)

The thickness of this soft soil layer varies among different locations and can extend up to 30m. CIS is generally overlain by either Port Melbourne Sands or a thin covering of filling and generally underlain by Fishermens Bend Silt having an average thickness of 7m. More details about the geological setting of this area can be found in VandenBerg (1997). Ervin (1992) summarised some important engineering properties of CIS which are briefly described in the next section.

Table 2.1. Index properties of CIS (from Ervin, 1992)

Location	No. of Results	Liquid Limit		Plasticity Index		Linear Shrinkage %		Moisture Content %	
		Range	Ave	Range	Ave	Range	Ave	Range	Ave
Arts Centre (Concert Hall)	9	34-41	38	10-19	15.5	-	-	16-37	30
Arts Centre (North End)	-	21-77	-	4-43	-	-	-	-	-
Arts Centre	- (1)	-	60	-	-	-	-	-	-
City Road	- (1)	91-98	-	-	-	-	-	-	-
Clarke Street	7 (1)	-	78	-	-	-	-	-	-
Grant Street	36	26-148	88	8-89	54	2-21	15.5	30-108	-
Cecil Street	13	44-119	79	25-69	46	10-19	15.5	-	-
Charles Grimes Bridge	14	46-78	58	23-51	34	9-17.5	11.5	41-90	68
Lorimer Street	- (2)	68-107	81	35-81	53	-	-	55-89	71
Lorimer Street	- (3)	65-110	83	-	55	-	-	65-80	72
Footscray Road	-	71-76	74	48-59	53	17-21	19	-	-
Appleton Dock	6	62-102	75	30-45	46	8-18	15.5	44-84	64
Dynon Road	- (1)	-	82	-	-	-	-	-	-
Todd Road	3	83-87	85	52-56	54	19-19.5	19	76-85	79
Port Melbourne	5 (4)	38-135	77	21-56	39	-	-	-	-
Lower Yarra Crossing	- (1)	-	116	-	-	-	-	-	-
Howe Parade	3	68-84	78	41-53	48	17-18	18	67-101	83
Webb Dock	10	61-116	85	41-73	55	14-21.5	19	49-81	-

References:- (1) Donald et al, (1976), (2) McDonald (1985), (3) McDonald & Cimino (1984), (4) Walker & Morgan (1977)

2.2.1 Classification

CIS is described as silty clay (Donald et al., 1976) but includes bands of clayey silt, silt, sandy silt and sandy clay. It is generally dark grey, brown or dark grey in appearance. Atterberg limits (Table 2.1) as well as some general index properties of CIS have big scatter, which in turn contributes to big scatter in its strength properties. The Atterberg limits of CIS have been found to vary significantly within a very short distance. Donald et al. (1976) reported liquid limit varying from 57 to 96 over a depth range of 7.0m to 8.2m and in three bores spaced 2.0m from each other. CIS consists mostly of kaolinite and illite (Donald et al., 1976). CIS is also known to contain pyrite, a sulphate bearing phase, at certain locations. Due to the presence of pyrite, excavated CIS requires appropriate treatment before its safe disposal in order to be compliant with strict requirements of Environmental Protection Authority (EPA).

2.2.2 Strength

Ervin (1992) summarized some important engineering properties of CIS. CIS was classified as a normally consolidated to slightly overconsolidated clay. Its consistency varies from very soft to soft near the surface to stiff at greater depth. It was reported that Cone Penetration Test (CPT) results of CIS showed a linear increase in cone resistance with depth. The measured friction ratio from CPT response of CIS varied from 1 to 4%. It was suggested that the variation was due to the high sensitivity of CIS with lowest friction ratio being associated with highest degree of sensitivity.

Ervin (1992) commented that confined compression tests tended to provide more consistent strength data of CIS. The variation of undrained shear strength of CIS with depth for two different locations is shown in Figure 2.2. Based on confined compression results, the shear strength of CIS in South Melbourne was found to fall within the range of $\frac{q_c}{10}$ to $\frac{q_c}{15}$ while the strength of CIS at Webb Dock was approximately $\frac{q_c}{15}$, where q_c is the cone resistance value (Figure 2.2). Ervin (1992) also summarized the results of ten consolidated undrained triaxial tests to provide typical effective strength properties of CIS. It was found that the values of ϕ' ranged from 27 to 50^0 and c' ranged from 0 to 17 kPa. By ignoring the unrealistic values of $c' = 0$ and $\phi' = 50^0$, the average value of c' and ϕ'

were found to be 5.9 kPa and 34.2^0 respectively. Further results from six other sites, with 28 tests results in total, produced averages of c' =6.8 kPa and ϕ'=29.6^0 (inclusive of ϕ'=16 to 37^0 and ignoring the unrealistic values). Ervin (1992) also reported that the stiffness measurements obtained throughout Melbourne varied considerably. Data obtained in South Melbourne suggested $\frac{E_i}{q_c}$ = 12 to 16 whereas in Port Melbourne it ranged from 12 to 25 with an average of 18, where E_i is secant undrained Young's modulus corresponding to 1% strain.

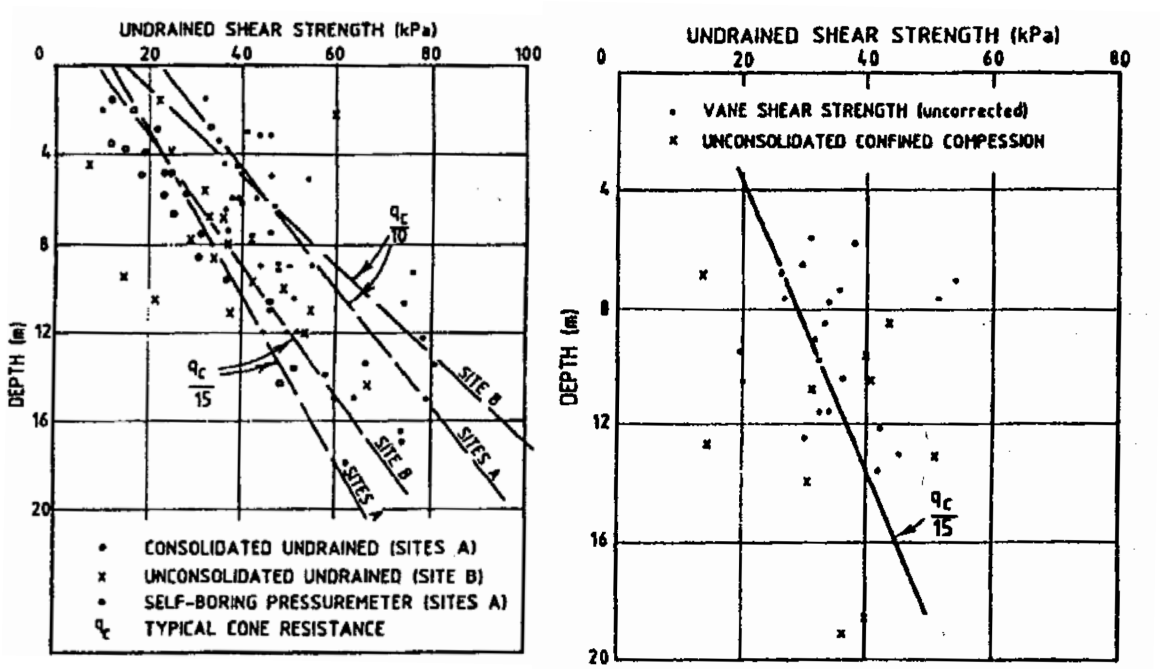


Figure 2.2. Undrained shear strength of CIS at (a) South Melbourne (b) Webb Dock (from Ervin 1992)

2.2.3 Compressibility

CIS is a highly compressible soil and has a record of highly variable settlement under loads. It exhibits significant secondary compression or creep behaviour even at very low applied loads (Ervin, 1992). McDonald (1985) observed about 700mm of settlement from a 3.5m of filling over a period of 3 years whereas 1200mm of settlement was reported where a wick drain was used. Throne (1981) and McDonald & Cimino (1984) reported a regional creep settlement rate of 10mm per year and 5-10mm per year respectively at a South Melbourne site. The settlement of CIS with load ranging between overburden pressure and pre-consolidation pressure had been found predominantly to be due to creep. For load beyond the pre-consolidation pressure, settlement consists of both primary consolidation and secondary consolidation settlements. Ervin (1992) reported that for CIS no strong correlation exists between the compression index (C_c) and liquid limit. This is in contrast to the suggestion of Terzaghi & Peck (1967) that soil compressibility can be correlated with soil plasticity. It was found that a much better correlation exists between moisture content and compression index. For saturated soils, moisture content is directly correlated with void ratio and therefore, it can be inferred that a good correlation exists between the compression index and void ratio of CIS. The variation of compression index of CIS with moisture content is presented in Figure 2.3 which shows that compressibility increases with an increase in moisture content. This finding indicates that CIS may be naturally structured since Leroueil et al. (1983) showed that the compressibility of structured soil correlates well with sensitivity and void ratio rather than with plasticity.

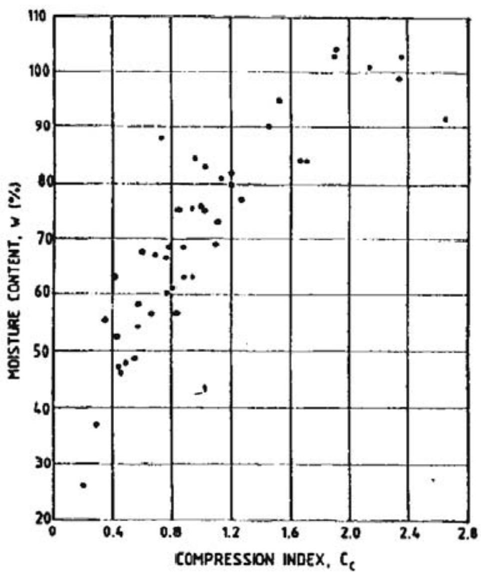


Figure 2.3. Compression index vs. moisture content for CIS (from Ervin 1992)

2.3 Fundamentals of chemical stabilization of soil

Soil stabilization is based on the principle of producing high strength cementitious reaction products within soft soil mass through the introduction of different cementitious additives. Different types of additives can be used for this purpose depending upon the type of soil, target strength and also the required rate of strength development.

Portland cement, lime, and latent hydraulic or pozzalonic materials react with water under certain conditions forming similar reaction products (i.e. C-S-H, C-A-S-H) which possess high strength. Cement yields high-strength reaction products immediately on reacting with water. Lime reacts with water to form calcium hydroxide, which has no strength enhancing effect in itself, but forms high-strength products in a secondary reaction with certain pozzolanic soils or added pozzolanic materials. Latent hydraulic materials, such as Ground Granulated Blast Furnace Slag (GGBFS), have to be activated with a suitable activator in order for the reaction products to form, while pozzolanic materials require the availability of calcium hydroxide throughout the reaction process.

The reactions that provide effective soil stabilization are (i) ion exchange between calcium ions from lime or cement and ions in the clay (ii) the reaction of cement with water in the soil and (iii) pozzolanic reactions between $Ca(OH)_2$ from lime or cement and pozzolanic minerals from the soil or added pozzolanic materials.

The reactivity of cement, latent hydraulic and pozzolanic materials depends, among other things, on the ratio of lime to silica (CaO: SiO_2) (Janz and Johansson, 2002). The larger this ratio, the more hydraulic and the more reactive the material is. Table 2.2 summarises the types of strength-enhancing reactions that take place for stabilization with different types of additives. In the following sections the reactions involved in the stabilization of soil with different additives are briefly reviewed.

Table 2.2. Strength-enhancing reactions (from Janz & Johansson 2002)

Binder	Reaction	CaO/SiO2	Co-reagent	Time-scale
Cement	Hydraulic	≈ 3	Water	Days
Ground granulated blast furnace slag (GGBFS)	Latent hydraulic	≈ 1	Water + Ca(OH)2 from cement or lime	Weeks
Lime	Pozzolanic	>50	Water + pozzolanic soil or pozzolanic additive	Months
Silica fume	Pozzolanic	≈ 0	Water + Ca(OH)2 from cement or lime	Months

2.3.1 Stabilization with cement

When cement comes in contact with water in the soil, hydration begins immediately giving a rapid gain in strength. The reaction products are mainly C-S-H gel, the main strength enhancing product, and calcium hydroxide $Ca(OH)_2$.

Table 2.3. Important constituents of cement (from Janz and Johansson, 2002)

Name of the constituent	Chemical formula	Abbreviation
Tricalcium silicate	$3CaO.SiO_2$	C_3S
Dicalcium silicate	$2CaO.SiO_2$	C_2S
Tricalcium aluminate	$3CaO.Al_2O_3$	C_3A
Tetracalcium aluminoferrite	$4CaO.Al_2O_3.Fe_2O_3$	C_4AF

The most important constituents of cement are given in Table 2.3. C_3S is the most important strength enhancing component of cement and the reactions involving this major component is given below:

$C_3S + H_2O \rightarrow C_3S_2H_x$ (Hydrated gel) + $Ca(OH)_2$--(2.1)

In pozzolanic clays and silts, a secondary reaction can take place between the soil silica/alumina and $Ca(OH)_2$ which provides a further strength gain. However, this reaction is much slower than the hydration of cement. The mechanism of pozzolanic reaction is as follow:

$Ca(OH)_2 = Ca^{++} + 2(OH)^-$--(2.2)

$Ca^{++} + 2(OH)^- + SiO_2$ (soil silica) →C-S-H (secondary cementitious product)----------(2.3)

$Ca^{++} + 2(OH)^- + Al_2O_3$ (soil alumina) →C-A-H (secondary cementitious product)-----(2.4)

2.3.2 Stabilization with lime

Lime reacts very rapidly with water in the soil forming $Ca(OH)_2$ with the generation of a great deal of heat. This reaction gives no strength gain but merely results in dewatering and a sharp rise in temperature. The strength gain comes about by secondary pozzolanic reactions between siliceous and aluminous compounds in the soil and the $Ca(OH)_2$ formed by hydration of the lime and also by ion exchange in clay soils.

The reactions involving lime are given below:

$CaO + H_2O \rightarrow Ca(OH)_2$--(2.5)

$Ca(OH)_2$ + pozzolana + H_2O → C-S-H (C-A-S-H) ---(2.6)

The dewatering due to increase in temperature can provide a temporary stabilizing effect, but this does not last long as the water content of the soil rises again when the reaction ceases. This temperature rise plays an important role in initiating the secondary pozzolanic reaction since the pozzolanic reaction is strongly temperature-dependent.

2.3.3 Stabilization with latent hydraulic and pozzolanic additives

Blast furnace slag, being a latent hydraulic additive, requires activation. The activation can be brought through the use of different types of alkalis such as $Ca(OH)_2$. The reaction products of lime and Portland cement contain $Ca(OH)_2$, making these products excellent activators for slag. Activation by $Ca(OH)_2$ from lime or cement gives a latent hydraulic reaction. After the slag is activated, the reaction takes place largely spontaneously with its own lime content.

Pozzolanic additives such as fly ash and silica fume require an external source of $Ca(OH)_2$. The $Ca(OH)_2$ may be supplied by hydration of cement or lime. Hence latent hydraulic and pozzolanic additives can never be used alone but can be used only in combination with cement and/or lime. The reactions involving these additives are given below:

Cement + H_2O → C-S-H + $Ca(OH)_2$---(2.7)

$CaO + H_2O \rightarrow Ca(OH)_2$--(2.8)

$Ca(OH)_2$ + pozzolana + H_2O → C-S-H (C-A-S-H)--(2.9)

2.4 Factors affecting the properties of stabilized soil

There are numerous factors that affect the properties of the stabilized soil mass. Some of these most important factors are physio-chemical properties of the soil, quality and quantity of the stabilizing agent, and mixing and curing conditions (Porbaha et al., 2000). The effects of different factors are briefly reviewed in the following sections.

2.4.1 Soil type

Various physical and chemical properties of soil such as grain size distribution, water content, Atterberg limits, type of clay minerals, cation exchange capacity, amount of soil silica and alumina, pH of the pore water and organic content influence the properties of the stabilized soil mass (Porbaha et al., 2000). Porbaha et al. (2000) reported that special considerations are needed to treat soil containing high amount of organic matter and excessive amount of sulphates since they can retard the hydration reactions significantly. Broms (1984) reported that presence of even a relatively small amount of organic matter in the host soil can have a large negative impact on the strength development. Miura et al.

(1986) reported that although for organic soils the increase in strength with cement stabilisation is often very low, cement is more effective than lime in improving the strength and deformation properties for soils high in organic content. Wissa et al. (1965) suggested that montmoriliionite clay minerals react more easily than illite with added cementitious additives. Bell (1993) and Woo (1971) reported that the quantity of stabilizing reagent required to achieve a target strength increases as the clay content of a particular soil increases. Woo (1971) also reported that the effectiveness of cement decreases with an increase in the plasticity index of the host soil. Figure 2.4 (Taki & Yang, 1991) shows the effect of grain size on the strength development and it can be seen that for a given cement content, the strength increase for coarse grained soil is significantly higher than that in fine grained soils. On the other hand, the effect of clay content on lime stabilization appears to be opposite to its influence observed for stabilization with cement. Since in the case of lime stabilization, strength is developed through the participation of clay particles in pozzolanic reactions with lime, the effectiveness of lime stabilization has been found to be enhanced with increasing clay content (Bergado et al., 1996).

From the above discussion it is obvious that the property of soil significantly affects the effectiveness of stabilization. The importance of taking the effect of soil characteristics in the design of a suitable binder into consideration can be clearly seen from Figure 2.5 which shows the effect of two different types of cements (ordinary Portland cement and slag-cement) on the strength development characteristics of two different types of clay studied by Kawasaki et al. (1981). It can be seen from this figure that the relative improvement effect of two different types of cement is significantly different for two different types of soils.

Apart from the physical and mineralogical characteristics of soils, pH of soil also plays important role in soil stabilization. Bergado et al. (1996) suggested that long term pozzalonic reactions are favoured by high pH values of the soil due to increased solubility of the silicates and aluminates of the clay particles at elevated pH.

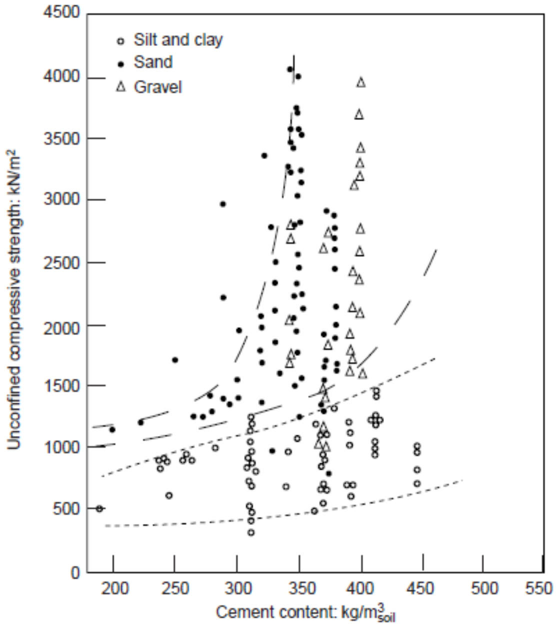


Figure 2.4. Effect of soil type on compressive strength of soil-cement (from Taki & Yang, 1991)

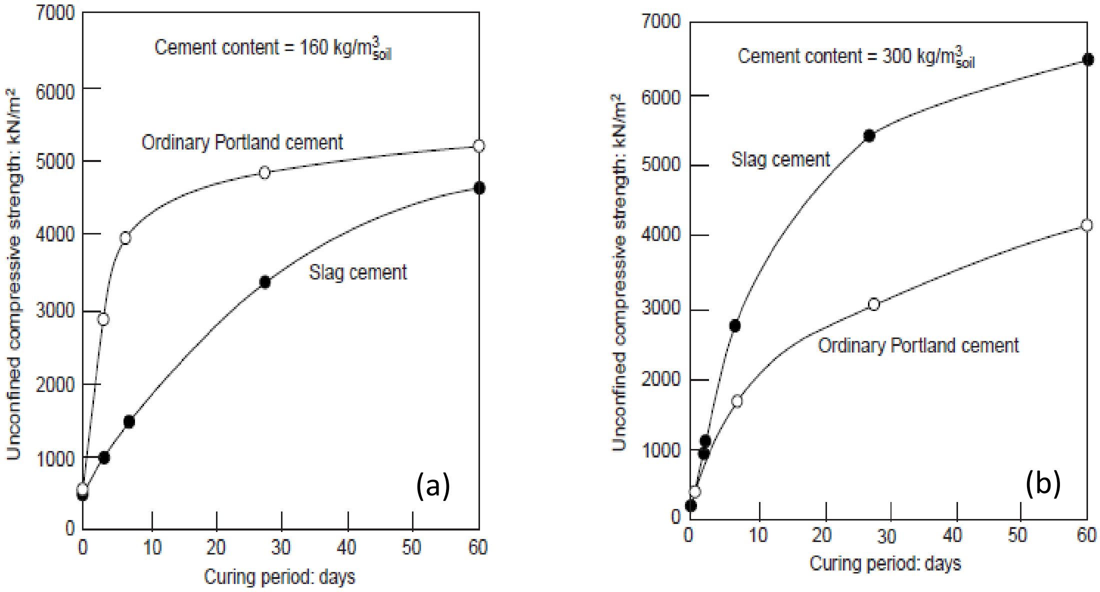


Figure 2.5. Effect of cement type on compressive strength of soil-cement (a) soil from Kanagawa, Japan (b) soil from Saga, Japan (from Kawasaki et al., 1981)

2.4.2 Type of stabilizing agent

When the cementitious stabilization of soil was first introduced, lime was the only binder used but since the mid 1980s cement has been widely used. It has been found that considerably higher strength can be achieved with cement (either alone or mixed with lime) than the strength achievable through lime stabilization (Ahnberg et al., 2003). It has also been found that industrial by-products such as GGBFS, fly-ash can also be used for stabilization purpose depending on the characteristics of the soil and strength requirements. Although the final reaction products that from within the treated soil mass treated with different types of binders are similar, the strength development can be significantly affected by the type of additive used and the type of soil being treated (Figure 2.6).

For soils treated with cement, the hydration reaction is comparatively faster and the strength development in the earlier phases of the hydration is significantly higher than the rate at which strength develops in the soil treated with latent hydraulic materials such as GGBFS or pozzolanic materials such as fly-ash (Bergado e t al., 1996). Ahnberg et al. (2003) suggested that when high early strength of the treated soil is required, the binder should contain at least a certain amount of cement. They found that although long-term strength achieved with latent hydraulic material or pozzolanic material can be comparable to the strength that can be achieved with cement treatment for a given total quantity of binders, the rate of strength development with cement treatment is much faster. Bergado et al. (1996) suggested that when a soil is treated with lime, there is an upper limit on the strength that can be achieved and that upper limit is controlled by the pozzalonic properties of the soil being treated. Ahnberg et al. (2003) also found that when a particular soil is stabilized with lime alone, the properties of the stabilized soil are largely determined by the properties of the host soil.

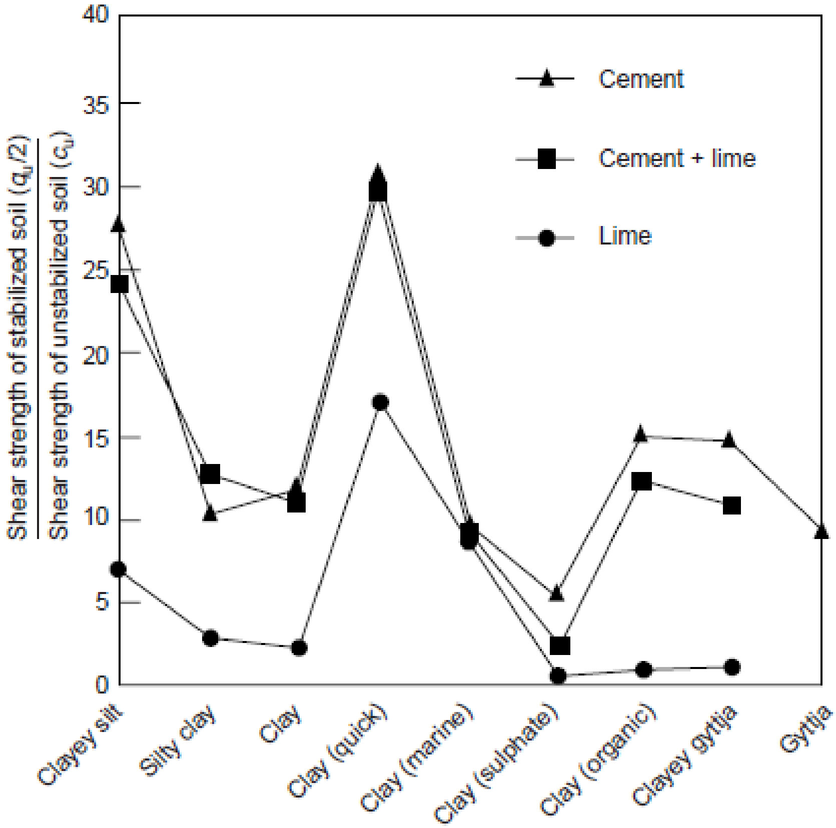


Figure 2.6. Effect of different stabilizers on the compressive strength development in stabilized Swedish soils (from Ahnberg et al., 1995)

In the recent times different industrial by products such as GGBFS and fly-ash are becoming popular due to the comparative environmental benefits these by-products offer. Wild et al. (1998) found that lime-activated slag is very effective in improving the strength and deformation properties of sulphate bearing soft soil. It was also found that lime-slag treatment provides added benefit of suppressing the sulphate-induced heaving of the stabilized soil mass. In their study Wild et al. (1998) investigated the effect of lime content on the activation of slag by studying the effect of partial substitution of lime by slag for a given total quantity of binder on the strength development in treated sulphate-bearing soft soils. It was reported that for a given total binder content, the strength increased with an increase in slag/lime ratio up to a threshold value and a further increase in the above ratio caused the 28-day strength to decrease. The increase in strength with an increase in the slag/lime ratio was due to the fact that when adequate lime is present in the reaction environment to fully activate the slag, the strength increases with an increase in slag content due to the formation of higher amount of cementitious reaction products. However, when the slag/lime ratio increased beyond a threshold value, the strength started to decrease due to the inability of the added lime to fully activate the added slag. However, Wild et al. (1998) suggested that when treating sulphate bearing soil with lime activated slag, the lime content needs to be selected judiciously so that an excess amount of lime does not remain in the reaction environment in which case the excess lime can react with the sulphates to produce expansive reaction products that can increase the heaving potential of the treated soil.

2.4.3 Quantity of stabilizing agent

Horpibulsuk et al. (2010) found that for a given mixing water content, strength development with increasing cement content can be divided into three zones: active zone, inert zone and deterioration zone. In the active zone the strength increases with an increase in cement content whereas in the inert zone the rate of strength increment with increasing cement content is insignificant and a further increase in cement content causes the strength to decrease. Horpibulsuk et al. (2010) argued that the strength decrease observed in the deterioration zone is mainly due to the absence of enough amount of water to react with the added cement. Bergado et al. (1996) found a different type of zoning for the purpose of characterizing the strength development with increasing cement content to be appropriate for cement treated Bangkok clay (Figure 2.7). Bergado et al. (1996) reported that there exists an initial inactive zone marked by a threshold cement content below which strength improvement is insignificant. Kamruzzaman (2002) also found a zoning scheme similar to that of Bergado et al. (1996) to be appropriate for cement treated Singapore marine clay. Although both Kamruzaman (2003) and Bergado et al. (1996) identified the presence of an initial inactive zone, such zones was not identified by Horpibulsuk et al. (2010). However, Kukko (2000), based on the study of different Finish soils treated with cement and different types of inorganic industrial by-products, suggested the presence of an initial inactive zone as suggested by Kamruzaman (2003) and Bergado et al. (1996). On the other hand, Consoli et al. (2007) concluded that addition of an even small amount of cement (1%) caused a noticeable increase in strength of the cement stabilized Brazilian soil.

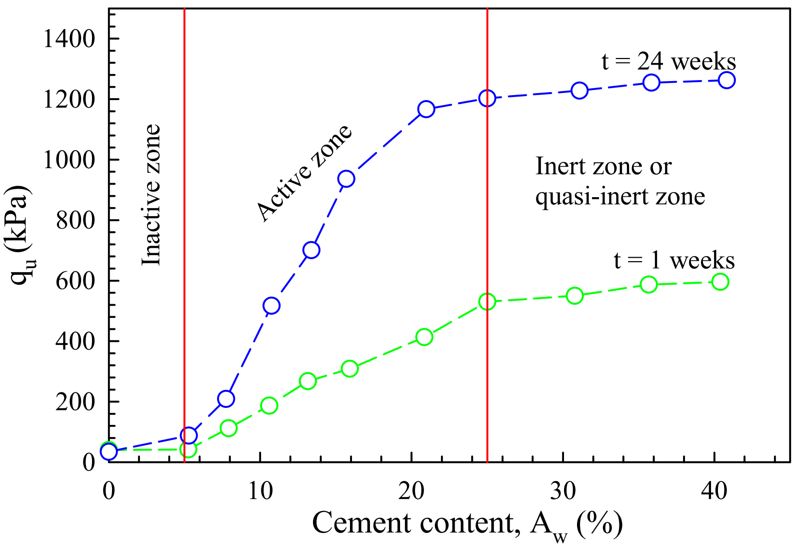


Figure 2.7. Influence of cement content on the Unconfined Compressive Strength of cement treated Bangkok clay (replotted from Bergado et al., 1996)

2.4.4 Curing time

The rate of increase of strength of soft soils stabilized with different types of binders gradually decreases with the passage of time. Figure 2.8 (Croft, 1968) shows the effect of curing time on the strength development of different types of laboratory prepared samples treated with lime. It can be observed from this figure that the rate of strength development significantly slows down with the progression of curing. The Cement Deep Mixing Association of Japan (Cement Deep Mixing Association of Japan, 1994), on the basis of the mean values from linear regression analyses, proposed the following relationships for the strength development of cement treated soils at different curing periods:

$q_{u28} = (1:49 \text{ to } 1:56).q_{u7}$

$q_{u91} = (1:85 \text{ to } 1:97).q_{u7}$

$q_{u91} = (1:20 \text{ to } 1:33).q_{u28}$

where q_{u7}, q_{u28} and q_{u91} are the unconfined compressive strengths of the treated soil after 7, 28 and 91 days respectively. The above relations clearly show that the rate of increase of strength gradually diminishes with the progression of curing. Based on Abraham's law (a

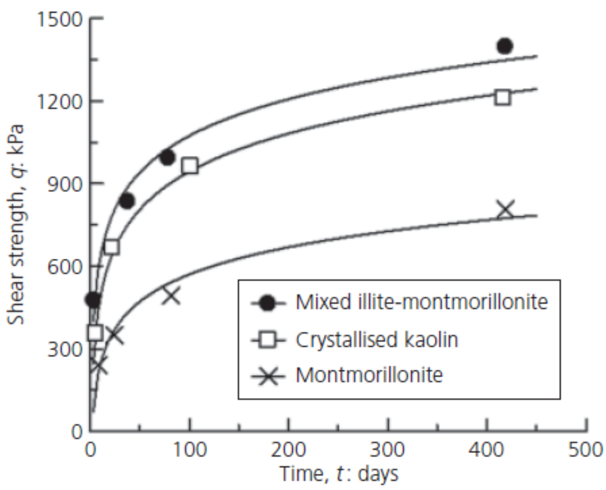


Figure 2.8. Peak strengths of three lime treated soils (from Croft, 1968)

widely used relationship in concrete science), Nagaraj et al. (1997) also proposed, a relationship between strength at any arbitrary time and the strength at a curing period of 14 days for cement stabilized soft soils by the following logarithmic relationship $\frac{S_D}{S_{14}} = -0.20 + 0.458 \ln(D)$ where S_D is the strength at time D days and S_{14} is the strength observed at 14 days. Horpibulsuk et al. (2003) and Ahnberg (2006) also proposed similar relationship between strength and logarithm of time. The logarithmic relationship clearly indicates about the progressively decreasing rate of strength improvement with time. The gradual decrease in the rate of increase of strength can be attributed to the progressive slowing down of different cementitious reactions due to the exhaustion of different reactants in the reaction environment.

2.4.5 Mixing water content

An increase in the mixing water content significantly decreases the strength of the stabilized soil. Endo (1976) studied the effect of initial water content ranging from 60 to 120% on laboratory prepared samples of marine clay treated with 5-20% cement and cured for 60 days (Figure 2.9). It was observed that the strength decreased significantly with an increase in the initial water content of the mix. Kamruzaman (2002) studied the effect of water content for a given amount of cement content on the strength development of cement treated soft Singapore Marine clay and found that the strength decreased significantly with an increase in initial moisture content. Yin & Lai (1998) also reached similar conclusion about the effect of mixing water content on the strength development based on a study of cement treated soft Hong Kong marine deposit. Horpibulsuk et al. (2003), through a comprehensive study of the strength development of Ariake clay treated with various proportions of water and cement, showed that clay–water/cement ratio can be uniquely related with the strength of cement stabilized soft soils. They emphasised that just like the water cement/ratio controls the strength of concrete (given by Abraham's Law), clay-water/cement ratio controls the strength in stabilized soil mass. They found that as long as the clay-water/cement ratio was kept constant, the strength, compressibility and stress-strain behaviour remained practically unchanged for Ariake clay treated with different amounts of cement. Consoli et al. (2007) studied the effects of moisture content, initial void ratio and cement content on strength development characteristics of a cement treated Brazilian clay. Although they found that the strength was considerably affected by mixing moisture content, they could not find any unique relationship between water/cement ratio and UCS but they found that a very good correlation exists between the identity (void ratio/cement) and UCS.

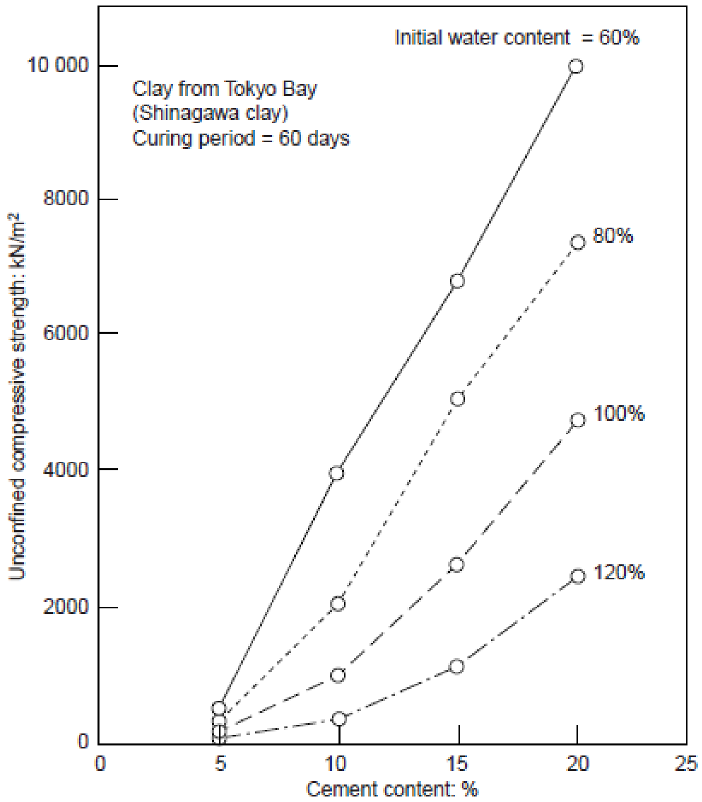


Figure 2.9. Effect of mixing water content on the compressive strength of cement treated clay (from Endo, 1976)

2.4.6 Curing temperature

Curing temperature has been found to affect the strength development process where the cementitious reactions are favored by high temperature (Broms, 1984). Figure 2.10 (Enami et al., 1985) shows the effect of curing temperature on the improvement of a silty soil treated with ordinary Portland cement and it can be seen that the strength increases almost linearly with increasing temperature at different curing periods. Ruff & Ho (1966) reported that when all other factors were identical, increasing curing temperature caused an increase in strength for lime treated soft soil. Broms (1984) attributed the positive effect of curing temperature to the increased solubility of the soil pozzalons. Kukko (2000) studied the effect of curing temperature on the strength development process of different Finish soils stabilized with cement as well as with different inorganic industrial by-products and found that temperature had prominent effect on the strength development in all the cases. It was suggested by Kukko (2000) that the study of long term strength development can be accelerated by studying the strength development under curing at elevated temperature.

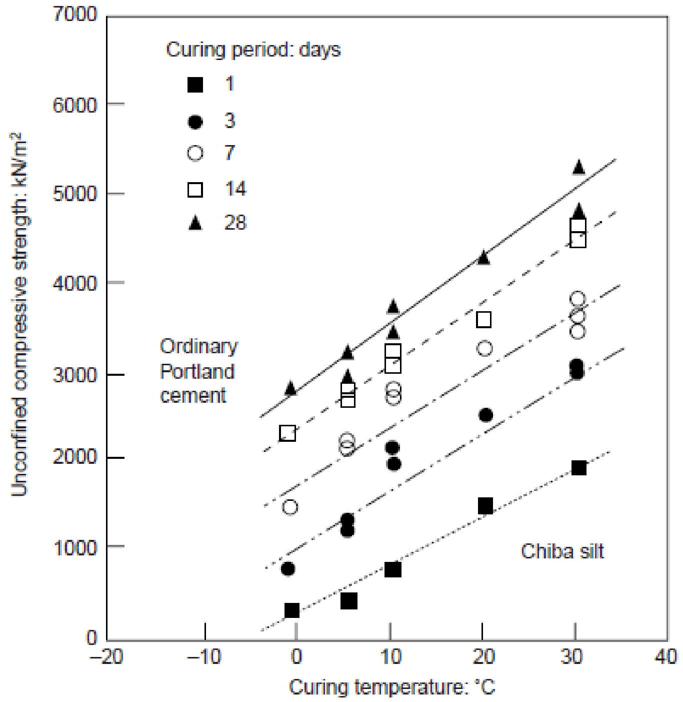


Figure 2.10. Effect of curing temperature on the compressive strength of cement treated silty soil (from Enami et al., 1985)

2.4.7 Field mixing condition

The mixing condition at field can not be controlled as precisely as it can be done in the laboratory. Horpibulsuk et al. (2011) found that the strength that can be attained in the field by both dry mixing method and wet mixing method can be significantly lower than the strength that can be achieved in the laboratory. They reported a ratio of field strength/laboratory strength in the range of 0.6 to 0.7. This result clearly indicates that in the design of cementitious binders for use in soil stabilization projects, the effect of field mixing conditions needs to be taken into account along with various other factors described earlier.

2.5 Mechanical characteristics of structured soils

Artificial cementations alter the mechanical behaviour of the host soil significantly. Artificially cemented soils fall in the category of structured soils as per the definition given by Mitchell (1976) who used the term “soil structure” to describe the combined effect of soil fabric and cementation bonds. Leroueil & Vaughn (1990) have shown that irrespective of its origin, different structured soils ranging from sensitive natural soils to soft rocks to artificially cemented soils exhibit a common pattern of mechanical behaviour. In general there is a scarcity of literature on the detailed mechanical behaviour of artificially cemented soils and therefore as implied by Leroueil & Vaughn (1990), important knowledge derived from the literature on naturally structured soils can be valuable in interpreting the experimental behaviour observed for artificially cemented soils. With that view, the literature on the mechanical behaviour of artificially cemented soils is complemented by discussions of literature on naturally structured soils wherever felt necessary.

The classical models of soil mechanics attempts to explain the mechanical behaviour of soils based on the initial density and its subsequent changes due to applied stresses. However, it has been found that the mechanical behaviour of naturally or artificially structured soils can not be adequately explained based only on this primary concept (Schnaid et al., 2001). Almost all geotechnical problems involve soils that are either naturally or artificially structured and therefore in recent times the study of the effect of structure on the soil behaviour (e.g., Burland, 1990; Leroueil & Vaughn, 1990; Cotecchia & Chandler, 1997; Kavvadas, 1998; Cotecchia & Chnadler, 2000) has gained significant momentum. From an in-depth analysis of a large body of experimental data, Leroueil & Vaughan (1990) have demonstrated that the influence of structure on the mechanical response of structured soils is as important as initial void ratio and stress history and concluded that the behaviour of structured soils in the laboratory and in the field cannot be properly understood unless the effects of structure are taken into account. The important impact of soil structures on the mechanical response of structured soils are briefly described in the following sections:

2.5.1 Compression behaviour of structured soils

Readily recognizable effect of structure on the mechanical behaviour of soil can be

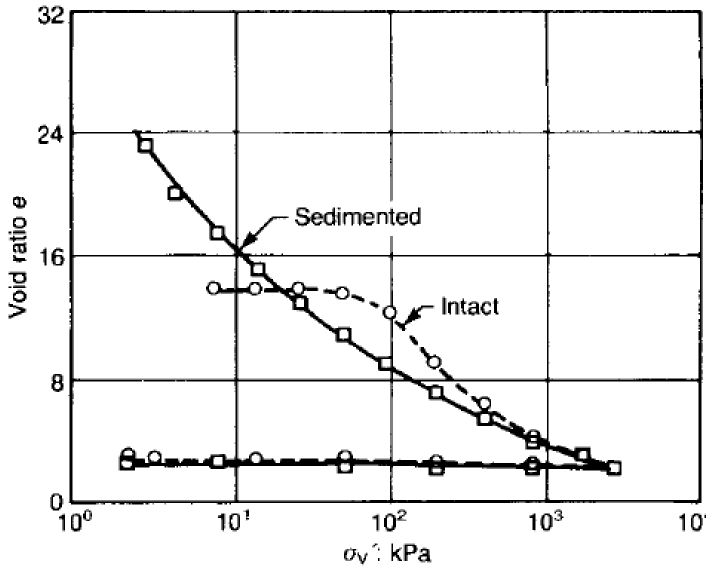
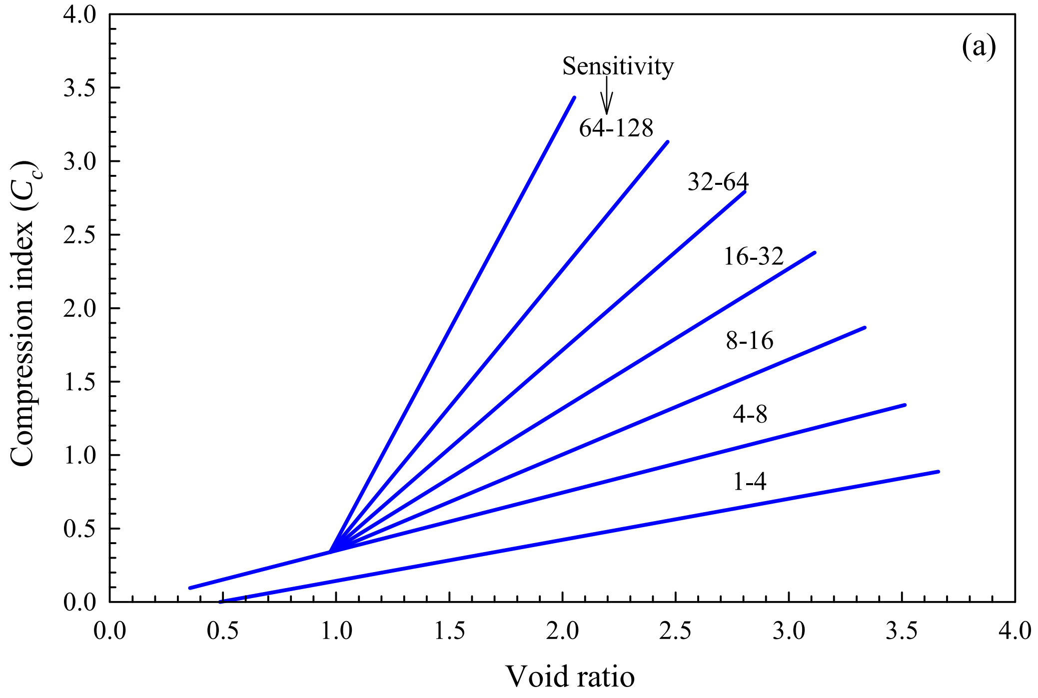


Figure 2.11. One-dimensional compression curves of intact and laboratory sedimented samples of Mexico City clay (from Mesri et al., 1975)

observed on the compression characteristics of structured soils (Leroueil & Vaughn, 1990). The presence of structures helps a soil to exist at void ratio which is significantly higher than the void ratio at which an equivalent non-structured soil can exist (Figure 2.11) under a given pressure. Leroueil & Vaughn (1990) also suggested that the structure remains almost intact up to the yield stress and as the stress approaches the yield point, major breakdown of structure takes place causing abrupt yielding of the soil. In the post yield region the structure is gradually removed from the soil and at large strain the influence of structure becomes insignificant. Although the compression behaviour up to the yield point for cemented soils is significantly stiffer than that of corresponding reconstituted soils, the plastic compression of structured soils is considerably higher than that of reconstituted soils.

Delage & Lefebvre (1984) and Delage (2010) discussed the effect of structure on the compression behaviour of structured soils in great details based on micro-structural and Mercury Intrusion Porosimetry (MIP) studies. Delage & Lefebvre (1984) and Delage (2010) showed that structured soil possesses two different types of pores: one group includes larger inter-aggregate pores formed by cementitious bridges spanning among clay aggregates and the other group includes much smaller intra-aggregate pores. When a structured soil is compressed, the pores with the largest size are the first to collapse and smaller intra-aggregate pores are affected only after the majority of the larger inter-aggregate pores resulting from the presence of structures have been collapsed. Leroueil et al. (1983) showed that the compressibility behaviour of structured soil can not be correlated with the plasticity of soils in the way the compressibility of reconstituted soils is related to soil plasticity. In the stress zone where the effect of structure is dominant, compressibility shows good correlation with sensitivity and void ratio (Figure 2.12) rather than with plasticity. On the other hand, when the structure is significantly removed, de-structured soils, like reconstituted soils, exhibit strong correlation with the plasticity of soils. Tanaka et al. (2001) also reported observation similar to that of Leroueil et al. (1983) on the compressibility behaviour of three different naturally structured clays: Singapore clay, Bangkok clay and Ariake clay. Nagaraj et al. (1990) suggested the use of two



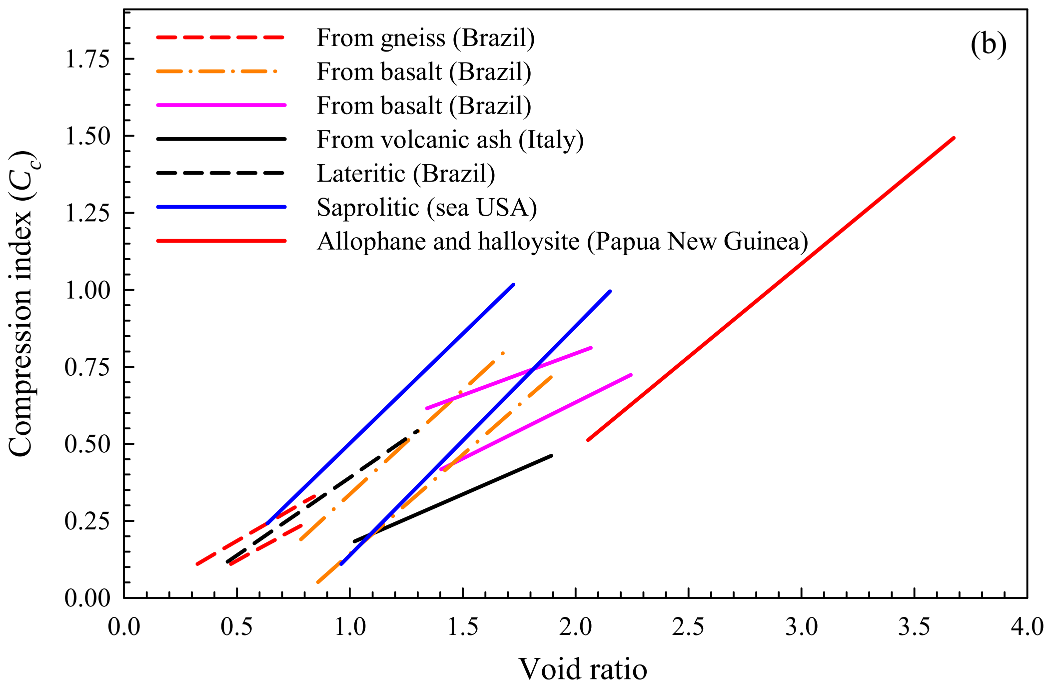


Figure 2.12. Relationships between compression index C_c and initial void ratio:

(a) for structured soft clays (b) for residual soils (replotted from Leroueil and Vaughn, 1990)

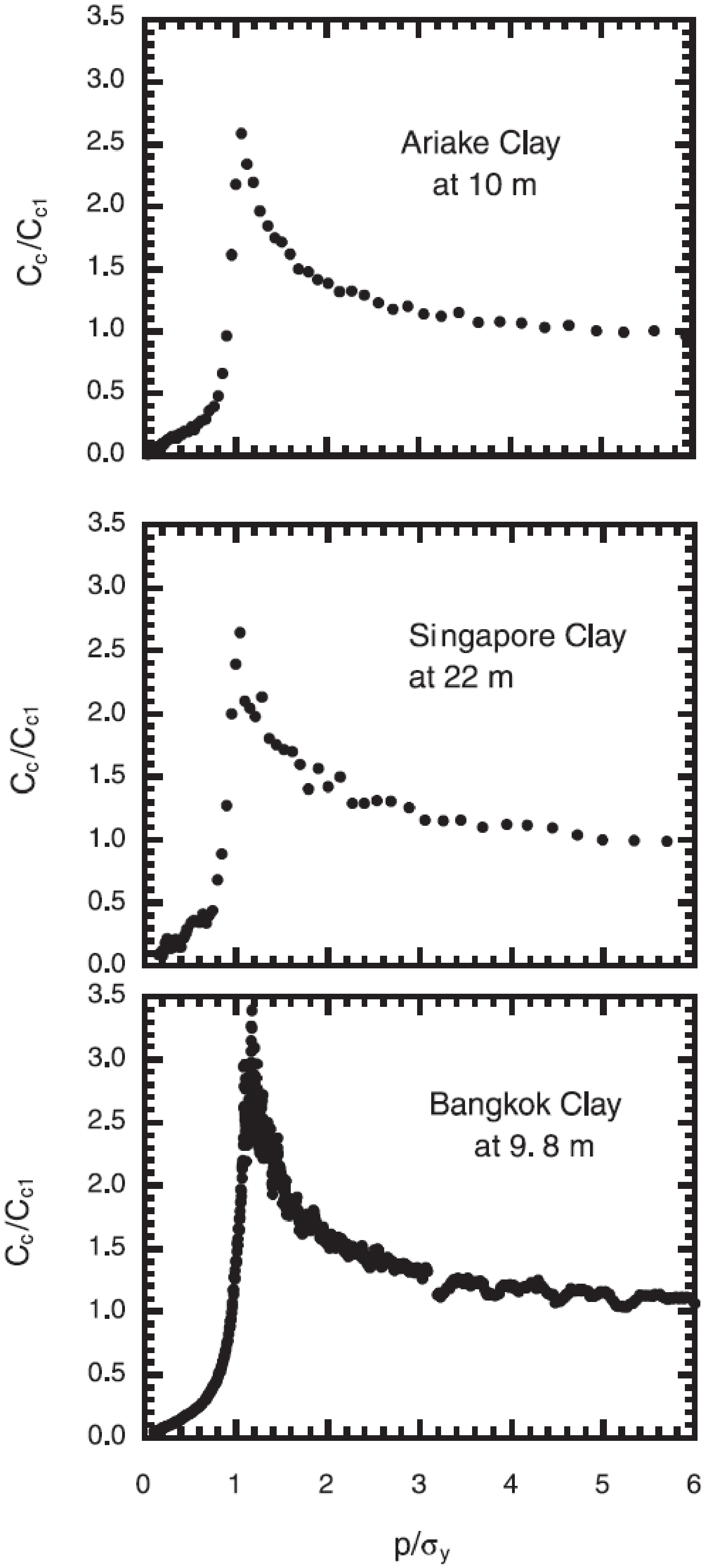


Figure 2.13. Relationship between normalized compression index and consolidation pressure for three different clays (from Tanaka et al., 2001)

Note: C_c = instantaneous compression index,
C_{c1}=Compression index at de-structured state
p = vertical stress, σ_y = yield stress

different compression indices for representing the virgin compression behaviour of structured soils. One compression index relates to the compression zone where de-structuration is dominant and the other compression index corresponds to compression in the de-structured zone. The above discussion on the compressibility behaviour of structured soils clearly indicates that the initial part of the compression is characterized predominantly by the breakdown of soils structures whereas the compressibility at large stress is controlled by the characteristics of the clay aggregates.

For naturally structured soils, the pressure dependence of compressibility is readily apparent due to rapid breakdown of weak natural structures and almost complete de-structuration is possible within the maximum pressure range typically employed in the laboratory to study the compression behaviour of soils. The variation of normalized compression index with normalized pressure for three naturally structured clays i.e., Singapore, Bangkok and Ariake clays is shown in Figure 2.13. It can be seen from the figure that although the compressibility varies considerably with pressure in the vicinity of yield stress, it becomes practically constant at large pressure. On the other hand, the variation of compression index of artificially cemented soils with pressure is not as immediately obvious as it is for naturally structured soils. This may be due to the fact that the maximum pressures that are typically applied in the investigation of the compression behaviour of artificially cemented soils are not large enough to cause significant de-structuration of the treated material. As a result there has been a general tendency to represent the compression behaviour of artificially cemented soils by a constant compression index (e.g., Uddin, 1995; Horpibulsuk et al., 2005). One notable study that investigated the pressure-dependent variability of compression index of artificially cemented soils is Kamruzzaman (2002) who studied the compressibility behaviour of cement treated Singapore marine clay comprehensively. Kamruzzaman (2002) concluded

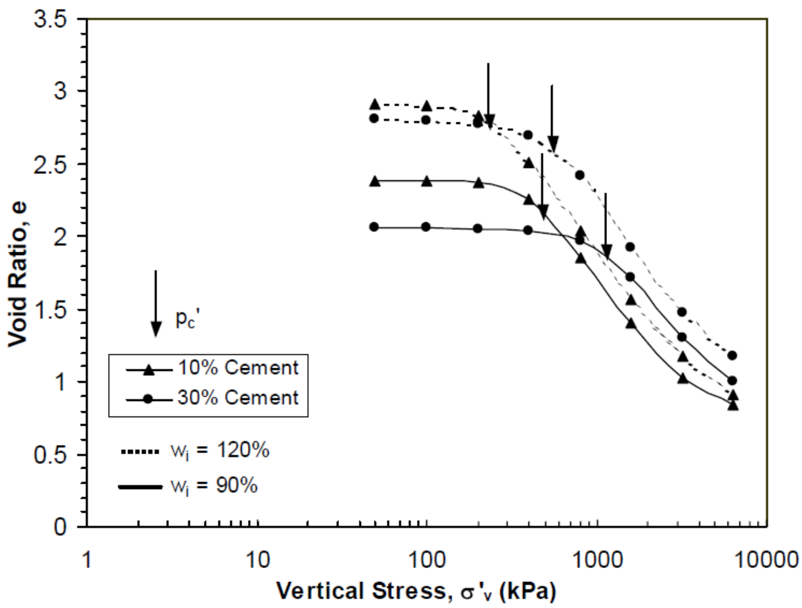


Figure 2.14. Effect of cement content on pressure-void ratio behaviour under 1-D consolidation (from Kamruzzaman, 2002)

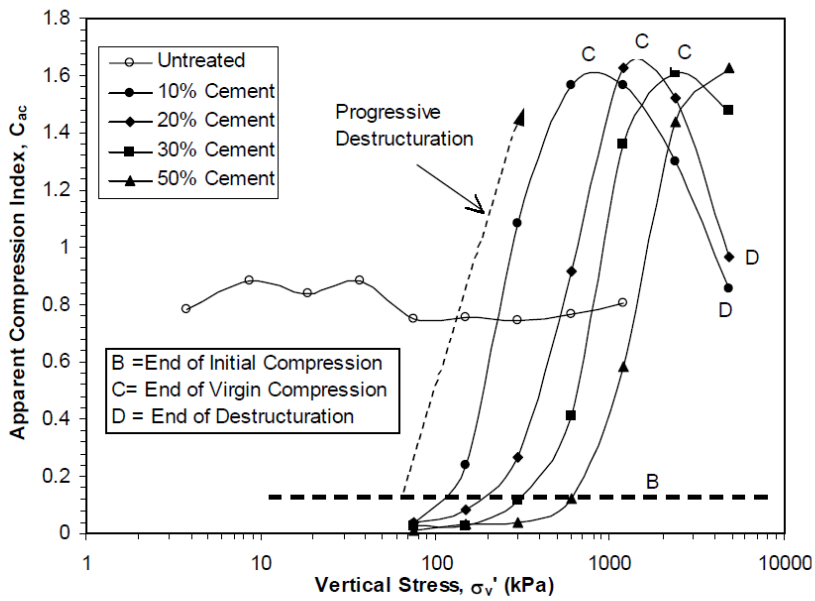


Figure 2.15. Effect of cement content on the variation of instantaneous compressibility index with log-pressure (from Kamruzzaman, 2002)

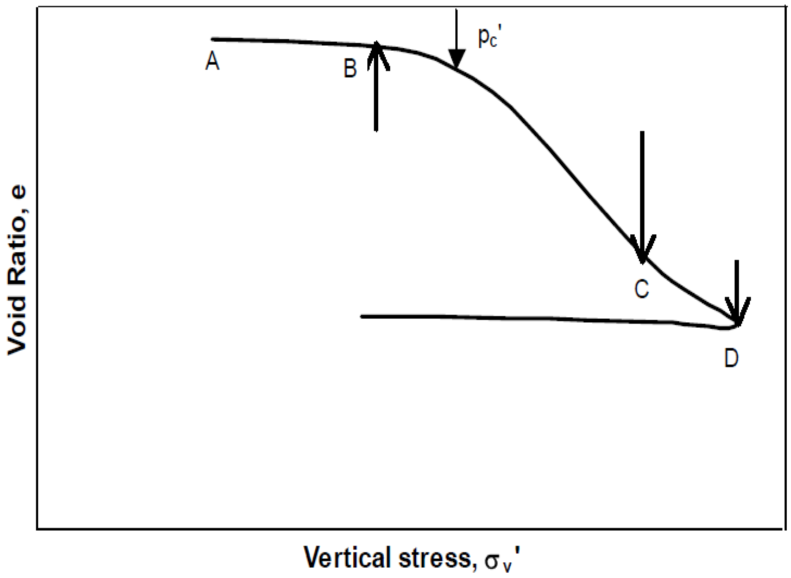


Figure 2.16. Schematic representation of pressure-void ratio curve for artificially cemented soils (from Kamruzzaman, 2002)

that the compressibility of artificially cemented soils, like naturally structured soils, display a strong dependence on pressure (Figure 2.14) and the compressibility at any particular pressure depends on the degree of de-structuration already caused to the treated soil mass. The variation of compressibility index with pressure for cement treated Singapore marine clay is shown in Figure 2.15 (in Figure 2.15 apparent compression index is the instantaneous value of the compression index) and this variation exhibit strong resemblance to the same variation observed for naturally structured Singapore, Bangkok and Ariake clays as presented in Figure 2.13. Kamruzzaman (2002) schematized the compression behaviour of artificially cemented soils as shown in Figure 2.16 where point C represents the point where compressibility is the maximum and point D corresponds to the stress condition at which the effect of structure becomes negligible. Kamruzzaman (2002) also studied the pore collapse mechanism of cement treated Singapore marine clay through Scanning Electron Microscopy (SEM) and Mercury Intrusion Porosimetry (MIP) studies of samples stressed at different stress levels. It has been found that the pore collapse mechanism of artificially cemented Singapore marine clay reported by Kamruzzaman (2002) is very similar to the pore collapse mechanism observed by Delage & Lefebvre (1984) and Delage (2010) on sensitive Canadian clays. The comparative study of the compression behaviour of naturally structured soils and artificially cemented soils presented above further reinforces the proposition of Leroueil & Vaughn (1990) that irrespective of the origin of the structures, mechanical behaviour of different structured soils exhibit a general pattern of behaviour.

Accurate mathematical representation of compression behaviour of naturally structured soil has also been an important area of research (e.g. Hardin, 1989; Liu & Carter, 2000; Chai et al., 2004). Proper mathematical description of the compression behaviour of structured soils is of fundamental importance in practical geotechnical engineering since nearly all settlement estimates of structures are based wholly or partially on one-dimensional compression behaviour (Lambe 1964; Skempton & Bjerrum 1957). Moreover, as pointed by Hardin (1989), a method to accurately predict the one-dimensional strain is also important to research aimed toward the development of three-dimensional constitutive equations for soils. The fact that conventionally used semi-logarithmic or bi-logarithmic relationships between pressure and void ratio are inadequate to describe the compression behaviour of structured soil has been the motivation behind this ongoing research. However, it appears that this aspect of artificially cemented soil has not been as comprehensively researched as it has been done for naturally structured soils possibly due to the apparently linear appearance of the compression curves of artificially cemented soils in a semi-logarithmic space over a limited stress range. Since the compressibility of both naturally structured soils and artificially cemented soils are controlled by similar mechanisms, a generalized compression model of structured soil should ideally be capable of reproducing the compression behaviour of a wide spectrum of structured soils including naturally structured soils and artificially cemented soils. Such a model should also be capable of characterizing the nature of structure/cementation through its parameters. With the view of finding such a generalized compression model, a review of available models is presented and the applicability of those models to describe the compression behaviour of artificially cemented soils is discussed.

The compression models found in geotechnical engineering literature can be broadly grouped into the following categories: (i) linear relationship between void ratio and logarithm of pressure (Terzaghi, 1925), (ii) linear relationship between logarithm of void ratio and logarithm of pressure (Butterfield, 1979) and (iii) different types of power formulations relating void ratio or a related variable with pressure (Janbu,1963; Hardin, 1989; Den Haan, 1992). Lately, another type of power formulation had been introduced by Liu & Carter (2000) where the void ratio of a structured soil at any given stress was divided into two parts: one part corresponded to the equivalent reconstituted soil and the other part was attributed to the soil structure. Two different decay laws were employed to describe the changes in these two components of the void ratio with pressure. For the reconstituted part, a linear relationship was assumed between the void ratio and logarithm of pressure while the additional void ratio associated with soil structure was assumed to decay with pressure according to a hyperbolic function. A single decay parameter, which Liu & Carter (2000) called soil destructuring index, was used in the hyperbolic decay function.

As stated earlier type-(i) and type-(ii) formulations fail to reproduce the virgin compression behaviour of structured soils accurately due to the use of a constant value of the compression index. It was discussed earlier how compressibility of structured soils is controlled by two different mechanisms in two different stress ranges. A capable compression model should therefore include at least two parameters to control the distinct compression mechanisms operating in the de-structuration and de-structured zones respectively. In the model proposed by Liu & Carter (2000), the decay law proposed for the variation of additional void ratio with pressure is equivalent to a linear relationship between the logarithm of additional void ratio associated with structure and the logarithm of pressure. However, from an analysis of a large body of data obtained from literature, it has been found that such linearity is not always observed. Moreover, the formulation of Liu & Carter (2000) requires the availability of information on the compressibility behaviour of reconstituted soils and such data is not available in most of the cases. In addition, the de-structuring parameter used in the Liu & Carter (2000) model has been found to vary considerably with the initial moisture content of the reconstituted soils used for the determination of compression index of the equivalent reconstituted part. Among the compression models available in literature, the generalized power formulation given by Den Haan (1992) has been found to be capable of reproducing the compression behaviour of a wide range of structured soils. The various power formulations proposed earlier have been shown by Den Haan (1992) to be special cases of the generalized power formulation given by the author. The power formulations other than that of Den Haan (1992) can then be expected not to be able to reproduce the virgin compression behaviour of a wide range of soils since they corresponds to certain fixed values of different model parameters of the generalized formulation given by Den Haan (1992). For example, Liu & Znidarcic (1991) reported that the power formulation given by them as well as the one given by Hardin (1989) may not be able to reproduce the compression behaviour of sensitive soils correctly. Although very capable, the formulation given by Den Haan (1992) suffers from the fact that in its most generalized from it includes four different parameters whose determination is a difficult exercise since the parameter determination requires regression analysis to be carried out multiple times based on trial and error. Use of a large number of parameters and considerable level of difficulty associated with their determination are likely to reduce the model's appeal for routine geotechnical analyses.

2.5.2 Shearing behaviour of structured soils

Soil structure has considerable influence on the shearing behaviour of structured soils. Shearing behaviour of structured soils has been found to be strongly influenced by the level of pre-shear confinement since different confining pressures represent different level of de-structuration at the beginning of the shearing. Coop & Atkinson (1993) presented a behavioural framework for the stress-strain behaviour of cemented soils. Their conceptual framework is presented in Figure 2.17. If the soil is consolidated to a stress less than the yield stress, the initial stress-strain response will be controlled by the behaviour of the cementitious bonds and the peak strength will be mostly derived from the resistance offered by the cementation bonds. This proposition is well supported by different experimental observations. Experimental study on the shearing behaviour of artificially cemented soils (Uddin, 1995; Kamruzzaman, 2002; Horpibulsuk et al, 2004) showed that at low level of consolidation pressure, the change of confining pressure does not affect the stress-strain and strength (Figure 2.18) behaviour to any significant extent. This is a clear indication that at low level of consolidation pressure, the strength is controlled by the structure and not by the change in void ratio associated with a change in consolidation pressure. Another notable experimental fact which corroborates the postulate of Coop & Atkinson (1993) is that when a soil with a relatively intact structure is sheared, the peak dilatancy is observed after the peak strength has been reached (Maccarini 1987, Anagnostopoulos et al. 1991, Coop & Atkinson 1993, Cuccovillo & Coop 1997, Schnaid et al. 2001). The presence of structure at the peak strength level inhibits the dilatancy of the soil since the presence of cementitious bonding reduces the mobility of soil particles (Yu et al., 2007).

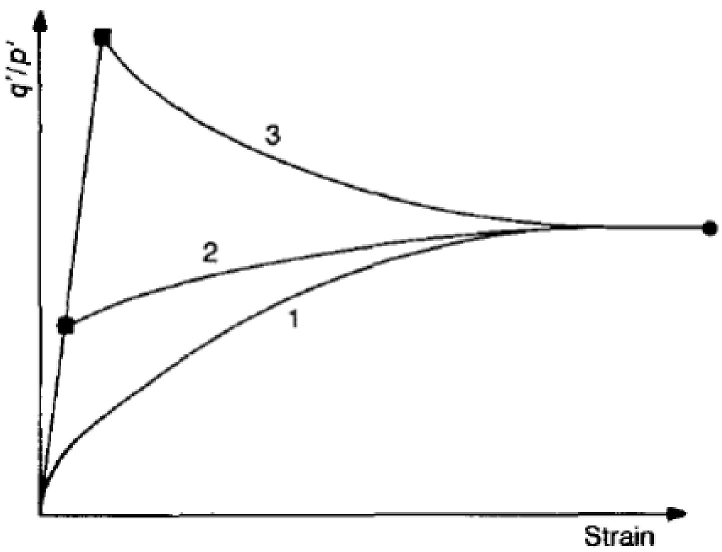


Figure 2.17. Idealized stress-strain behaviour of cemented soils

(from Coop and Atkinson, 1993)

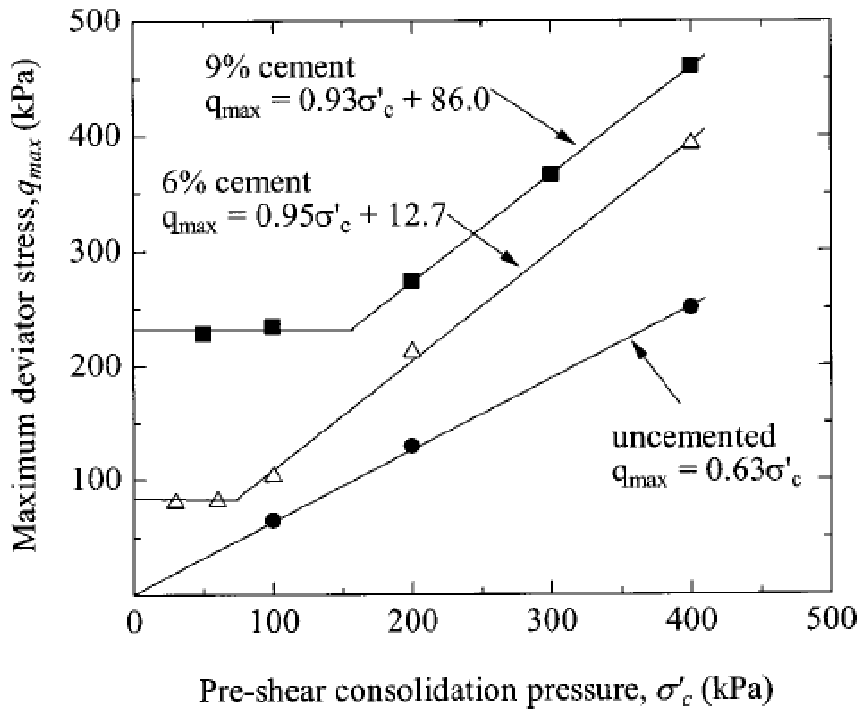


Figure 2.18. Peak strength at different consolidation pressures for Ariake

clay treated with 6% and 9% cement content (from Horpibulsuk et al., 2004)

At low level of confinement the effect of structure is also seen to have prominent effect on the stiffness. Structured soils display an initial very stiff and linear stress-strain behaviour under shearing loads. Elliot & Brown (1985), Airey (1993), Leroueil & Vaughn (1990), and Coop & Atkinson (1993) have suggested that the deformation of the structure up to the limit of the linearity is mostly reversible and the start of non-linearity of the stress-train curves indicates the initiation of breakdown of cementitious structures causing irreversible deformation. The initial stiffness is largely dependent on the type of cementation present and increasing degree of cementation increases soil stiffness (Uddin, 1995; Kamruzaman, 2002; Horpibulsuk et al, 2004; Schnaid et al. 2001).

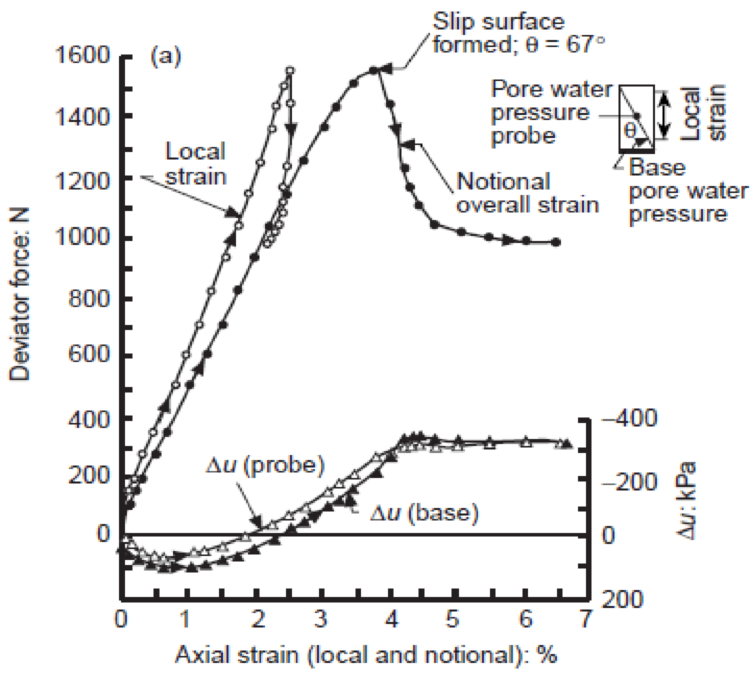


Figure 2.19. Peak and post-rupture strength for an intact sample of Todi clay

(from Burland et al., 1996)

Due to the influence of structures, the shearing response of structured soils at low confinement is characterized by brittleness at peak and strong post-peak dilation of the samples (Elliot & Brown 1985, Coop & Atkinson, 1993). In the post-peak region, the stresses are heavily localized and the stress-deformation condition in the localized zones is significantly different from the global stress-deformation behaviour deduced from the boundary measurements. Figure 2.19 (Burland et al., 1996) presents the stress strain behaviour of an intact sample of stiff Todi clay to highlight the effect of stress localization in structured soils. This figure shows the strain level at which slip surface forms and also the stress-strain behaviour measured at global level and that measured in the vicinity of the slip surface. Due to formation of fractures, the behaviour at large strain is characterized by the relative movement of the rigid blocks of the fractured sample and the characteristics strength under this condition has been described by Burland (1996) as post-rupture strength of the soils. It has been found that a unique strength envelope corresponding to the post-rupture strength could be established for structured soils in the deviator stress vs. mean effective stress space. This post-rupture strength envelope is very close to critical state line at low level of confinement whereas it lies below the critical state line at large pressure due to particle reorientation (Figure 2.20). Airey (1993) reported that although an approximate post-rupture strength envelope can be identified in deviator stress vs. mean effective stress plane for structured soils, defining a unique envelope in the specific volume vs. mean effective stress plane was difficult due to localization of deformations (Figure 2.21).

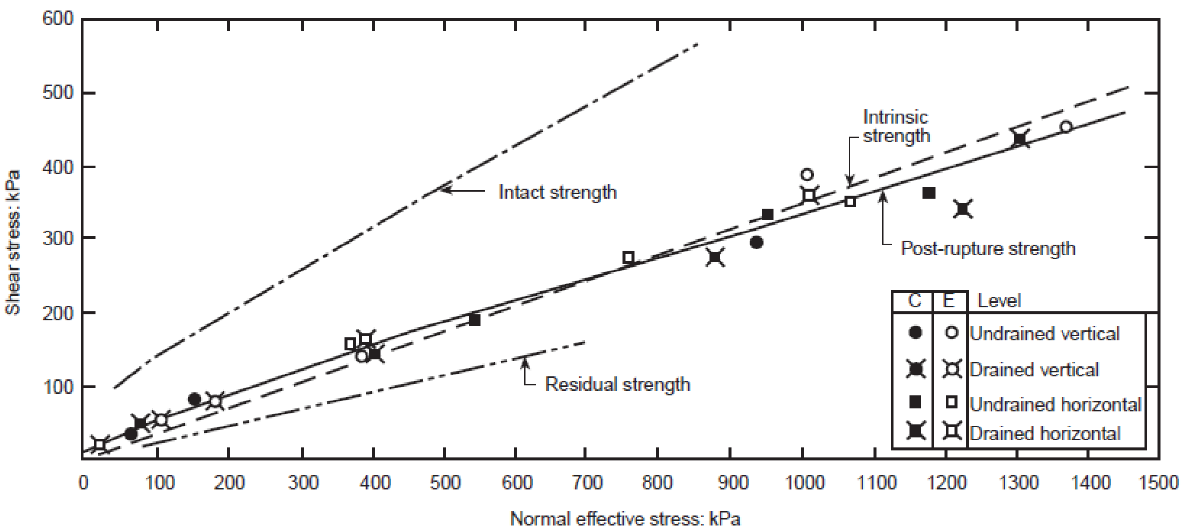


Figure 2.20. Mohr-Coulomb strength envelopes for London clay from Ashford Common (from Burland et al., 1996)

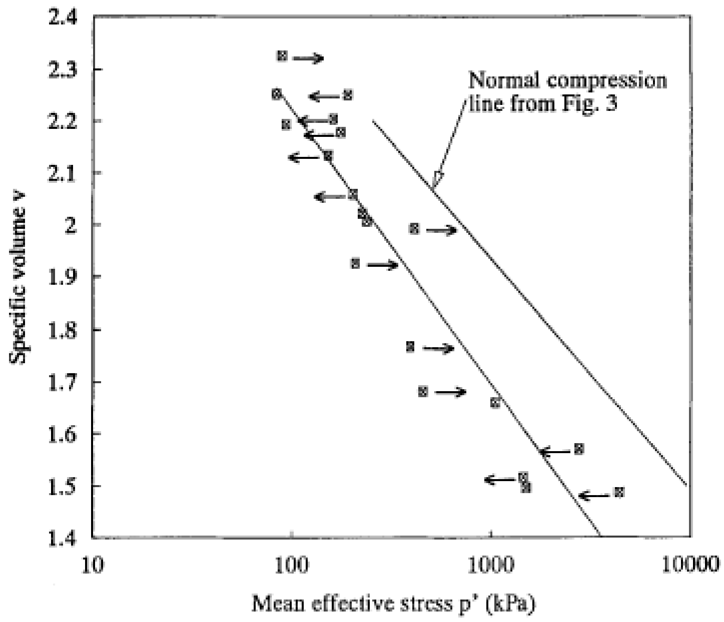


Figure 2.21. End of test points and estimated critical state line

(from Airey, 1993)

Coop & Atkinson (1993) postulated that when the soil is significantly de-structured from the consolidation phase, the shearing behaviour will predominantly be frictional in nature and the idealized stress-strain behaviour corresponding to this type of situation is shown by Case-1 in Figure 2.17 (Coop & Atkinson, 1993). In the intermediate range where the soil is not significantly de-structured from the consolidation phase, the behaviour is controlled by an interaction of cementation bonds and the frictional response of already de-structured soil particles (Case 2 in Figure 2.17). Uddin (1995), Horpibulsuk et al. (2004) and Kamruzzaman (2002) have shown that at elevated consolidation pressure, the stress-strain and strength behaviour is significantly influenced by a change in confining pressure. This observation indicates that at large pressure the effect of variation of void ratio has significant influence on the shearing response. Porbaha et al. (2000) also suggested that the mode of failure is strongly affected by the level of confinement and with an increase in the confinement level the shearing response gradually becomes ductile. Azman et al. (1994)

also concluded that at confinement level in excess of yield stress of the treated soil, the stress-strain relationship is no longer characterized by brittleness.

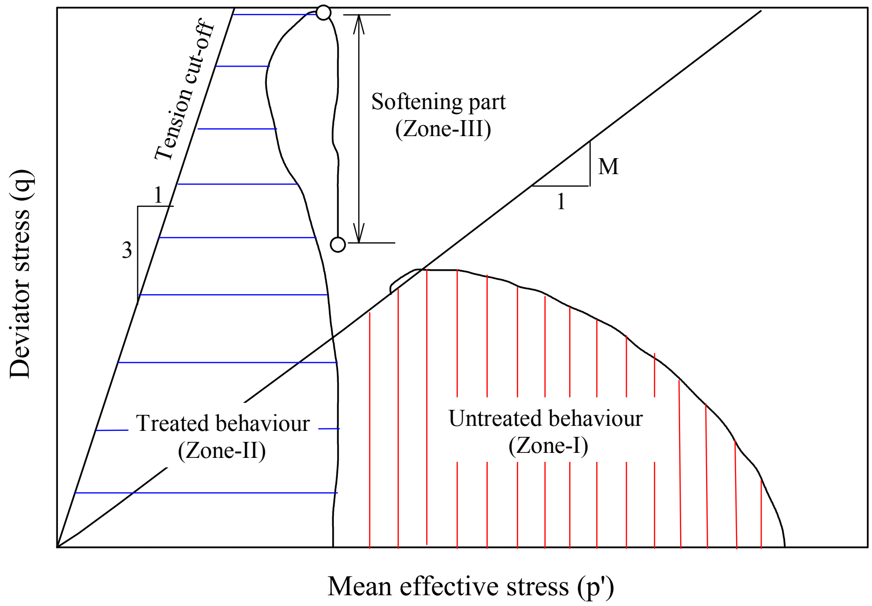


Figure 2.22. Undrained behaviour of cement treated clay in deviator stress vs. mean effective stress space (replotted from Bergado et al., 1996)

Bergado et al. (1996) presented a behavioural framework for the undrained shearing behaviour of artificially cemented soils based on the comprehensive experimental investigation on the shearing behaviour of cement treated Bangkok clay. The behavioural framework proposed by Bergado et al. (1996) is presented in Figure 2.22. According to this framework, the shearing response of cemented soils can be subdivided into three different zones in a deviatoric stress vs. mean effective stress space. In the Zone-II where the confinement level is low, the effect of treatment is the most prominent. The left boundary of the stress-path is marked by the tension cut-off line. At low level of confinement the stress path rises almost vertically to the peak strength implying highly elastic behaviour. Once the peak strength is reached the soil samples develop fractures and become unstable. The stress path falls almost along a vertical path to a strength envelope which is above the critical state line of the original soil. The strength envelope corresponding to the large strain condition has been identified by Bergado et al. (1996) as the de-structured strength envelope. The slope of the de-structured strength envelope increases with the increase in cement content and curing time. On the other hand when the confining pressure increases, the structure is destroyed significantly from the consolidation phase of the loading causing the behaviour of the de-structured material to be very close to that of untreated material. The difference in the behaviour of de-structured material and that of the original untreated material is dependent on the degree of the mineralogical alteration of the original soil caused by cement treatment.

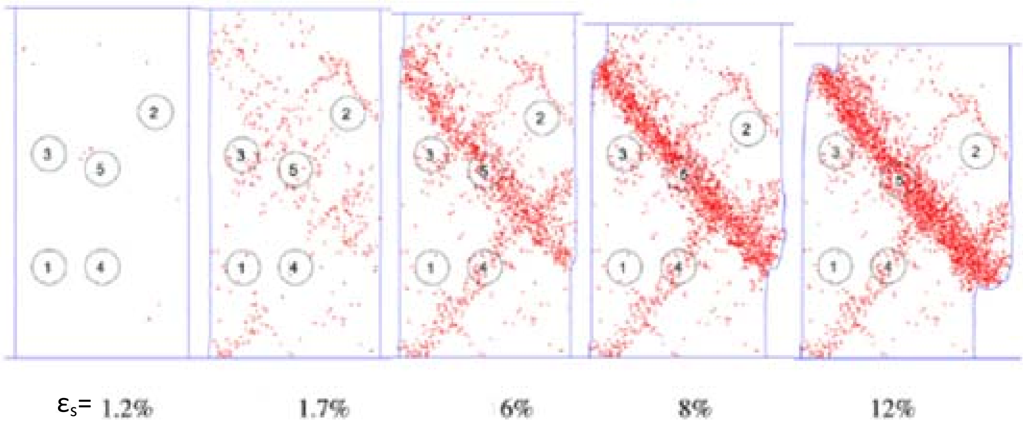


Figure 2.23. Distributions of bond breakage in the two bonded samples at different axial strains at σ3 = 100 kPa (from Jiang et al., 2011)

Elliot & Brown (1985) also suggested that the shearing behaviour of cemented soft rock can be divided into three different types of behaviour: brittle, transitional and ductile depending on the level of pre-shear confinement and the brittleness of stress-strain response decreases with an increase in confining pressure.

The stress-strain behaviour discussed so far has also been well substantiated by the numerical simulation of bi-axially stressed assemblage of bonded soil particles by DEM method (Figure 2.23). Jiang et al. (2011) showed that when an assemblage of bonded soil particles is sheared, the initial shearing response is significantly influenced by the constitutive models used to describe the behaviour of the cementitious bonds and also by the properties assigned to the cementitious bonds. With the progression of shearing, micro-cracks from due to the breakdown of bonds and the density of the cracks gradually increases giving rise to macro-cracks at different level of strains depending on the confinement. Jiang et al. (2011) observed that at low level of confinement, significant localization of stress takes place near the vicinity of the peak stress and the stress and deformation taking place within those localized zones differ significantly from the global stress-deformation deduced from the boundary measurements. Due to the formation of major cracks, the deformation in the post-peak region is governed by the response of the separated clusters of bonded particles within the slip surfaces. At low level of confinement, these clusters of bonded particles behave like individual soil grains and give rise to strong dilation due to absence of appreciable confinement. On the other hand with an increase in confining stress, the samples were found to deform more homogeneously and the detached clusters of bonded particles were not able to give rise to dilation due to presence of significant level of confining stress.

2.5.3 Volumetric response under shearing

For stabilized soils, the volumetric behaviour in response to shearing is found to be dependent on the level of confining pressure. Uddin (1995) reported that at low level of confinement, stabilized soils exhibit strong volumetric dilation under drained shearing. On the other hand, for samples sheared from low level of confinement under undrained condition, a noticeable reduction of excess pore water pressure takes place after an initial positive pore pressure development (Uddin, 1995; Kamruzzaman, 2002). This type of pore pressure response of structured soil is similar to the behaviour of mechanically overconsolidated reconstituted soils. Although the behaviour up to the peak stress level is characterized by the frictional response in the case of overconsolidated reconstituted soils and by the behaviour of the cementation bonds in the case of cemented soils, the post peak behaviour for both the cases is characterized by the presence of strong discontinuities. Due to lack of confinement, the particles within the fracture planes tends to increase in volume under continued shearing which is manifested as either an increase in volume under drained shearing or a decline in excess pore water pressure under undrained shearing. Although the pore pressure behaviour at low level of confinement is similar for mechanically overconsolidated reconstituted soils and cemented soils, the pore pressure behaviour at higher level of confinement is significantly different.

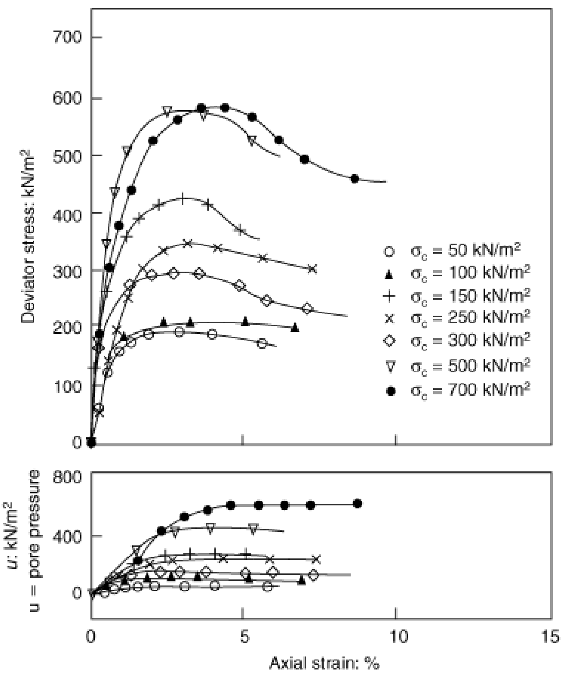


Figure 2.24. Undrained triaxial compression tests on marine clay mixed with 5% cement (from Endo, 1976).

When reconstituted soils are sheared from elevated confining pressure, the pore pressure is seen to increase with the increase in shear strain and the increase in pore pressure is accompanied by strain hardening behaviour of soil. On the other hand, when cemented soils are sheared from a level of consolidation pressure that can not completely eliminate the structure, some surprising shearing response is observed (Yu et al., 2007).

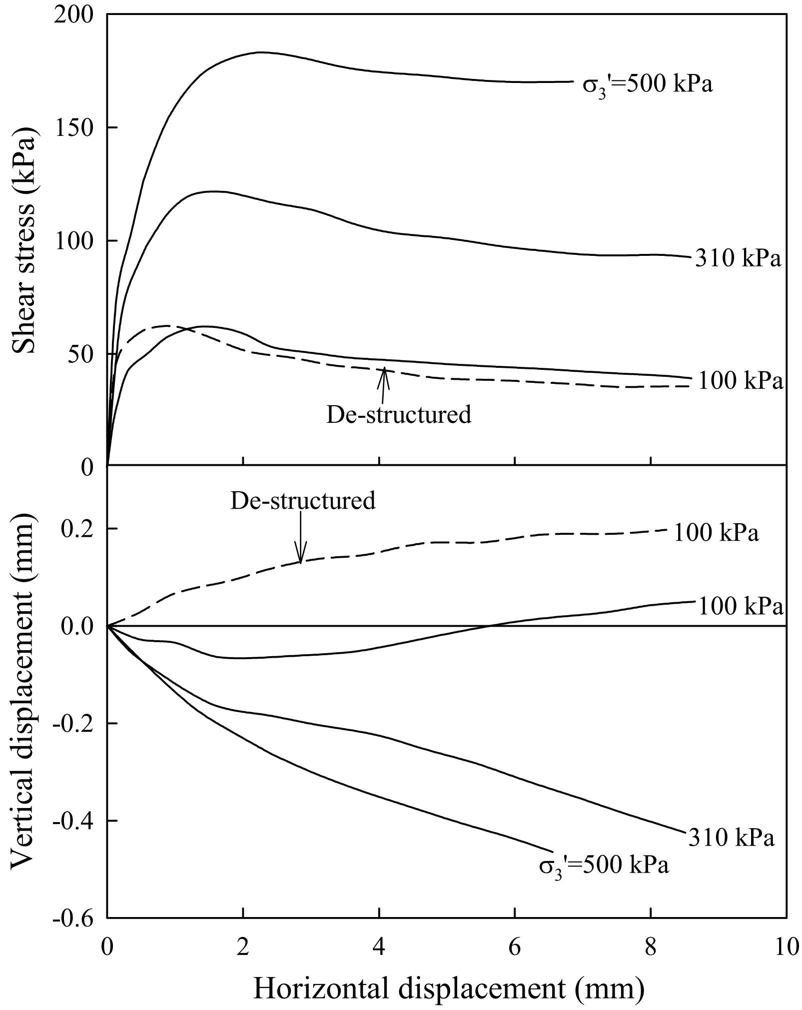


Figure 2.25. Drained direct shear tests on intact London clay

(replotted from Georgiannou et al. 1993)

At this intermediate level of consolidation pressure, post-peak strain softening is frequently accompanied by an increase in pore water pressure under undrained shearing (Figure 2.24) and compressive volumetric strain under drained shearing (Figure 2.25). Nagaraj & Miura (2001) argued that although the volume of the structured soil tends to decrease in the post-peak region of straining due to de-structuration of the soil, the loss of strength due to breakdown of cementitious structures can not be compensated by the incremental contribution of mobilized friction of already de-structured soil particles. As a result, the softening is seen to be associated with volumetric compression in drained shearing and with an increase in pore water pressure under undrained shearing. Yu et al. (2007) argued that since the overall plastic modulus in the post-peak region is made up of two competing components (hardening due to reduction of void ratio and softening due to removal of cementitious structures), the overall stress-strain response and volumetric behaviour will be determined by the net effects of these two factors and therefore, strain softening can in some cases be accompanied by volumetric compression under drained shearing or by the development of positive excess pore pressure under undrained shearing.

2.5.4 Stiffness and strength parameters

The earlier researches almost invariably found that the peak strength and elastic stiffness improve as a result of artificial cementation depending on the type and quantity of cementing materials used. It has also been found that the shear strain at failure decreases with an increase in the degree of cementation (Probaha et al., 2000). Although there is a general agreement among the earlier researches about the influence of cementation on the strength, stiffness and failure strain, the observation on the strength parameters is not conclusive. Wissa et al. (1965) reported that the residual strength envelope is independent of the amount of cementation and it can be described by a single strength envelope. Clough et al. (1981), based on the studies on artificially and naturally cemented soils, concluded that the slope of the failure envelope does not change to any appreciable extent due to cementation but the cohesion intercept increases with an increase in the degree of cementation. Kasama et al. (2000) carried out consolidated undrained triaxial compression tests on cement-admixed Ariake clay and concluded that the failure line for cemented soil is parallel to that of un-cemented soils but the cohesion intercept increases with an increase in the degree of cementation. Consoli et al. (2007) found that friction angle is almost independent of cement content whereas cohesion increases with an increase in the degree of cementation. Contrary to the above observations, Uddin (1995) and Kamruzzaman (2002) found that the friction angle significantly increases with an increase in the cement content. Horpibulsuk et al. (2004) found that both the cohesion intercept and the friction angle corresponding to peak stresses increase with an increase in cement content. Horpibulsuk et al. (2004) also found that the cohesion intercept at residual state was zero for all the cement contents but the residual friction angle was found to increase with increasing cement content. Uddin (1995) suggested that the peak strength envelope is curved and at large confining pressure the failure envelope tends to merge with the envelope of untreated soil. Shibuya & Ozawa (1996) also found that it was difficult to characterize the failure points by a single straight line failure envelope.

2.5.5 Constitutive modeling of structured soils

The influence of structure on the mechanical behaviour of structured soil is also evident from the fact that the unusual behaviour of structured soils, such as strain softening in conjunction with volumetric compression, can not be reproduced by the classical soil mechanics theories which were developed based on the behaviour of reconstituted soils observed in the laboratory (Schnaid et al., 2001). Although not routinely used in the geotechnical analyses, many constitutive models that can reproduce the mechanical behaviour of cemented soils have been proposed (Gens & Nova, 1993; Lagioia & Nova, 1995; Kavvadas & Amorosi, 2000; Rouainia & Muir Wood, 2000; Nova et al., 2003; Baudet & Stallebrass, 2004). Gens & Nova (1993) have shown that in order to successfully reproduce the behaviour of structured soils, the classical plasticity based theories need to be modified to include the effect of structure. They showed that a simple modification of classical elasto-plastic theories is possible by including the effect of structure on the yielding (Figure 2.26), hardening and flow behaviour of structured soils. A suitable de-structuration law is required to describe the progressive change of structure and the effect of progressive de-structuration needs to be incorporated in the subsequent hardening and flow behaviour of the continuously de-structuring material. Based on the concept of Gens & Nova (1993), many constitutive models have been proposed with different degrees of sophistications. These models (e.g., Yu et al., 2007) have in general been found to be capable of reproducing the main features of the mechanical behaviour, including some of the surprising behaviour described earlier, of naturally structured soils as well as of artificially cemented soils.

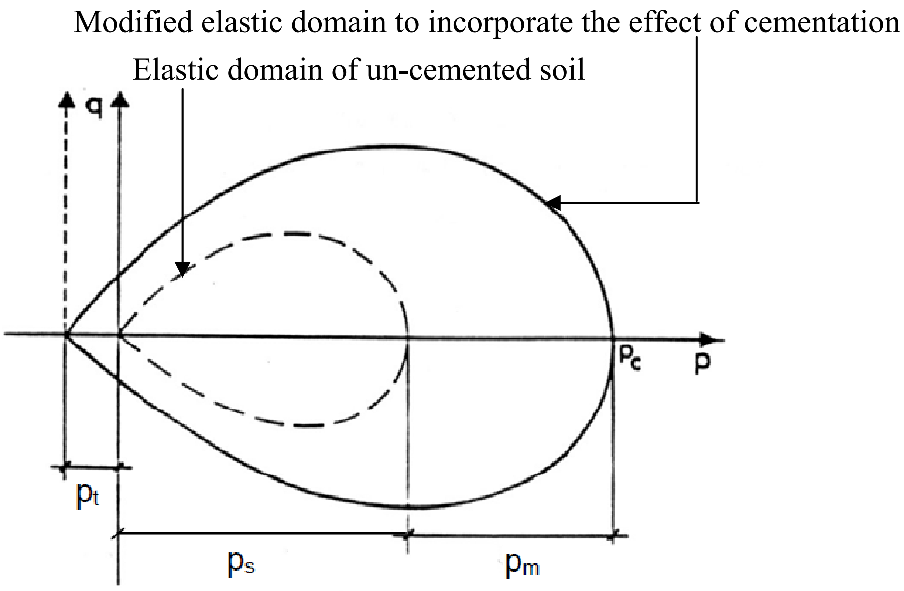


Figure 2.26. Modification of elastic domain due to cementation

(from Gens and Nova, 1993)

2.6 Previous Researches on Coode Island Silt

The research on improving the engineering properties of CIS by using chemical additives had been started in the early 2000s at Monash University and is still going on. Several of the past investigations in this area are briefly reviewed in the next section.

2.6.1 Soon (2003)

From the review of available literature, it appears that Soon (2003) was the first to explore the possibility of improving soft CIS through cementitious stabilization. The objective of that investigation was mainly to explore the effectiveness of cement in improving the properties of CIS taken from different locations within Melbourne. Different combinations of flay-ash/ cement and GGBFS/cement were also used to investigate the effectiveness of different types of binders. It was reported by Soon (2003) that the strength and stiffness increased invariably with an increase in cement content and curing period and the strength was found to decrease with an increase in water content. All these results are consistent with the results of other investigations described earlier. In this study, the effects of partial substitution of cement with fly-ash and GGBFS on the UCS values were also investigated. It was found that when 25% of the cement was replaced by fly-ash, the strength developed at 28 days of curing was less than the strength observed for CIS treated with cement alone. However, at 25% replacement, the strength achieved at 270 days of curing with the cement and fly-ash combination was found to be comparatively higher. On other hand, a replacement of 50% of the cement by fly-ash produced UCS values less than 50% of the value achieved for treatment with cement alone. Therefore, it is clear from this result that there is an upper limit on the amount of cement that can be replaced by fly-ash. This is logical since fly-ash can not continue the cementation reactions without the presence of $Ca(OH)_2$ and the strength characteristics observed with 50% replacement ratio may be due to the fact that the amount of $Ca(OH)_2$ generated from the hydration reaction of cement was not adequate to react with the added quantity of fly-ash. On the other hand GGBFS was found to perform better than fly-ash as a replacement material. It was reported by Soon (2003) that when cement was replaced by GGBFS even up to as high as 75%, the strength developed at 28 days and beyond was comparable to the strength observed for the case where CIS was treated with cement only. It was discussed earlier that once activated, GGBFS can continue its own hydration reactions with its own $Ca(OH)_2$. For GGBFS: cement ratio of 75:25, the amount of $Ca(OH)_2$ generated from the hydration of 25% cement was possibly enough to activate the 75% GGBFS. As a result the hydration reactions was not adversely affected as it was for the case where cement was replaced with fly-ash for which the presence of lime in adequate quantity at all time is a mandatory requirement for cementation reactions to progress.

2.6.2 Rex et al. (2008)

In this project, soft CIS was treated with lime-activated GGBFS. The CIS sample used in this project was taken from a site in the Docklands precinct from a shallow depth of approximately 5.0m. The results from this investigation are presented in Figure 2.27. It is found that a lime content as low as 5%, when used in combination with 10 to 15% slag, was adequate to improve the strength properties of CIS significantly. It was found that higher slag content invariably produced higher strengths at all curing periods whereas lime was found to have comparatively much lower influence on the strength development and the influence of lime was also found to be affected by the quantity of slag used.

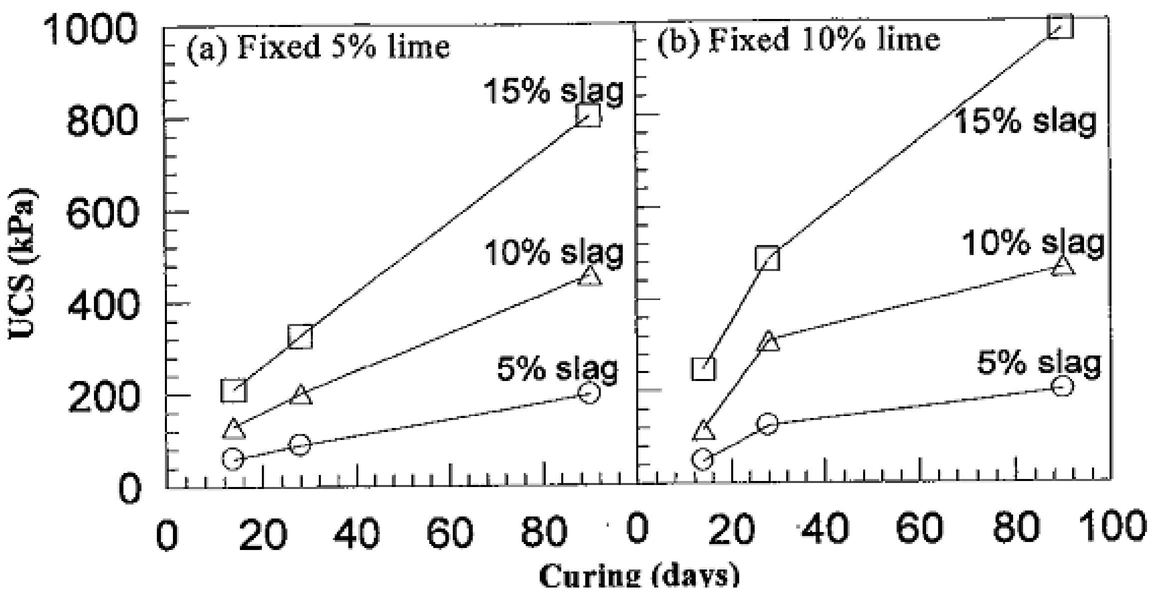


Figure 2.27. UCS of lime-slag treated CIS (from Rex et al., 2008)

2.6.3 Stanley (2010)

Bulk samples for this study were taken from a construction site of West Gate Freeway upgrade project (junction of Lorimer and Montague Street) from a depth of 12m. In sharp contrast to the results reported by Rex et al. (2008), Stanley (2010) found that CIS sampled from this particular location did not show any noticeable improvement of strength when treated with lime contents of 5 to 10% along with different amounts of slag (Figure 2.28a and b). However, it was found that the same CIS responded well to a dose of 15% lime when combined with different amounts of slag (Figure 2.28c). Stanley (2010) found that the CIS sampled from 12.0m depth contained 3% pyrite, a sulphate bearing phase. Based on the mineralogical investigation of treated lime-slag treated CIS, it was concluded that pyrite interfered with the cementitious reactions considerably. When oxidized and subsequently mixed with water, pyrite creates an acidic environment and reduces the pH of the reaction environment. Due to lowering of the pH, the cementitious reactions can not progress. Stanley et al. (2011) concluded that when pH of the reaction environment drops, C-S-H, the main strength enhancing hydration product, transforms into a deleterious reaction product thaumasite in the presence of aluminium and carbonate.

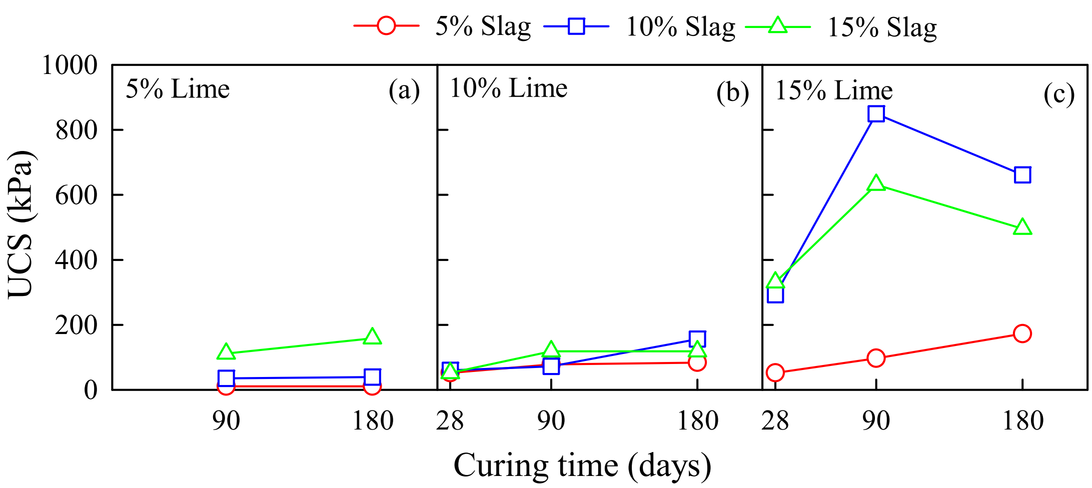


Figure 2.28. Variation of UCS of CIS treated with different combinations of lime and slag (replotted from Stanley, 2010)

2.7 Summary

CIS is a soft sedimentary soil possessing extremely low shear strength and very high compressibility. The problem with this soil is further complicated by the fact that various engineering properties (e.g., plasticity, shear strength, stiffness and compressibility) as well as mineralogical composition of CIS display a wider range of scatter making its characterization a difficult task. To avoid any potential risk, conventionally expensive pile foundations extended to Melbourne mudstone layer is used for supporting even lightly to moderately loaded structures built on lands underlain by this problematic soft soil. Moreover, projects involving bored piles or deep excavations require expensive treatment of a large amount of excavated materials prior to its safe disposal.

Limited studies on the stabilization of CIS with various cementitious additives have been found to be promising. Considering the issues associated with the conventional practice and the promise shown by the preliminary researches on the cementitious stabilization of CIS, there is an immediate need for maturing the cementitious stabilization technique for its use as a potential cost-effective and environment friendly alternative to traditional foundations solutions in infrastructure projects built on CIS deposits.

From the review of literature, it has been found that mechanical behaviour of naturally structured soils as well as that of artificially cemented soils is much more complicated than laboratory prepared reconstituted soils. Advanced mechanical characterization of the geomaterial is a prerequisite for accurate load-deformation analysis of any geo-infrastructure. Therefore, there is a need for undertaking elaborate laboratory investigations to gain an in-depth understanding on the mechanical behaviour of the treated CIS. Previous researches on the cementitious stabilization of CIS have been mainly based on the results of Unconfined Compressive Strength (UCS) tests and other simple index tests. UCS can also be considered merely an index test which does not generate information required for advanced geotechnical analyses of real life projects. Considering the limitation of earlier studies on the cementitious stabilization of CIS, an in-depth experimental investigation on the detailed mechanical characterization of the stabilized CIS is proposed in this project. It is important to mention at the outset that due to naturally occurring spatial variability of the properties of CIS, the knowledge gained from this research project can not be directly applied to real-life projects. However, the knowledge generated from this project will help the geotechnical practitioners to anticipate the type of mechanical behaviour the stabilized materials may exhibit.

In addition to studying the mechanical behaviour of stabilized CIS, developing a simple compression model for structured soils has been selected as a major focus of the current project since the review of literature revealed the need for the development of such a model. It was shown that the compression behaviour of naturally structured soils and that of artificially cemented soils are controlled by similar mechanisms. From the discussion of available models, it was demonstrated that relatively simple models are not capable of reproducing the compression behaviour of both naturally structured soils and that of artificially cemented soils. On the other hand, more advanced models require the use of large numbers of difficult-to-determine parameters. From the discussion of different available models it is obvious that there is a need for developing a simple compression model which can reproduce the compression behaviour of a wide range of structured soils accurately by using model parameters that can be easily determined. To address the above need, development of a compression model which can simulate the compression behaviour of both naturally structured soils and different types of artificially cemented soils by employing easily determinable yet meaningful parameters has been selected as a key area of the proposed research.

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CHAPTER 3: EXPERIMENTAL INVESTIGATIONS

3.1 Introduction

Under the current research, a comprehensive testing program is designed to investigate the effectiveness of cementitious stabilization for the improvement of strength and deformation properties of pyrite bearing soft Coode Island Silt (CIS). The experimental investigation consists of three different types of tests for studying the mechanical behaviour of the stabilized soft CIS. The first phase of the experimental investigation deals with the study of strength development of treated CIS through Unconfined Compressive Strength (UCS) tests. In this part of the investigation, the effect of additive contents and curing periods (1, 3 and 6 months) are investigated. The second phase of this investigation deals with the study of the compressibility characteristics of the stabilized soil through one-dimensional compression tests on samples treated with different combinations of additives and cured for 1,3 and 6 months. The last phase involves studying the undrained shearing behaviour of CIS treated with different quantities of additives and consolidated to a wide range of consolidation pressures. For the triaxial testing program the combinations of binders is narrowed down to only three different combinations and a single curing period based on the results of UCS testing program. In the following sections, the details of the materials used, the specimen preparation methods employed and the details of the experimental program are described.

3.2 Selection of binder for the current project

In selecting a binder the main criterion is the ability of the binder to meet the performance requirements in terms of strength and compressibility properties. However, economic and environmental considerations also play important roles in the selection of suitable binders.

Considering all of these factors, Ground Granulated Blast Furnace Slag (GGBFS) appears to be an attractive choice in Australian context. An enormous quantity (about 3 million tons per year) of GGBFS is annually generated as by-product from the iron and steel manufacturing industries in Australia (Gregory & Jones, 2005). In Australia, GGBFS is mainly used for manufacturing aggregates and as a replacement of clinker in the production of slag-cements. However, Gregory & Jones (2005) reported that GGBFS is underutilized in Australia (utilisation of GGBFS products in Australia at the time of reporting was around 69%). An increased use of GGBFS in soil stabilization can reduce the demand for cement and thereby can offer significant economic as well as environmental benefits since cement manufacturing industry is one of the major CO_2 emitters (Gregory & Jones, 2005). Moreover, the use of GGBFS in place of cement will reduce the demand for naturally occurring resources required for production of cement. Unlike naturally occurring materials, resource that gets progressively exhausted, GGBFS production represents a continually renewable supply of material that is dependant only on continuance of the iron and steel manufacturing industry. Under the current scenario, where the demand for steel and iron are increasing continuously, especially driven by the increasing demand from China and India, two of the fastest growing economies in the world, the steel and iron industries in Australia are certain to become more active in the coming days. Increased production of iron and steel implies a higher production of GGBFS. Therefore, there is a need for finding new ways to make use of GGBFS. It is therefore obvious that, from an Australian perspective, GGBFS appears to be a candidate of choice as a cementitious binder in ground improvement works provided it can impart desirable properties in the improved soil. Earlier researches carried out at Monash University on the improvement of CIS with lime-activated GGBFS demonstrated that this binder is certainly capable of meeting the performance requirements. Since the use of

GGBFS for treating CIS has been found to be promising and increased use of this industrial by-product can bring significant economical and environmental benefit to Australia, immediate attention should be directed towards advancing the use of GGBFS in cementitious stabilization of soft soils. Considering all these factors, GGBFS has been chosen as the primary cementing binder in the current project.

3.3 Materials

3.3.1 Coode Island Silt (CIS)

CIS is widespread throughout the Yarra Delta region, South Melbourne, Port Melbourne and Footscray. Although it is a predominantly a silty clay, it also includes bands of clayey silt, sandy silt and sandy clay. The physical characteristics of CIS as well as its engineering properties vary highly with location and depth. A detailed description of the CIS has been presented in Chapter 2 under section 2.2.

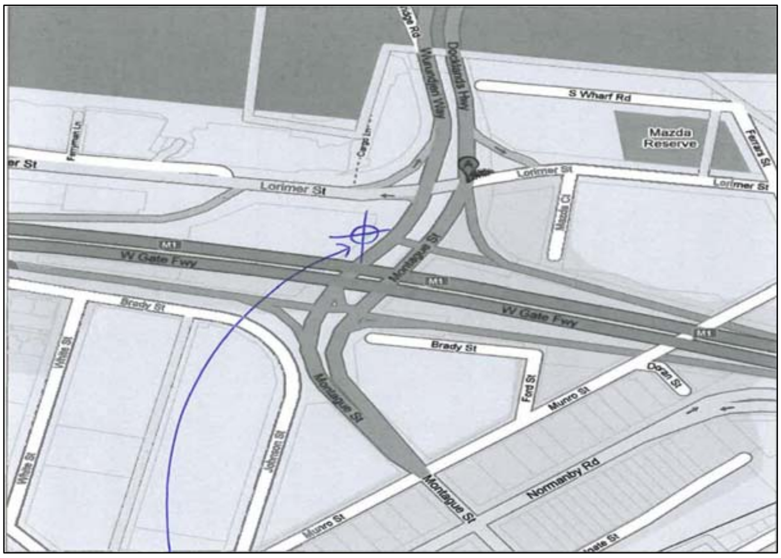

Figure 3.1. Location of CIS sample collection

Bulk samples for this experimental study was taken during the piling work at the construction site of West Gate Freeway upgrade project (junction of Lorimer and Montague Street, Figure 3.1) from a depth of approximately 12m. A typical geological section of the sampling area can be found in VandenBerg (1997). The samples were collected from one bore hole to minimise variability in soil content and about 100kg of soil was collected. The collected soil sample was transferred to Monash University Civil Engineering laboratory and stored in sealed boxes in the laboratory to preserve the original condition of the soil as much as possible.

The Atterberg limits of untreated CIS indicated that it was highly plastic clay (LL=83, PL=42, PI=41). The natural moisture content was estimated to be around 68% (some loss in moisture can be expected during transportation and storing). All these estimated values fall within the range of CIS data reported by Ervin (1992) for different locations. The mineralogical composition of CIS used in this investigation is given in Table 3.1.

Table 3.1. Mineralogical composition of CIS

Mineral	Contents (%)
Quartz	24
K-feldspar	2
Na/Ca-feldspar	3
Mica/ Illite	10
Kaolinite	23
Smectite	32
Sulphide	4
Anatase	1
Halite	1

3.3.2 Hydrated Lime

Hydrated lime was used in this project for activation of GGBFS. The properties of the hydrated lime used in this project are given in Table 3.2.

3.3.3 Ground granulated blast furnace slag (GGBFS)

Slag, a latent hydraulic binder, produces cementitious reaction products when activated by a suitable activator such as lime. The chemical composition of the slag used for the current project is given in Table 3.2.

Table 3.2: Properties of hydrated lime and GGBFS (as provided by the suppliers)

Constituents	Hydrated lime	GGBFS
SiO_2 (%)	1–2	35–37
Al_2O_3 (%)	0–2	13.5
MgO (%)	-	5.9
$Mg(OH)_2$ (%)	0.5–1.5	-
CaO (%)	-	41–43
$Ca(OH)_2$ (%)	85–95	-
Fe_2O_3 (%)	0–0.7	0.3
SO_3 (%)		2.9
MnO (%)		0.4
Loss on ignition (%)		0.9
Specific gravity	2.1–2.3	2.8–3.1
pH	≈12	
Fineness index (m^2/kg)		430.0

3.4 Preparation of samples

The testing program requires preparation of a large number of samples to be tested at different curing periods. In order to study the change in properties with different additive contents and at different curing periods, the variation among the samples needs to be minimized as much as possible.

The raw CIS was initially mixed in a mechanical mixer with the amount of water required to achieve a moisture content of 100%. The required amount of hydrated lime and GGBFS was mixed subsequently in another mixer with a water/(lime + slag) ratio of 1 for about 5 minutes to form a uniform slurry. The additive slurry was then transferred to the mixer in which the raw CIS was being mixed with water. An additional amount of water was added to the mix to achieve a final moisture content of 130% (based on the dry mass of the soil + lime + slag mix). An overall moisture content of 130%, which is approximately two times the liquid limit of the natural CIS, was used to improve workability of the mix required for the preparation of high quality samples. Trials with lower water content resulted in nonuniformity of the samples. The CIS and the additives were continuously mixed in the mechanical mixer for about half an hour to ensure that the final mixture was homogeneous.

After completing the mixing, the slurry was put in PVC moulds of 50mm diameter and 110mm height. In order to ensure easy extraction of the samples from the moulds after the required curing periods, a thin layer of petroleum jelly was applied on the inside wall of the moulds. Before putting the slurry into the mould the bottom of the mould was enclosed by attaching a plastic sheet. The slurry was put into the mould by using a spatula in three layers and after putting each layer, vibration was applied on the outside wall of the mould to remove entrapped air bubbles from the slurry already deposited in the mould. When the moulds were completely filled with slurry, the top ends were covered with another plastic sheet. At that stage all the samples were clearly marked and were put in plastic bags which were then transferred to the humid chamber for solidification of the samples by the process of cementation. The initial moisture content of the prepared mix was determined by drying it in an oven at 105^0C for 24 hours. Every effort was made to maintain uniformity in the process of placing the treated soil slurry in the PVC moulds. It was found that the mass of all the finished samples inclusive of PVC moulds at the initial condition was within the range of 385 ± 5gm and the moisture content of the samples was within the range of 130 ±3.

3.5 Laboratory testing

3.5.1 Initial Consumption of Lime (ICL) test

The optimum amount of lime to be used in this study was determined by carrying out Initial Consumption of Lime (ICL) test (BS 1924: Part 2:1990). It has been reported by Stanley (2010) that a lime content of 5% or less is ineffective in improving the strength of lime-slag treated pyrite bearing CIS. Based on the recommendation of Stanley (2010), the lowest level of lime content to be used in ICL test was fixed at 6%. The amount of lime was varied from 6% to 15% and the resulting pH was measured.

3.5.2 Unconfined Compressive Strength (UCS) test

At the end of pre-determined curing periods, the samples were extruded from the PVC moulds by pushing the samples with thumbs from one end. The extruded samples were then trimmed to approximately 100mm length and the moisture contents of the samples were determined by oven drying the trimmings. For each combinations of additive and curing time, at least two samples were tested. When the observed strength varied by more than 10%, a third sample was tested and in those cases the results of the two tests with the least amount of variation was considered for reporting. The trimmed samples were loaded in unconfined condition using a load frame. The applied load and resulting displacements were measured using a load cell and a Linear Variable Displacement Transducer (LVDT) respectively. The data was recorded using GDSLab software with the help of a GDS data acquisition system. The specimen was sheared at a strain rate of 1mm/min (1% per minute) based on BS 1377-part 7 (1990). The details of the test variables are given in Table 3.3.

Table 3.3. Details of UCS testing program

Test type	Curing time (Months)	Additive content (% of dry mass of CIS)	
		Lime (%)	Slag (%)
Unconfined compressive strength tests (UCS)	1	10	10, 15, 20
		15	10, 15, 20
		20	10, 15, 20
	3	10	10, 15, 20
		15	10, 15, 20
		20	10, 15, 20
	6	10	10, 15, 20
		15	10, 15, 20
		20	10, 15, 20

3.5.3 Oedometer consolidation test

One-dimensional compression test was carried out on the treated samples at 1, 3, and 6 months of curing. Corresponding water contents of the samples can be found in Figure 4.4. The tests were carried out according to AS1289.6.6.1. LoadTrac-II (from Geocomp Corporation, USA; shown in Figure 3.2), a fully automated consolidation testing system having a maximum pressure capacity of 10000 kPa (based on a 50mm diameter sample), was used for the consolidation tests. In all of the tests the samples were subjected to maximum stress of 8000 kPa. The LoadTrac-II system consists of a consolidation cell to retain the specimen in place, a loading mechanism, load and displacement transducers, a microprocessor for test control and data acquisition software ICON which also includes an interface to set the test control parameters. A standard stainless steel consolidation ring of size 50mm (diameter) x 20mm (height) was used for all the tests. The sharp edge of the ring was pushed against the samples extruded from the PVC mould and then both ends of the sample were carefully trimmed. Before pushing the ring against the extruded sample, the inside of the ring was lubricated by applying very thin layer of petroleum jelly on the inside of the ring. In LoadTrac-II, the systems takes total control over the test once the test is started after putting the soil specimen in place and specifying the appropriate control parameters. The end of primary consolidation associated with a particular loading step is automatically detected by the system. When the displacement of the sample under a specific load step does not change over a period of five minutes, the system identifies the condition as end of primary consolidation for that particular load step and applies the next prescribed stress immediately. Once the test is finished, the software can be used to automatically generate reports including different consolidation properties of the tested specimen and associated compression curves. For the tests conducted at one month of curing, a load increment ratio of 1 was used. However, at 3 and 6 months of curing, it was found from trial tests that if a load increment ratio of 1 is used, the resulting compression curve does not accurately produce the compressibility behaviour in the vicinity of the yield stress. At 3 and 6 months of curing, an incremental stress of 200 kPa (selected based on trial and error) was applied from much before the expected yield stress and up to 3200 kPa in order to obtain compression curves having smooth transition in the vicinity of the yield stress. Starting from 3200 kPa, a larger incremental stress was applied. The details of one-dimensional consolidation testing program are given in table 3.4.

Figure 3.2. LoadTrac-II testing system

Table 3.4. Details of oedometer consolidation testing program

<table>
<tr><th rowspan="2">Test type</th><th rowspan="2">Curing time (Months)</th><th colspan="2">Additive content</th><th rowspan="2">Load steps (kPa)</th></tr>
<tr><th>Lime</th><th>Slag</th></tr>
<tr><td rowspan="9">Oedometer consolidation test</td><td rowspan="3">1</td><td>10</td><td>10, 15, 20</td><td>12.5, 25, 50, 100, 400, 800, 1600, 3200, 6400, 8000</td></tr>
<tr><td>15</td><td>10, 15, 20</td><td>-do</td></tr>
<tr><td>20</td><td>10, 15, 20</td><td>-do</td></tr>
<tr><td rowspan="3">3</td><td>10</td><td>10, 15, 20</td><td>12.5, 25, 50, 100, 200, 400, 600, 800, 1000, 1200, 1400, 1600, 1800, 2000, 2200, 2400, 2600, 2800, 3000, 3200, 4000, 5000, 6000, 7000, 8000</td></tr>
<tr><td>15</td><td>10, 15, 20</td><td>-do</td></tr>
<tr><td>20</td><td>10, 15, 20</td><td>-do</td></tr>
<tr><td rowspan="3">6</td><td>10</td><td>10, 15, 20</td><td>-do</td></tr>
<tr><td>15</td><td>10, 15, 20</td><td>-do</td></tr>
<tr><td>20</td><td>10, 15, 20</td><td>-do</td></tr>
</table>

3.5.4 Isotropically Consolidated Undrained (CIU) triaxial shearing test

Isotropically consolidated undrained triaxial shearing tests with pore pressure measurements were carried out on lime-slag treated CIS according to BS1377-part 8(1990) and Head (1986). UCS and one-dimensional consolidation testing indicated that the lime contents do not significantly affect the strength and compression properties of lime-slag treated CIS. Based on these observations, only a single lime content of 10% was used in this part of the investigation in order to minimize the number of tests. Due to limitation of time, the effect of curing period on undrained shearing behaviour could not be investigated and the behaviour at only a single curing period of 1 month was investigated. Back pressure saturation system was used to saturate the samples. The cell pressure was always maintained to be 10-15 kPa higher than the back pressure during the saturation phase. A backpressure value in the range of 400-500 kPa was required to achieve Skempton's B parameter value >0.90. It was found difficult to achieve the value of the B parameter greater than 0.95 even when a back pressure of 800 kPa was applied. A value of the B parameter in excess of 0.9 was considered to indicate full saturation of the sample. Upon saturation, the sample was isotropically consolidated to different pre-determined levels of consolidation pressures. At the end of the consolidation phase the samples were sheared under undrained condition at a rate of 0.05mm/min. This specific rate strain was carefully selected based on t_{100} during consolidation at different consolidation pressures. Details of the CIU testing program are given in Table 3.5.

Table 3.5. Details of CIU testing program

Test type	Curing time (months)	Additive content		Pre-shear consolidation pressure (kPa)
		Lime	Slag	
Isotropically consolidated undrained triaxial compression test	1	10	10,15,20	50, 100, 200, 400, 800, 1600, 2400

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CHAPTER 4: .UNCONFINED STRESS-STRAIN-STRENGTH BEHAVIOUR OF LIME-SLAG TREATED CIS

4.1 Introduction

Unconfined Compressive Strength (UCS) is the most commonly used, one of the quickest and most economic tests to measure undrained shear strength of soils. Undrained shear strength is ubiquitously used to determine the short-term stability of any geo-structure. In soil stabilization with additives, UCS is widely employed to assess the effectiveness of stabilization (Nagaraj and Miura, 2001). This research project focused on utilization of lime and slag, an industrial by-product, for the improvement of a problematic soft soil Coode Island Silt (CIS) which is widespread in Melbourne, Australia. Main variables that affect the strength development in lime-slag stabilization of soft soils are the quantities of additives and curing time. The influences of lime content, slag content and curing period on the strength development of lime-slag treated CIS was investigated in this project with the help of a comprehensive UCS testing program.

The testing program comprised of UCS tests on CIS treated with different combinations of additives and cured for different periods. Three different lime contents of 10, 15 and 20% were combined with each of the three different slag contents of 10, 15 and 20% giving a total of nine different additive combinations. The minimum quantity of lime to be used in this investigation was fixed by carrying out Initial Consumption of Lime (ICL) test. CIS treated with these nine different combinations of lime and slag was tested at one, three and six months of curing. The findings from these tests are presented in the following sections.

4.2 Initial Consumption of Lime (ICL) test

The optimum amount of lime to be used in this study was determined by carrying out Initial Consumption of Lime (ICL) test (BS 1924: Part 2:1990). Since it has been found from earlier studies (Stanley, 2010) that a lime content ≤ 5% is ineffective in improving strength of lime-slag treated pyrite bearing CIS, the lowest level of lime content to be used in ICL test was fixed at 6%. The amount of lime was varied from 6 to 15% and the resulting pH was measured. The result of this test is shown in Figure 4.1 which shows that the pH value reaches approximately 12.4 at a lime content of 7%. Although 7% lime was found to provide a pH value of 12.4, the minimum amount of lime used in this study was 10%. A significant amount of lime is consumed in the initial flocculation and agglomeration processes and if an amount higher than that determined by ICL test is not provided, the amount of lime available after fulfilling the demand of the initial flocculation and agglomeration processes may not be adequate to maintain the pH required for the activation of the added slag.

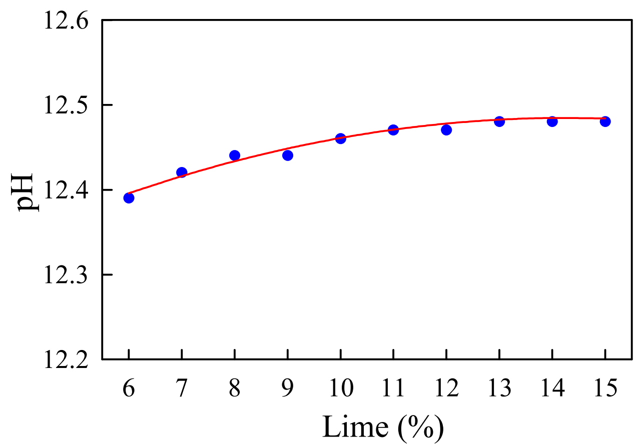

Figure 4.1. Result of Initial Consumption of Lime (ICL) test

4.3 Results and discussions

4.3.1 Unconfined stress-strain behaviour of lime-slag treated CIS

The unconfined stress-strain behaviour of lime-slag treated CIS is shown in Figure 4.2. It can be seen from Figure 4.2(i) that for a fixed lime content of 10%, the peak strength increases with an increase in slag content at one month curing. It can also be seen that the stiffness of the treated material increases with an increase in slag content. Similar type of stress-strain behaviour can also be observed for increasing lime contents (15 and 20%) at one month of curing as shown in Figure 4.2(ii) and Figure 4.2(iii). The increase in strength and stiffness with increasing slag content can be attributed to the formation of higher amount of cementitious reaction products within the treated soil mass. It can be seen that at one month curing, a brittle type of failure is observed for 15 and 20% slag treated materials whereas a ductile type of behaviour is observed for the soil treated with 10% slag. After reaching the peak stress within a range 3-4% strain, 15 and 20% slag treated samples fail in an abrupt manner. Although 15 and 20% slag treated samples exhibit clear peaks, no such peak is obvious for 10% slag treated CIS. It can also be seen that the strain at peak stress tends to increase with an increase in slag content at one month of curing.

The stress-strain behaviour for various lime-slag treated CIS at three months of curing is shown in Figure 4.2(iv)-(vi) whereas the stress-strain response at six months of curing is presented in Figure 4.2(vii)-(ix). It can be observed that there is a significant increase in strength and stiffness from one month to three months for all the additive combinations investigated. However, the rate of increase of peak strength and stiffness from three months to six months is found to be comparably much lower than that observed from one month to three months. Similar to the observation made at one month, the peak strength and stiffness at three and six months is also seen to increase with increasing slag content.

From the stress-strain curves presented in Figure 4.2, it can be observed that the failure at three and six months is more progressive in nature compared to the stress-strain behaviour observed at one month of curing where the samples failed abruptly approximately around the peak stress value. Under unconfined shearing condition majority of the strength is derived from the resistance offered by the cementation bonds since frictional mobilization is insignificant due to the absence of confinement. The change of brittle type of stress-strain behaviour at one month to more progressive strain softening behaviour at three and six months may have been caused by the difference in the progressive failure mechanism of the cementation bonds. When a cemented soil is sheared, the cementitious bonds progressively collapse giving rise to micro-cracks at the initial stage of the loading. With the progression of straining, the density of these micro-cracks increases and ultimately large macro-cracks appears due to the cumulative effects of these micro-cracks. It was observed that samples tested at the end of one month of curing developed almost vertically oriented fractures running throughout the height of the samples whereas at three and six months of curing the there was progressive spalling off of materials from the middle portion of the tested samples. The observed stress-strain behaviour suggests that at one month of curing majority of the cementation bonds collapsed at around peak stress value. For the samples tested at the end of one month of curing, it can be inferred that the rate of formation of micro-cracks from the collapse of cementitious bonds accelerated near the peak stress value and culminated in an abrupt formation of large macro-cracks which caused the sample to lose all of its strength in an abrupt manner. On the other hand, at three and six months of curing, all of the cementation bonds possibly did not fail at peak stress value and therefore further straining was possible beyond the peak stress due to the presence of some intact bonds.

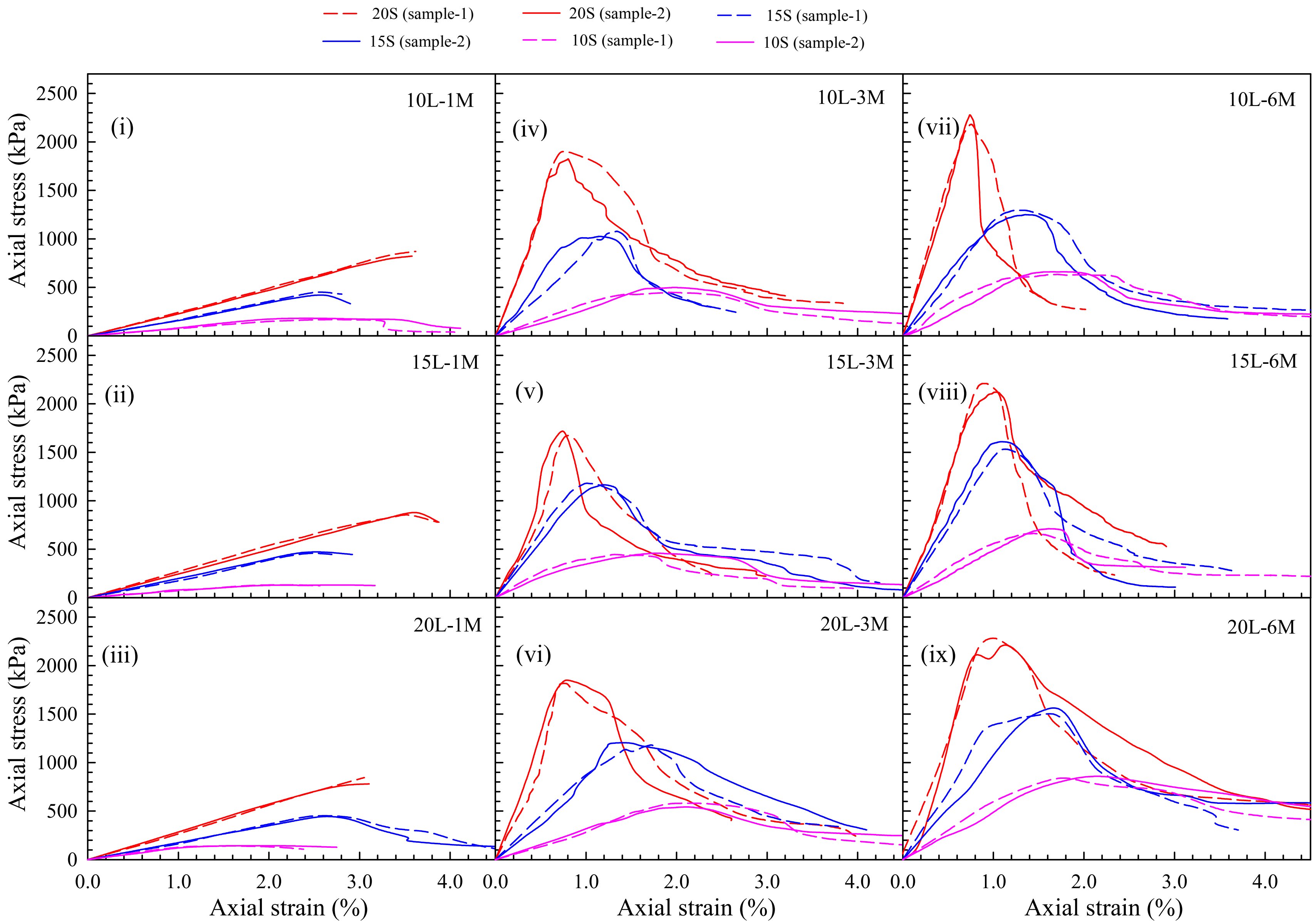

Figure 4.2. Unconfined stress strain behaviour of lime-slag treated CIS

Note: throughout the discussion of the results, L = % of lime, S = % of GGBFS and M = curing period in months

The change of brittle type of failure at one month to more progressive failure could possibly be linked to the strength of the cementitious bonds developed at those different curing periods. It has been suggested in literature that cementation provides an artificial confinement to the cemented soils and the degree of confinement increases with increasing degree of cementation (Kasama et al., 2000). At one month of curing, the strength of cementation bonds is much lower than the strength at three and six months of curing and therefore the magnitude of the cementation induced confinement will increase with increase in curing period. The effect of confinement on stress-strain behaviour of cemented soils have been investigated by many researchers and most of these investigations found that as the confinement increases the brittleness at peak stress gradually decreases and the stress-strain behaviour becomes more ductile at elevated confinement. The change of brittle type of stress-strain behaviour at one month of curing to more progressive failure observed at three and six months of curing may have been caused by the increased degree of artificial confinement provided by the enhanced strength of the cementation bonds.

From the stress-strain plots presented in Figure 4.2, it can also be seen that with the progression of curing, the strain at peak stress reduces significantly. At one month of curing, strain at peak stress was within a range of 3-4% whereas at three months and six months the strain at peak stress is less than 2% for most of the cases. Moreover, it is also observed that at three and six months of curing the strain at peak stress reduces with increase in slag content which is in sharp contrast to the observation made at one month. This observation suggests that the increase in stiffness with slag content is more pronounced at longer curing periods.

4.3.2 UCS behaviour of lime-slag treated CIS

The variation of UCS with lime and slag contents and curing periods are shown in Figure 4.3 and the numerical values are presented in Table 4.1. Figure 4.3(a) shows the influence of lime content on the unconfined strength for different fixed slag contents at different curing periods. It can be seen that UCS value at one month of curing remains almost unchanged with lime content for all the slag contents investigated. On the other hand slag is seen to have significant influence on the UCS values [Figure 4.3(b)] at one month. For example, at one month of curing and for all the different lime contents, an increase of slag from 10 to 15% causes almost a three-fold increase of UCS and a further increase of slag from 15 to 20% causes approximately a two-fold increase of UCS. The independence of UCS of the lime content may be due to the fact that the minimum amount of lime used was significantly higher than the required minimum amount determined by ICL test. It appears that 10% lime is adequate to meet the demand of the initial flocculation and agglomeration processes as well as to activate even the highest amount of slag (20% in this case) used in this investigation. It can also be inferred that for the soil treated with 20% lime (i.e., highest amount of lime) and 10% slag (i.e., lowest amount of slag), an excess lime may have been present in the reaction environment. The experimental results on UCS suggest that neither did the excess lime contribute to the strength development nor did it adversely affect the strength to any significant extent. The strong dependence of UCS on the slag content at one month of curing suggests that hydration of slag progressed significantly at one month of curing. In lime-slag stabilization of soft soil, the role of lime is primarily limited to the strength developed through the initial flocculation and agglomeration processes and the role of slag in these initial processes is unimportant. Therefore, it is evident that had slag not participated in the strength developed at one month of curing, such pronounce influence of slag on the strength would not have been observed.

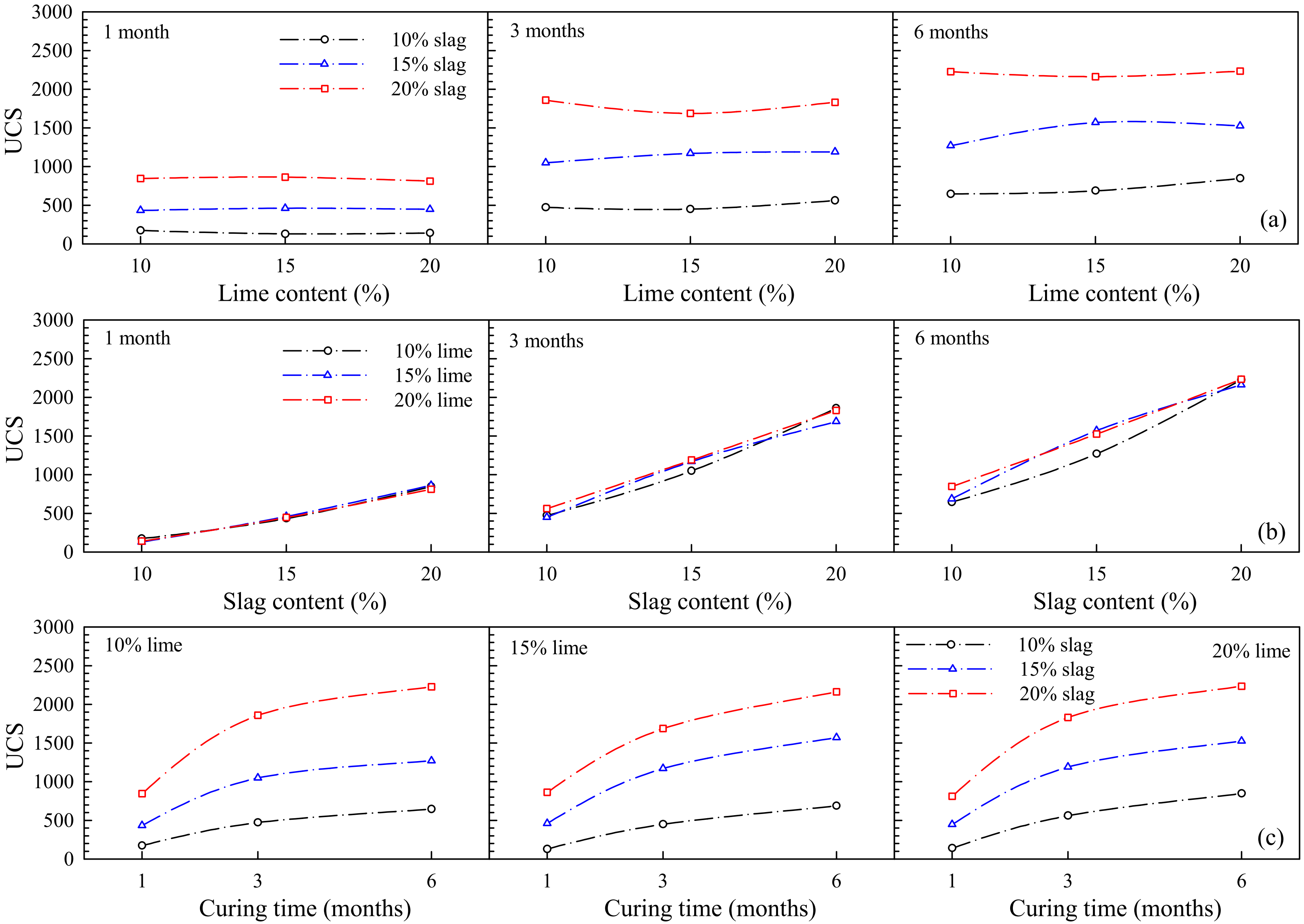

Figure 4.3. : (a) Influence of lime content on UCS (b) influence of slag content on UCS (c) influence of curing time on UCS

Table 4.1. Summary of UCS and E_{50} of lime-slag treated CIS

1 Month									
UCS (kPa)					E_{50} (MPa)				
	Slag (%)	10	15	20		Slag (%)	10	15	20
Lime (%)	10	175	434.6	847	Lime	10	7.94	16.9	24.1
	15	131	462.8	864		15	7.79	18.6	25.7
	20	142	447.9	813		20	12.6	17.7	28.1
3 Months									
UCS (kPa)					E_{50} (MPa)				
	Slag (%)	10	15	20		Slag (%)	10	15	20
Lime (%)	10	473	1050	1859	Lime	10	30.7	103	246
	15	451	1172	1688		15	47	125	183
	20	562	1191	1832		20	30	82.9	217
6 Months									
UCS (kPa)					E_{50} (MPa)				
	Slag (%)	10	15	20		Slag (%)	10	15	20
Lime (%)	10	647	1272	2228	Lime	10	55.1	120	311
	15	688	1572	2164		15	54.7	154	241
	20	848	1526	2236		20	55.8	125	255

The effect of curing time on the UCS is shown in Figure 4.3(c). It can be observed from this figure that curing has significant influence on UCS up to three months of curing and after three months the effect of curing on the unconfined strength decreases considerably for all the different combinations of lime and slag. This type of effect of curing on the strength development of cement treated soil has been reported by many earlier researchers (Uddin, 1995; Kamruzzaman, 2002). It is well known fact that the rate of hydration gradually decreases with the passage of time. With the progression of curing, the constituents of slag and lime which take part in the cementitious reactions get exhausted and the rate of formation of new cementitious products gradually slows down. Due to the decreasing rate of formation of new cementitious products, the rate of increase in strength was also seen to decrease.

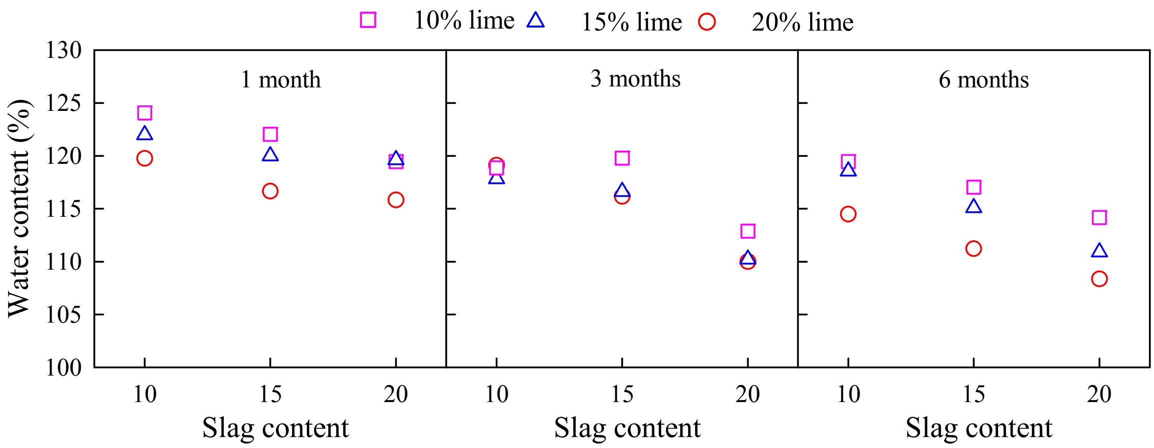

Figure 4.4. Moisture content of lime-slag treated CIS

Apart from the influences of the experimental variables described earlier on the UCS, the post-cured moisture content may have some impact on the observed UCS. A reduction in moisture content may be associated with a reduction in void ratio and a reduction in void ratio may positively contribute towards unconfined strength. In order to isolate this secondary effect of strength gain through the change in void ratio from the strength developed through the formation of cementation bonds, the variation of moisture content at different curing periods is analysed here. The moisture contents of the tested samples are presented in Figure 4.4. It can be observed that the moisture content tends to decrease with an increase in curing period. The initial moisture content of all the samples prepared was approximately 130% whereas the moisture content at one month curing lie within a band of 115-125% and at three months curing it is within a range of 110-120%. From three months to six months there is no appreciable change in moisture content. It is also observed that both the increasing lime content and increasing slag content tend to cause a decrease in moisture content. However, the pattern of decrease of moisture content with increasing slag content appears to be more consistent than the pattern observed for increasing lime content. The decrease in moisture content with an increase in slag content and curing time may be due to the formation of higher amount of cementitious products within the treated samples. However, it can be observed that the variation of moisture content with lime content, slag content and curing period is not very significant and the influence of the variation of moisture content on the observed strength behaviour may be negligible.

4.3.3 Unconfined stiffness of lime-slag treated CIS

It is well known that the stiffness of soil is controlled by many factors such as confinement level, strain level, stress path among many others. Although stiffness measured in unconfined condition may not be representative of the actual stiffness magnitudes to be used in analysis, the study of unconfined stiffness can provide a qualitative picture on the effect of different experimental variables on the stiffness properties of lime-slag treated CIS.

The stiffness is reported here in terms of E_{50}, a secant stiffness measure corresponding to a stress value of 50% of the peak stress. Figure 4.5(a) shows the effect of lime content for different fixed slag contents on the measured E_{50} values at various curing periods. It can be seen that the stiffness of treated material is practically independent of lime content for all the slag contents and at all curing periods. Figure 4.5(b) shows the effect of slag content on the E_{50} values for different fixed lime contents at different curing periods. It can be seen that the stiffness increases significantly with slag content and approximately a linear relationship can be found between slag content and E_{50} values. The influence of slag on the E_{50} is found to be more prominent at three and six months of curing compared to its influence observed at one month of curing. The rate of increase of E_{50} with increasing slag content at three and six months is significantly higher than the rate of increase observed at one month of curing. This may be due to the fact that the contribution of cementitious reaction products generated through the initial flocculation and agglomeration processes to the overall strength of the cementitious bonds at shorter curing period is significantly

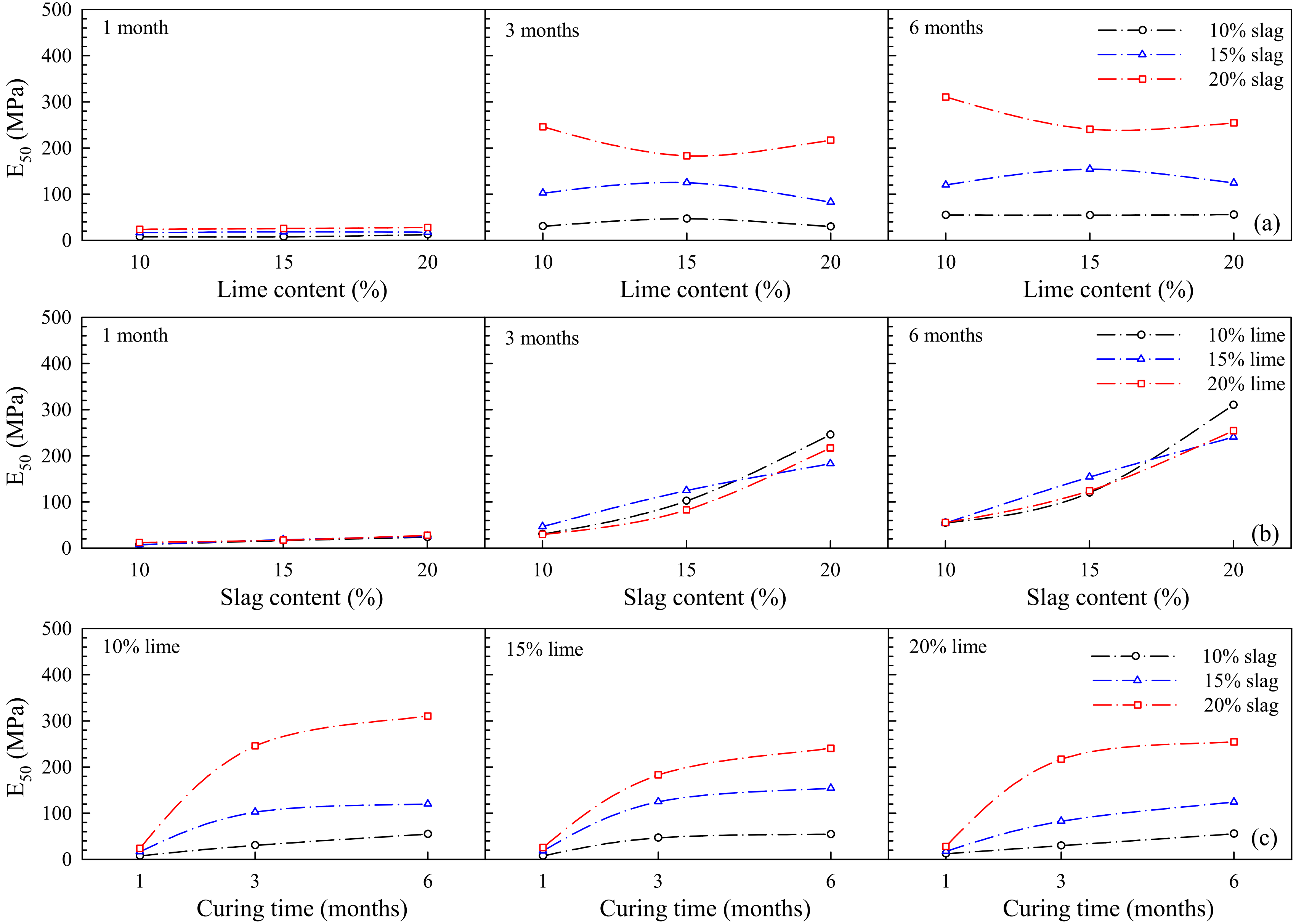

Figure 4.5. (a) Influence of lime content on E_{50} (b) influence of slag content on E_{50} (c) influence of curing time on E_{50}

higher than their contribution to total strength at longer curing periods such as three months and six months. The role of slag in the initial agglomeration and flocculation processes is insignificant. Therefore, it is evident that the contribution of slag hydration products to the overall strength of the cementitious bonds increases with an increase in the curing period and this may explain the influence of slag on the E_{50} values observed at different curing periods.

Figure 4.5(c) shows the effect of curing time on the E_{50} values for different combinations of lime and slag. It is observed that the effect of curing is most prominent from one to three months of curing and thereafter increasing curing does not change the E_{50} appreciably. This decrease in the rate of increase of E_{50} with curing time can be attributed to the gradual slowing down of the pozzalonic reactions.

From the discussion on the effects of lime content, slag content and curing time on the strength and stiffness, it is seen that all these experimental variables influence the strength and stiffness in a similar fashion. As was discussed earlier, the strength and deformation up to the peak value is controlled mainly by the mechanical behaviour of the cementitious bonds. Since strength and stiffness are both controlled by the same variable (mechanical properties of the cementitious bonds), it is expected that both strength and stiffness will be affected by all these experimental variables in a similar way.

4.3.4 Correlation between unconfined E_{50} and UCS

Although for routine geotechnical design calculations, strength is the main design parameter, for many applications accurate determination of deformation becomes equally important. For artificially cemented soils, it is likely that the applied maximum stress will be within the elastic range. An empirical correlation between UCS and unconfined E_{50} can therefore be of great practical significance since if the UCS is known from some other tests which does not produce stress-strain data, the stiffness values can be roughly approximated if such a correlation exists. With this view, an empirical relationship between UCS and E_{50} values for CIS treated with all different combinations of lime and slag is sought here.

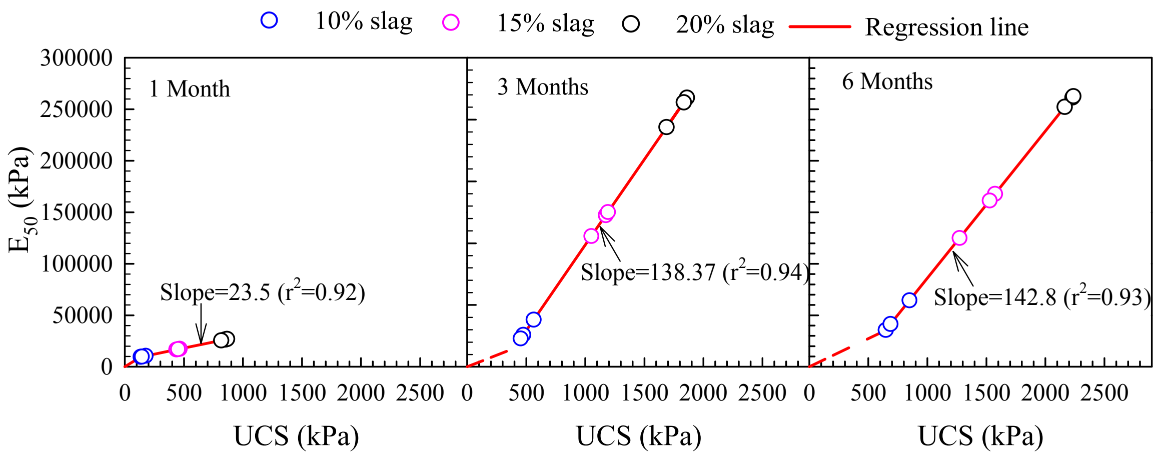

Figure 4.6. E_{50} and UCS correlations at different curing periods

It was discussed earlier that all the experimental variables (lime content, slag content and curing time) affect the strength and stiffness in a similar way and therefore it is highly likely that a correlation exists between strength and stiffness values. It can be seen from Figure 4.6 that a strong correlation in fact exists between UCS and E_{50} values as was expected. However, it is observed that the slope of the E_{50} vs. UCS curve increases significantly from one month to three months but the increase in the above slope from three months to six months is only marginal. Since the elastic stiffness is controlled by the mechanical properties of the cementitious bonds, the difference in the slope of the E_{50} vs. UCS line at different curing period can be attributed to the mechanical properties of the cementitious products formed at those different curing periods. At shorter curing period most of the reaction products remain in an amorphous phase whereas at longer curing periods the reaction products becomes more crystalline possessing comparatively much higher strength. As a result the stiffness of soil skeleton at longer curing period can be expected to be much higher due to the increasing degree of stiffening effects derived from much stronger cementitious bonds.

4.4 Summary

A comprehensive UCS testing program was undertaken to study the influences of lime content, slag content and curing time on the strength development of lime-slag treated CIS. The major findings from this investigation can be summarized as follows:

- The strength and deformation properties of soft pyrite bearing CIS could effectively be improved by treating it with lime and slag.

- When the lime content used is higher than the required minimum amount determined ICL test, lime is seen to have very insignificant influence on the strength and stiffness values over a curing period of six months.

- It was seen that the influence of slag on the strength and stiffness is very prominent for all the lime contents and at all the curing periods investigated. It was found that the influence of slag on strength and stiffness becomes more prominent at longer curing periods.

- Curing period also affects the strength and stiffness values significantly. The effect of curing from one month to three months is more prominent than its effect from three to six months due to progressive slowing down of pozzalonic activity. It was found that curing also changes the stress-strain behaviour noticeably. Whereas at one month of curing samples failed in an abrupt manner near the peak stress value, at three and six months of curing there was significant post-peak strain softening of the samples and the samples failed in a progressive manner at longer curing periods. Increasing curing period was also found to decrease the strain at peak stress.

- Proper caution should be exercised in the use of lime-slag stabilized CIS in conditions involving low level of confinement. The failure of the samples at shorter curing period has been observed to be of brittle type and therefore, care should be taken so that the design load does not induces strains in excess of the strain at peak strength. If such a loading is imposed, there is a risk of sudden failure of the loaded soil mass.

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CHAPTER 5: A NEW GENERALIZED VIRGIN COMPRESSION MODEL FOR STRUCTURED SOILS

5.1 Introduction

Compressibility behaviour of soil is of fundamental importance in geotechnical engineering. In routine geotechnical practice, almost all settlement calculations of foundations are based on the one-dimensional compression behaviour of soils. The compression behaviour of soils is also used to describe the hardening characteristics of yield surface in the formulation of many of the plasticity based soil constitutive models. Its importance in geotechnical engineering is evident from numerous literatures published on this particular topic throughout the history of soil mechanics (Terzaghi, 1925; Skempton, 1944; Janbu, 1963; Butterfield, 1979, Mesri & Choi, 1985; Hardin, 1989; Burland, 1990; Leroueil & Vaughn, 1990; Den Haan, 1992; Hong et al., 2012)

It was discussed in details in Chapter 2 how the compression behaviour of structured soils differ from that of corresponding reconstituted soils. It was showed that two distinctly different mechanisms control the compression behaviour of structured soils in different stress ranges. While the compression behaviour immediately after the yield is governed by the collapse of the soil structure, the behaviour at large stress is controlled by the mineralogy of the soil aggregates. Based on a review of existing compression models for structured soils, the need for the development of a simple compression model incorporating easily determinable parameters which can characterize these distinct mechanisms was demonstrated. To fulfil the identified need, a simple compression model is developed in this chapter that can reproduce the compression behaviour of a wide range of structured soils including both naturally and artificially structured soils. The development and validation of the proposed model and associated parametric studies have been based on the experimentally observed virgin compression behaviour of naturally structured soil although the current project deals with the mechanical behaviour of artificially cemented soils. Natural soil has been chosen instead of artificially cemented soil mainly due to the scarcity of literature data on the compressibility behaviour of artificially cemented soils at their de-structured states. Since it is well established that structured soils, irrespective of the origin of the structure, exhibit a common pattern of behaviour (Leroueil and Vaughn, 1990), knowledge gained from the study of naturally structured soils can be employed to interpret the behaviour of artificially cemented soils.

5.2 Formulation of a generalized virgin compression model

Mesri & Choi (1985) proposed a technique to incorporate the variability of soil compression index with increasing pressure into the calculation of primary compression settlement of embankments. They observed that, since the compression index of natural soils is a pressure-dependent variable quantity, it is difficult to incorporate it in conventional settlement calculations where a constant compression index is used. They demonstrated, however, that the use of a modified compression index made it possible to make realistic settlement calculations within the traditional theoretical framework for such cases. In their method, different lines were drawn from the yield point (σ_y') to various points on the experimental compression curve and the slopes of these lines defined the modified compression indices C_c' at different pressures. The calculated values of C_c' were then used to determine primary compression settlements at different pressures. The graphical technique proposed by Mesri and Choi (1985) to determine the variable compressibility index of soils provides us with an idea to formulate an analytical pressure-void ratio relationship incorporating the pressure dependent compressibility of soils.

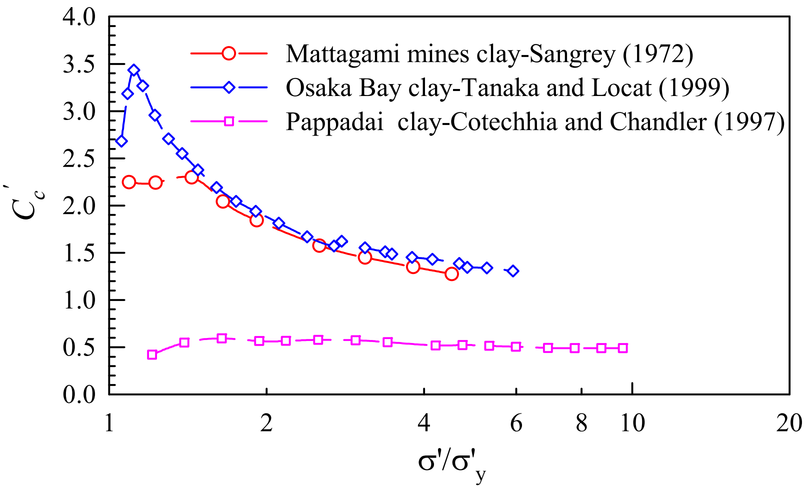


Figure 5.1. Variation of C_c' with pressure for several natural soils

The variation of C_c' with pressure for a number of natural soils were analysed (Figure 5.1). Based on these analyses, the following observations are made:

- The value of C_c' is undefined at the yield point which makes it difficult to model the variation of C_c' with pressure starting from it.

- The variation of C_c' with pressure is not smooth for some of the soils and this makes it difficult to mathematically reproduce this variation by simple functions.

5.2.1 Mathematical formulation of the virgin compression model

The difficulties discussed above regarding representing the variation of C_c' with pressure by a simple mathematical function can be eliminated by selecting the initiation point for drawing the slopes at 1 kPa of pressure. To ensure that this modified C_c' equals to zero at the yield point and thus eliminate the necessity of an additional initial value parameter, the void ratio at yield (e_y) is selected as the void ratio of the initiation point. The modified C_c' thus effectively becomes a secant parameter and is termed as secant compression index C_{cs}' (Figure 5.2a). It is represented mathematically by:

$$C'_{cs} = \frac{e_y - e}{log\,(\sigma')} \text{---} (5.1)$$

Where σ'= effective vertical pressure for 1-D compression and mean effective pressure for other triaxial loading conditions, σ'_y = yield stress and e = void ratio corresponding to pressure σ'.

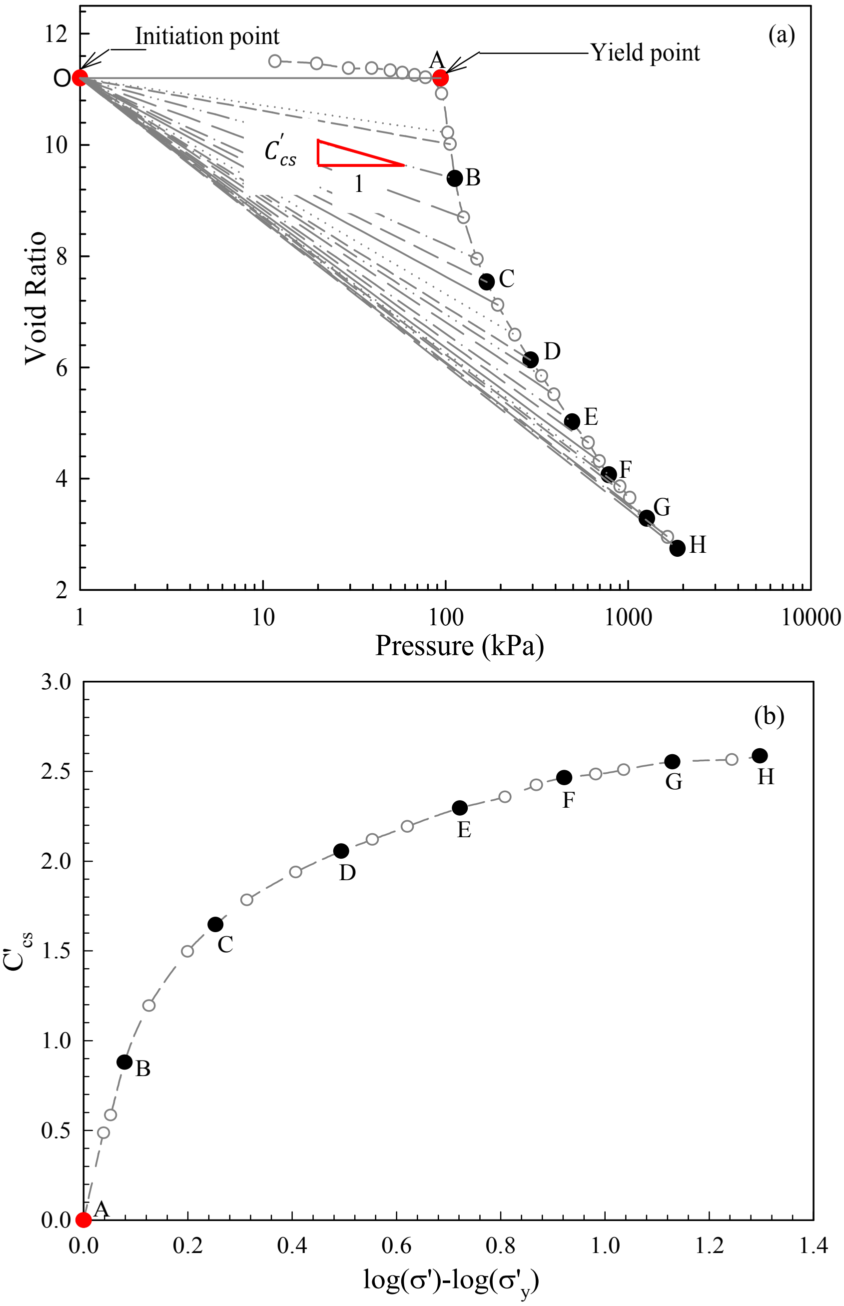


Figure 5.2. (a) $e - log(\sigma')$ curve showing variation of C'_{cs} with pressure

(b) hyperbolic variation of $C'_{cs}$with $[log(\sigma') - log(\sigma'_y)]$

The compression data of Mexico City clay, widely used in past studies (Terzaghi, 1953; Mesri et al., 1975), has been selected to first illustrate the variation of $C_{cs}^{'}$ with pressure. Different lines were drawn connecting the initiation point "O" to different points on the compression curve AH as shown in Figure 5.2(a). The variation of $C_{cs}^{'}$ with log-pressure starting from yield is shown in Figure 5.2(b). It can be seen that the quantity $C_{cs}^{'}$ gradually increases with pressure but its rate of increase gradually diminishes eventually becoming insignificant when the soil is heavily de-structured (e.g., points G, H on Figure 5.2b). This variation of $C_{cs}^{'}$ with pressure can be modelled with a very high degree of accuracy by a simple two-parameter rectangular hyperbola of the form:

$$C_{cs}^{'} = \frac{[\log(\sigma^{'}) - \log(\sigma_y^{'})]}{a' + b'[\log(\sigma^{'}) - \log(\sigma_y^{'})]} \text{---}(5.2)$$

Where, a' and b' are two hyperbolic fit parameters. The reciprocal of a' represents the initial slope of the hyperbola and reciprocal of b' represents the asymptotic value of the hyperbola. Soils having higher rate of post-yield de-structuration will have steeper initial slope of the above hyperbola and therefore smaller value of the parameter a'. On the other hand, soils having higher compressibility in the de-structured state will have higher asymptotic value of the hyperbola and therefore smaller value of the parameter b'. The applicability of Equation 5.2 to model the variation $C_{cs}^{'}$ with pressure is demonstrated for a number of soils in Figure 5.3 which shows that the fit between the experimental data and the simulation is excellent for all the cases (the values of the parameters as well as information on the quality of simulation can be found in Table 5.1).

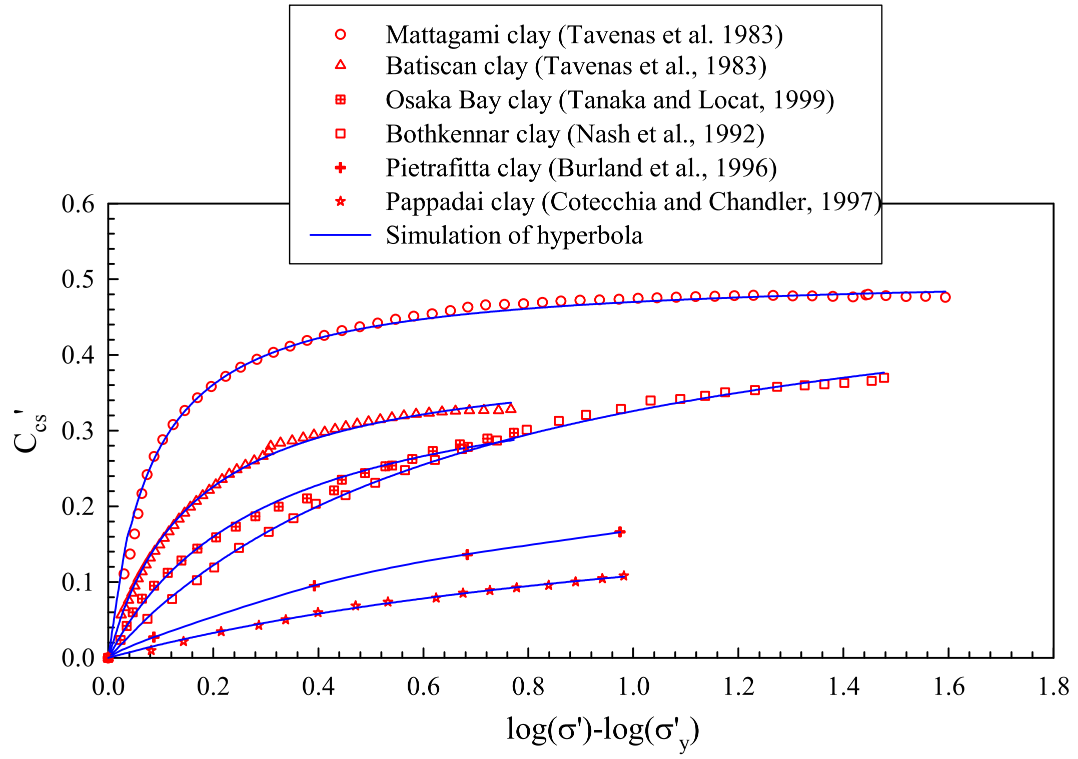


Figure 5.3. Simulation of the variation of $C_{cs}^{'}$ with log-pressure by a rectangular hyperbola

Combining Equations 5.1 and 5.2, the variation of void ratio with pressure can be represented by: $e = e_y - \frac{[\log(\sigma^{'}) - \log(\sigma_y^{'})]}{a' + b'[\log(\sigma^{'}) - \log(\sigma_y^{'})]} \times \log(\sigma^{'})$

$$=> e = e_y - \frac{\log\left(\frac{\sigma'}{\sigma_y'}\right)}{a' + b'\log\left(\frac{\sigma'}{\sigma_y'}\right)} \times \log(\sigma^{'}) \text{ --(5.3)}$$

While the above approach has been found to be highly accurate in reproducing the compression curves and characterizing the de-structuration behaviour of structured soils, it has been found that the above formulation needs further modification before it can be employed to carry out comparative study of the de-structuration behaviour of different types of soils. The modification is necessary to address the influence of differing yield stresses of different soils on the model parameter $a^{'}$. To illustrate this particular issue,

Figure 5.4 is presented. In Figure 5.4(a), the red coloured curve presents the actual compression data of Leda clay as reported by Yong & Nagaraj (1977). The variation C'_{cs} with $\log(\sigma')$ for the actual compression data is represented by the red-coloured curve in Figure 5.4(b). Now let us consider a hypothetical situation where another soil has exactly similar $\frac{\Delta e}{\Delta \log(\sigma')}$ $vs \log(\sigma')$ profile as Leda clay but has a different yield stress. Such two imaginary curves are represented by the blue and pink coloured curves respectively in Figure 5.4(a). The corresponding C'_{cs} profiles are represented by blue and pink coloured curves in Figure 5.4(b). It can be seen from Figure 5.4(b) that although the response of the soils to incremental pressure in the post-yield stress regime is identical for all the cases presented in Figure 5.4(a), the hyperbolas representing the different cases are significantly different. Consequently the parameters characterizing the hyperbolas will be significantly different for different cases and this issue must be addressed adequately before the compression equation presented earlier can be accepted to be of generalized nature. A simple technique can be employed to overcome this problem by bringing the yield points of all the compression curves of different soils to a common point without changing the $\frac{\Delta e}{\Delta \log(\sigma')}$ profiles and thereby keeping the integrity of the de-structuration characteristics intact of the respective soils. Let us represent the pressure and void ratio at the reference point mentioned above by σ_y^T and e_y^T respectively. For bringing the yield points of different compression curves to the common reference point (σ_y^T, e_y^T), a quantity equal to $[\log(\sigma_y^T) - \log(\sigma'_y)]$ needs to be added to each of the experimental $\log(\sigma')$ values and another quantity $[e_y^T - e_y]$ to each of the experimental values of the void ratio e. The values of $[\log(\sigma_y^T) - \log(\sigma'_y)]$ and $[e_y^T - e_y]$ are constant for any particular soil and are represented by C_σ and C_e respectively in the following sections. It is important to mention here that C_σ and C_e are not intended to be used as material parameters to characterize the compression behaviour rather they are merely some dummy quantities required to bring the virgin compression data of different soils to a common reference point so that the virgin compression behaviour of various soils can be objectively compared. The procedure for bringing all the compression curves to a common reference point is illustrated in Figure 5.5(a).

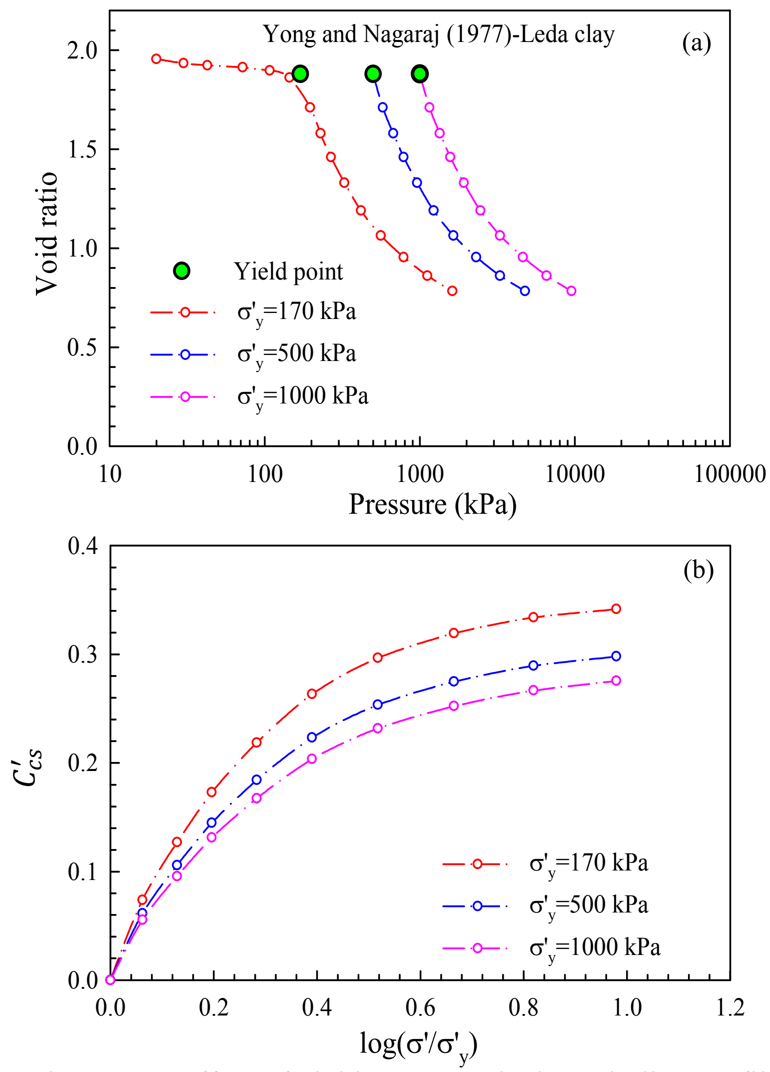


Figure 5.4. Effect of yield stress on the hyperbolic profiles

(a) compression curves (b) hyperbolic C'_{cs} vs. log-pressure profiles

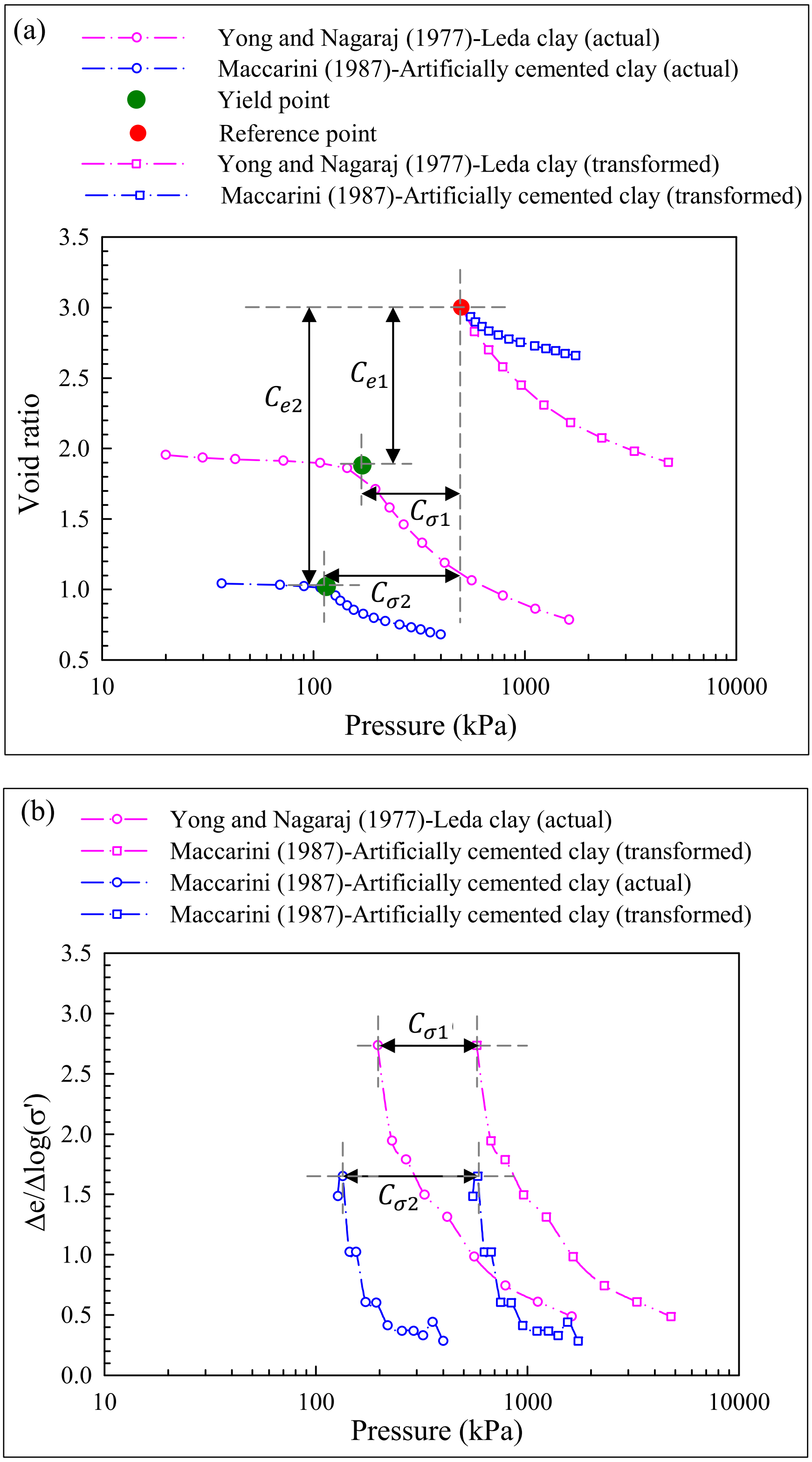


Figure 5.5. (a) Transformation of post-yield part of the compression curves

(b) $\frac{\Delta e}{\Delta \log(\sigma')}$ profiles for actual and transformed data

Figure 5.5(b) shows the $\frac{\Delta e}{\Delta \log(\sigma')}$ vs. log-pressure profiles of the soils presented in Figure 5.5(a) based on the original and transformed values of the pressures and void ratios. It can be seen that for both the original and the transformed data, the variation of $\frac{\Delta e}{\Delta \log(\sigma')}$ with log-pressure are identical with the only difference being that the $\frac{\Delta e}{\Delta \log(\sigma')}$ vs. logarithm of pressure profile derived from transformed data is displaced along the pressure axis by the quantity C_σ.

The values of the transformed pressures can be calculated by the following equation:

$$\log(\sigma^T) = \log(\sigma') + C_\sigma$$

=> $\sigma^T = 10^{[\log(\sigma')+C_\sigma]}$ --(5.4)

Similarly the transformed void ratios can be calculated as follows:

$e^T = e + C_e$---(5.5)

Let the values of secant compression indices calculated based on the transformed pressure and void ratios be defined by C_{cs}^T and the associated hyperbolic parameters by a and b.

Once the necessary transformations of the experimental data have been carried out, the variation of C_{cs}^T with log (σ^T) can be modelled by the following equation:

$C_{cs}^T = \frac{[\log(\sigma^T)-\log(\sigma_y^T)]}{a+b[\log(\sigma^T)-\log(\sigma_y^T)]}$---(5.6)

The transformed void ratios corresponding to different transformed pressure values can be calculated by the following equation:

$$e^T = e_y^T - \frac{[\log(\sigma^T) - \log(\sigma_y^T)]}{a + b[\log(\sigma^T) - \log(\sigma_y^T)]} \times \log(\sigma^T) \text{---} (5.7)$$

Since $[\log(\sigma^T) - \log(\sigma_y^T)] = [\log(\sigma') - \log(\sigma_y')]$ and $\log(\sigma^T) = \log(\sigma') + C_\sigma$, $e = e^T - C_e$, the experimental void ratios in terms of actual experimental pressure can be simulated by the following expression:

$$e = e_y - \frac{[\log(\sigma') - \log(\sigma_y')]}{a + b[\log(\sigma') - \log(\sigma_y')]} \times [\log(\sigma') + C_\sigma] - C_e \text{---} (5.8)$$

Equation 5.8 can also be used in the calculation of one-dimensional primary consolidation settlement as follows:

$$\Delta e = e_y - e = e_y - [e_y - \frac{[\log(\sigma') - \log(\sigma_y')]}{a + b[\log(\sigma') - \log(\sigma_y')]} \times \{\log(\sigma') + C_\sigma\} - C_e]$$

$$S = \frac{\Delta e}{1 + e_0} H = [\frac{\{\log(\sigma') - \log(\sigma_y')\}}{a + b\{\log(\sigma') - \log(\sigma_y')\}} \times \{\log(\sigma') + C_\sigma\} - C_e] \times H \text{ -------------------------- } (5.9)$$

Where S =settlement of the foundation within the virgin compression range

H=height of the consolidating soil layer

e_0 =initial void ratio

σ' =applied vertical pressure.

5.2.2 Determination of model parameters

The hyperbolic relationship presented in Equation 5.6 can be reorganized in the following straight line form:

$$\frac{[\log(\sigma^T) - \log(\sigma_y^T)]}{C_{cs}^T} = a + b[\log(\sigma^T) - \log(\sigma_y^T)]$$

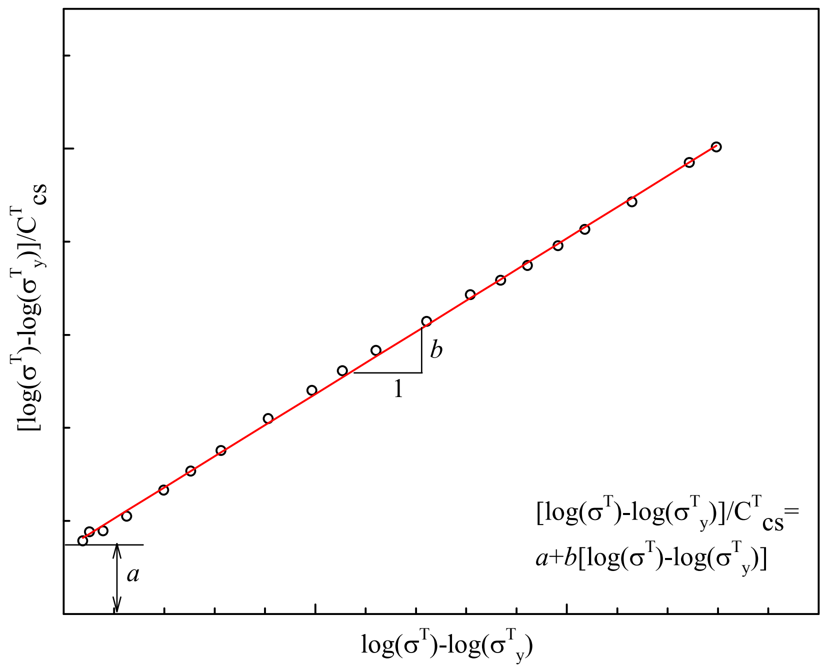


Figure 5.6. Determination of model parameters

By plotting the quantity $\frac{[\log(\sigma^T)-\log(\sigma_y^T)]}{C_{cs}^T}$ in the Y axis against the quantity $[\log(\sigma^T)-\log(\sigma_y^T)]$ in the X axis and fitting a straight line, as shown in the Figure 5.6, the parameter a can be determined from the Y axis intercept and the parameter b from the slope of the straight line. For obtaining realistic values of the parameters, at least few data points within the pressure range where the slope of the pressure-void ratio relationship in a semi-logarithmic plot becomes approximately constant are required. For the soft soils analysed in this study, it has been found that a pressure equal to five times the yield stress is adequate to produce the values of the parameters that can estimate the void ratios at higher stress ranges with an acceptable degree of accuracy. However, for stiff soils de-structuration is much slower and the stress range at which the slope of the compression curve in a semi-logarithmic plot becomes approximately linear may be significantly higher. Therefore, for stiff soils compression data in a stress range much higher than the five times the yield stress may be required for accurate determination of the parameters.

5.2.3 Validation of the proposed virgin compression model

In this section the capability of the proposed compression model to reproduce the non-linear virgin compression curves of various naturally structured soils are investigated. It is also explored here whether the model parameters are able to appropriately characterize the compression behaviour of structured soils in the de-structuration and de-structured ranges of the compression behaviour. For this validation study, the pressure and void ratio at the reference yield point has been selected as 5000 kPa and 5.0 respectively.

5.2.3.1 Capability of the model to simulate the virgin compression curves of structured soils

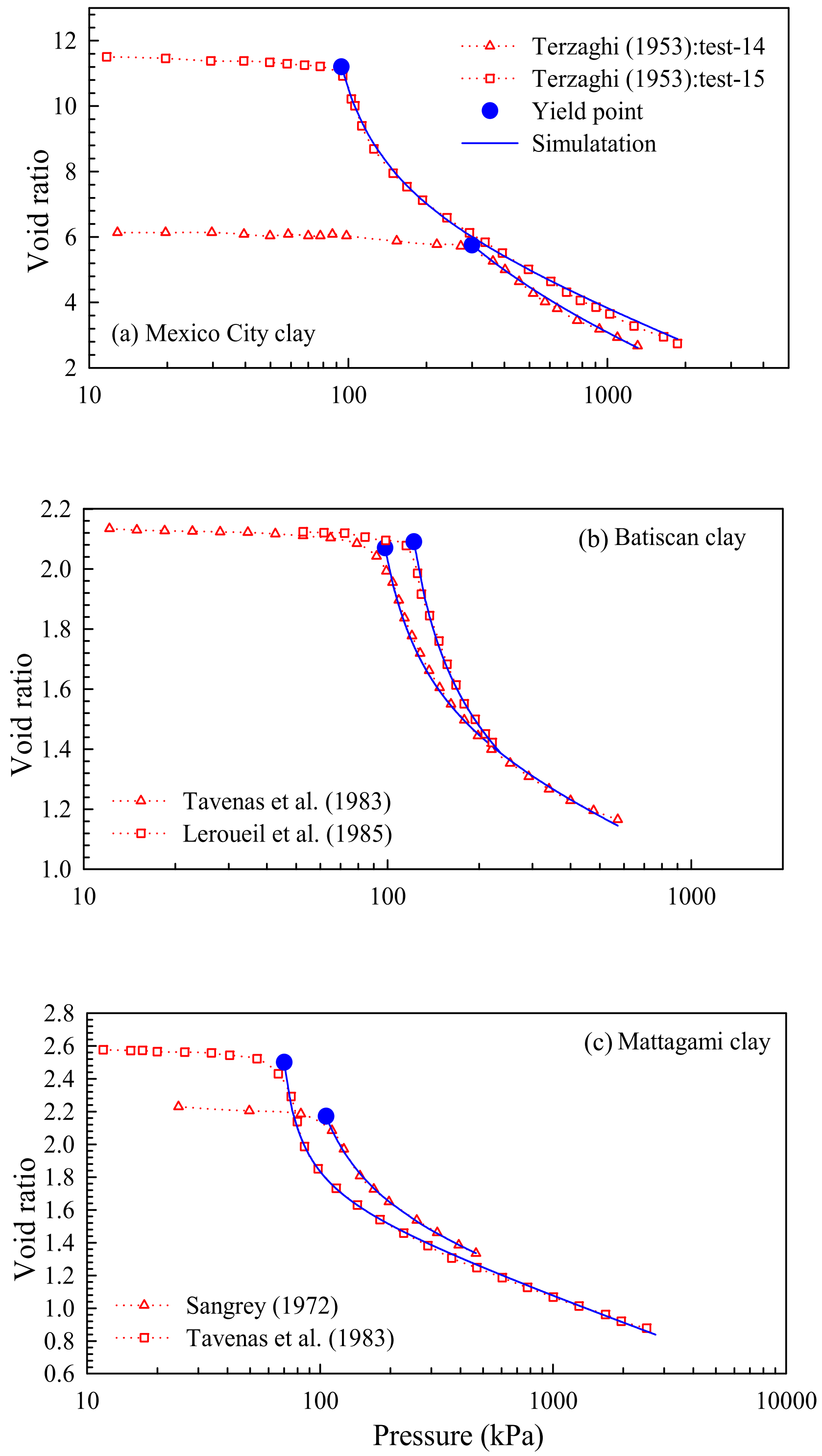
The validity of Equation 5.8 to model the compression behaviour of a number of natural soils possessing a wide range of sensitivities (4-125), yield stresses (65-2935 kPa) and yield void ratios (0.8-11.2) has been tested. A summary of the hyperbolic fit parameters *a* and *b* for these soils is presented in Table 5.1. All the experimental compression curves are presented along with the simulated curves in Figure 5.7. In addition, the absolute maximum error in the calculation of void ratio is reported in terms of percentage of void ratio at yield (e_y). Figure 5.7 shows that virgin compression of different structured soils exhibits various degrees of non-linearity and the proposed model can reproduce the virgin compression behaviour for all those soils with a very high degree of accuracy. Table 5.1 shows that for most of the cases R^2 value is greater than 0.99 for the assumed hyperbolic variation of secant compression index with log-pressure and the absolute maximum error in the calculation of void ratio at any pressure is less than 3% for all the soils analysed. It is to be noted that Figure 5.7(g) represents the experimental data reported by Hong et al. (2012) along with the simulation by the proposed method. It was shown by Hong et al. (2012) that in a bi-logarithmic plot, the compression behaviour of St.-Hilaire clay and

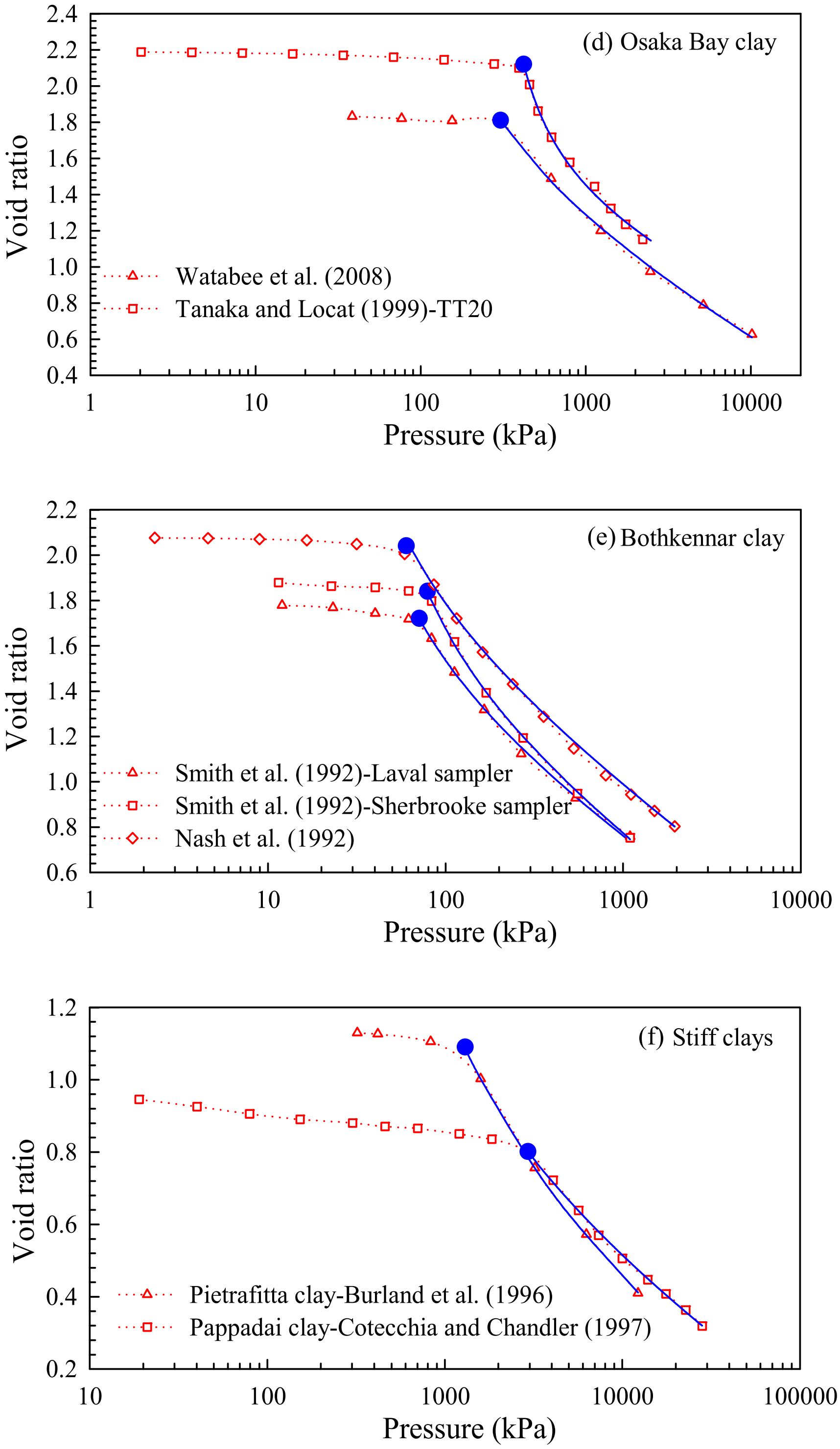
Grande-Baleine clay exhibit bi-linear behaviour whereas Osaka bay clay and Bothkennar clay show linear behaviour in the virgin compression range. It can be seen from Figure 5.7(g) that the proposed model can reproduce the compression behaviour of soils exhibiting either linear or bi-linear behaviour with very high degree of accuracy.

Table 5.1. Summary of fit parameters for various natural soils

Name of the clay	Reference	LL	PL	PI	LI	W_n (%)	S_t	σ'_y	e_y	a	b	R^2	Error (%)
Mexico city clay	Terzaghi (1953): Test-14	425-550*	57-150*	300-493*	-	439-574*	6-16**	300	5.75	0.51	0.54	0.999	2.867
	Terzaghi (1953): Test-15	500*	150*	-	-	421-574*	6-16**	94	11.2	0.15	0.4922	0.995	2.473
Mattagami clay	Sangrey (1972)	78	24		1.04	80	80	106	2.17	1.08	3.56	0.996	0.629
	Tavenas et al. (1983)	48-74	25-28	20-49	1.4-2.3	48-108	-	70	2.5	0.48	2.97	0.999	2.744
Batiscan Clay	Tavenas et al. (1983)	35-54	22-24	17-31	1.5-2.6	71-80	-	98	2.07	0.79	3.85	0.999	0.823
	Leroueil et al. (1985)	43		21	2.7	73.8-84.9	125	122	2.09	0.60	3.61	0.988	1.242
Osaka Bay clay	Watabe et al. (2008)	100	38	-	-	69	-	304	1.81	2.92	2.44	0.989	1.139
	Tanaka and Locat (1999)-TT20	108.7	40.2	68.5	0.52	83	>10	420	2.12	1.08	3.21	0.988	2.213
Bothkennar clay	Smith et al. (1992) -Laval sampler	80±6	31±4	-	-	70±3	4	71	1.72	2.72	2.72	0.993	1.346
	Smith et al. (1992) -Sherbrooke sampler	80±6	31±4	-	-	70±3	4	79	1.84	2.09	2.64	0.994	0.661
	Nash et al. (1992)	85	37	47	-	67	-	65	2.02	2.74	2.36	0.997	1.106
Pappadai Clay	Cotecchia and Chandler (1997)	65	30	35	-	31	-	2935	0.80	5.66	4.05	0.980	2.148
Pietrafitta clay	Burland et al. (1996)	87	52.5	34.5	-	41.9	-	1300	1.09	3.50	3.31	1.000	0.213

*Data obtained from Ou (2006), ** Data obtained from Terzaghi (1955)


Terzaghi (1953):test-14
Terzaghi (1953):test-15
Yield point
Simulatation
Void ratio
(a) Mexico City clay
Tavenas et al. (1983)
Leroueil et al. (1985)
(b) Batiscan clay
Sangrey (1972)
Tavenas et al. (1983)
(c) Mattagami clay
Pressure (kPa)


(d) Osaka Bay clay
Void ratio
Pressure (kPa)
Watabee et al. (2008)
Tanaka and Locat (1999)-TT20
2.4
2.2
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
1
10
100
1000
10000
(e) Bothkennar clay
Void ratio
Pressure (kPa)
Smith et al. (1992)-Laval sampler
Smith et al. (1992)-Sherbrooke sampler
Nash et al. (1992)
2.2
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
1
10
100
1000
10000
(f) Stiff clays
Void ratio
Pressure (kPa)
Pietrafitta clay-Burland et al. (1996)
Pappadai clay-Cotecchia and Chandler (1997)
1.2
1.0
0.8
0.6
0.4
0.2
10
100
1000
10000
100000

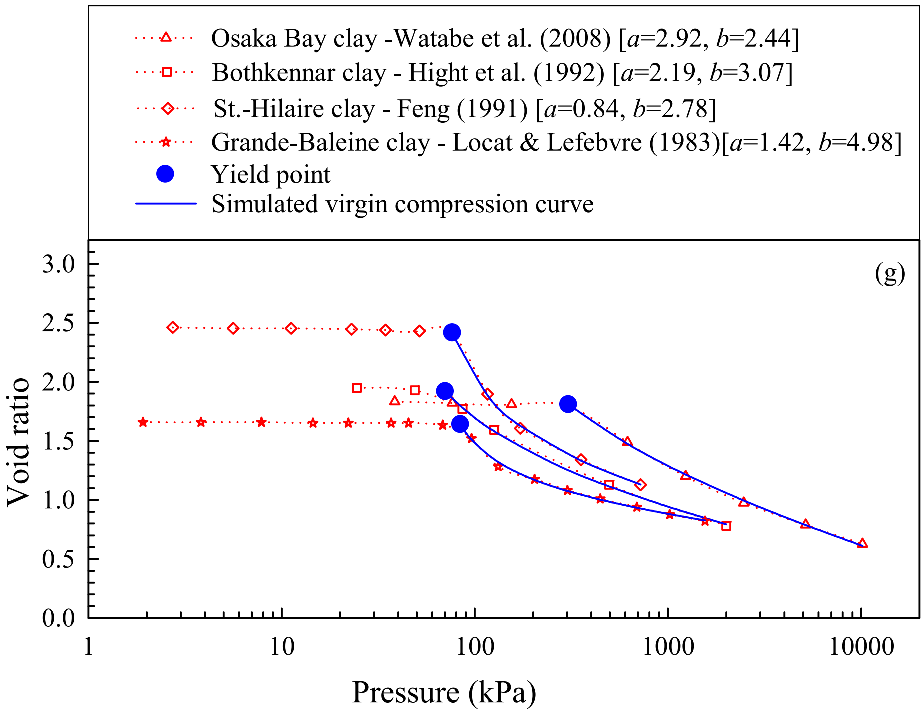


Figure 5.7. Validation of the proposed compression model for various natural soils

5.2.3.2 Performance of the model parameters to characterize the compression behaviour in the de-structuration and de-structured zones

It has already been demonstrated that the proposed formulation can reproduce the compression curves of a diversified group of soils with a very high degree of accuracy. In this section, it will now be investigated whether the model parameters correctly capture the compression characteristics within the de-structuration and de-structured zones appropriately.

In order to demonstrate the capability of the model parameters to convey meaningful characteristics of the modelled soils, compression data of a randomly selected group of soil is chosen. All the experimental compression curves are presented in Figure 5.8(a). It can be observed from this figure that it is difficult to compare the behaviour of one soil with another soil since most of the soils presented in Figure 5.8(a) exhibit non-linear behaviour and it is not possible to compare their compressibility characteristics by employing conventional constant compression indices. Figure 5.8(b) represents all the curves in terms of transformed values. It is obvious that Figure 5.8(b) provides with a very objective and rational basis for comparing the de-structuration characteristics of all the selected soils. Figure 5.8(c) presents the variation of instantaneous compression indices with pressure for all the soils presented in Figure 5.8(a). While Figure 5.8(b) is intended to establish correspondence between the parameter a and the de-structuration behaviour of different soils, Figure 5.8(c) will help to explore the correspondence between the de-structured compressibility and the parameter b.

The values of the parameters a and b associated with the different soils are given in Table 5.2. The curves presented in Figure 5.8(b) has been assigned different numbers and are ordered in terms of the ascending values of the parameter a, i.e, the soil with the lowest value of the parameter a has been designated with the lowest integer value. Figure 5.8(b) clearly shows that as the numbers designating the soils increases, the rate of de-structuration becomes more and more gradual. This observation clearly proves that the model parameter a can accurately characterize the de-structuration behaviour of structured soils.

The de-structured compressibility is inversely proportional to the parameter b, i.e, the smaller the value of the parameter b, the higher is the de-structured compressibility. It can be seen from Figure 5.8(c) that at large pressure the instantaneous compression indices become approximately constant. These constant values of the instantaneous compressibility indices at large pressure can be treated as the compressibility of the soils at their de-structured states. Now it is investigated whether the values of the parameter b correctly reflects the relative magnitudes of de-structured compressibility of the soils presented in Figure 5.8(a). It can be seen from Figure 5.8(c) that de-structured compressibility is the highest for Soil-5 for which the value of the parameter b is the lowest (b=1.48). On the other hand the de-structured compressibility is the lowest for Soil-8 which is represented by the highest value of the parameter b at 10.57. It can be observed that although the instantaneous compression indices attain an approximately constant value at large pressure, they exhibit a fluctuating pattern which makes it difficult to order them precisely in terms of magnitude. A better approach can therefore be to divide the soils into groups having approximately similar values of the de-structured compressibility. If the soils are grouped according to decreasing magnitudes of de-structured compressibility, a reasonable grouping could be as follows: Group-1(4 and 5)> Group-2(3 and 7)> Group-3(1, 2 and 6)> Group-4(8). The ranges of values of parameter b for these groups are: Group-1(1.48-1.72), Group-2(2.05-2.62), Group-3(3.06-4.27), Group-4(10.57). It can be clearly seen that the values of the parameter b for different groups maintain exactly the same sequence as the sequence observed for the magnitudes of de-structured compressibility of the groups as derived from the experimental compression curves. This observation clearly demonstrates the ability of the model parameter b to convey the compressibility characters of the soils in de-structured state.

Table 5.2. Details of the soils used to examine the capability of model parameters

SL. No.	Soil Name	Reference	Yield		Model parameters	
			σ'_y (kPa)	e_y	a	b
1	Louseville clay	Tavenas et al. (1983)	167	1.91	0.48	4.27
2	Mattagami Mines clay	Sangrey (1972)	106	2.17	1.08	3.56
3	Yamashita clay	Tanaka et al. (2001)	440	2.52	1.21	2.62
4	Ariake Clay	Tanaka et al. (2001)	53	3.43	1.23	1.72
5	Fjord sediments	Perret et al. (1995)-Test 20	32	3.08	2.14	1.48
6	Lab sedimented clay	Leonards & Altschaef (1964)	58	1.51	3.77	3.06
7	Guang Shen clay	Wang and Wei (1996)	28	2.12	4.25	2.05
8	Drammen Clay	Bjerrum-1967 (Lean)	98	0.90	8.42	10.57

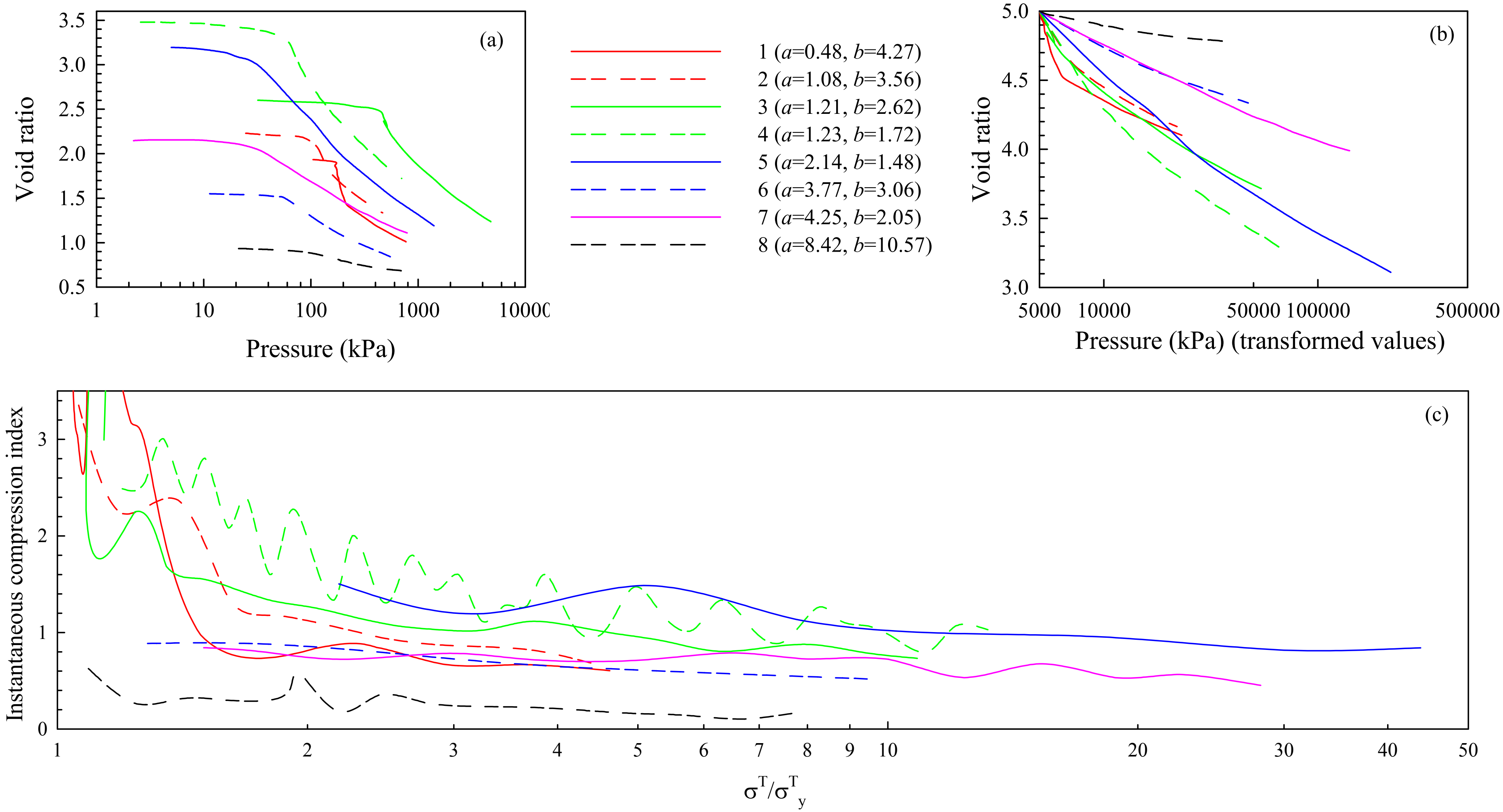


Figure 5.8. Demonstration of the capability of the model parameters (a) actual compression curves

(b) transformed compression curves (c) instantaneous compression indices of different structured soils in the post-yield region

5.2.4 Parametric study

Delage (2010) has shown through mercury intrusion porosimetry and scanning electron microscopic studies that the compression behaviour immediately after the yield is governed by the progressive collapse of larger inter-aggregate macro pores formed by inter-aggregate bridges while the micro-pores within the clay aggregates are generally unaffected. The micro-pores within the clay aggregates starts getting affected only after the macro-pores have been significantly collapsed. Nagaraj et al. (1990) showed that the virgin compression of structured soils is better represented by a bi-linear relationship in a semi-logarithmic plot where the first linear part is controlled by the structure and the later is controlled by the soil composition. Recently Hong et al. (2012) reported that a bi-linear representation of the compression curve in a bi-logarithmic space is more appropriate for some of the structured soils. The above findings clearly indicate that a realistic compression model should include at least two parameters to represent the compression behaviour of structured soils over a wide stress range and this is one of the most important features of the proposed pressure-void ratio relationship.

For parametric study, Batiscan Clay (Tavenas et al., 1983) having a=0.79 and b=3.85 has been arbitrarily selected. In Figure 5.9(a), the parameter a is varied within the range of 0.1 to 10 while keeping the value of b fixed at 3.85. In Figure 5.9(b), the value of a is kept constant at 0.79, while the parameter b is varied from 1.5 to 10. The influences of the parameters a and b on the compression behaviour are summarized below:

- A smaller value of a is associated with rapid de-structuration and sharper transition between the de-structuration and post-de-structuration ranges. This type of behaviour can be observed for highly sensitive soils with weak structures such as Mexico City clay. For increased values of the parameter a, the rate of de-structuration and the transition to the de-structured state is more gradual as can be observed for stiff clays such as Pietrafitta clay and Pappadai clay (for values of the parameters refer to Table 1).

- Soils having higher compressibility in their de-structured states will have smaller values of the parameter b. For example, the post de-structuration compression index (C_c) of Mexico City clay is very high ($C_c \approx 3$) and the b value from the simulation has been found to be comparatively smaller ($b \approx 0.5$).

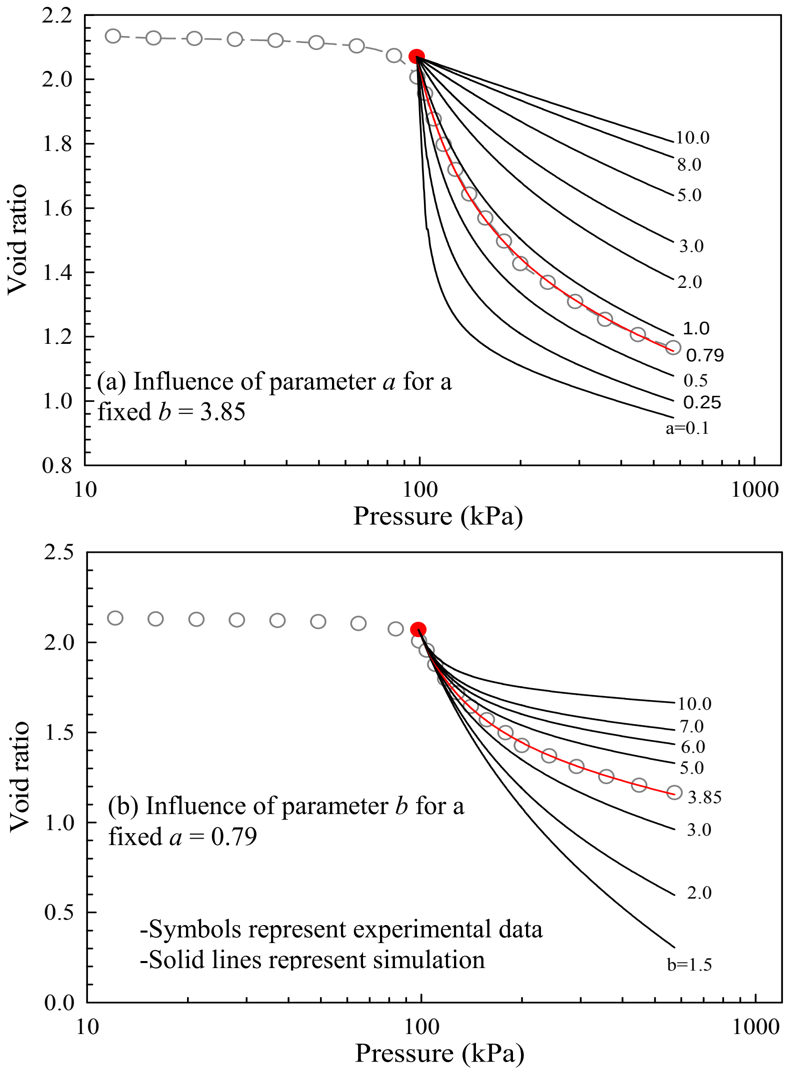


Figure 5.9. Parametric study (a) Influence of parameter a for a fixed b = 3.85 (b) influence of parameter b for a fixed a = 0.79

5.2.5 Factors affecting the model parameters

Leroueil et al. (1983) showed that compressibility of structured soils can be correlated with the sensitivity and void ratio where compressibility increases with an increase in either the void ratio or the sensitivity or the both. For the soils analysed, the parameter a, which controls the compression behaviour in the de-structuration zone, has also been found to be influenced by both the sensitivity and the void ratio where the parameter a decreases with an increase in either the void ratio or the sensitivity.

For the soils analysed, the value of the parameter a is found to be the lowest for Mexico City clay (Test-15 in Figure 5.7a) implying its rate of post-yield de-structuration to be the highest among all the soils analysed. Although sensitivity of Mexico City clay is much lower than that of some other clays (Batiscan Clay, Mattagami clay), its extremely high rate of post-yield de-structuration can be attributed to its unusually high initial high void ratio. Figure 5.7(a) shows two different compression curves for Mexico City clay denoted by Test-14 and Test-15. The value of the parameter a for Test-14 is much higher than that for Test-15 which is possibly due to the considerably lower initial void ratio of the sample used in Test-14. The effect of void ratio on the parameter a can also be observed for Mattagami clay (Figure 5.7c). The value of the parameter a for the sample of Mattagami clay reported by Tavenas et al. (1983) is significantly smaller than that of the sample reported by Sangrey (1972). Higher void ratio at yield of the sample used by Tavenas et al. (1983) could be one of the reasons for its comparatively higher rate of post-yield de-structuration and therefore a comparative smaller value of the parameter a.

If the case of Mexico City clay is kept out of consideration due to its incomparable void ratio and only the test data of soft clays for which sensitivity values are available is considered, it can be seen that the value of the parameter a decreases with increasing sensitivity. The yield void ratios of the clays considered here vary within a narrow range (1.72-2.17). Among the test results presented on soft clays, Bothkennar clay (Figure 5.7e) has the lowest sensitivity at 4 (Smith et al., 1992) and it has the highest value of the parameter a (2.09-2.72). The sensitivities of Mattagami clay (Figure 5.7c) and Batiscan clay (Figure 5.7b) are 80 and 125 respectively and the values of the parameter a for them are 1.08 and 0.60 respectively.

For a given soil, the sensitivity may be affected by the degree of sampling disturbance and parameter a has also been found to be able to capture this disturbance-induced variation of sensitivity. To elaborate this, the results reported by Smith et al. (1992) on two samples of Bothkennar clay sampled from similar depth (5.3-6.2m) but using different samplers is considered. The value of the parameter a for the sample collected by Sherbrooke sampler is lower than that of the sample collected by Laval sampler. Smith et al. (1992) hinted that the sample collected by Sherbrooke sampler was less disturbed than the one collected by Laval sampler. Due to relatively intact state of the sample collected by Sherbrooke sampler, its rate of post-yield de-structuration can be expected to be higher and this is reflected by its comparatively lower value of the parameter a.

The values of the parameter a for stiff clays presented in Figure 5.7(f) have been found to be considerably higher than that of the soft clays discussed above indicating that the rate of post-yield de-structuration of these stiff clays are much slower. Among the two stiff clays presented, both the Pappadai clay and the Pietrafitta clay have similar plasticity indices but their initial void ratios are significantly different. The value of the parameter a for Pietrafitta clay is much smaller than that of Pappadai clay possibly due to the higher initial void ratio of Pietrafitta clay.

It has been found that although the value of the parameter a for any given soil varies depending on the initial void ratio and sensitivity, the parameter b for any particular soil varies within a much narrower range (Table-1). For the case of Mexico City clay, the parameter a for Test-14 was larger than three times the value obtained for Test-15. However, their b values are very close to each other (0.49-0.54) which implies that their compression behaviour in their respective de-structured states are similar possibly due to their similar mineralogical compositions. The influence of soil composition on the values of the parameter b can also be observed for other soils. It was found that for Bothkennar clay (Smith et al., 1992) the parameter a was affected by the degree of sample disturbance while the parameter b for both the samples were found to vary in a much narrower range (2.64-2.72). Another compression result on Bothkennar clay (Nash et al., 1992) having similar index properties produces a value of 2.34 for the parameter b. If the natural variability in the soil composition and the experimental error in the determination of plasticity index are taken into consideration, the b value obtained for the result reported by Nash et al. (1992) can be considered to be close to the value obtained for the samples used by Smith et al. (1992). The values of the parameter a for the two stiff clays discussed earlier were significantly different but their b values have been found to vary within a much narrower range (3.31-4.05) most likely due to their very similar plasticity indices (34.5-35).

5.3 Summary

A new pressure-void ratio relationship for structured soils in the virgin compression range has been developed. This relationship has been developed based on a hyperbolic variation of a newly proposed secant compression index with pressure and contains two parameters a and b. The values of the parameters can be determined from routine laboratory compression test. It has been demonstrated that parameter a controls the post-yield de-structuration behaviour while the other parameter b reflects the compressibility in the de-structured state of the soil. A lower value of parameter a indicates very rapid post-yield de-structuration whereas a lower value of the parameter b implies higher compressibility in the de-structured state of the soils. It has been found from the analysis of compression data of several structured soils that the value of the parameter a decreases with increasing values of sensitivity and initial void ratio. Although for a given soil the parameter a has been found to vary depending on sensitivity and void ratio, the parameter b has been found to vary within a much narrower range.

5.4 References

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CHAPTER 6: COMPRESSION BEHAVIOUR OF ARTIFICIALLY CEMENTED SOILS

6.1 Introduction

Artificial cementation modifies the compression behaviour of the parent soil significantly. Readily noticeable change in the compression behaviour due to artificial cementation is the increase in yield strength. In addition to modifying the yield strength, cementation also alters the post-yield compression behaviour significantly. In contrast to the compression behaviour of un-cemented soils, whose behaviour can generally be represented by a constant compression index, the compression index of cemented soil is pressure-dependent and its variation with pressure is characterized by the nature and degree of cementation. Although there have been extensive studies on the progressive de-structuration and its effect on the resulting non-linear (in semi-logarithmic space) virgin compression behaviour of natural soils (Loreueil et al., 1983; Mesri and Choi, 1985; Leroueil and Vaughan, 1990, Burland, 1990; Hong et al., 2012), very few studies (Kamruzzaman, 2002) have focused on the same non-linearity that may be encountered in the virgin compression behaviour of artificially cemented soils.

In this chapter, findings from a detailed investigation on the compressibility behaviour of an artificially cemented soil (lime-slag treated CIS) up to a high pressure (8000 kPa) are critically analyzed with particular focus on the post-yield de-structuration process and resulting non-linearity of the virgin compression behaviour. The analyses are carried out with the help of a new compression modeling framework developed through a comprehensive study of the compression behaviour of a wide range of naturally structured soils originating from diversified geological backgrounds. The framework proposed for natural soils has been found to be equally applicable for artificially cemented soils. The analyses also include the study of the compression behaviour of some other soils treated with Portland cement (data obtained from literature). It will be shown that the overall compressibility behaviour and de-structuration characteristics of different types of cemented soils can be objectively compared in quantitative terms with the help of the proposed framework.

6.2 Compression behaviour of lime-slag treated CIS

The main factors that influence the mechanical behaviour of a particular treated soil are additive contents and curing time. For lime-slag treated CIS, the effects of lime content, slag content and curing time on the compression behaviour are discussed in the following sections. For studying the effect of any particular experimental variable on the compressibility behaviour, the compression data over the whole stress range is first presented in a conventional $e - \log(\sigma')$ space and then only the virgin compression range of the data is presented on a transformed pressure-void ratio space in a separate plot. It has been demonstrated in Chapter 5 that presenting the compression data on a transformed pressure-void ratio space provides a rational basis for carrying out comparative study of the virgin compression behaviour of different types of structured soils. The technique for presenting the virgin compression data on transformed space has been discussed in details in Chapter 5. The data in the X axes of these transformed pressure-void ratio spaces has been presented in terms of $\log(\frac{\sigma^T}{\sigma_y^T})$ instead of $\log(\sigma^T)$ noting that the quantities $\log(\frac{\sigma^T}{\sigma_y^T})$ and $\log(\frac{\sigma'}{\sigma_y'})$ are equal and therefore the actual experimental compression curves and the transformed compression curves become identical in $e - \log(\frac{\sigma^T}{\sigma_y^T})$ space. Moreover, presenting the data in $e - \log(\frac{\sigma^T}{\sigma_y^T})$ space also eliminates the necessity of dealing with imaginary stress values associated with transformed pressure-void ratio space. When presented in $e - \log\left(\frac{\sigma^T}{\sigma_y^T}\right)$ space, the relative values of the compressibility of different materials also maintain exactly the same order as it is seen on a $e - \log(\sigma^T)$ space.

6.2.1 Results

6.2.1.1 Effect of lime content on the compression behaviour of lime-slag treated CIS

The effect of lime content on the compressibility behaviour is presented in Figure 6.1 and 6.2. While Figure 6.1 shows the experimental data on conventional $e - \log(\sigma')$ space, Figure 6.2 shows the virgin compression part of the data on the transformed pressure-void ratio space. Figure 6.3 shows the effect of lime content on the yield strength determined by Casagrande method. Figure 6.1 shows that lime has only a minor influence on the compression behaviour of lime-slag treated CIS. The most noticeable influence of lime is observed at one month of curing and its influence gradually diminishes with the passage of time. Figure 6.3(a) shows that, at one month of curing, yield strength is practically independent of lime content for 10% slag treated CIS. The same figure shows that for the soil treated with 15% slag, yield strength decreases slightly whereas for 20% slag treated soil, yield strength decreases considerably with an increase in lime content at one month of curing. The effect of lime content on the post-yield compressibility behaviour can be studied with the help of the transformed compression curves presented in Figure 6.2. Figure 6.2(i) shows that at one month of curing and for a fixed slag content of 10%, soil treated with 15 and 20% lime contents exhibit similar post-yield compressibility which is lower than the compressibility observed for 10% lime treated samples. Figure 6.2(ii) shows that at one month of curing, for soil treated with 15% slag, 10 and 15% lime produces almost identical compressibility which is higher than the compressibility observed for 20% lime treated samples. Figure 6.2(iii) shows the compressibility behaviour of soil treated with a fixed slag content of 20% and various lime contents. The effect of lime content on the post-yield compressibility for this case is very similar to its effect observed for CIS treated with 15% slag.

Figure 6.1(iv)-(vi) show the compression curves for CIS treated with different combinations of lime and slag at three months of curing. It can be seen from these figures that whatever effect lime had on the compression behaviour at one month almost disappears at three months of curing. From Figure 6.3(b) it can be seen that the yield strength appears to be almost independent of lime content at three months of curing. The post yield compressibility behaviour of samples treated with different lime contents but a fixed slag content is almost identical as evidenced by the merging of the compression curves in Figure 6.1(iv)-(vi). The corresponding compression curves in transformed space are presented in Figure 6.2(iv)-(vi) which show that at three months of curing post-yield compressibility marginally increases with an increase in lime content. At six months of curing, the tendency of the compression curves to band together increases from that observed at three months. Figure 6.3(c) shows that at six months of curing there is slight tendency of the yield strength to increases with an increase in lime content. From Figure 6.2(vii)-(ix) it can be observed that although all the transformed compression curves for CIS treated with a particular slag content and various lime contents are located within a narrow band at six months of curing, compressibility tends to increase marginally with an increase in lime content.

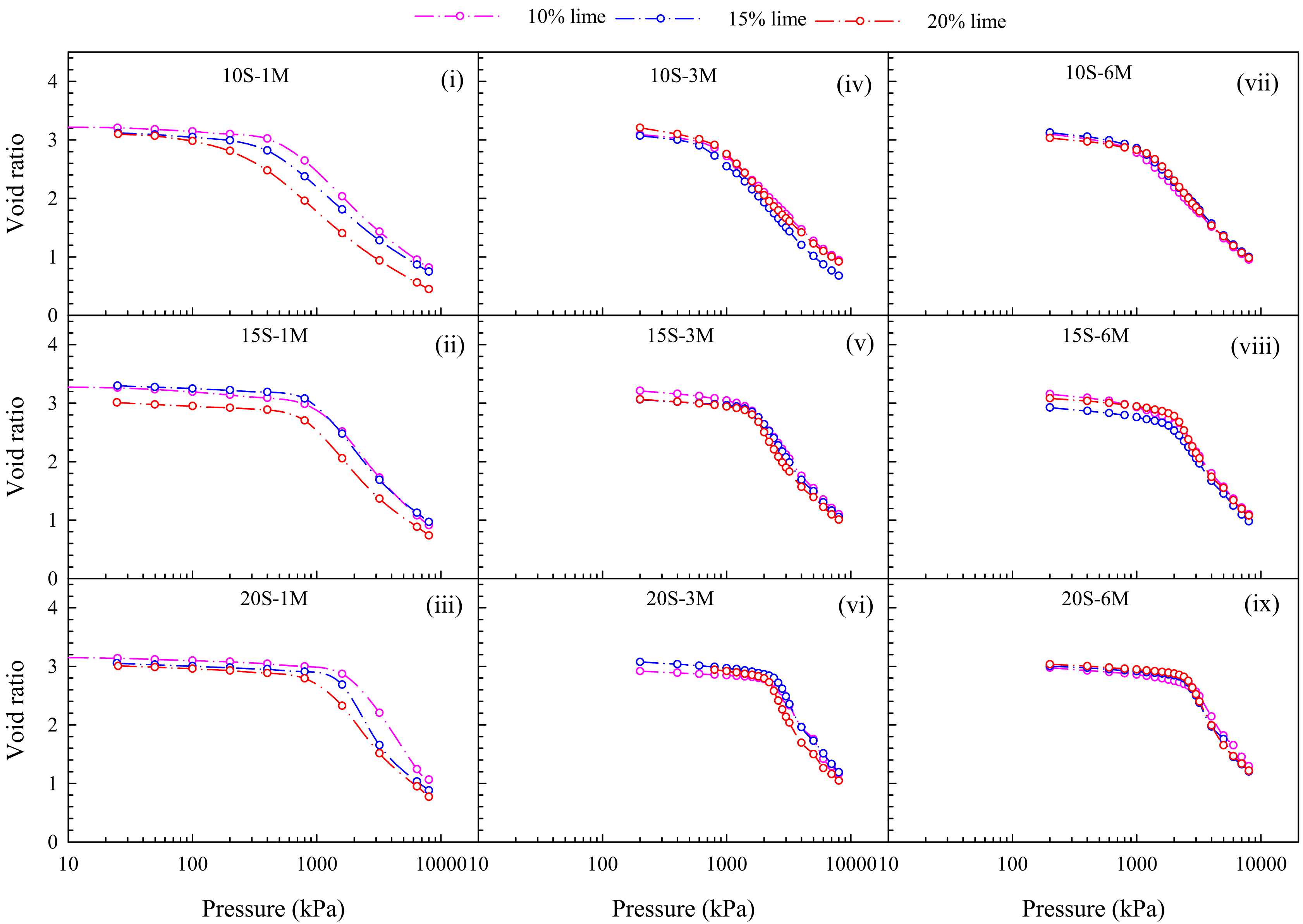

Figure 6.1. Effect of lime contnet on the compression behaviour of lime-slag treated CIS

Notes:

- Throughout the discussion of the results, L = % of lime, S = % of GGBFS and M = curing period in months
- Stanley (2010) found the specific gravity of CIS treated with various combinations of lime and GGBFS and cured for different periods vary within a narrow range of 2.5-2.6. Specific gravity of treated CIS was not measured in this study and a constant value of 2.55 has been used in all cases for the determination of void ratio.

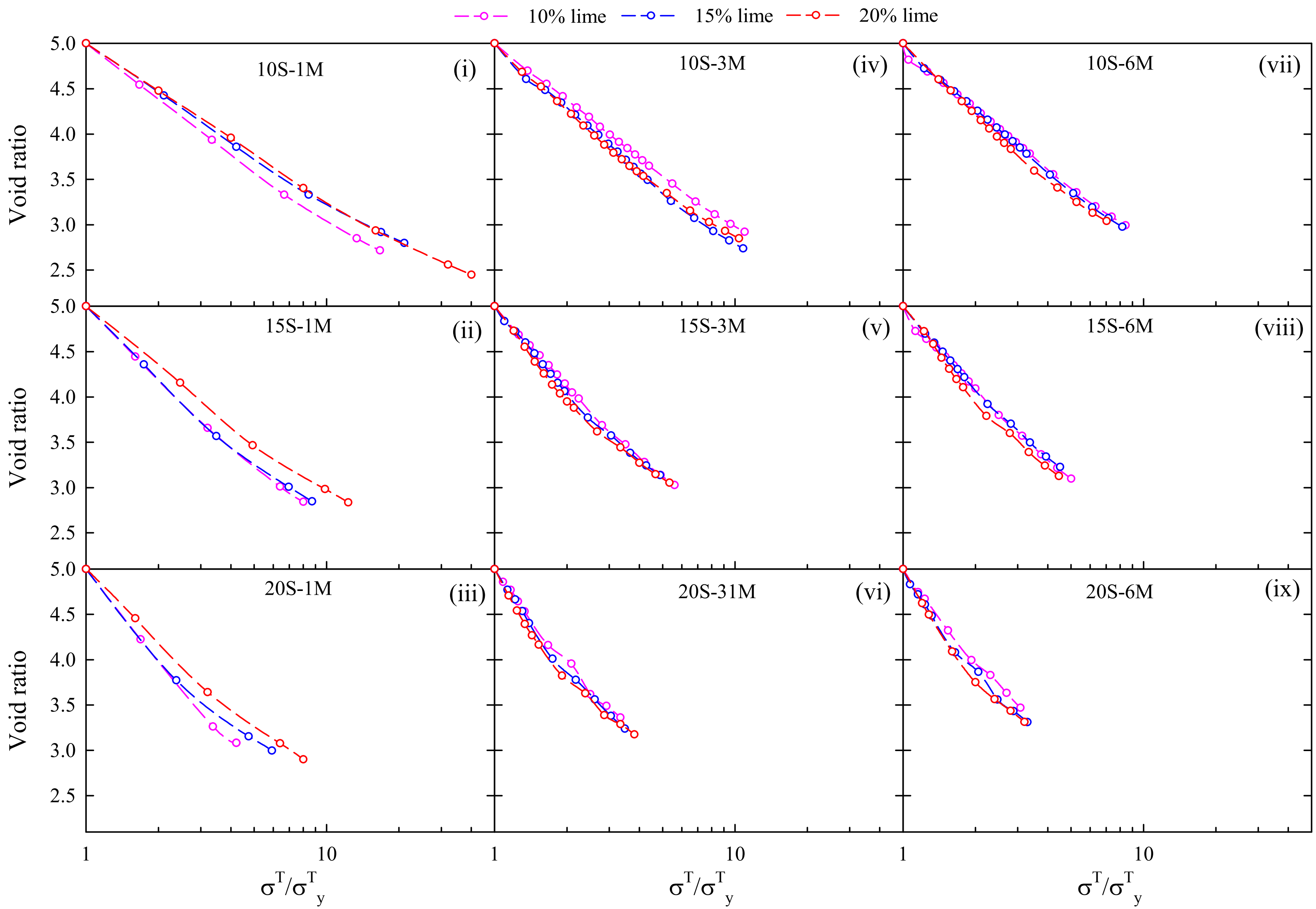

Figure 6.2. Effect of lime contnet on the virgin compression behaviour of lime-slag treated CIS on transformed pressure-void ratio space

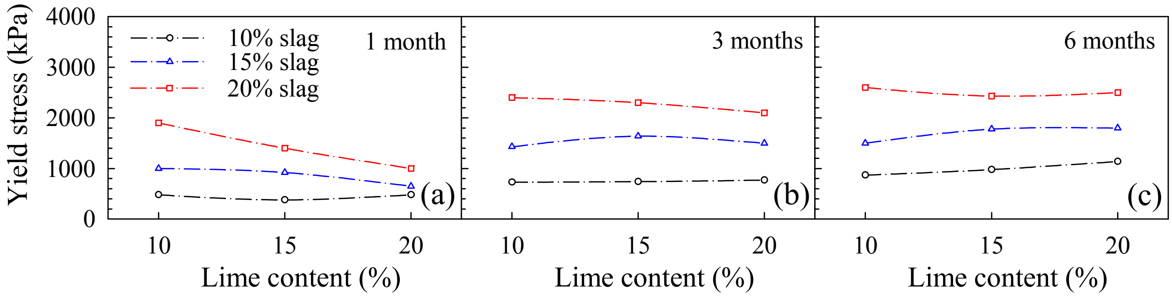

Figure 6.3. Effect of lime content on the 1-D yield stress of lime-slag treated CIS

The main observations from the effect of lime content on the post-yield compressibility behaviour of lime-slag treated CIS can be summarized as follows:

- Lime has only minor influence on the compressibility behaviour of lime-slag treated CIS and the way lime influences the compressibility behaviour changes with curing period.

- The influence of lime is most prominent at one month curing. At three and six months, the influence of lime on the yield strength and post-yield compressibility decreases noticeably from that observed at one month. At one month of curing, the yield strength and post-yield compressibility is found to decrease considerably with an increase in lime content whereas an increasing lime content tend to marginally increase the yield strength and post-yield compressibility at three and six months of curing.

- It has been found that whenever a change in lime content causes an increase in yield strength, it also causes an associated increase in the post-yield compressibility and vice versa.

1-D yield stress, post-yield compressibility and UCS are all controlled by the same mechanical properties and distribution of the cementitious bonds within the treated soils mass and therefore, lime can be expected to have similar effect on all of them. However, the observed influence of lime on the 1-D yield strength and post yield compressibility behaviour at one month of curing is somewhat different from its influence observed on UCS values at the same curing period. In Chapter 4 it was found that the quantity of lime does not influence the UCS values to any significant extent for the range of lime contents investigated whereas 1-D yield stress and post-yield compressibility have been found to decrease with an increase in lime content at one month of curing. This observation on the effect of lime content on the compressibility behaviour, especially on the yield strength, indicates that when the raw CIS is treated with an amount of lime which is in excess of what is actually required may have some negative impact on the strength of the developed cementitious bonds. Bell (1996) suggested that since lime neither has appreciable friction nor cohesion, excess lime serves as lubricant and thereby reduces the strength. Kumar et al. (2007) made the platy shape of the un-reacted lime particles responsible for the drop of strength observed when an excessive amount of free lime is present in the reaction environment. Kolawole (1998) and Dash and Hussain (2012) also reported that strength reduced appreciably when the soils were treated with an excessive amount of lime.

The observed difference in the influence of lime on the UCS and compressibility behaviour may be due to the difference in the boundary conditions employed in the UCS and 1-D compression tests. In UCS tests, load is applied under zero lateral confinement condition whereas in oedometer compressibility test, load is applied under laterally constrained condition. In oedometric compression, lateral confinement may create an opportunity for particle crashing to take place after the breakdown of cementitious structures but the particle crashing mechanism may not be possible under uni-axial unconfined loading condition employed in UCS testing. In the absence of other convincing evidences, it is speculated that with the help of possible particle crashing mechanism that may support some additional loads, slightly stronger bonds (which is possibly the case for soil treated with a lower amount of lime) may exhibit more prominent influence on the 1-D yield strength in oedometric compression than on the unconfined strength values.

6.2.1.2 Effect of slag content on the compression behaviour of lime-slag treated CIS

The effect of slag content on the compression behaviour can be seen in Figure 6.4 and 6.5 and its effect on the yield stress can be observed in Figure 6.6. It can be observed from these figures that, yield strength increases with increasing slag content for all the lime contents used and for all the curing periods investigated. From Figure 6.6(a) it can be seen that at one month of curing, the influence of slag on the yield strength is considerably affected by the lime content. It is found that at one month of curing, the rate of increase of yield strength with slag content for 10% lime treated sample is higher than the rate of increase observed for 15 and 20% lime treated samples. However, at three and six months of curing, the influence of slag on the yield strength appears to be almost independent of lime content as can be seen from Figure 6.6(b) and (c). It can also be seen from Figure 6.4 and Figure 6.5 that, in addition to yield stress, post-yield compression behaviour is significantly affected by the slag content for all the cases. It is seen that although higher slag produces higher yield strength, the compressibility of treated CIS increases considerably with an increase in slag content. It can also be seen from Figure 6.4 that at large stresses the compressibility characteristics of CIS treated with different amounts of slag tend to become similar.

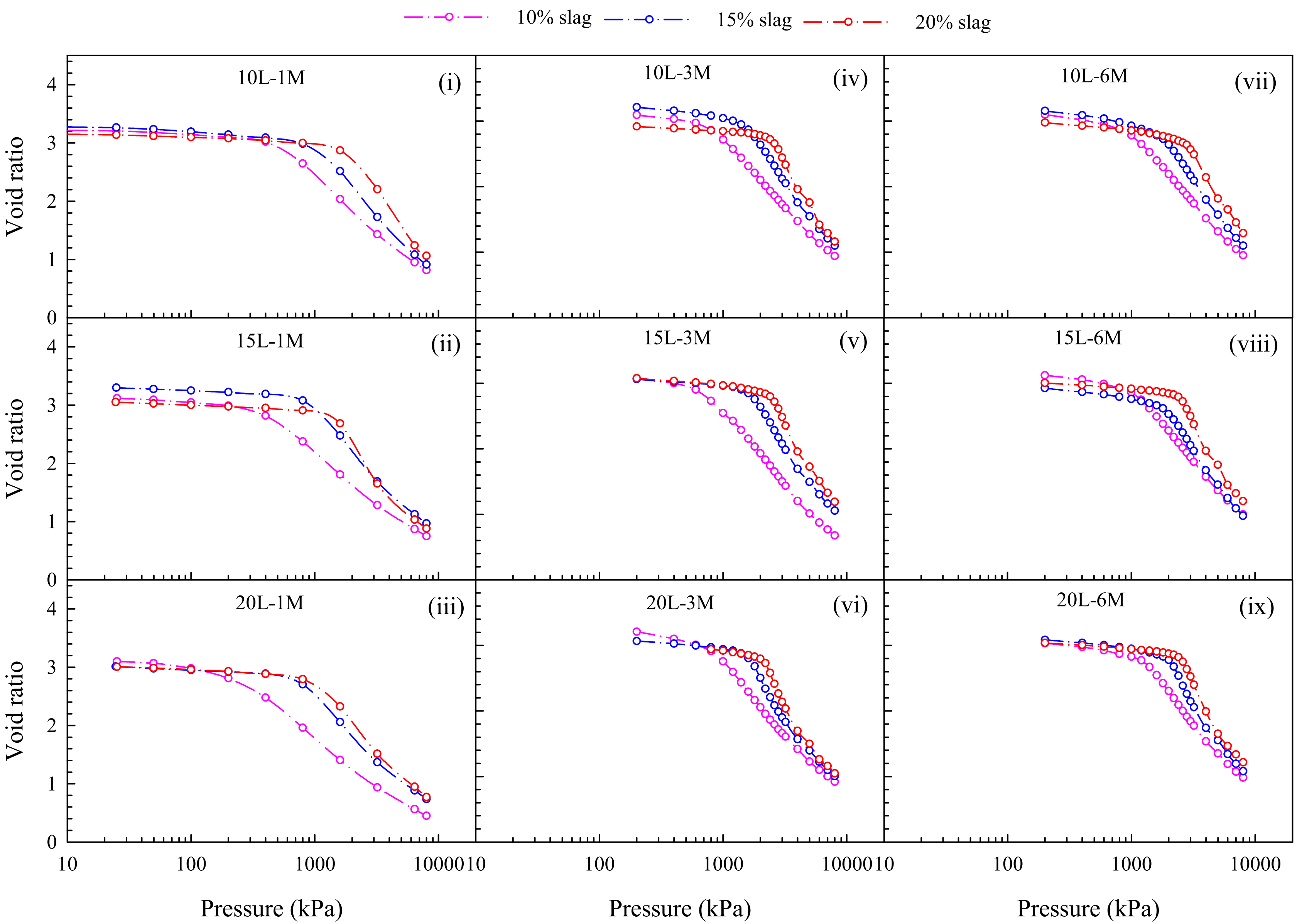

Figure 6.4. Effect of slag content on the compression behaviour of lime-slag treated CIS

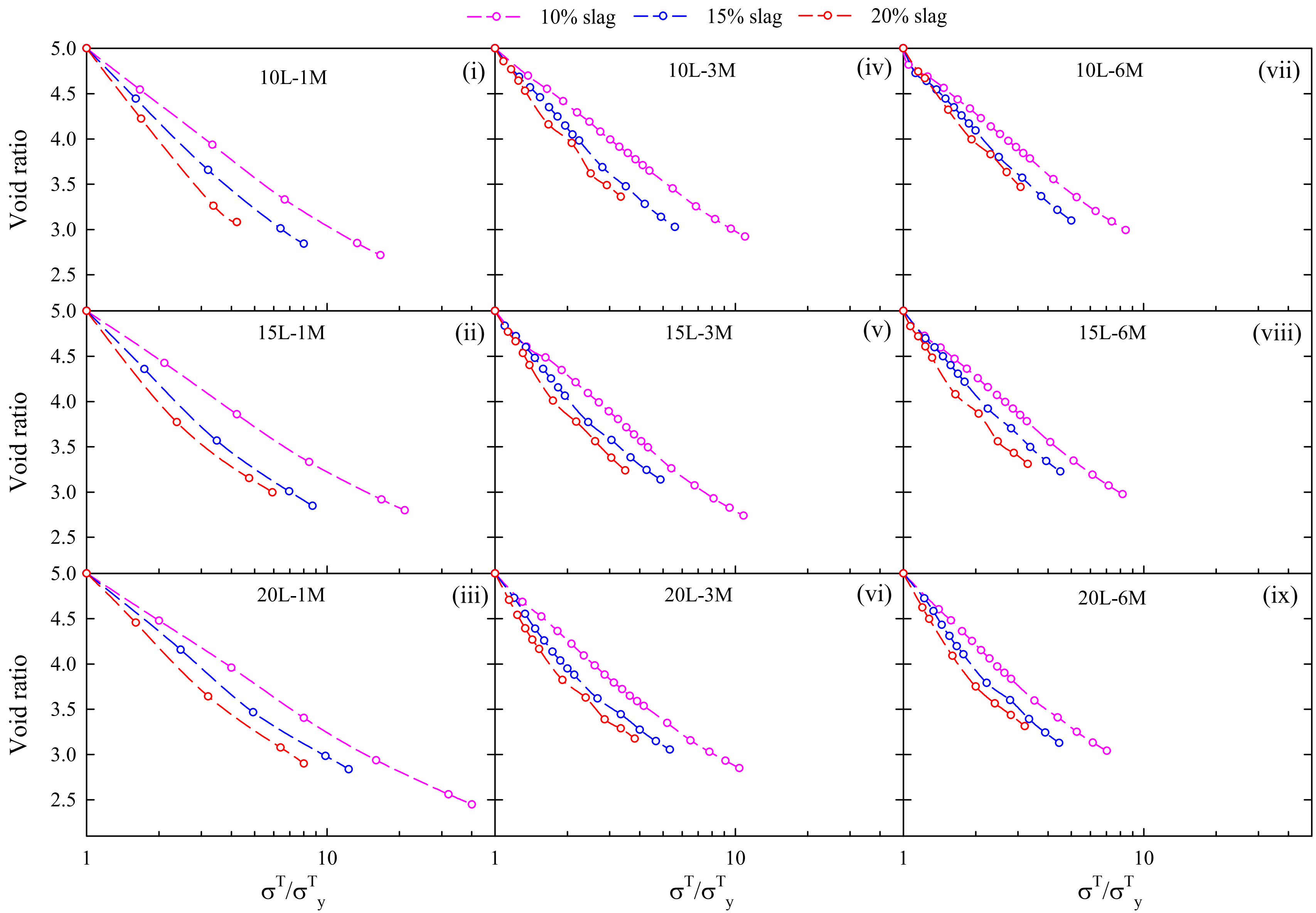

Figure 6.5. Effect of slag contnet on the virgin compression behaviour of lime-slag treated CIS on transformed pressure-void ratio space

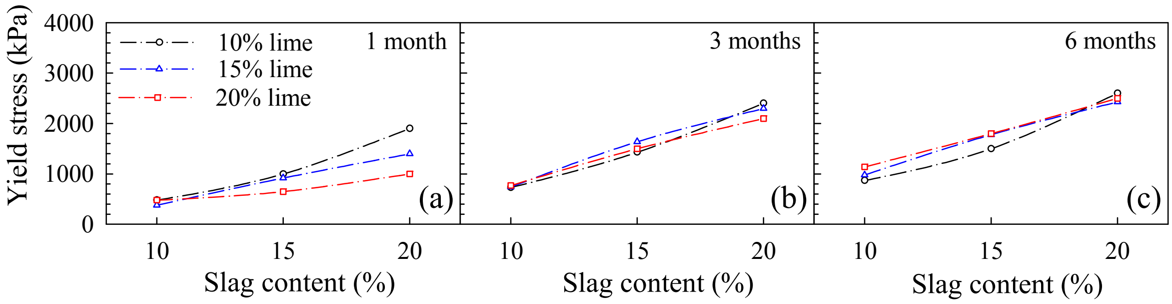

Figure 6.6. Effect of slag content on the 1-D yield stress of lime-slag treated CIS

The main observations from the study of the effect of slag content on the compression behaviour can be summarized as follows:

- Increasing slag content increases yield strength.
- Increasing slag content increases post-yield compressibility of the treated soil mass.
- At shorter curing period, the influence of slag is affected by the lime content used. At shorter curing period, the rate of increase of yield strength with slag content is higher for samples treated with lower amount of lime.
- At longer curing period, yield strength is mostly controlled by slag content and is almost independent of lime content.

6.2.1.3 Effect of curing time on the compression behaviour of lime-slag treated CIS

The effect of curing period on the compression behaviour is presented in Figure 6.7 to Figure 6.9. It can be seen from Figure 6.7 that the effect of curing on both the yield strength and post-yield compressibility behaviour gradually diminishes as the curing progresses. Figure 6.9 shows that the rate of increase of yield stress from one month to three months is significantly higher than the rate of increase observed from three months to six months. It can also be observed that the rate of increase of yield strength with curing is affected by the lime content. Figure 6.9 clearly shows that for 15 and 20% lime treated samples [Figure 6.9(b) and Figure 6.9(c) respectively], yield strength increases more rapidly with an increase in curing from one month to three months than for 10% lime treated samples [Figure-6.9(a)]. The effect of curing on the post-yield compressibility behaviour can be conveniently studied with help of the transformed compression curves presented in Figure 6.8. It is observed from Figures 6.8(i), (iv) and (vii) that for 10% lime treated samples curing has very marginal influence on the post-yield compressibility behaviour where longer curing produces slightly higher compressibility. On the other hand, the influence of curing becomes more pronounced with an increase in lime content. It can be seen from Figure 6.8(ii) that for a fixed lime content of 15% and a slag content of 10%, the compressibility increases significantly from one month to three moths and thereafter the rate of change with curing becomes negligible.

However, for the fixed lime content of 15%, the effect of curing on the post-yield compressibility is only marginal when the sample is treated with either 15% or 20% slag as can be seen from Figures 6.8(v) and (viii) respectively. For 20% lime treated samples, the effect of curing on the post-yield compressibility behaviour is the most obvious. It can be seen from Figure 6.8 (iii), (vi) and (ix) that when the soil is treated with a fixed lime content of 20% and different slag contents, post-yield compressibility at three months is significantly higher than the compressibility observed at one month of curing. However, for the soil treated with 20% lime, the effect of curing becomes negligible beyond three months of curing as was the case with the soil treated with 15% lime.

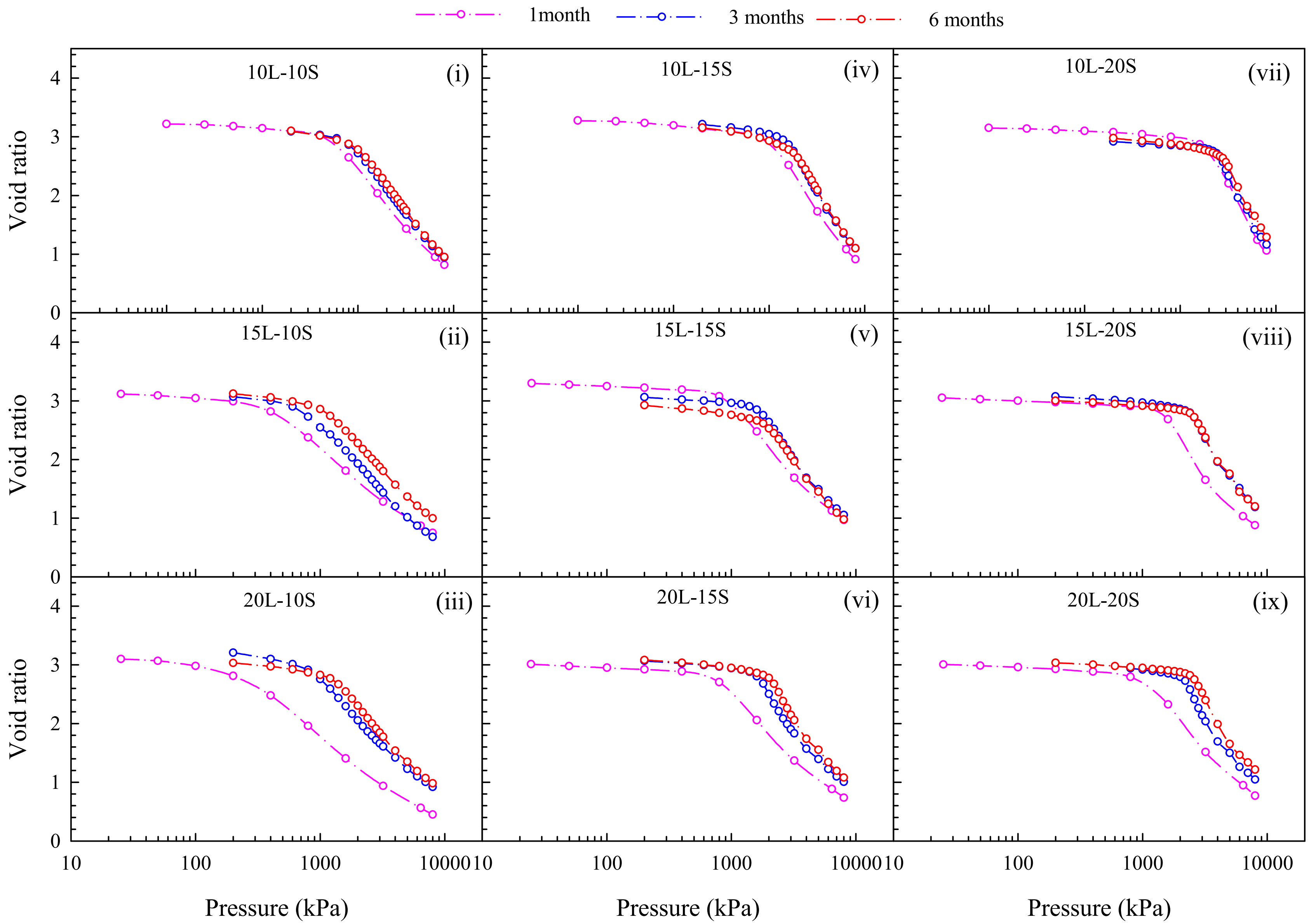

Figure 6.7. Effect of curing time on the compression behaviour of lime-slag treated CIS

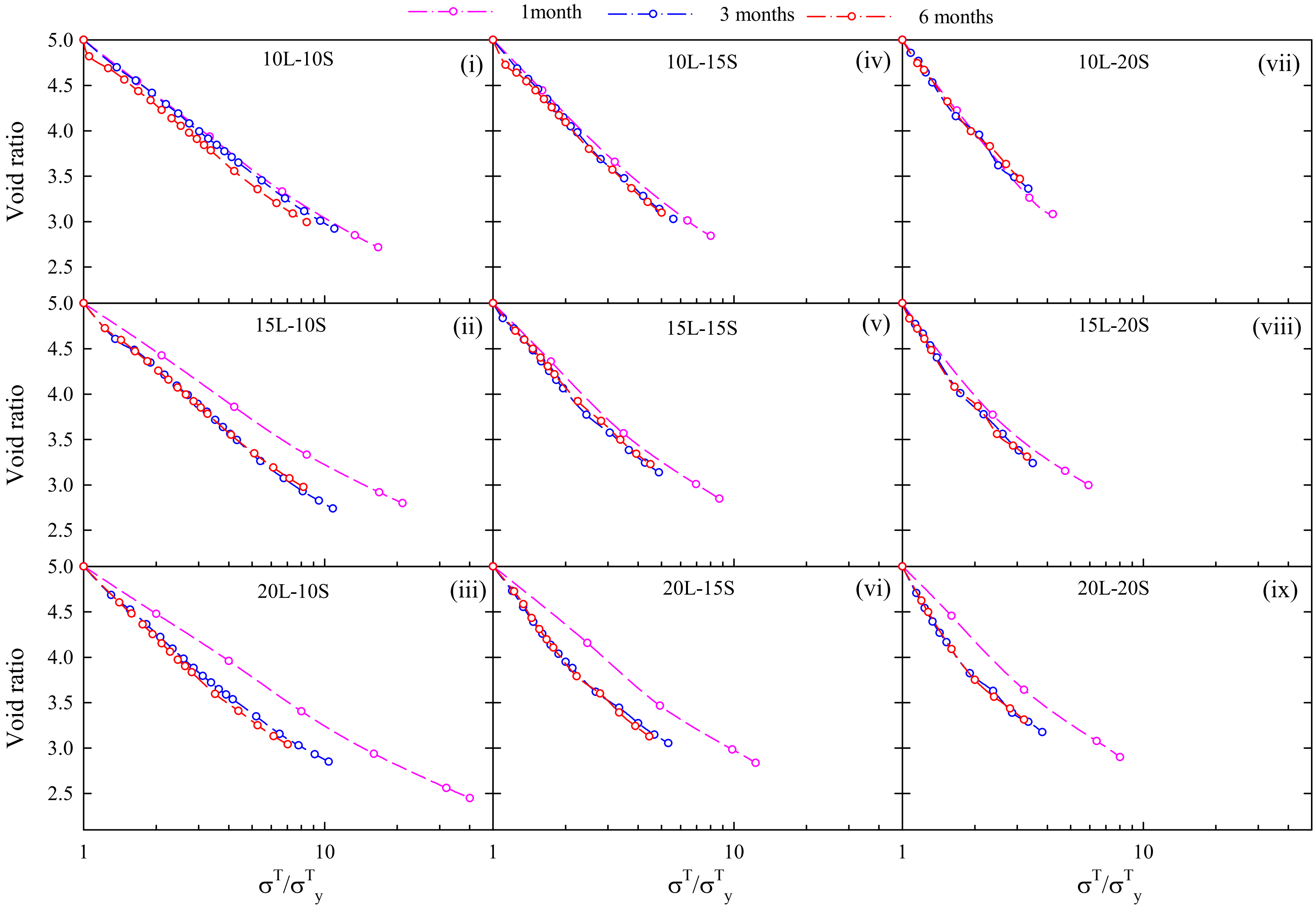

Figure 6.8. Effect of curing time on the virgin compression behaviour of lime-slag treated CIS on transformed pressure-void ratio space

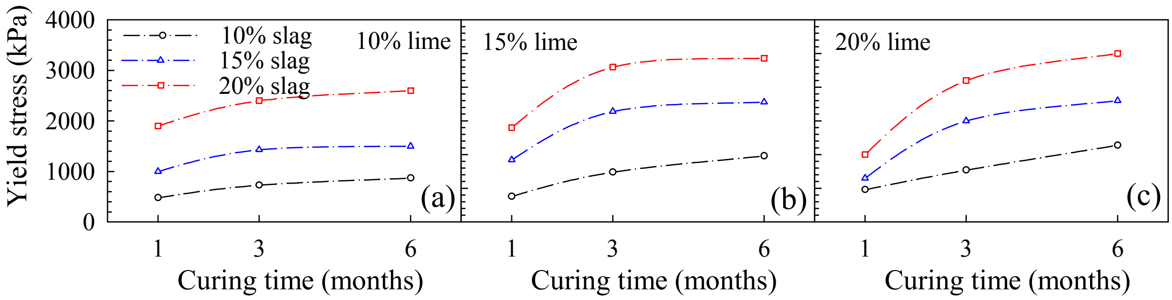

Figure 6.9. Effect of curing time on the 1-D yield stress of lime-slag treated CIS

The main observations from the study of the effect of curing period on the compression behaviour can be summarized as follows:

- Both the yield strength and compressibility increases with an increase in curing period
- The effect of curing significantly decreases with an increase in curing period beyond three months.
- The effect of curing becomes more pronounced with an increase in lime content

6.2.2 Discussion

From the above discussion on the compressibility behaviour of lime-slag treated CIS, it is found that the experimental variable that has the greatest influence on the yield strength and post-yield compressibility behaviour is the amount of slag. The influence of slag is followed by the influence of curing time whose effect is most prominent at early curing periods and lime content has been found to have the least influence on the overall compressibility behaviour of lime-slag treated CIS. It is also seen that an increment in yield strength is generally associated with an increase in the post-yield compressibility. In the following sections the possible mechanisms behind these experimental observations are discussed.

The compression of structured soils is essentially a process of gradual collapse of pores within the treated soil mass in response to applied loading (Delage and Lefebvre, 1984; Delage, 2010). The main findings from the studies of Delage and Lefebvre (1984) and Delage (2010) which can be helpful in explaining the observed compression behaviour of lime-slag treated CIS are summarized below:

- Structured soils contain two types of pores: large inter-aggregate macro pores formed by soil aggregates interconnected by cementitious bridges and much smaller intra-aggregate micro pores.

- When loading is applied to a structured soil, at any given stress only a particular group of pores are affected and the group of pores to collapse at any instance is the ones with the largest size. Provided the cementing material remains unchanged, the magnitude of collapse stress for a particular group of pores increases with a decrease in the size of these pores. At the initial stage of the loading, the compression behaviour is controlled by the progressive collapse of larger inter-aggregate pores and when the larger inter-aggregate pores have been collapsed sufficiently, the compression behaviour is controlled by the collapse of the intra-aggregate micro-pores.

- Remolding of structured soils does not change the intra-aggregate micro-pore structure of structured soils to any noticeable extent. This observation about the effect of remolding action on the intra-aggregate micro pore structure has also been confirmed by Lapierre et al. (1990).

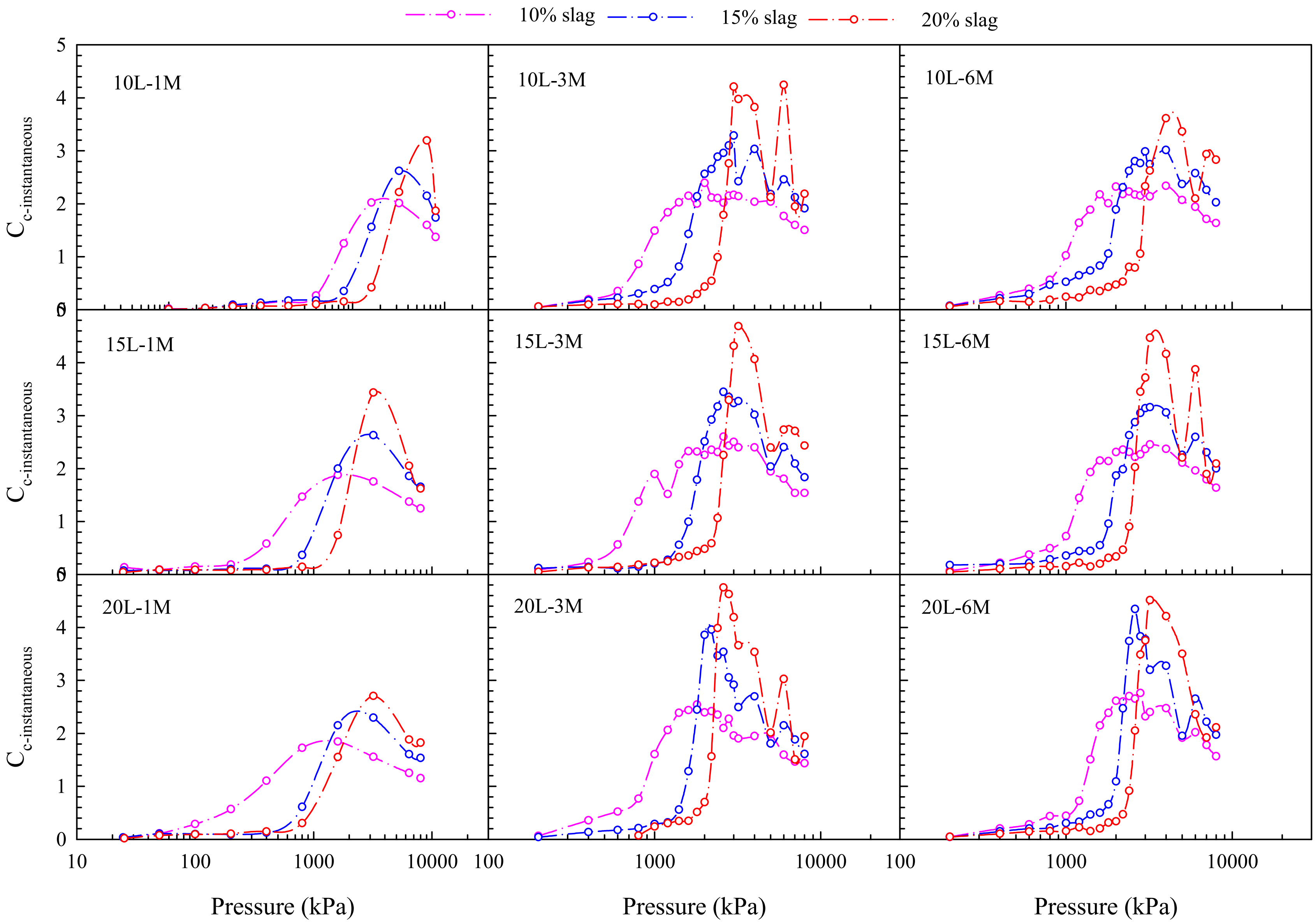

Figure 6.10. Instantaneous compression indices of CIS treated with different combinations of lime and slag at different curing periods

Delage and Lefebvre (1984) and Delage (2010) showed for a wide range of structured soils that the local value of the compression index at any given pressure is determined by the pore volume occupied by the group of pores which are at the threshold of being collapsed by the next increment of stress. Observing the generality of the above findings for a large group of structured soils, Delage (2010) went on to comment that *"Actually, compression of structured soils with a rigid porous matrix can be considered as a new PSD (particle size distribution) measurement technique. It is very probable that this trend will be found to be valid for other geomaterials with rigid structure, as those described by Leroueil and Vaughan (1990) for instance."*

The progressive collapse mechanism of pores and their effect on the compressibility behaviour can be investigated with the help of a study on the progressive change in the compression index with increasing pressure. The compression index of structured soils continuously changes with the removal of cementitious structures and when the structure is significantly removed, the compression index approaches approximately a constant value which is close to the value of the compressibility index of the corresponding reconstituted soil. Since the micro-pore structure within aggregates does not change considerably due to remoulding (Delage et al., 1984 and Lapierre et al., 1990), it is reasonable to assume that the de-structuration taking place until the soil compression index approaches approximately a constant value is primarily due to the breakdown of larger inter-aggregate pores. Therefore, a study on the effect of different experimental variables on the variation of instantaneous compression indices with pressure can be helpful in explaining the de-structuration characteristics of the cementitious bridges forming the larger inter-aggregate pore structures. It can provide information about the amount of volume that collapse at any particular incremental stress and also the stress range within which the compressibility behaviour is characterized by the progressive de-structuration of the larger inter-aggregate pores. Figure 6.10 shows the variation of instantaneous compression indices of lime-slag treated CIS with pressure at different curing periods. The instantaneous compression index ($C_{c-instantaneous}$) is the instantaneous slopes of the $e - log(\sigma')$ curves at different applied pressures.

Figure 6.10 shows that for all the treated soil samples, the $C_{c-instantaneous}$ vs. pressure profile has, in general, two clearly identifiable inflection points. The first inflection point represents the instance where the compressibility increases suddenly. This point can be taken as the yield strength where the cementitious structures start getting affected irreversibly. The second inflection point corresponds to the pressure at which the compressibility is the highest. After this second inflection point the rate of de-structuration gradually decreases. It is obvious that at the second inflection point majority of the cementation bonds associated with larger inter-aggregate pores are already damaged. It is reasonable to assume that the influence of structure on the compression behaviour is most dominant within the pressure range spanning between these two inflection points. The second inflection point can be considered as a transition point from where the behaviour is gradually more and more characterized by the response of the clay aggregates and gradually less and less by the collapse of the larger inter-aggregate pores. Therefore, the breadth of these two inflection points in the pressure axis can be taken as a very good indicator of the breakdown characteristics of the inter-aggregate cementitious structures. The narrower is the breadth of this pressure range, the more brittle and more compressible will the material behaviour be within that particular pressure band.

Figure 6.10 shows that lime has only minor influence on the $C_{c-instantaneous}$ vs. pressure profile. As discussed earlier, lime, when used in quantities above the lime saturation value, is not likely to affect the degree of cementation significantly and as a result the effect of lime content on the alteration of the property of the treated material should only be minimal. It can, therefore, be expected that the yield strength and the post-yield de-structuration behaviour of soil treated with different amounts of lime will be very similar which is clearly reflected in the $C_{c-instantaneous}$ vs. pressure plots.

It is observed from Figure 6.10 that slag influences the instantaneous compression indices significantly. The figure shows that before the first inflection point, the $C_{c-instantaneous}$ vs. pressure curves for the samples treated with lower amount of slag lie above those of higher slag treated soils. This observation implies that elastic compressibility of soils treated with lower amount of slag is higher. It can also be seen that the location of the first inflection point shifts rightward with an increase in slag content which is an indication of increasing yield strength with increasing slag content. As per the suggestion of Delage et al. (2010), an increase in yield strength is caused by the reduction of the size of the pores which collapse at yield stress and the result indicates that the size of the pores which collapse at yield stress may decrease with an increase in slag content. It can be observed that the width of the pressure band within two inflection points decreases with an increase in slag content implying that for soils treated with higher amount of slag, the collapse stress values for a large number of pores lie within a much narrower range.

The effect of curing on the $C_{c-instantaneous}$ vs. pressure behaviour can be observed by comparing the sub-figures of Figure 6.10 along the rows. It can be observed that there is a significant change in the $C_{c-instantaneous}$ vs. pressure profile from one month to three months but the change becomes insignificant for an increase in curing period from three months to six months. An increase in curing period from one month to three months tends to shift the location of the first inflection point rightward and also to shorten the width of the pressure bands within the two inflection points. The progressive slowing down of the rate of change of the $C_{c-instantaneous}$ vs. pressure profile with time can be attributed to the gradual cessation of cementitious reactions.

From the above discussion it can be observed that the yield strength and post-yield de-structuration are largely controlled by the degree of cementation. The higher is the degree of cementation, the higher is the yield strength and the narrower is the width of the pressure band within the two inflection points. If the proposition of Delage and Lefebvre (1984) and Delage (2010) is assumed to be valid for lime-slag treated CIS, it is reasonable to infer that the degree of cementation is likely to alter the compressibility behaviour by altering the characteristics of the network of pores developed within the treated soil mass. If the above framework is employed to explain the compressibility behaviour of lime-slag treated CIS, following conclusions regarding the influences of different experimental variables on the probable pore size distribution can be drawn:

- With a variation of lime content the pore size distribution does not change significantly.
- The influence of slag content on the yield strength value and on the width of the pressure band spanning between the two inflection points indicate that soils treated with higher amount of slag produces more uniform distribution of relatively smaller size pores.
- An increase in curing period increases the uniformity of pores and decreases the size of the pores which are affected at the yield stress. However, the influence of curing time on the pore size distribution gradually diminishes.

Ahnberg and Johansson (2005) showed that for different soils treated with different quantities of additives, the developed strength correlates well with the quantity of cementitious reaction products formed within the treated soil mass. It was observed in Chapter 4 that the UCS values were affected mostly by the quantity of slag and its influence on UCS was followed by the influence of curing time while lime content was found to have the least influence on the developed strength. Horpibulsuk et al. (2010) reported that when a particular soil is treated with a fixed amount of water and different amounts of cement, the amount of cementitious products formed within the treated soil mass increases with an increase in cement content causing an increase in the strength of the treated material. Based on the observations of Ahnberg and Johansson (2005) and Horpibulsuk et al. (2010), it can be inferred that the quantity of reaction products formed within the treated soil mass increases with an increase in slag content and curing time. On the other hand, lime is not likely to influence the quantity of reaction products considerably.

Horpibulsuk et al. (2010) also studied the effect of cement content on the progressive formation of cementation reaction products within the pore spaces of the treated materials. It was found in their investigation that with the increase in the quantity of cementitious reaction products, the size of the larger inter-aggregate pores decrease noticeably due to deposition of increasing amount of cementitious reaction products within the pore spaces. They reported that as the quantity of cementitious reaction products increased, the total volume occupied by the larger inter-aggregate pores of size >0.1μm decreased while the total volume occupied by the smaller inter-aggregate pores of size <0.1μm increased considerably. Choquette et al. (1987) studied the effect of lime stabilization on the pore size distribution of lime treated marine clay from eastern Canada and reported that the volume occupied by larger micro-pores was seen to decrease significantly with an increase in the quantity of hydration reaction products formed within the treated soil mass. The reduction in the size of the larger sized micro-pores was attributed to the fact that hydration reaction products partition each larger pore into a number of smaller ones. In a similar observation, Sasanian (2011), for another Canadian clay treated with Portland cement, made the same partitioning effect of cement hydration products responsible for the reduction in the volume occupied by larger sized inter-aggregate pores and an increase in the volume occupied by smaller inter-aggregate pores. These experimental findings clearly indicate that formation of increasing quantity of cementation products will result in the gradual shifting of the pore size distribution curves towards the smaller size. Due to the reduction in the size of the pores, the collapse stress of the pores will increase causing an increase in the yield stress. Moreover, due to an increase in the number of smaller pores, the density of the smaller sized inter-aggregate micro-pores will increase and an increasingly large number of pores are likely to collapse within a narrow pressure range. This type of alteration of the pore size distribution is likely to cause a reduction in the width of the pressure band between two inflection points described earlier and a significant increase in the compressibility when the applied stress approaches the collapse stress value of the more uniformly distributed group of smaller pores.

From the above discission it appears that the progressive pore collapse mechanism suggested by Delage and Lefebvre (1984) and Delage (2010) can be employed to explain the compressibility behaviour of lime-slag treated CIS. This inference is based on the observation that the probable pore size distribution deduced based on the interpretation of the compressibility data with the help of the postulate of Delage and Lefebvre (1984) and Delage (2010) appears to be supported by the findings of several other independent investigations on the effect of the degree of cementation on the resulting pore size distribution.

6.3 Analysis of compression behaviour of lime-slag treated CIS under the proposed compression modeling framework

In this section the compression behaviour of lime-slag treated CIS discussed in earlier sections is analyzed under the compression modeling framework proposed in Chapter 5.

6.3.1 Validation of experimental data on lime-slag treated CIS

The compression data of CIS treated with different combinations of additives at different curing periods is plotted in $e - \log\left(\frac{\sigma^T}{\sigma_y^T}\right)$ space in Figure 6.11. On the same plot, the $C_{cs}\, vs. \log\left(\frac{\sigma^T}{\sigma_y^T}\right)$ curves for all the cases are also presented. The model predictions corresponding to $e - \log\left(\frac{\sigma^T}{\sigma_y^T}\right)$ space are also presented in the above figure by solid lines. The main objective of presenting the experimental data in the above form is to validate the applicability of assumed hyperbolic relationship between C_{cs}^T and $\log(\sigma^T)$ and the capability of the proposed model to simulate the experimentally observed compression curves of lime-slag treated CIS. It is also examined here whether the hyperbolic parameters can accurately characterize the de-structuration behaviour of lime-slag treated CIS. The values of the parameters, quality of fit and the magnitude of errors corresponding to different simulated cases are presented in Table 6.1. The error has been reported in terms of the absolute value of the error as percentage of the void ratio at yield.

It can be seen from Figure 6.11(i) that for a fixed lime content of 10% and a curing period of one month, the rate of post-yield de-structuration significantly increases with an increase in slag content. From the corresponding $C_{cs}\, vs. \log\left(\frac{\sigma^T}{\sigma_y^T}\right)$ profiles it can be seen that the initial slope of these hyperbolic profiles also increases significantly with an increase in the slag content. Higher initial slope of the C_{cs} profile is associated with a

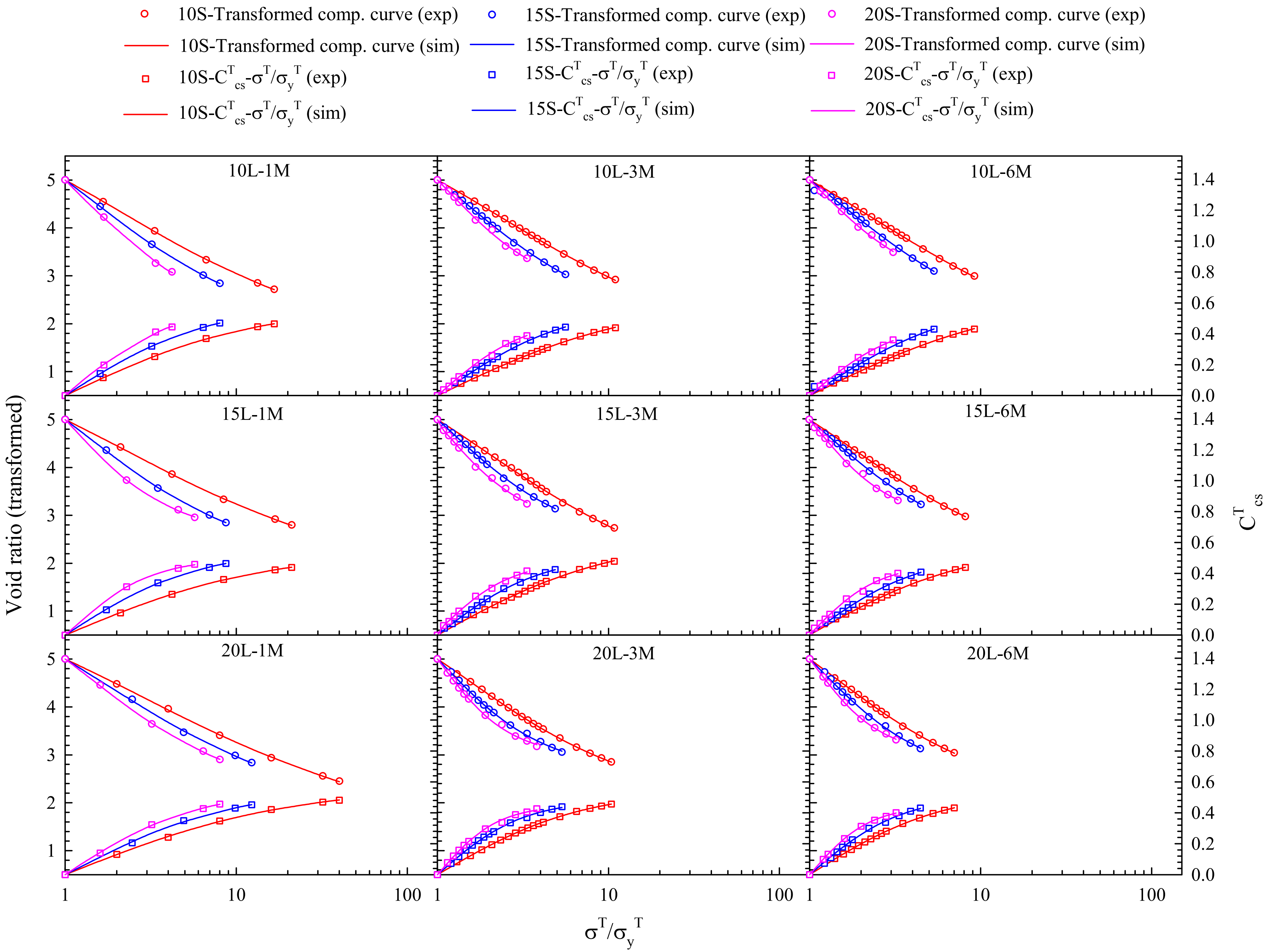

Figure 6.11. Experimental and simulated transformed compression curves and corresponding C_{cs}^T profiles

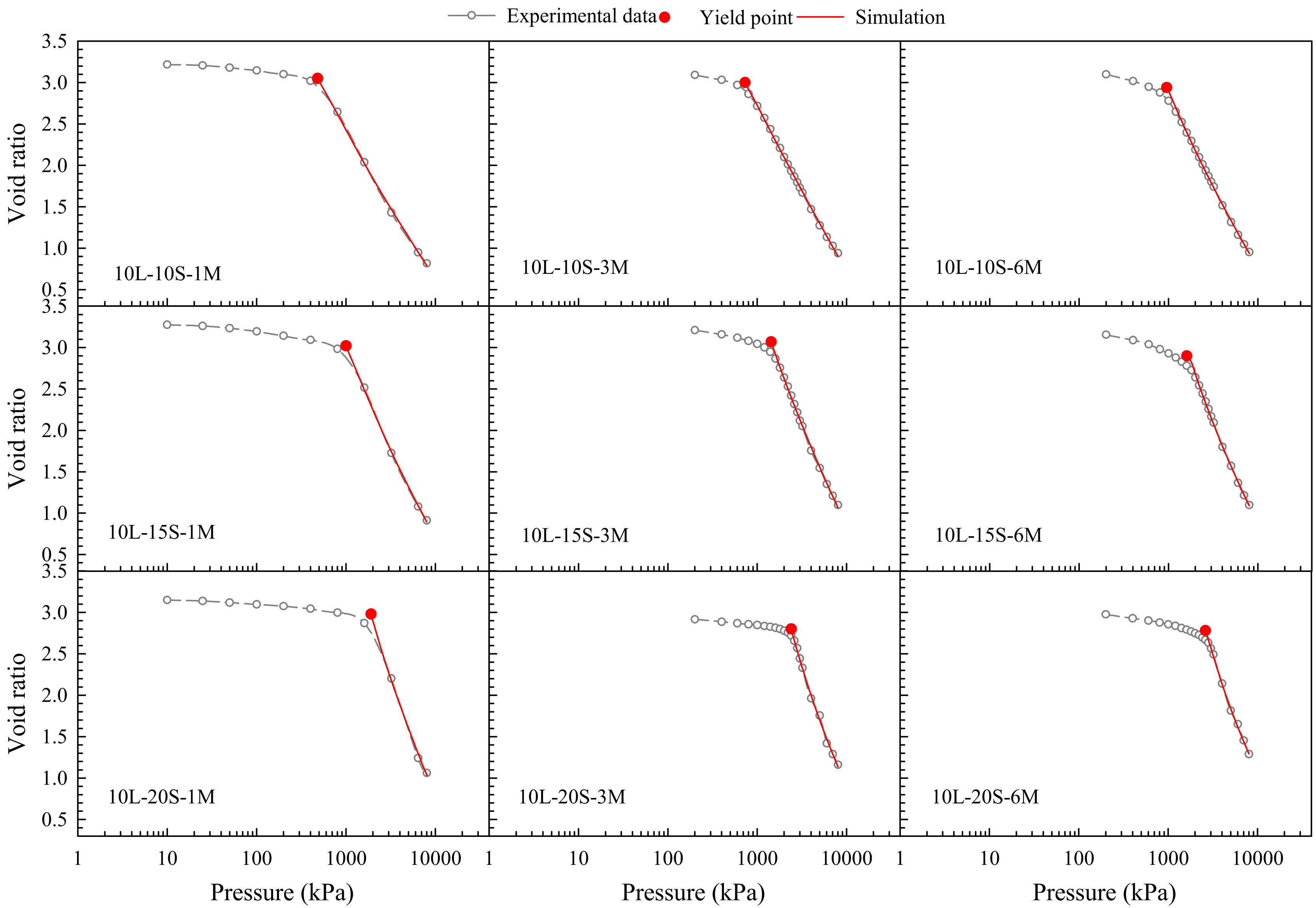

Figure 6.12. Comparison between experimental and simulated compression curves for 10% lime treated samples

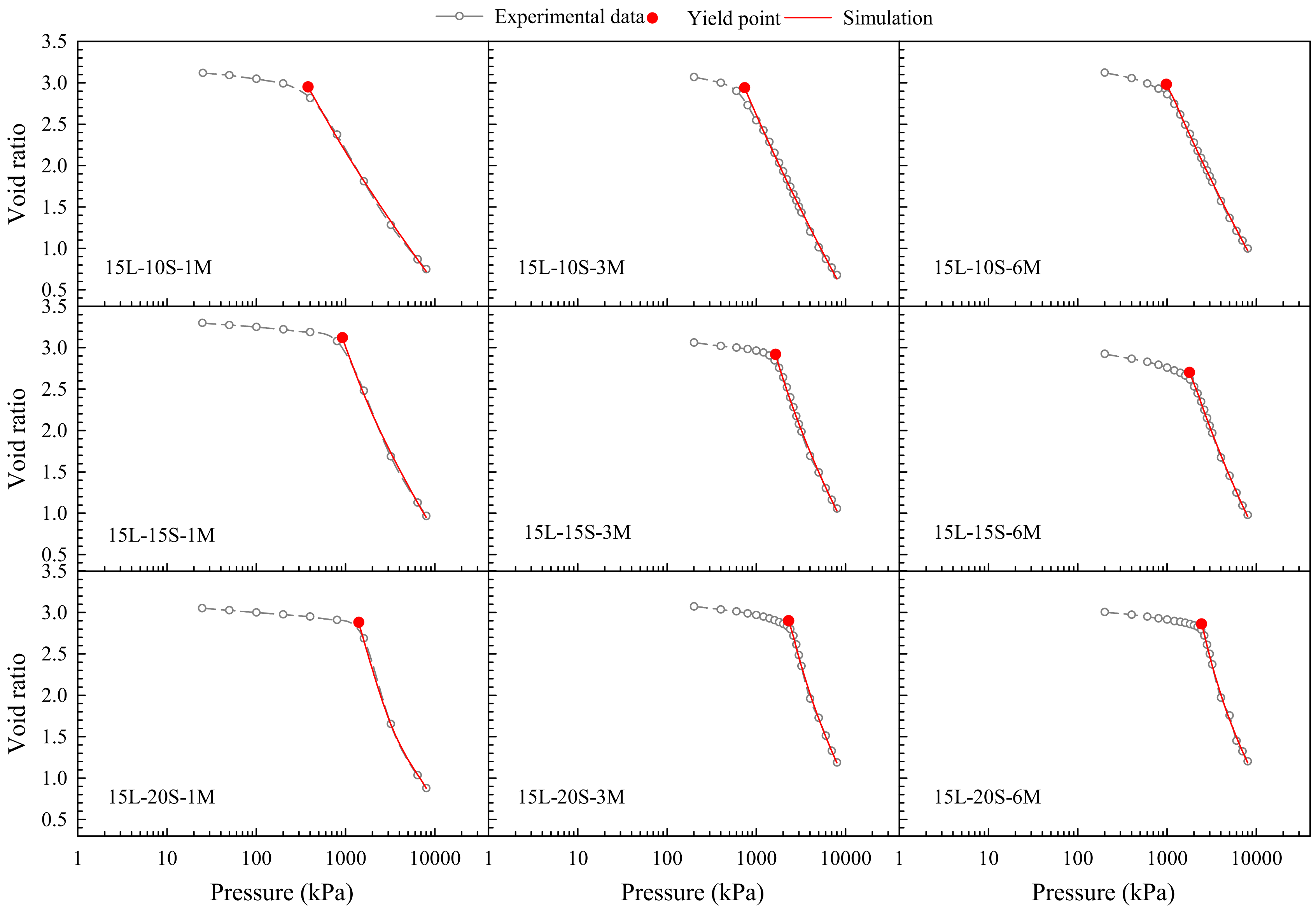

Figure 6.13. Comparison between experimental and simulated compression curves for 15% lime treated samples

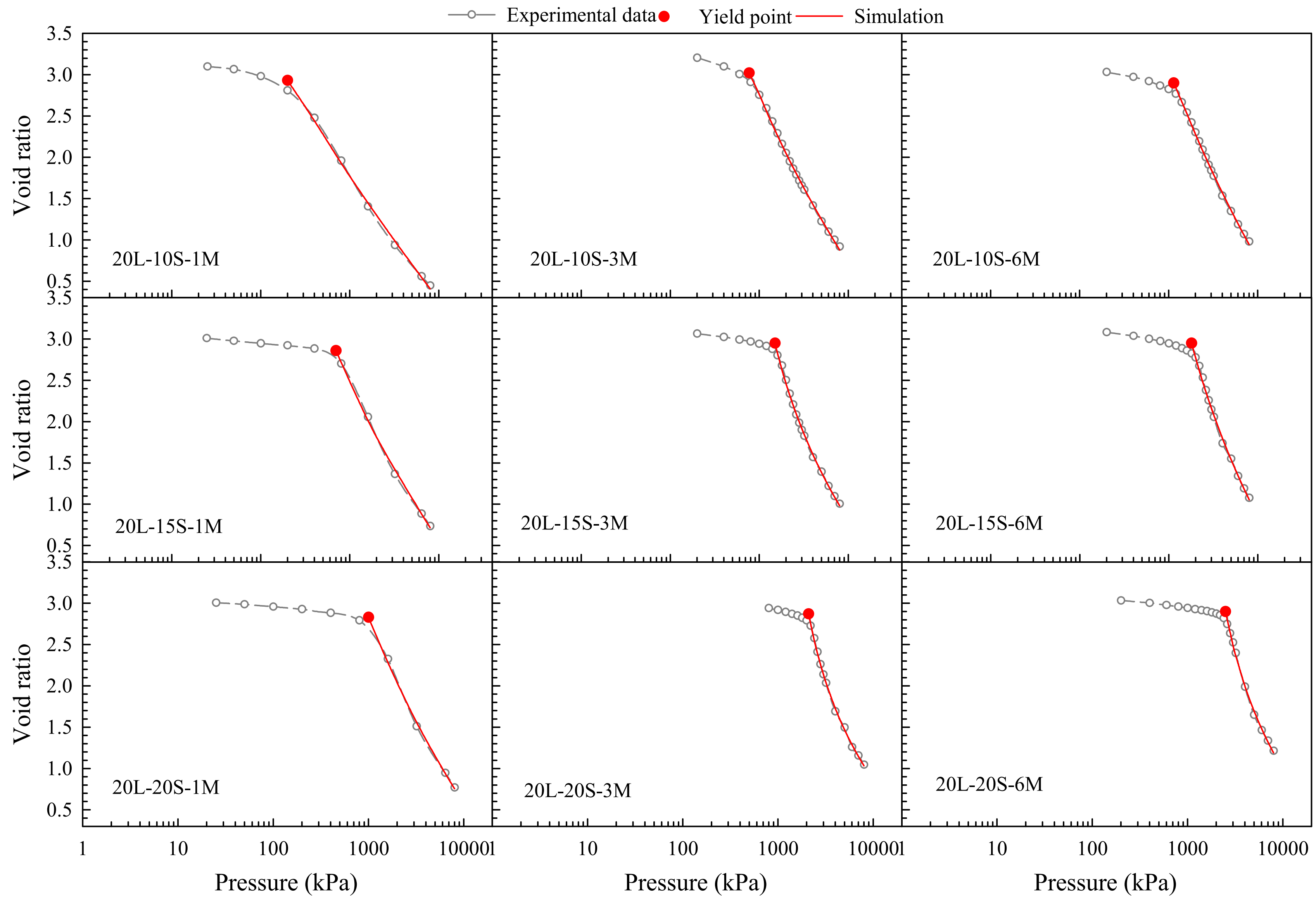

Figure 6.14. Comparison between experimental and simulated compression curves for 20% lime treated samples

Table 6.1. Parameters for simulating the compression behaviour of lime-slag treated CIS

Test ID	σ_y' (kPa)	e_y	a	b	Error (%)
10101	480	3.1	1.64	0.79	1.39
10103	730	3.02	1.70	0.61	1.24
10106	950	2.96	1.42	0.76	4.16
10151	1000	3.07	1.22	0.76	0.77
10153	1430	3.07	1.17	0.66	1.48
10156	1600	3	1.07	0.78	3.34
10201	1900	2.98	0.96	0.70	1.86
10203	2400	2.8	0.91	0.79	1.97
10206	2600	2.82	0.90	0.90	1.13
15101	380	2.95	1.85	0.86	1.61
15103	740	2.94	1.44	0.65	2.27
15106	980	3.02	1.43	0.69	1.68
15151	920	3.12	1.18	0.88	1.49
15153	1640	2.92	1.02	0.83	1.80
15156	1780	2.75	1.11	0.73	1.25
15201	1350	2.88	0.79	1.21	0.25
15203	2300	2.95	0.79	0.94	2.03
15206	2430	2.89	0.74	1.04	1.57
20101	200	3	1.90	0.86	1.77
20103	770	3.07	1.31	0.86	1.28
20106	1140	2.94	1.27	0.78	1.31
20151	650	2.9	1.48	0.83	1.97
20153	1500	2.95	0.84	1.09	1.73
20156	1800	2.95	0.91	0.88	2.06
20201	1000	2.87	1.18	0.87	1.65
20203	2100	2.87	0.62	1.25	1.28
20206	2500	2.9	0.64	1.18	1.57

Note: First two digits of test ID represent lime content, next two digits represent slag content and the last digit represent curing period (e.g., Test ID 10101 represent 10% lime, 10% slag and curing period of 1 month).

lower value of the de-structuration parameter a. For the case presented in Figure 6.11(i), the values of the parameter a for 10, 15 and 20% slags are 1.11, 0.83 and 0.64 respectively. Therefore, it is evident that the parameter a can correctly characterizes the relative rate of de-structuration of the soils treated with different amounts of slag. From other subplots of Figure 6.11, it can be clearly seen that for all other cases the assumed hyperbolic relationship between C_{cs}^T and log (σ^T) fits the experimental data very well. It can also be seen that the performance of the model to simulate the transformed compression curves is also excellent in all the cases. In all of the cases the relative rate of de-structuration of CIS treated with different combinations of additives and cured for different length of times is appropriately reflected by the relative slopes of the C_{cs} vs $\log(\sigma^T)$ hyperbolas and the relative values of the parameter a (Table 6.1). The capability of the model to simulate the actual experimental compression curves of CIS treated with different slag contents along with fixed lime contents of 10, 15 and 20% is shown in Figure 6.12, 6.13 and 6.14 respectively. It can be clearly seen that the performance of the model to simulate the actual compression curves of lime-slag treated is excellent for all the cases.

6.3.2 Compressibility characteristics in terms of model parameters

It was demonstrated in Chapter 5 that the compressibility characteristics of cemented soils can be expressed in terms of two hyperbolic model parameters a and b. While the parameter a characterizes the de-structuration behaviour, the other parameter b characterizes the compression behaviour in the de-structured state of the soil. It was found that the value of the parameter a decreases with an increasing rate of de-structuration while the value of the other parameter b decreases with an increase in the value of the compressibility at de-structured state. In this section the capability of these parameters to model the overall virgin compression behaviour of lime-slag treated CIS in different stress ranges is investigated by comparing the effect of different experimental variables on the virgin compression behaviour already discussed in Section 6.2 and the effect of the same variables on the model parameters a and b.

6.3.2.1 Parameter a

Influence of lime: It was observed in Section 6.2 that at one month of curing the rate of post-yield de-structuration decreases with an increase in lime content. It was found that at one month of curing, for CIS treated with a 10% fixed slag content, 15 and 20% lime

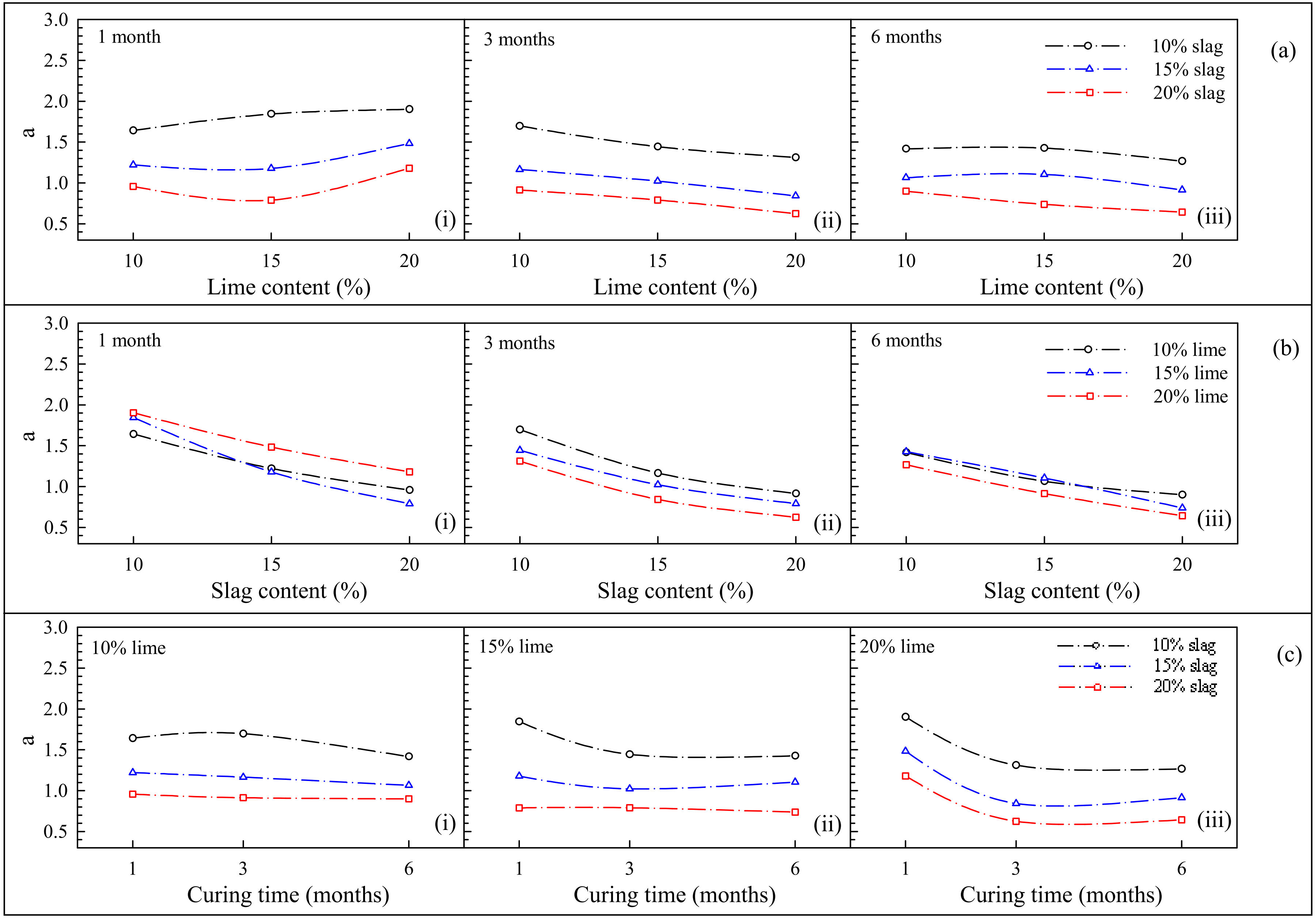

Figure 6.15. Variation of parameter *a* with (a) lime content (b) slag content (c) curing time

contents produced similar de-structuration behaviour which was less than the rate of de-structuration observed for 10% lime treated sample. It can be seen from Figure 6.15(a)-(i) that at one month of curing and for a fixed slag content of 10%, the value of the de-structuration parameter a is similar for 15 and 20% lime contents which is higher than the value of the parameter for 10% lime treated soil. On the other hand, at one month of curing, for CIS treated with fixed slag contents of either 15 or 20%, the rate of de-structuration observed for 10 and 15% lime contents were similar which was higher than the rate observed for 20% lime treated sample. It can be observed from Figure 6.15(a)-(i) that at one month of curing for samples treated with a fixed slag content of either 15 or 20%, the value of the parameter a for 10 and 15% lime treated samples is similar and is smaller than its value for 20% lime treated sample. From the study of the effect of lime content on the compressibility behaviour it was observed that at three and six months of curing, the role of lime reverses from its role at one month. While the post-yield de-structuration rate was found to decrease with an increase in lime content at one month of curing, the rate slightly increased with increasing lime content at three and six months of curing. This trend is clearly captured by the model parameter a as can be seen from Figures 6.15(a)-(i), (ii) and (iii) which show that while the value of the parameter a slightly increases with increasing lime content at one month of curing, its value shows a decreasing trend with increasing lime content at three and six months of curing.

Influence of slag: It was observed that the rate of post-yield de-structuration increases significantly with an increase in slag content at all curing periods and for all the lime contents (Figure 6.4 and Figure 6.5). This experimental observation is clearly reflected in the variation of the de-structuration parameter a with slag content. From Figure 6.15(b) it can be observed that the value of the parameter a decreases with an increase in slag content for all the cases. The observation that the role of lime reverses at three and six months of curing from its role at one month can again be observed in Figure 6.15(b) which shows that the line representing 20% lime lies above the lines for other lime contents at one month curing [Figure 6.15(b)-(i)] but the line for 20% lime occupies the bottom position at three and six months of curing as can be seen from Figure 6.15(b)-(ii) and (iii) respectively.

Influence of curing time: The effect of curing period on the parameter a for CIS treated with a fixed lime content of 10% and different slag contents is shown in Figure 6.15(c)-(i). It was observed that for CIS treated with a fixed lime content of 10% and different proportions of slag, the effect of curing on the de-structuration behaviour is insignificant [Figure 6.8(b)-(i), (iv) and (vii)]. This observation on the effect of curing on the de-structuration behaviour is correctly reflected by the variation of the model parameter a with curing period as can be seen from Figure 6.15(c)-(i).

This particular figure shows that when the soil is treated with a fixed lime content of 10% and different fixed slag contents, there is no noticeable change in the value of the parameter a due to a variation in the curing period.

The effect of curing period on the parameter a for soil treated with a fixed lime content of 15% and different slag contents is shown in Figure 6.15(c)-(ii). For soils treated with a fixed lime content of 15%, the effect of curing was seen to be most prominent when it was combined with a slag content of 10% where the post-yield de-structuration increased markedly from one month to three months but remained almost unchanged from three months to six months [Figure 6.8-(ii)]. On the other hand, for soils treated with 15% lime along with either 15 or 20% slag, the effect of curing was found to be almost negligible

[Figure-6.8(v) and (viii) respectively]. All these observations correspond appropriately with the variation of the parameter a as shown in Figure 6.15(c)-(ii). It can be seen from this figure that for sample treated with a fixed lime content of 15% and a slag content of 10%, the value of a decreases from one month to three months and remains almost unchanged from three months to six months. On the other hand, for soils treated with 15% lime and either of 15 and 20% slag contents, the parameter a does not show any noticeable variation with curing period.

The effect of curing period on the parameter a for soil treated with a fixed lime content of 20% and different amounts of slag is shown in Figure 6.15(c)-(iii). For soil treated with 20% lime and different amounts of slag it was observed that for all the cases the rate of post-yield de-structuration increased significantly from one month to three months and remained almost unaffected by a further increase in curing period [Figure 6.8(iii), (vi) and (ix)]. It can be seen from Figure 6.15(c)-(iii) that for soil treated with 20% lime and different fixed amounts of slag, the parameter a decreases from one month to three months but remains almost unchanged from three to six months.

From the above discussion it is obvious that the observed variation of the parameter a with different experimental variables establishes a clear correspondence with the observed post-yield de-structuration behaviour of CIS treated with different amounts of lime and slag and cured for different periods. It is worthwhile to mention here that the relative rate of post-yield de-structuration for different cases could also be roughly represented by assigning constant compressibility indices to various cases. However, a constant compression index is not able to characterize the distinct mechanisms controlling the compression behaviour in the de-structuration zone and de-structured zones respectively and it is also not capable to reproduce the non-linear virgin compression behaviour observed for many cases.

6.3.2.2 Parameter b

It was discussed in details in Chapter 5 that while the compression behaviour immediately after the yield is dictated by the nature of the cementitious structures, the compressibility behaviour in the de-structured state is controlled by the soil mineralogy. Delage and Lefebvre (1984) and Lapierre et al. (1990) suggested that once the larger inter-aggregate pores are significantly collapsed, the compression behaviour is controlled by the collapse of the smaller intra-aggregate pores. They also suggested that the pore structure for reconstituted soils and that of the soil from which the larger inter-aggregate pores have been collapsed is very similar. While for naturally structured soils, the intra-aggregate pore is likely to be characterized by the soil mineralogy, for artificially cemented soils different experimental variables may have different degrees of influences on the resulting intra-aggregate pores. As a result the effect of different experimental variables on the parameter b, which characterize the behaviour in the stress range where the behaviour is dominated by the collapse of the smaller intra-aggregate pores, will be dependent on the type of influences these experimental variables have on them.

The values of the parameter b for all the cases investigated are shown in Figure 6.16 in the form of a bar-chart. The dashed line in this figure shows the median value. It can be seen from this figure that with few exceptions, the value of the parameter b varies within a reasonably narrow range centering on the median value. The exception is more prominent for the soil treated with 20% slag. The issue with the 20% slag treated soils is that the yield stresses of these soils are relatively high and therefore very few data points after the yield stress could have been obtained from the experiments even by using the maximum capacity of the testing equipment. Since the value of the parameter b is related to the asymptotic value of C_{cs}, the quality of prediction is bound to deteriorate when the prediction is based only on few data points over a limited range of pressure. If the case of

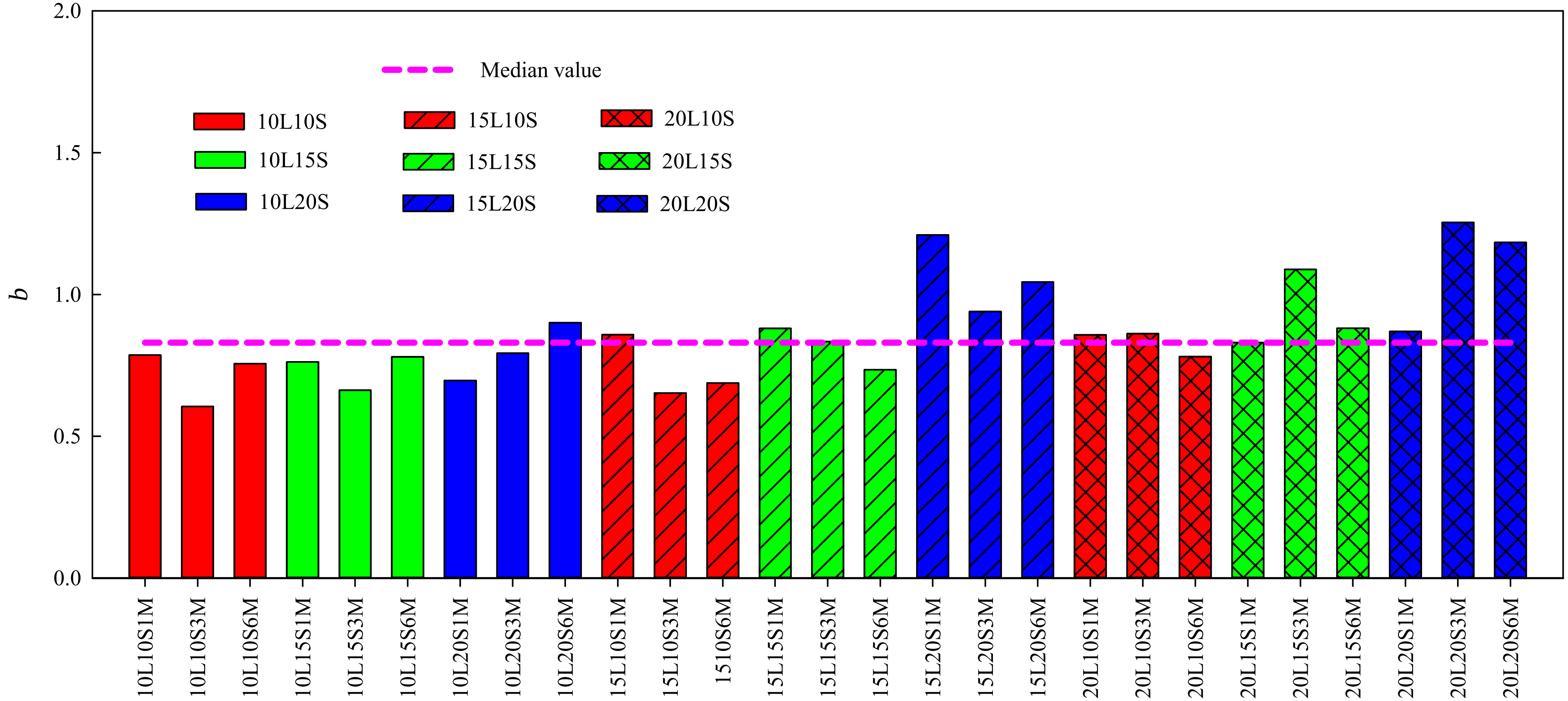

Figure 6.16. Values of the parameter *b* for CIS treated with different combinations of lime and slag at different curing periods

20% slag is kept aside, it can be seen that the range of variation of the parameter *b* becomes even narrower. This type of variation of *b* is consistent with the observation made in Section 6.2 that all the compression curves exhibits a tendency to merge together at higher pressure [Figure 6.1, Figure 6.4 and Figure 6.7]. It is not known to what extent different experimental variables affected the smaller intra-aggregate pores. However, the variation of the parameter *b* within a narrow range indicates that the effect of different experimental variables on the intra-aggregate pores may not be as prominent as their effects on the larger inter-aggregate pores. Horpibulsuk et al. (2010) also reported that the degree of cementation hardly affects the amounts of volume occupied by the intra-aggregate micro-pores, the size of which was <0.1μ for the case they presented.

6.4 Validation of the proposed compression model against literature data on Portland cement treated soils

It has been shown that the proposed model can successfully reproduce the non-linear compression curves of artificially cemented soils while correctly characterizing the de-structuration behaviour and behaviour at the de-structured states. Having validated the model for a wide range of naturally structured soil (in Chapter 5) and lime-slag treated CIS, the model is now validated against the experimental compression data obtained from literature on several different types soils treated with Portland cement. Along with the literature data, two cases of lime-slag treated CIS have also been considered here and the data on CIS considered here represents the cases for which the post-yield de-structuration was the highest and the lowest.

All the original compression curves for the cases considered here are presented in Figure 6.17(a). It can be seen from Figure 6.17(a) that many of the compression curves clearly exhibit non-linear virgin compression behaviour in a semi-logarithmic plot. The plot of instantaneous compression indices vs. logarithm of pressure for all these soils as presented in Figure 6.17(b) demonstrates the extent of non-linearity of compression behaviour of all these treated soils. In order to assess the de-structuration behaviour of all these soils, the compressibility data is presented on a transformed pressure-void ratio space in Figure 6.17(c). The variation C_{cs}^{T} with pressure for all these cases is also shown in Figure 6.17(d). In Figure 6.17(b) and Figure 6.17(c), points represent actual experimental data and solid lines represent simulated curves. The values of the parameters a and b for all these simulations are presented in Table 6.2. It can be observed from Figure 6.17(b) and Figure 6.17(c) that the quality of simulation for both the transformed compression curves and for the variation of C_{cs}^{T} with pressure is excellent. All the actual experimental compression curves are presented in Figure 6.18 along with the simulation of the virgin compression part. Excellent agreement between the experimental compression curves and simulated ones observed in Figure 6.18 once again clearly demonstrates the capability of the proposed compression model.

The compression curves presented in Figure 6.17(a) has been assigned different integer values according to the values of their a parameter. Soil with the highest value of the parameter a has been assigned the lowest integer value. Although it is difficult to compare the relative rate of de-structuration of different soils from the original compression curves presented in Figure 6.17(a), their relative rates of de-structuration can easily be compared with the help of Figure 6.17(c). It can be observed from Figure 6.17(c) that the relative values of the parameter a are able to correctly reflect the relative rate of post-yield de-structuration of all those soils which is evidenced by the fact that with the increase in the number assigned to different soils (i.e., decrease in the value of the parameter a) the rate of post-yield de-structuration increases.

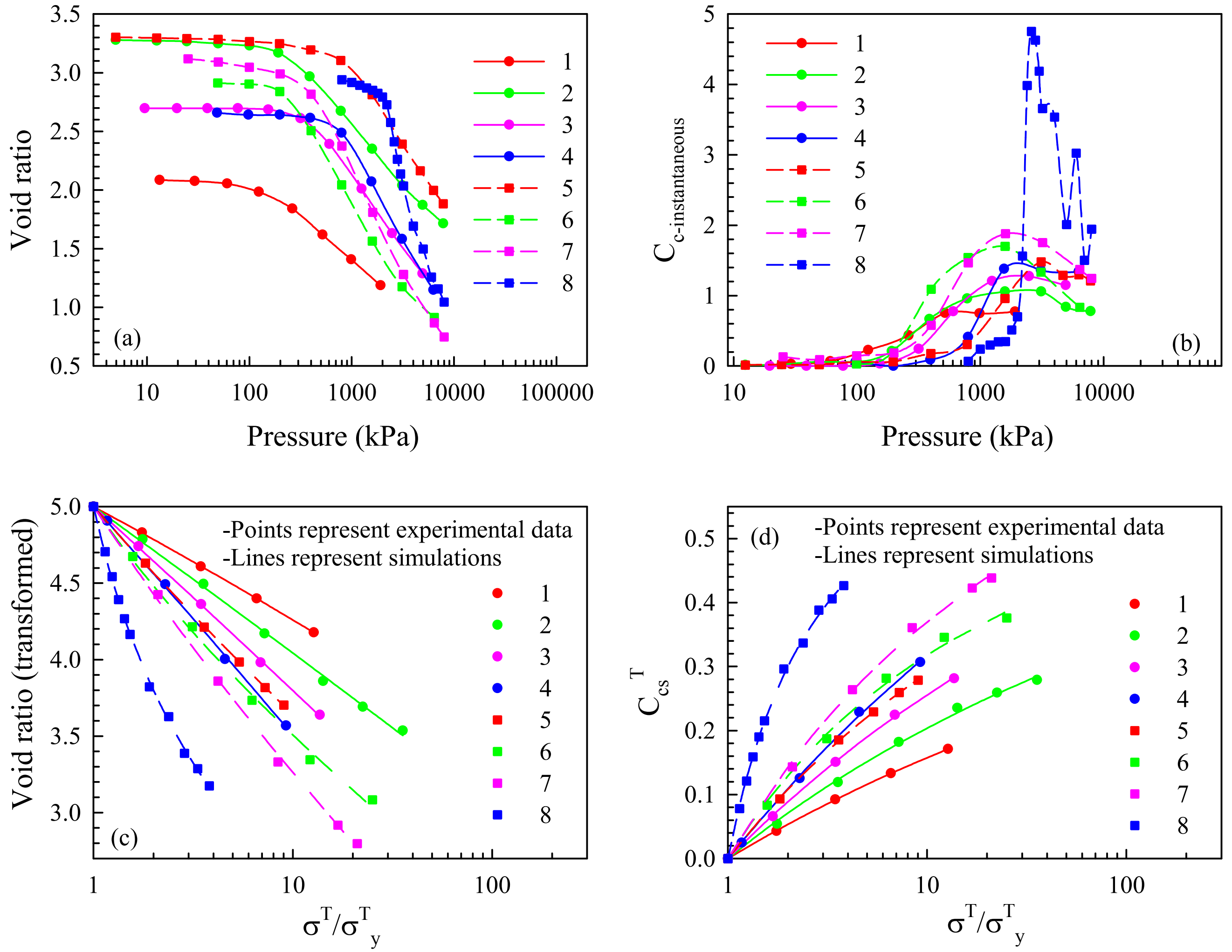

Figure 6.17. Validation of the proposed compression model for different artificially cemented soils (a) compression curves (b) instantaneous compression indices (c) transformed compression curves (d) C_{cs}^{T} profiles

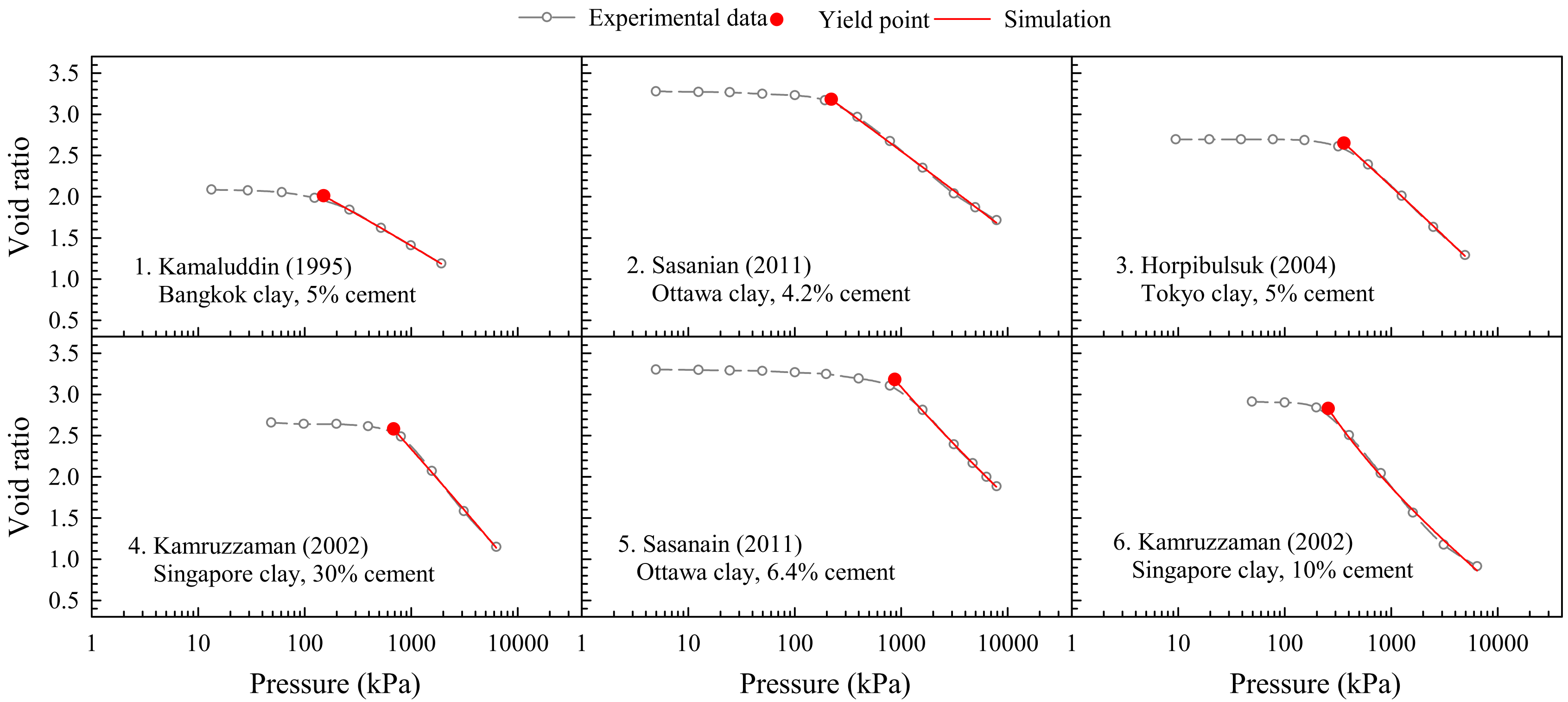

Figure 6.18. Comparison between experimental and simulated compression curves for different artificially cemented soils

Table 6.2. Values of parameters a and b for various artificially cemented soils

Sl.	Reference	Soil name	Additive	a	b
1	Kamaluddin (1995)	Bangkok clay	5% Cement	5.32	1.02
2	Sasanian (2011)	Ottawa clay	4.2% cement	3.93	0.98
3	Horpibulsuk (2004)	Tokyo clay	5% cement	3.15	0.75
4	Kamruzzaman (2002)	Singapore clay	30% cement	2.58	0.57
5	Sasanain (2011)	Ottawa clay	6.4% cement	2.48	0.97
6	Kamruzzaman (2002)	Singapore clay	10% cement	1.95	1.20
7	Chowdhury (2013)	Coode Island Silt	15% lime+10% slag	1.85	0.86
8	Chowdhury (2013)	Coode Island Silt	20% lime+20% slag	0.62	1.25

The compressibility at the respective de-structured states of the artificially cemented soils considered here can be observed from Figure 6.17(b). Figure 6.17(b) shows that the compressibility of the different artificially cemented soils at their respective de-structured states is incidentally very close to each other. The values of the parameter b for all these cases are presented in Table 6.2 and it can be found that the values of the parameter b varies within a narrow range to reflect the proximity of the de-structured compressibility behaviour of all those treated soils. It can also be found from Table 6.2 that the relative magnitudes of the parameter b largely reflect the relative magnitudes of de-structured compressibility observed from Figure 6.17(b) although some discrepancy may be found due to unavailability of adequate data points in the de-structured zone for many of the soils.

6.5 Summary

The compressibility behaviour of lime-slag treated CIS was investigated through a comprehensive experimental program. In this investigation minimum amount of lime used was higher than the saturation lime content determined by ICL test and it was found that lime has very insignificant influence on the overall compressibility behaviour for the range of lime contents investigated. Among the variables investigated, slag is found to have the greatest influence on the compressibility behaviour. Increasing the amount of slag increases the yield strength and the rate of post-yield de-structuration irrespective of the magnitudes of other testing variables. The effect of curing was found to be dominant from one month to three months after which the effect of curing significantly decreased. One interesting finding from this investigation is that compressibility increases significantly with an increase in yield strength of the treated material.

The compression modeling framework developed in Chapter 5 based on the observation of compression behaviour of a wide range of naturally structured soils has been found to be equally applicable for the study of the non-linear virgin compression behaviour of artificially cemented soils. It was found that the gradual pore collapse mechanism under compressive loading proposed by Delage and Lefebvre (1984) and Delage (2010) is likely to be able to explain the compression behaviour of lime-slag treated CIS. It was clearly demonstrated that the parameter a can characterize the experimentally observed de-structuration behaviour correctly. Parameter b, which characterize the compression behaviour at the de-structured state, was found to vary within a relatively narrow range for CIS treated with different combinations of lime and slag and cured for different periods. For CIS treated with 20% slag, the value of the parameter b was found to vary considerably from its value for all other cases. This variation for 20% slag treated sample is likely to be caused by the unavailability of adequate data points in the post-yield region. Variation of parameter b for all other cases within a narrow range seems to indicate that the effect of different experimental variables on the smaller intra-aggregate pores may not be as prominent as their influences on the larger inter-aggregate pores.

It is evident that the proposed framework can be confidently employed to study the effect of different experimental variables on the de-structuration behaviour as well as on the behaviour in the de-structured states of artificially treated soils. For example, with the help of the proposed framework, it can be studied how a variation of moisture content, stabilizer type, or any other experimental variable affects the behaviour in the de-structuration and de-structured ranges respectively.

6.6 References

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CHAPTER 7: UNDRAINED SHEAR BEHAVIOUR OF LIME-SLAG TREATED CIS

7.1 Introduction

Short term stability is most often the governing criteria for any geo-infrastructure founded on saturated clayey soils. For short-term stress-deformation analysis of structures founded on stabilized soil mass, it is critically important to understand the undrained shear behaviour of both the treated soil and the host soil. In order to comprehend the undrained shearing behaviour of lime-slag treated CIS, a comprehensive series of Isotropically Consolidated Undrained (CIU) triaxial shear testing with pore pressure measurement was undertaken. The testing program included various combinations of additives and confining pressures. The salient features of the mechanical responses of lime-slag treated CIS identified from the results of this experimental investigation are presented in this chapter and the possible mechanisms behind these observations are explained.

Artificial cementation alters the mechanical behaviour of parent soil significantly. For un-cemented soils, the response to any type of loading, either compression or shearing, is controlled by the interaction of soil particles at their contacts and the behaviour is mainly frictional in nature. On the other hand, the presence of cementing bonds makes the mechanical behaviour of cemented soils more complex than un-cemented soils. The shearing response of cemented soil is governed by a continuously changing complex interaction between the soil particles/clusters and the inter-particular cementation bonds. If the mechanical behaviour of cementitious bonds and that of soil particles are considered independently, it can be seen that while the cementitious bonds exhibit approximately a rigid type of behaviour failing in a brittle manner when the threshold strength value of the bond is reached (Nagaraj and Miura, 2001), the behaviour of an assemblage of soil particles is more ductile in nature which is controlled by the forces operating at the inter-particle contacts. At the initial stage of the loading, the overall mechanical behaviour of cemented soils is primarily governed by the response of the cementitious bonds and their progressive collapse mechanism (Coop and Atkinson, 1993). At the other extreme, when the soil is significantly de-structured (structured soils, in most of the cases, may not be able to reach fully de-structured state), the behaviour is controlled mainly by the frictional response of the de-structured soil particles/clusters and broken cementation bonds (Coop and Atkinson, 1993; Ismail et al., 2002). In the intermediate stress range, the cementation bonds are gradually eliminated with the increase in strain and the stresses previously carried by these cementation bonds is progressively transferred to the soil particles. For a cemented soil, the incremental mechanical response at any instance of the straining is, therefore, governed by the load transfer/sharing mechanism operating between the cementitious bonds and the de-structured soil particles at that particular condition.

The main variables that affect the shearing behaviour of treated soils are the amount of cementitious additives, curing time and the state of de-structuration of the treated soil resulting from consolidation prior to the application of shearing loads. Increasing amount of additives and increasing curing time can be expected to affect the behaviour of treated soil mass in a similar way in that increasing magnitudes of both of these variables increase the degree of cementation in the treated soil mass. Due to limitation of time, the effect of curing time on the undrained shearing behaviour of lime-slag treated CIS could not be explicitly investigated in this project. However, by studying the influence of different degrees of cementation on the mechanical behaviour, it is possible to draw qualitative inferences about the effect of curing on the undrained shear behaviour of lime-slag treated CIS. The effect of degree of cementation on the shearing behaviour was studied by treating the raw CIS with different amounts of cementing additives (10% fixed lime along with 10, 15 and 20% slag content by dry weight of the raw soil and cured for 1 month). A wide spectrum of pre-shear consolidation pressures (50, 100, 200, 400, 800, 1600 and 2400 kPa) was employed to study how differing pre-shear de-structured states affect the shear behaviour of the treated soils.

7.2 Results

The undrained shear behaviour of lime-slag treated CIS is reported in terms of stress-strain and excess pore pressure responses and stress path behaviour in the following sections.

7.2.1 Stress-strain behaviour of lime-slag treated CIS

The stress-strain behaviour of lime-slag treated CIS is presented in two different ways in Figure 7.1 and Figure 7.2 to highlight the effect of slag content and the effect of confining pressure respectively on the stress-strain behaviour.

It can be seen from Figure 7.1 that for all the levels of pre-shear consolidation pressures investigated, the main influence of slag is manifested as an increase in the shear strength and stiffness of the treated soil mass. It is seen that the peak strength increases invariably with an increase in slag content. This observation about the influence of slag content on the undrained strength is consistent with the observation made on the effect of slag content both on the UCS and 1-D yield stress values as reported in earlier chapters. Similar influence of slag content on the UCS, 1-D yield stress and undrained shear strength is expected since all these values are mainly controlled by the same variable which is the strength of the cementation bonds formed within the treated soil mass. It was discussed in earlier chapters that slag content is the prime determinant of the degree of cementation whereas lime plays more of a passive role in the lime-slag treatment of clays provided a

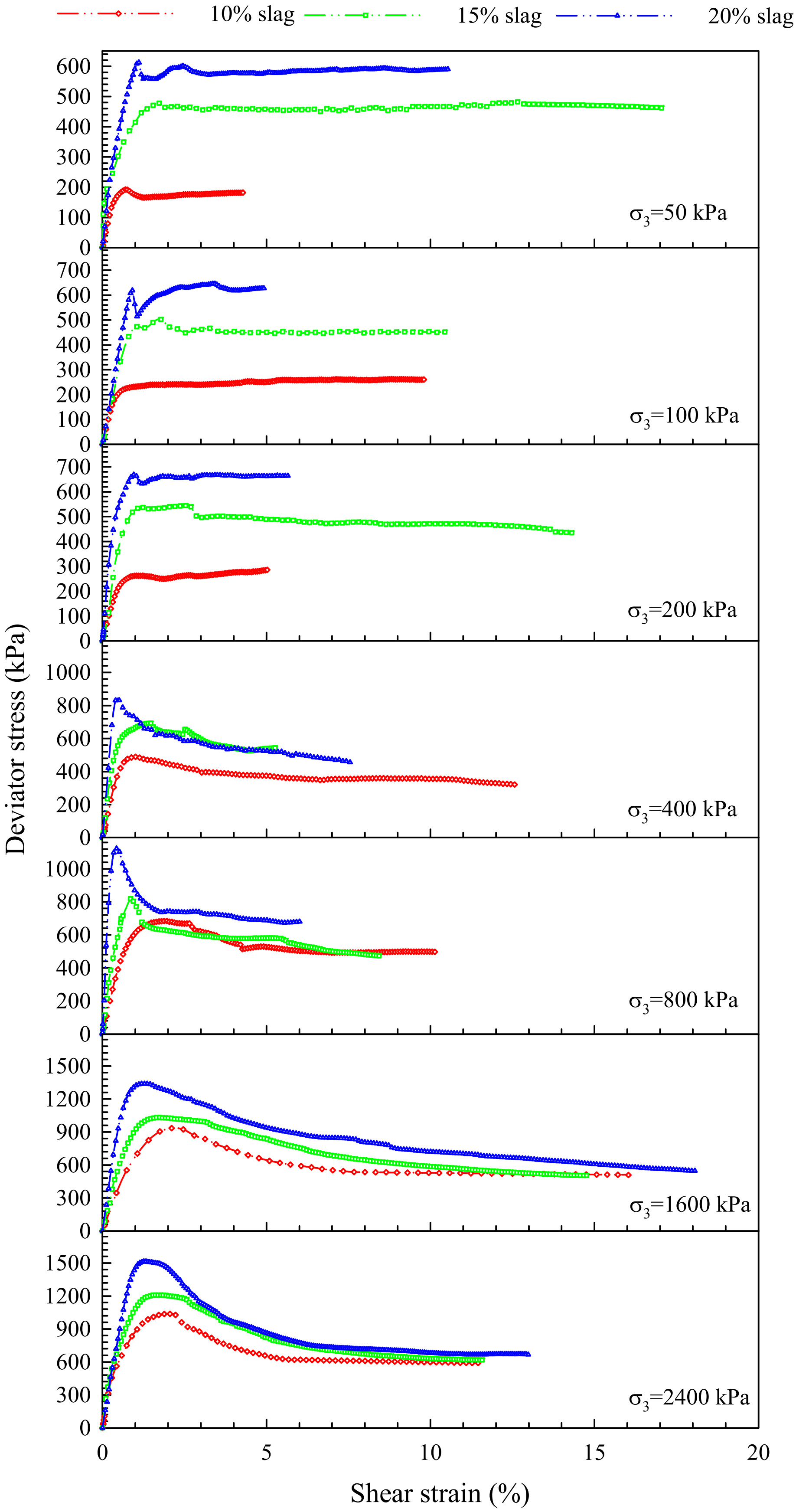

Figure 7.1. Effect of slag content on stress-strain behaviour of lime-slag treated CIS

(curing period = 1 month)

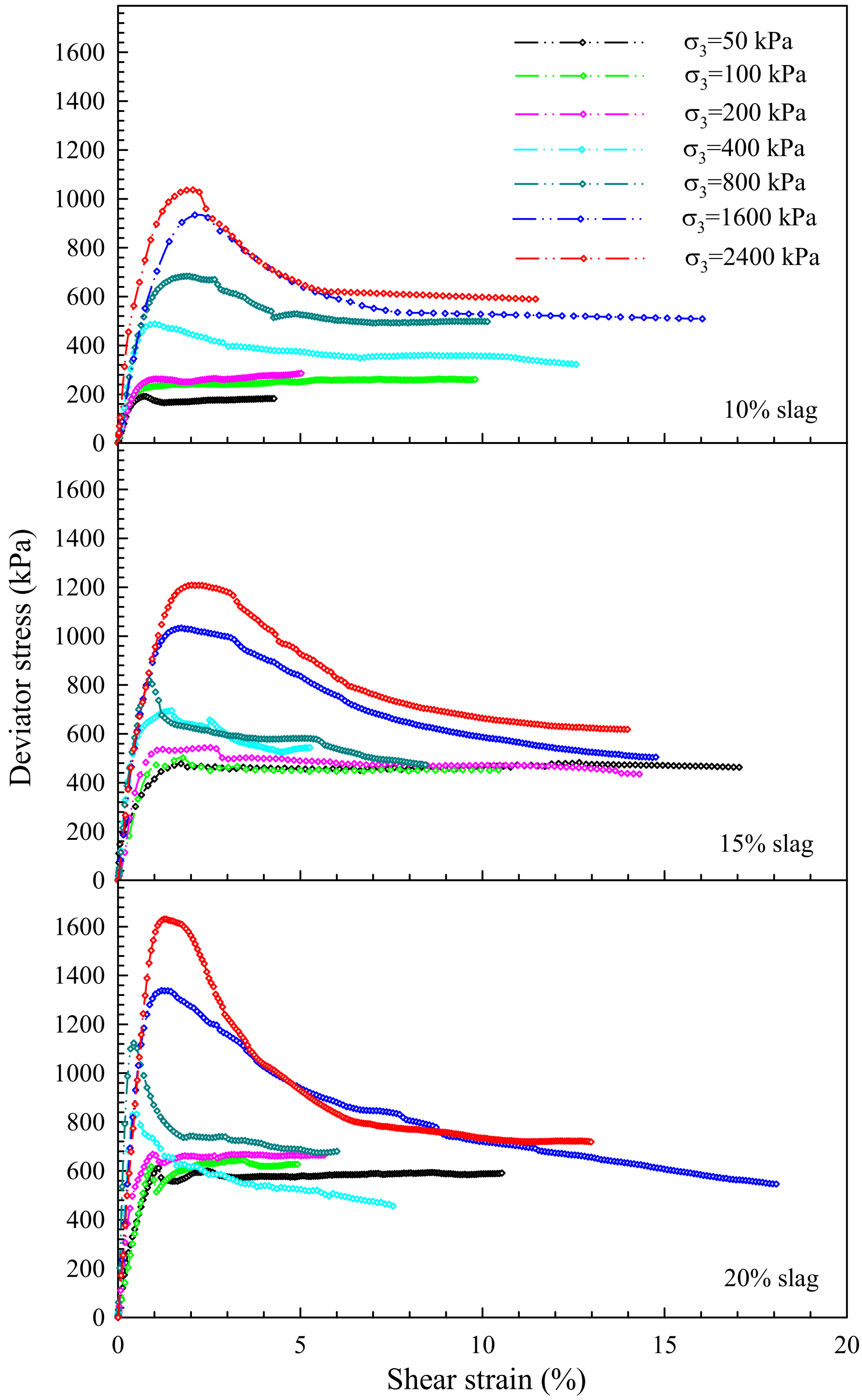

Figure 7.2. Effect of consolidation pressure on the stress-strain behaviour of lime-slag treated CIS (curing period = 1 month)

minimum amount lime is present in the reaction environment to maintain a threshold level of pH. Since the lime content used in this investigation is significantly higher than the optimum lime requirement determined by the ICL test, samples treated with higher amount of slag is likely to possess a higher amount of cementitious products which can be held responsible for the increase in the undrained shear strength and stiffness observed in samples treated with higher amount of slag.

The effect of pre-shear consolidation pressure on the stress-strain behaviour is presented in Figure 7.2. It can be observed from this figure that the pre-shear consolidation pressure has significant influence on the undrained shear strength and stiffness values as well as on the overall stress-strain characteristics. The figure shows that increasing confining pressure increases both the undrained shear strength and the stiffness values. This is expected since the void ratio of the soil decreases with the increase in pre-shear consolidation pressure and this reduction of void ratio with increasing consolidation pressure can explain the observed higher strength and stiffness magnitudes associated with higher level of effective pre-shear confinement. However, the effect of consolidation pressure on the strength and stiffness is found to be dependent on the level of effective pre-shear confinement. The effect of confinement at low level of pre-shear consolidation pressure (up to 200 kPa) is insignificant compared to the effect of confinement observed at confinement level in excess of 400 kPa. For all the slag contents, it has been found that for consolidation pressure up to 200 kPa, the maximum value of the deviator stress is only marginally affected by an increase in the pre-shear consolidation pressure. At this low level of confinement, the damage of the cementitious structures and associated change in volume of the sample from the consolidation phase is insignificant and the strength is mainly derived from the strength of the cementitious bonds with very little contribution coming from the mobilized friction. As a result, the strength and stiffness values vary within a very narrow range for a confining pressure up to 200 kPa. Similar observation has also been reported by Horpibulsuk et al. (2004), Kamruzzaman et al. (2002) and Uddin (1995) for different types of soil treated with Portland cement. In addition to influencing the undrained shear strength and stiffness magnitudes, confining pressures is found to influence the overall stress-strain characteristics significantly. It has been found that at low level of consolidation pressure, all the stress-strain curves reaches the maximum stress value at a relatively small strain and thereafter the stress values remain almost constant against increasing shear strain. On the other hand, samples sheared from higher level of consolidation pressure, stress-strain curves display progressive strain softening behaviour beyond the peak stress value. It can be seen that the quantity of slag also influences the strain softening behaviour of the treated soil mass. For CIS treated with 20% slag, the onset of strain softening is found to be noticeable at 400 kPa of consolidation pressure whereas samples treated with 10 and 15% slag exhibit only marginal strain softening at this particular level of pre-shear consolidation pressure. Strain softening for the soil treated with 15% slag becomes appreciable at 800 kPa of pre-shear confining pressure whereas CIS treated with 10% slag starts exhibiting significant strain softening behaviour only from 1600 kPa of consolidation pressure.

In order to explain the post-peak stress-strain behaviour of treated CIS samples, the experimental results reported by Elliott and Brown (1985) is invoked here. Elliott and Brown (1985) studied the triaxial shearing behaviour of cemented soft rock under different level of confining pressures and studied the progressive formation of fractures within the samples at different stages of the loading. They found that when the soft rock samples were sheared from low level of confining pressure, the initial part of the shear deformation was characterized by the recoverable closure of pores and pre-existing micro-cracks and elastic deformation of the cemented skeleton (this stress range was designated as pre-fracture zone). Once the limit of the reversible deformation was exceeded, the incremental deformation was characterized by the appearance of micro-cracks and this stress zone was identified as fracture zone. For samples sheared from low level of consolidation pressures, the micro-fractures collapsed into macro-fractures near the vicinity of the peak stress. On the other hand, for samples sheared from higher level of consolidation pressure, the density of micro-cracks increased and these micro-cracks were more uniformly distributed throughout the samples near the peak stress level and failure of the sample was found to be of barrelling type. Although the progress of the development of fractures with loading was not monitored in this investigation with appropriate instrumentation, a scrutiny of the stress-strain curves can be helpful in qualitatively identifying the strain levels at which cracks may have formed within the samples. It can be seen that for samples sheared from low level of consolidation pressures, there is noticeable discontinuity in the stress-strain curves in the vicinity of the peak stress value and the discontinuity is more prominent for CIS treated with higher amount of slag. In the absence of adequate data, it can be speculated that first appearance of noticeable discontinuity in the stress-strain curves indicates the probable formation of macro level fractures. It can be observed from Figures 7.1 and 7.2 that with an increase in consolidation pressure, the first appearance of discontinuity is increasingly shifted rightward from the peak state and at elevated consolidation pressure such as 2400 kPa, the said discontinuity completely disappears. It is important to mention here that although the stress-strain curves for samples sheared from 2400 kPa of consolidation pressures does not exhibit any noticeable discontinuity, post-test observations revealed the presence of distinct fracture planes in the specimens sheared from all levels of consolidation pressures including 2400 kPa.

For samples sheared from low levels of consolidation pressures, it is therefore difficult to explain the post-peak stress-strain behaviour of samples based on the measurements made at the boundary of the samples due to formation of significant discontinuities in the samples in the vicinity of the peak stress value. The deformation is heavily localized in thin shear bands and the stresses and deformations within this shear bands are significantly different from those in the other relatively intact parts of the tested samples. Burland (1990) discussed the post-rupture stress-deformation response of Todi clay with the help of the measurements made at the boundary of the sample as well as those measured at the locations close to the rupture surfaces. Burland (1990) showed that the stress-strain response on the failure plane differed significantly from the stress-strain behaviour deduced based on the boundary measurements. Burland (1990) suggested that once the shear banding takes place, clusters of bonded particles form within the shear band and the mechanical behaviour is heavily influenced by the response of these clusters of bonded particles. Jiang et al. (2011), based on numerical simulation by Distinct Element Method (DEM) of bi-axial tests on cemented soil also drew similar conclusions about the localization of deformation. For lime-slag treated CIS samples sheared from low level of effective confinement, the most plausible explanation behind the maintenance of almost constant level of stress beyond the initial peak value is that the loss of strength due to breakage of cementitious bonds is compensated by the ongoing dilation caused by the relative movement of the broken rigid clusters of bonded particles within the shear zones. Burland (1990) suggested that particulate response of broken cluster of bonded particles within the shear bands can even cause an increase in the shear stress value after the formation of fracture planes. It will be discussed later that for treated soil samples sheared from lower level of pre-shear consolidation pressures, pore pressure starts to decline immediately after the peak strength value is reached causing the mean effective confinement to increase with an increase in the shear strain. If the straining is continued, the clusters of bonded particles generated from the fracture of the treated soil mass will have to roll over the surrounding clusters under progressively increasing level of mean effective confinement. The resistance derived in this way in the post-peak strain range may be responsible for the maintenance of the constant level of shear stress values observed.

It is seen from Figure 7.1 and Figure 7.2 that for all the different slag contents used, the stress-strain curves display a tendency of increasing degree of strain softening with the increase in pre-shear consolidation pressures. This type of strain softening behaviour is in contrast with the behaviour of un-cemented soils for which strain softening is usually observed only under drained shearing for sample possessing high over-consolidation ratios but generally not observed under undrained shearing even at low level of consolidation pressure. While the strains-hardening/softening behaviour of mechanically overconsolidated un-cemented soil is dependent mainly on the over-consolidation ratio, the shearing response of cemented soil is determined by the state of de-structuration at the beginning of the shearing. The observed difference in the shearing behaviour between the cemented soil and un-cemented soil can, therefore, easily be attributed to the cementation bonds. Coop and Atkinson (1993) and Bergado et al. (1996) proposed conceptual frameworks, which are to some extent similar, to explain the possible stress-strain response of structured soils sheared from different levels of consolidation pressures. In the extreme case where the structure is completely destroyed during the consolidation process, the shearing response of the de-structured soil resembles that of un-cemented soil. However, from the de-structuration characteristics studied in earlier chapters, it is clear that a consolidation pressure of 2400 kPa is not able to completely remove the cementation induced structure from the treated soil mass. Elliott and Brown (1985) observed a transitional type of shearing response which is neither brittle nor completely ductile at this type of intermediate level of consolidation pressures which is not high enough to eliminate the structure completely. Therefore, as suggested by Yu et al. (2007), this type of surprising shearing behaviour can be expected for cemented soils sheared from an intermediate level of consolidation pressure where the overall shearing response is controlled by an interplay of the particulate and non-particulate responses.

Nagaraj and Miura (2001) presented a very simple reasoning to explain the stress-strain behaviour of cemented soils. They suggested that during shearing of cemented soils, a handover mechanism continually operates between cementation and frictional components of the shear resistance. As soon as some of the bonds are eliminated, the stress previously carried by the cementitious bonds is transferred to the soil particles and water filling up the void spaces. The incremental shear response at any instance of the loading is determined by the net effect of two simultaneously operating mechanisms: loss of strength due to elimination of cementation bonds and increasing contribution of inter-particulate friction. It can be seen from Figure 7.2 that the initial parts of the stress-strain curves are very stiff but the stress-strain curves start exhibiting non-linearity before the peak strength is reached. Elliott and Brown (1985) found that most of the deformations of the pores at the initial linear portion of the stress-strain curve are reversible and the start of the non-linearity prior to the peak strength signals the initiation of irreversible deformation. The onset of non-linearity can therefore be considered to be the point from where frictional mobilization starts to operate significantly. It can be argued that from the start of the non-linearity to the point of peak stress value, there is still significant contribution from the cementitious bonds and the combined contribution of cementitious bonds and friction help the stress value to increase with an increase in strain by overcoming the loss of strength resulting from the further breakdown of the remaining cementitious bonds. However, after the peak stress value is reached, the loss of strength due to breakdown of cementitious bonds can no longer be compensated for by the mobilized friction and therefore strain softening response ensues at the post-peak level. Ismail et al. (2002) suggested that the degree of post-peak strain softening will be controlled by the difference in the peak strength value and the strength at the de-structured state. Since the peak strength increases with increasing slag content, the amount of post peak strain-softening can be expected to increases with an increase in slag content.

The observed stress-strain behaviour discussed so far can also be explained with the help of the constitutive modelling framework proposed by Gens and Nova (1993) which was discussed in Chapter 2. For samples sheared from low level of consolidation pressure, this framework may not be applicable for explaining the post-peak behaviour due to inhomogeneous deformation in the post-peak range resulting from the formation of fractures. However, the post-peak stress-strain behaviour for samples sheared from higher level of consolidation pressure can, to some extent, be explained with the help of the above-mentioned constitutive modelling framework. For samples sheared from higher level of consolidation pressure, the yield surface was able to expand after the initial yielding (the first appearance of non-linearity in the stress-strain curves can be considered to be the initial yielding of the material) up to the peak stress value due to a net positive effect of the two competing mechanisms: expansion of the yield surface due to volumetric compression and shrinking of the yield surface due to removal of cementitious structures. However, in the post-peak region the reduction in the yield surface due to elimination of cementitious bonds can no longer be overcome by an increase in the size of the yield surface due to reduction of void ratio and as a result stress is seen to decrease with increasing shear strain.

7.2.2 Stress ratio-shear strain behaviour of lime-slag treated CIS

A convenient way of studying the shearing response of geomaterial is to study the progressive change of stress ratio with shear strain. The shearing response of soils is dependent on confining pressure and in undrained triaxial shearing, the mean effective confinement continuously changes due to the change in pore water pressure. The study of the progressive change in stress ratio with shear strain can be helpful in identifying which part of shearing is characterized by what type of mechanism.

The shear stress-strain data presented in Figure 7.2 is replotted in Figure 7.3 in terms of stress ratio (deviator stress/mean effective stress) vs. shear strain. It can be seen from this figure that although the samples sheared from higher consolidation pressure yielded higher undrained shear strength (Figure 7.2), the value of the maximum stress ratio is seen to decrease with an increase in consolidation pressure. For samples sheared from a confinement level much lower than the size of the yield surface of the cemented soil, the peak state is dominated by non-particulate response and as a result the peak stress ratio can be expected to be significantly higher than the maximum stress ratio that the soil can sustain in a purely frictional manner. Once the pre-shear confinement increases, the soil is significantly de-structured prior to shearing and the response at peak is increasingly characterized by the frictional behaviour of the soil. Therefore the decrease in the magnitude of maximum shear stress ratio with increasing consolidation pressure clearly indicates that at lower level of consolidation pressure, the contribution of cementation bonds to the peak stress is much more prominent for samples sheared from low level of consolidation pressures.

It is also seen that for samples sheared from low level of pre-shear consolidation pressure, the peak stress ratio takes place at a very small shear strain which is close to the strain at which peak stress takes place and the stress ratio increases almost linearly up to the peak

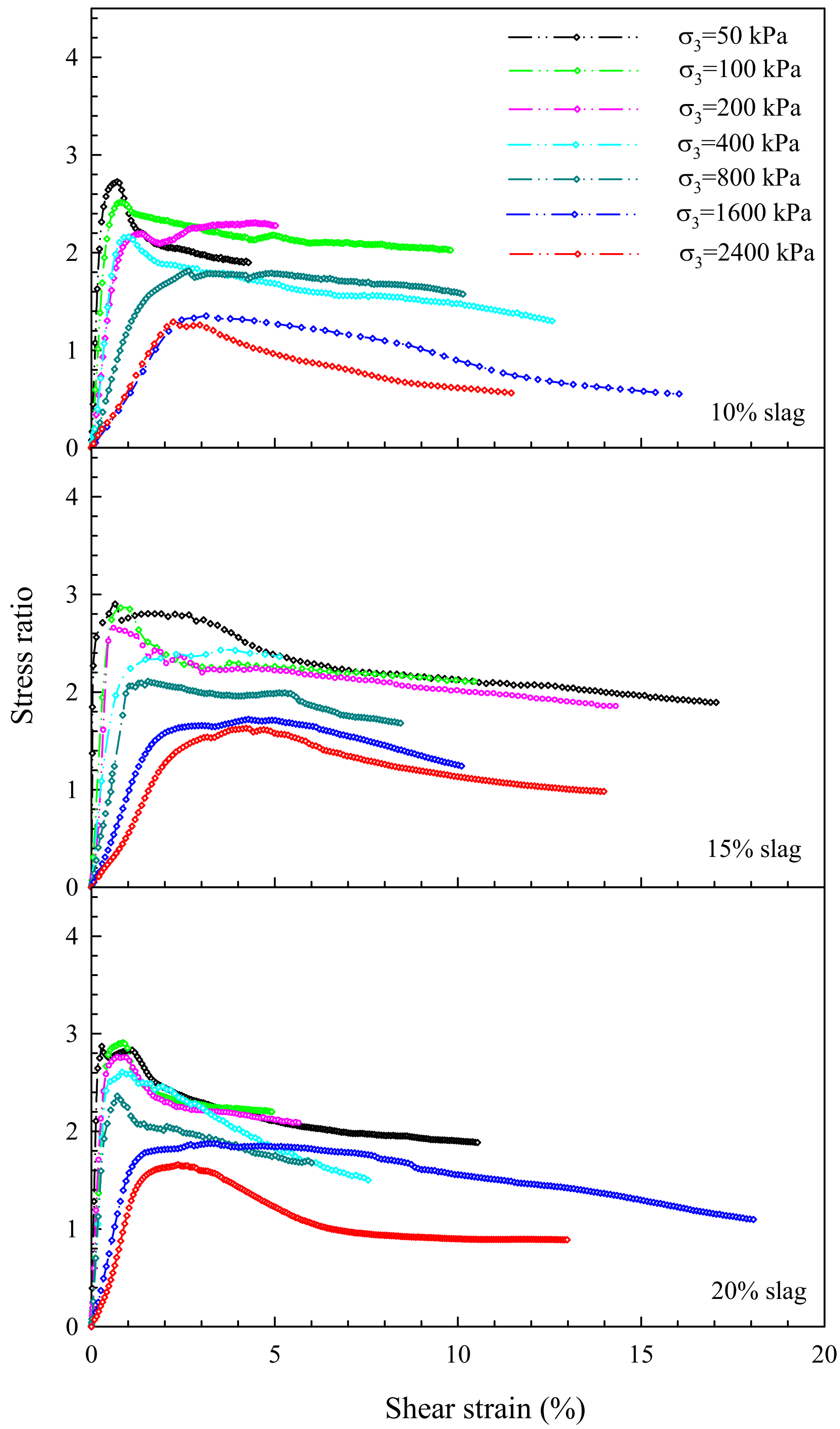

Figure 7.3 Effect of consolidation pressure on the stress ratio-strain behaviour of lime-slag treated CIS (curing period = 1 month)

value. On the other hand, the rate of increase of stress ratio with shear strain slows down with an increase in consolidation pressure. The relatively rapid increase of stress ratio with shear strain at low level of consolidation pressure can be attributed to the presence of higher amount of cementation bonds at the beginning of shearing. Due to increasingly reduced degree of participation of cementitious bonds in the mobilization of shearing resistance at higher consolidation pressures, the rate of increase in the stress ratio with shear strain slows down.

Another interesting observation can be made from the results presented in Figure 7.3. Although it was found that for soils sheared from low level of consolidation pressure, the stress remained almost constant after the peak value was reached, the stress ratio actually shows strain softening behaviour beyond the peak value. Since the stress ratio displays progressive softening even though shear stress was found to remain approximately constant, the mean effective stress must have increased beyond the peak stress value to cause a reduction in the stress ratio. The increase in mean effective stress can be attributed to the progressive dilation in the shear bands and this type of behaviour implies that for samples sheared from low level of consolidation pressure, fractures may have been initiated at the peak stress level.

For samples sheared from higher level of consolidation pressures, the peak value of the stress ratio is generally found to take place at strain level larger than the strain at which peak stress value is observed. This observation implies that although the samples sheared from higher consolidation pressures exhibits strain softening immediately after the peak stress is reached in conventional stress-strain plots, the stress ratio vs. shear strain diagram reveals that the soil skeleton is actually hardening in the earlier part of the strain softening range as observed in conventional stress-strain plots. Similar observation on the stress-strain behaviour of structured soils has been discussed by Burland (1990) who suggested that this type of behaviour implies that the sample deforms in a homogeneous manner beyond the peak stress value until the peak stress ratio is reached. This observation indicates that for samples sheared from higher level of consolidation pressures, shear banding may not initiate at peak stress level rather macro level fracture may appear at a shear strain larger than the strain at which peak stress takes place. Similar conclusion about the strain level at which macro fractures forms within the samples sheared from elevated consolidation pressures was also made based on the observation of the location of discontinuity in the stress-strain curves.

7.2.3 Elastic stiffness of lime-slag treated CIS

Based on the load-deformation response measured at the boundary of the samples by conventional triaxial testing method, it has been found that stiffness of treated CIS generally increases with an increase in slag content and consolidation pressure. However, the influence of different experimental variables on the stiffness is not discussed in details recognizing the limitations associated with the determination of stiffness based on boundary measurements. For many structures (e.g., walls supporting urban excavations) stipulated serviceability limits requires the structures to be far from failure. For such conditions, an accurate determination of the elastic stiffness of soils become critically important for realistic predictions of ground movements that may affects the adjacent structures (Clayton, 2011). It has been shown by various researchers that conventional method of stiffness determination based on boundary measurements produces highly inaccurate values of elastic stiffness (Burland, 1989, Clayton, 2011) due to different limitations associated with the measurement of stiffness in conventional triaxial tests. Among many such limitations important ones are misalignment of the loading ram during the consolidation phase, bedding error and compliance of the testing apparatus.

Kamruzzaman (2002) compared the elastic stiffness values calculated from displacements measured both locally and globally for cement treated Singapore marine clay and found the locally measured values to be very different from the values obtained from boundary measurements. Since any special arrangement was not made in this investigation for accurate measurement of elastic stiffness of lime-slag treated CIS, the description of soil stiffness based on boundary measurements will largely be qualitative in nature. The influences of lime content, slag content and curing period on the elastic stiffness have been already discussed qualitatively based on the results of unconfined stress-strain behaviour and that is why another qualitative study on the elastic stiffness behaviour of lime-slag treated CIS is not repeated here.

7.2.4 Pore pressure response to undrained shearing

The development of excess pore pressure in response to increasing shear strain is shown in Figure 7.4 and Figure 7.5. While Figure 7.4 shows the effect of slag content on the excess pore pressure development, Figure 7.5 shows the effect of pre-shear consolidation pressure on the excess pore pressure behaviour.

It can be observed from Figure 7.4 that the effect of slag content on the magnitude of maximum positive excess pore pressure is insignificant. For all the slag contents, it was found that the value of the maximum excess pore pressure was almost equal to the magnitude of pre-shear consolidation pressure for samples sheared from low level of consolidation pressures. However, at higher consolidation pressures, the maximum excess pore pressure was found to be noticeably smaller than the applied consolidation pressure. The effect of slag content on the excess pore pressure response is found to be more prominent at low level of pre-shear consolidation pressure. At a pre-shear effective confinement of 50 kPa, it is found that for soil samples treated with all different slag

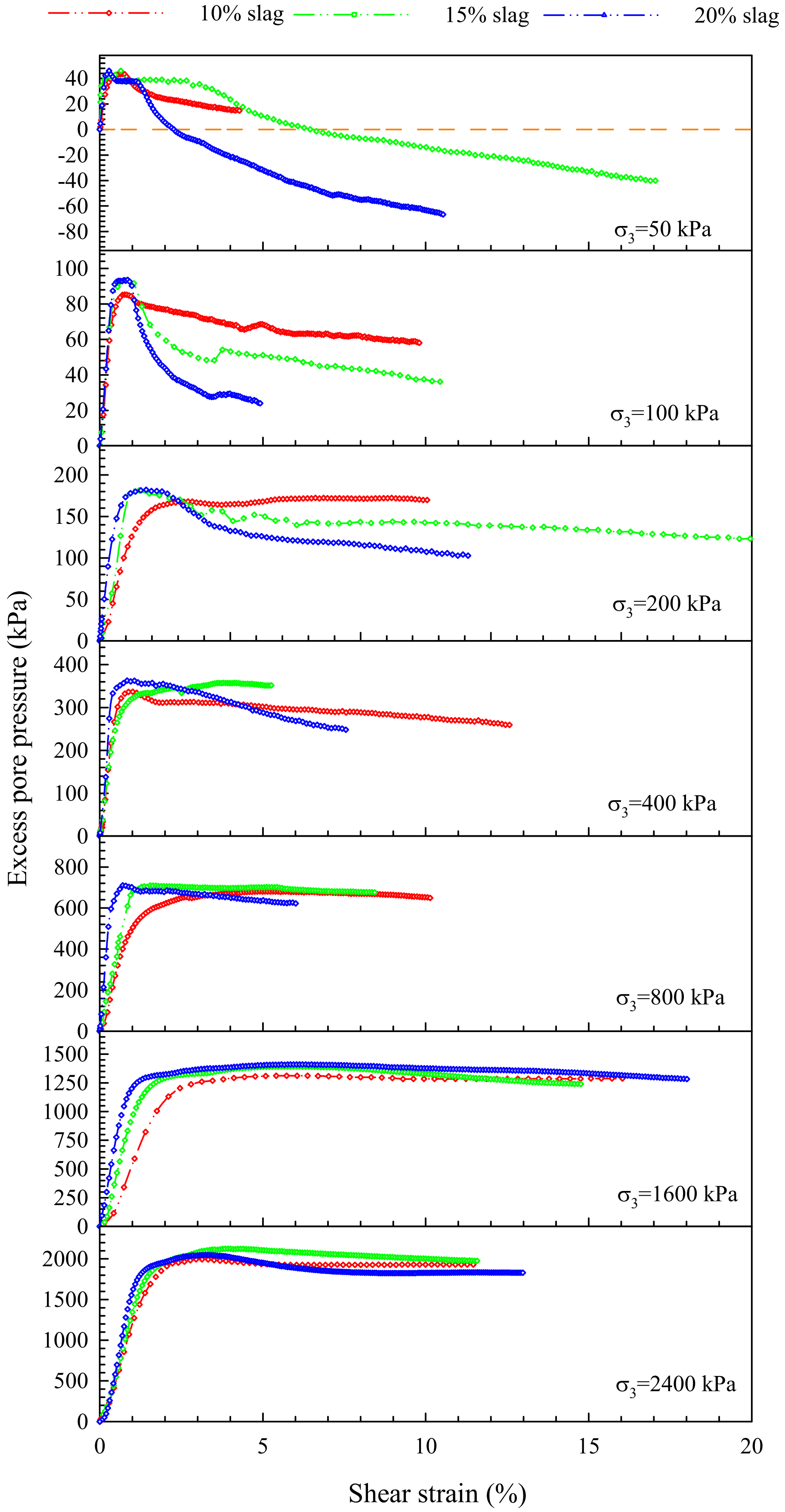

Figure 7.4. Effect of slag content on excess pore pressure – shear strain behaviour of lime-slag treated CIS (curing period = 1 month)

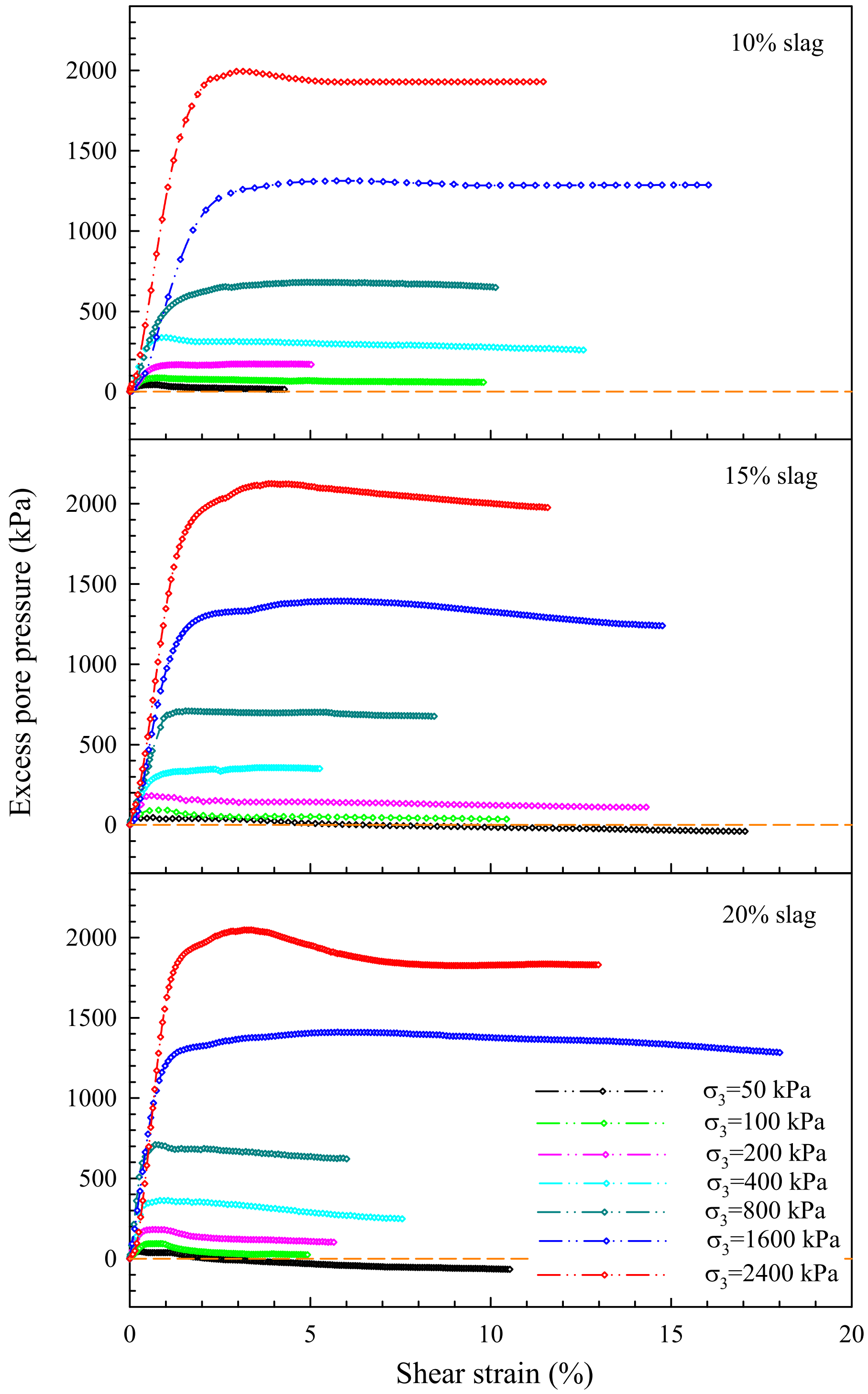

Figure 7.5. Effect of consolidation pressure on excess pore pressure – shear strain behaviour of lime-slag treated CIS (curing period = 1 month)

contents, the pore pressure initially rises to maximum positive value and thereafter shows a declining trend indicating dilation of the samples. The peak positive excess pore pressure takes place at a shear strain similar to that at which peak shear stress is seen to take place. The decline of pore pressure after this particular strain level is possibly due to the ongoing dilation caused by the movement of broken clusters of bonded particles within the shear bands. It is found that at consolidation pressure of 50 kPa, the excess pore water pressure for the sample treated with 20% slag becomes negative at a strain of approximately 2% whereas for the sample treated with 15% slag, the negative pore pressure starts accumulating from a strain level of approximately 7% and although the excess pore pressure shows a declining trend after reaching the maximum positive value for the sample treated with 10% slag, it does not attain any negative value for the range of strain investigated. The increasing degree of dilation with increasing slag content may be attributed to the influence of slag content on the characteristics of the formed shear surfaces and that of the clusters of bonded particles formed within the shear bands.

Significant post-peak dilation is observed for consolidation pressure up to 400 kPa and for samples sheared from 1600 kPa of consolidation pressure, the dilation almost completely disappears. The transition from dilating to contracting behaviour indicates that with the increase in confinement, the brittleness at peak stress level decreases, the breakdown of cementation bonds becomes more uniform throughout the samples and the soil body can deform as a whole. For samples sheared from elevated consolidation pressures, the excess pore pressure is seen to increase to peak value and it remains almost constant beyond the peak value. For samples sheared from elevated consolidation pressures, the strain at which peak excess pore pressure takes place has generally been found to be greater than the strain at which peak shear stress takes place. The peak excess pore pressure takes place at a shear strain range of 3-4% which is higher than the strain at which peak stress takes place (peak stress values are seen to take place at a shear strain ≤ 2%). This observation gives an indication that for samples sheared from higher levels of consolidation pressures, significant level of macro-level fracturing may not occur at the peak stress state. However, almost stationary values of the excess pore pressure beyond the peak value indicate that macro-cracks may have been formed after the peak excess pore pressure had been reached. Once the macro-cracks are formed, the deformation is characterized by the relative sliding of rigid blocks and this sliding is unlikely to induce any excess pore pressure. The bonded clusters of particles may also form in the shear bands when the samples are sheared from elevated level of consolidation pressures but it is likely that these clusters will find it difficult to survive at elevated confinement. The overall dilation caused by these clusters of bonded particles will therefore be increasingly suppressed at higher level of confinement and this suppressed dilation may not be able to decrease the pore pressure to any significant extent.

7.2.5 Characteristics of undrained stress-paths

The study of stress paths, which represent progressive and corresponding changes in deviatoric stresses and mean effective stresses, can be helpful in understanding the contribution of cementation bonds and frictional resistances in the mobilization of shearing resistance at different stages of the loading. In order to gain a holistic overview of the shearing behaviour of lime-slag treated CIS, the behaviour of the stress-paths is studied in this section with the help of Figure 7.6 and Figure 7.7.

It is seen that at all levels of pre-shear consolidation pressures, the stress path initially rises almost vertically and then bends towards either left or right depending on the effective pre-shear consolidation pressure. Under undrained shearing elastic volumetric strain is zero (for isotropic material) up to the initial yielding of the soil and therefore the mean effective

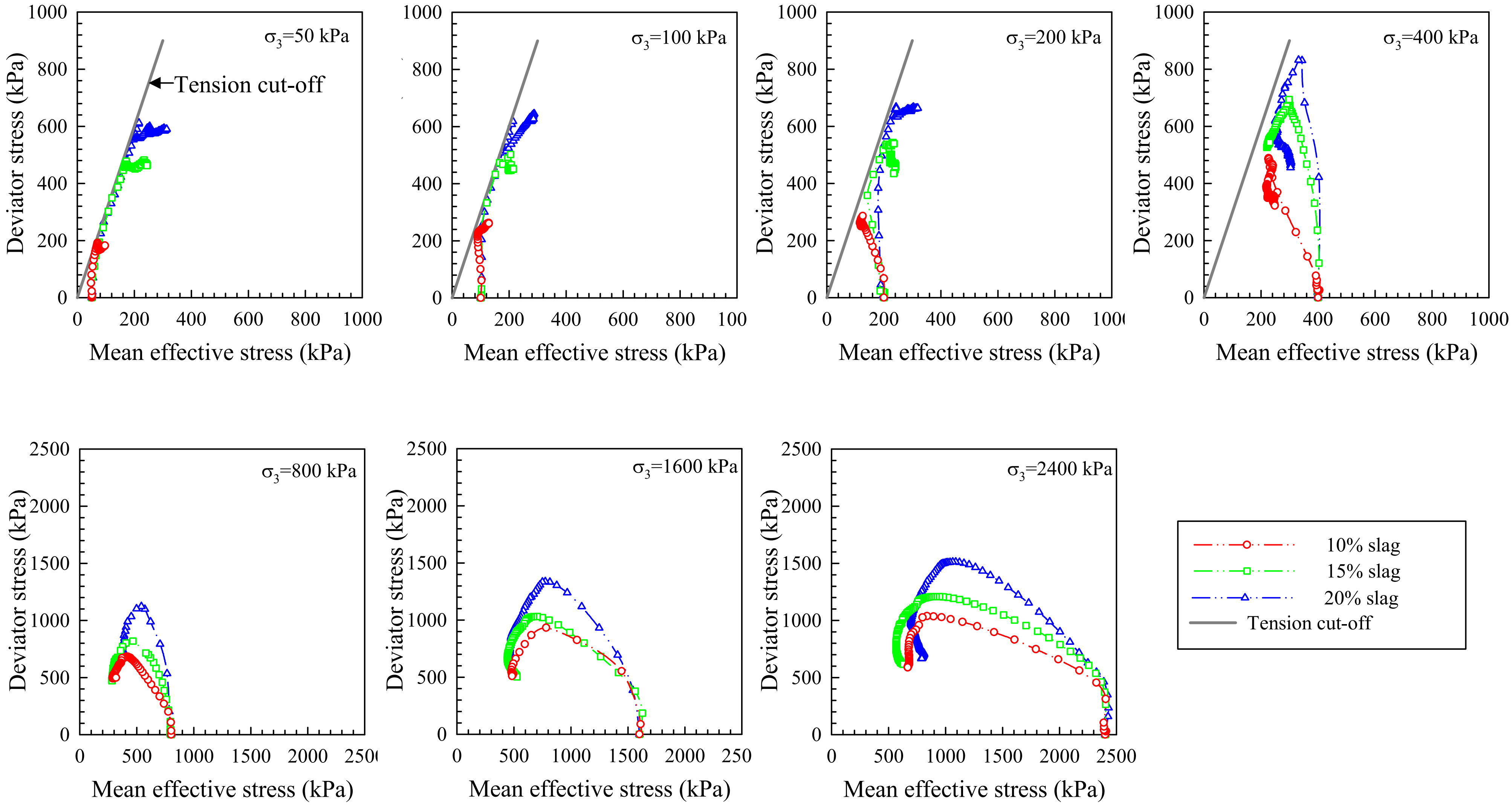

Figure 7.6. Effect of slag content on the undrained stress-paths of lime-slag treated CIS (curing period = 1 month)

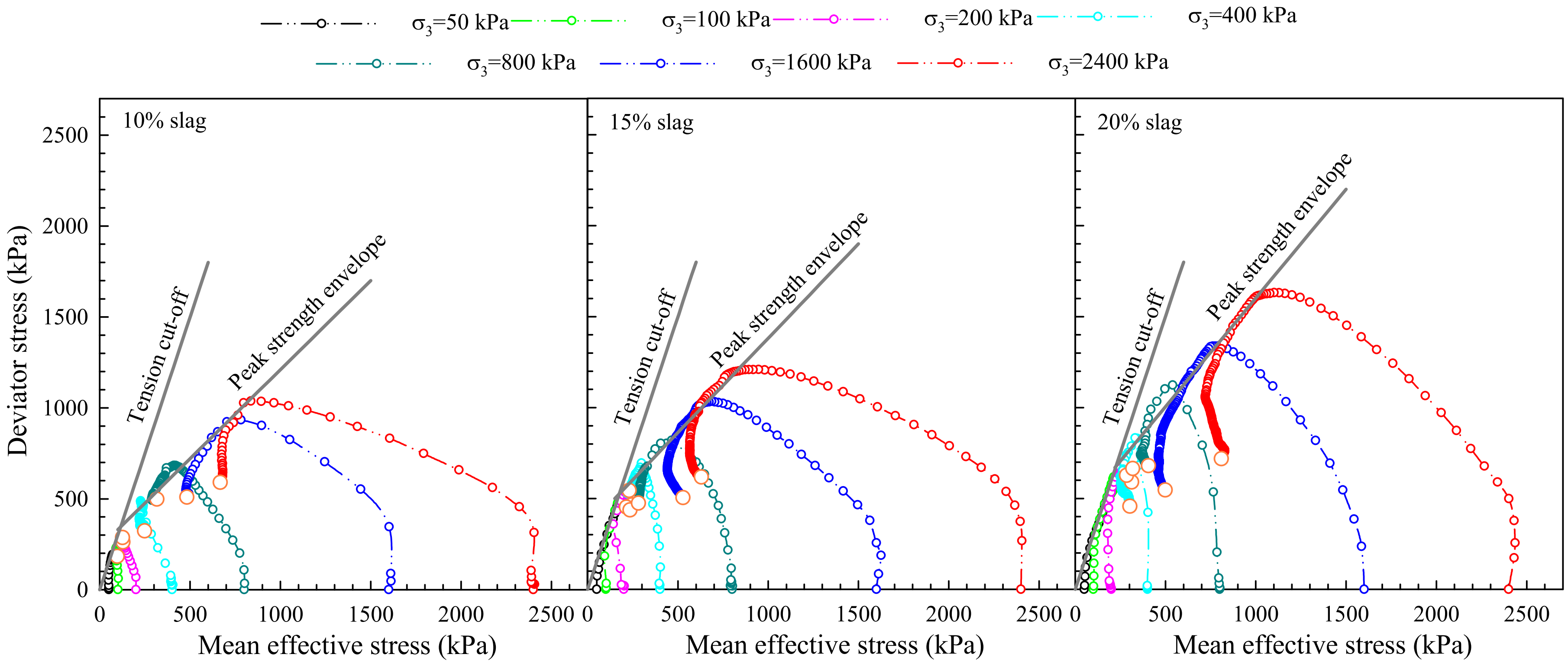

Figure 7.7. Effect of consolidation pressure on the undrained stress-paths of lime-slag treated CIS (curing period = 1 month)

stress does not change during the purely elastic part of the loading. A vertical rise of the stress path with almost constant mean effective stress therefore implies an initial elastic behaviour. It is found that lime-slag treated CIS sheared from consolidation pressure both less and greater than the yield stress exhibit an initial elastic part. It is a well-established experimental observation that when un-cemented soils are sheared from normally consolidated states, the soil displays plastic deformation from the very beginning of the shearing and the stress path starts bending towards the left from the onset of the shearing. The initial elastic response of lime-slag treated CIS observed at confining pressure as high as 2400 kPa can be attributed to the presence of cementation bonds still remaining in the sample at the beginning of the shearing.

It is seen that the height of the initial vertical section of the stress-path decreases with increasing consolidation pressure and this observation may be attributed to the higher degree of de-structuration caused to the samples by the higher level of consolidation pressure applied to the soils prior to shearing. Another important observation is that for soil samples sheared from relatively low level of consolidation pressures, the height of the initial vertical section of the stress path increases significantly with increasing slag content. However, as the confining pressure increases, the difference in the height of the constant p' section resulting from the differing slag contents decreases (Figure 7.6). The observation clearly indicates that the effect of slag content on the elastic behaviour of the samples significantly reduces due to the de-structuration caused by the high level of consolidation pressures.

Apart from the issues discussed above one very important observation is that the behaviour of stress-path changes significantly due to change in the pre-shear consolidation pressure. At low level of pre-shear consolidation pressures, the stress-path indicates dilating soil behaviour starting approximately from the peak strength whereas the soil behaviour gradually changes to contracting type at higher level of pre-shear consolidation pressures. In the following sections the mechanisms behind this dependency of stress-paths on the consolidation pressure is discussed.

It is seen from Figure 7.6 that that up to a pre-shear confining pressure of 200 kPa, the stress paths for all the slag contents initially rise up to the tension cut-off line and then bends towards the right showing a tendency for dilation of the samples. Once the stress path reaches the tension cut-off line initially, it is seen to run almost along the tension cut-off line up to the peak stress values. Along the tension cut-off line the effective radial confinement is zero and the mobilization of friction along the tension cut-off line should be negligible due to lack of appreciable mean effective confinement. Since the stress paths can propagate along the tension cut-off line, the majority of the strength over this section of the stress path is most likely derived from the cementation bonds that were still present when the stress path initially reached the tension cut-off line. Once the stress-path reaches the peak strength, the samples show a tendency of strong dilation as evidenced by the almost horizontal progression of the stress paths towards the increasing p' value. It was discussed in earlier section that the dilation is caused possibly due to the formation of clusters of bonded particles within the shear bands and their relative movement with increasing shear strain. Although it was seen from Figure 7.1 that when the soil samples are sheared from low level of consolidation pressures, the deviatoric stress values become approximately constant after reaching the peak strength value, the behaviour at large shear strain, which is characterized by the discontinuous response of the fragmented rigid blocks, may not be representative of the behaviour of the soil at their fully de-structured states.

For all the slag contents, the stress paths show a marked change in their behaviour when the magnitude of pre-shear confining pressure is increased from 200 kPa to 400 kPa. Up to 200 kPa of consolidation pressure, the stress path bends towards the right after the initial constant p' path but starting form 400 kPa of pre-shear confining pressure, the stress path bends towards the left after the initial vertical section. Once the stress path reaches peak strength envelope, it is seen to move down the peak strength envelope before falling down in a nearly vertical manner. The sharp fall of the stress-path may be due to the formation of significant micro-cracks within the samples. It is observed that with increasing confining pressure, the roundness of the stress-paths increases. At low level of confining pressure, the stress paths rises to the peak value almost vertically whereas at higher consolidation pressure such as 2400 kPa, the stress paths start to bend towards the left much before the peak value is reached. The implication of this observation is that the contribution of frictional resistance to the peak strength increases with increase in consolidation pressure. In order for friction to mobilize two conditions need to be met: sufficient grain-to-grain contacts needs to be established through the collapse of the cementitious bridges and the confining pressure needs to be sufficient so as to fulfill the basic condition for the frictional resistance to mobilize under increasing shear strain. At low level of confining pressure, when the resistance provided by the cementation bonds is exhausted, friction cannot mobilizes due to lack of confinement and as a result the sample fails very close to the elastic limit of the soil in a brittle manner. At higher consolidation pressure, when the cementitious bonds which were still intact at the beginning of the shearing withstanding the consolidation stresses are destroyed significantly due to shearing, a sufficient number of contacts among the soil particles can be established and frictional resistance can start to mobilize due to presence of sufficiently high level of confinement. Since the soil is sheared under undrained condition, the volumetric compression tendency of the samples is reflected as a gradual build up of pore water pressure and progressive reduction of mean effective stress until the stress state reaches the peak strength envelope.

It has been observed that for any given level of deviatoric stress, the generation of pore pressure increases with a decrease in slag content. This is evidenced by the fact that the effective stress paths for soils treated with lower amount of slag exhibit higher amount of deviation from the total stress path. This may be due to the higher degree of de-structuration caused to the samples treated with lower amount of slag at any given consolidation pressure. At any given level of deviatoric stress, the amount of cementitious bonds present in the sample is likely to be higher for soil treated with higher amount of slag. The presence of a higher amount of intact cementitious bonds in the samples treated with higher amount of slag may provide the material with enhanced level of bulk stiffness. As a result of this enhanced bulk stiffness, the amount of pore pressure generated at a given level of deviatoric stress decreases with increasing slag content. One very important implication of this observation is that the influence of cementation on the shearing response is not completely eliminated at the elastic limit of the soil since the effect of the degree of cementation is clearly seen in the stress-range where the soil behaviour is elasto-plastic.

7.2.6 Strength envelope of lime-slag treated CIS

The peak strength envelope of the limes-lag treated CIS is shown in Figure 7.8(a). The strength envelope is based on the maximum deviatoric stresses and corresponding mean effective stresses observed during the undrained shearing at different levels of consolidation pressure. It can be seen that the peak strength envelope for all the different slag contents is curvilinear. When the treated CIS sample is sheared from low level of consolidation pressure, the strength is mainly derived from the cementation bonds and the gradient of the strength envelope is much stiffer at lower mean effective stress level than the gradient observed at larger mean confining stresses. At low level of confining pressure, the peak strength data for all the soils fall on the tension cut-off line implying a $\frac{dq}{dp'}$ ratio of 3. The curvilinear peak strength envelope therefore implies that when the cementation bonds are intact, the sample can withstand a much larger shear stress for a given level of mean effective stress. Such curvilinear peak strength envelope for artificially cemented soils has also been reported by Uddin (1995) and Kamruzzaman (2004).

It is difficult to derive a unique set of peak strength parameters from the data presented in the $p' - q$ stress space in Figure 7.8(a) due to the non-linearity of the peak strength envelope. It was also found that it is also difficult to find a common tangent to the effective stress Mohr circles corresponding to different consolidation pressures. Uddin (1995) found that the use of $s' - t$ plot [where, $s' = (\frac{\sigma_1'+\sigma_2'}{2})$ and $t = (\frac{\sigma_1'-\sigma_2'}{2})$] can be more convenient than conventional Mohr-Coulomb plots for the determination of strength parameters of artificially cemented soils. Following that suggestion the peak strength data was plotted in $s' - t$ stress space as shown in Figure 7.8(b). Average lines through the data points were drawn based on eye-estimate. The lower parts of the lines are extended up to the t axis (these extensions are represented by dotted lines) to determine the t axis intercept. It can be seen from the figure that fairly straight lines could be drawn through the data points for all the slag contents. It can be seen from Figure 7.8(b) that the drawn straight lines are almost parallel to each other while having different intercepts on the t axis. The peak strength data plotted in Figure 7.8(b) gives an indication that the peak friction angle does not change significantly with slag content but the cohesion intercept increases with an increase in the degree of cementation. It was found that the effective peak friction angle (ϕ_p') varied within a range of 34-36.5^0. On the other hand, the effective cohesion intercept for 10, 15 and 20% slag treated CIS was found to be 48 kPa, 118 kPa and 164 kPa respectively.

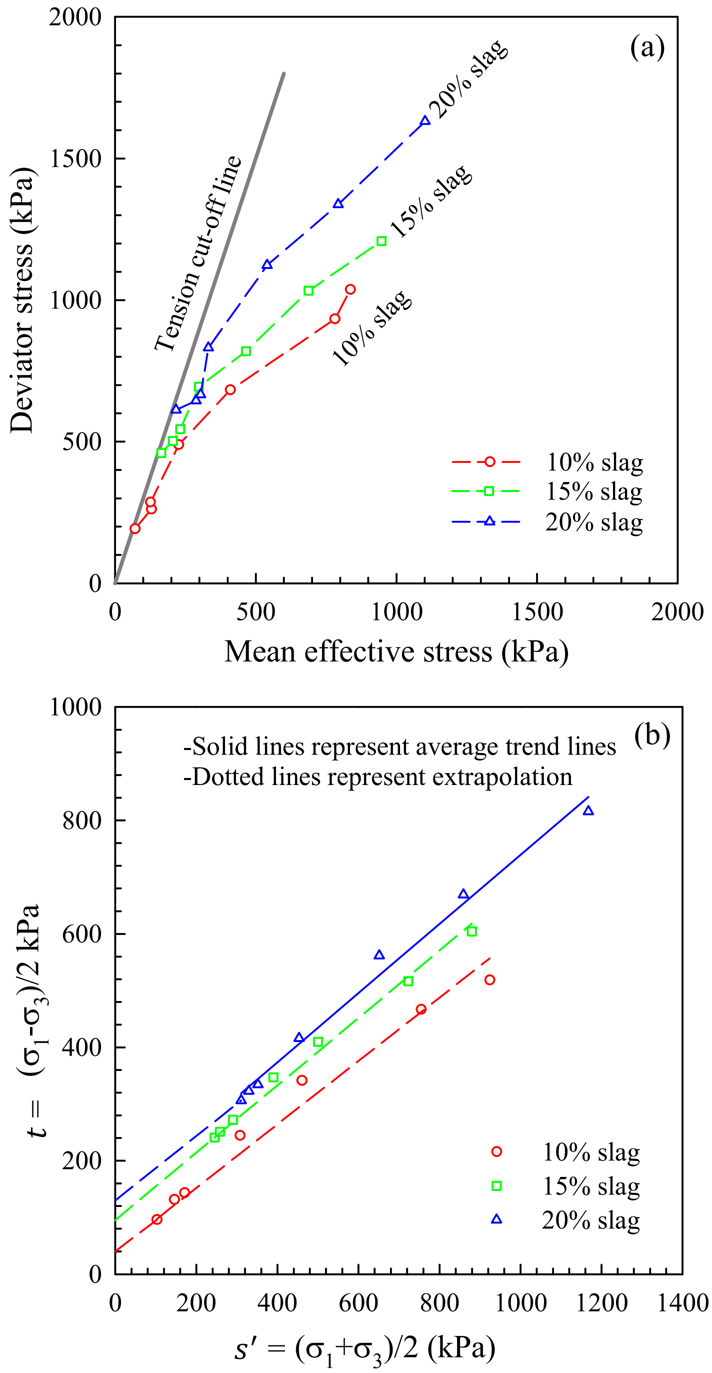

Figure 7.8. peak strength envelope (a) in $p' - q$ space (b) in $s' - t$ space

(curing period = 1 month)

Identification of de-structured state for lime-slag treated CIS is difficult due to the formation of fractures within the samples. It can be seen from Figure 7.1 that stress-strain curves tend to attain approximately a constant stress state at a strain level in excess of 15% and this is especially true for the CIS treated with 10% slag. However, the stress-ratio vs. shear strain plots presented in Figure 7.3 shows that for samples treated with a particular amount of slag but sheared from different consolidation pressures, the stress-ratios at large strain does not show a tendency to converge to a common value. The stress-path plots presented in Figure 7.7 shows that an approximately straight line could have been drawn by connecting the origin of the $p' - q$ stress space and the set of deviator and mean stresses corresponding to large strain condition for samples sheared from 1600 kPa and 2400 kPa of consolidation pressures. However, whether such line is representative of de-structured state of the soil is inconclusive and the true significance of this line can only be confirmed by carrying out undrained shearing test from much higher consolidation pressure. It is immediately obvious from Figure 7.7 that if such lines were drawn, the large strain strength data for soils sheared from consolidation pressure less than 1600 kPa would have lied much above that line. All these observations on the large strain behaviour clearly indicate about the difficulty associated with the characterization of large strain behaviour and therefore no particular set of parameters have been assigned to the large strain condition.

7.3 Summary

A series of isotropically consolidated undrained triaxial shear testing with pore pressure measurement was conducted on CIS treated with different amounts of lime and slag to understand the undrained shearing behaviour of lime-slag treated CIS. The main observations from this experimental investigation are summarized below:

- The stress-strain behaviour is found to be affected by both the slag content and pre-shear consolidation pressure. In the presence of adequate amount of lime, increasing slag content is found to increase both the strength and stiffness due to increased degree of cementation. Increasing curing period can be expected to have a similar influence on the stress-strain and stiffness behaviour. Increasing consolidation pressure has been found to increases both the strength and stiffness although the influence of confining pressure on the strength and stiffness is seen to be insignificant at low level of consolidation pressure. For soils sheared from low level of consolidation pressure, the peak stress ratio and peak stress is found to take place at similar strain levels and the decline of stress ratio beyond the peak stress indicates that shear band may start forming at the peak stress level. On the other hand, at higher consolidation pressure peak stress ratio is seen to take place at strain larger than the strain at which peak stress takes place indicating that the formation of macro-level fracture may be delayed to a strain level larger than the strain at which peak stress is observed.

- Excess pore pressure is seen to increase with an increase in pre-shear consolidation pressure. At low level of consolidation pressure, the excess pore pressure decreases after the peak positive value is reached possibly due to the dilation caused by the formation of clusters of bonded particles within the shear bands. The rate of dilation increases with an increase in slag content. At higher level of pre-shear consolidation pressure, the excess pore pressure is seen to increase up to a shear strain level larger than the strain at which peak shear stress takes place and this observation also implies that formation of major macro-cracks may be initiated at a strain level beyond the strain at which peak stress value is observed.

- The behaviour of undrained stress paths is seen to be dependent of consolidation pressure. At low level of consolidation pressure, the stress paths rise almost vertically to the peak stress value and thereafter show a strong tendency of dilation. This dilation is likely to be caused by the formation of shear bands and the formation of clusters of bonded particles within the shear bands at the peak stress level. At higher level of consolidation pressure, the stress-path bends towards the left after the initial vertical rise and much before the peak stress value is reached implying that the contribution of friction towards the peak strength increases with increasing confining pressure.

- It has been found that the peak failure envelope is non-linear in $p' - q$ space making it difficult to derive unique set strength properties of the treated soil. However, in $s' - t$ stress space fairly straight lines could be drawn through the data points corresponding to peak stress conditions. The parameters determined from $s' - t$ plots indicate that cementation does not affect the peak friction angle significantly but affects the cohesion value of the lime-slag treated CIS samples. It was found difficult to obtain a set of strength parameters corresponding to de-structured states possibly due to the formation of shear bands in all the samples tested. From the study of stress ratio vs. shear strain behaviour it was found that stress ratio at large strain did not converge to a common de-structured value.

- For practical geotechnical applications involving lime-slag treated CIS, significant caution needs to be exercised. For low confinement conditions, if the stress value approaches the peak strength value of the treated material, there is a danger of fracturing of the treated CIS. On the other hand, at higher confining pressure, there is a risk of strain softening beyond the peak stress value.

7.4 References

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CHAPTER 8: CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

Due to ever increasing demand for expansion of urban infrastructure, the demand for suitable land is increasing disproportionately. As a result, there is a growing need for developing infrastructure on lands previously deemed unsuitable from economic considerations. The problem associated with poor ground condition is prevalent in the prime locations of Melbourne CBD which is predominantly underlain by a problematic soft pyrite bearing CIS. Previous researches on the effectiveness of cementitious stabilization for improving the engineering properties of soft CIS has been found to be promising. While earlier researches on improving the engineering properties of CIS were mainly based on very simple laboratory tests, a more elaborate experimental investigation was undertaken under the current research project with the objective of mechanical characterization of the stabilized CIS. Several important conclusions were drawn based on the results of the current investigation and those conclusions are presented in this chapter.

8.1.1 Strength development characteristics of lime-slag treated CIS

Progressive strength development of CIS treated with different combinations of lime and slag was assessed by conducting UCS tests on treated samples at different curing periods. It was found that strength and deformation properties of soft CIS could effectively be improved by treating it with lime and slag. From an analysis of the results of current experimental investigation and previous investigations, it was found that the minimum amount of lime to be used in the stabilization of CIS should be higher than the lime saturation point determined by ICL test. It was found that when the amount of lime used in the stabilization is higher than the saturation lime content, the strength development becomes almost independent of lime content. On the other hand, slag content and curing period have been found to be the main determinant of strength development. It was found that the effect of slag content on the strength development become more prominent at longer curing period. The effect of curing was seen to be more prominent from one month to three months than its effect observed from three months to six months. This study found a strong correlation to exist between the strength and stiffness magnitudes for all the combinations of different experimental variables. However, the rate of increase of stiffness with strength was found to become more prominent at longer curing periods.

8.1.2 Generalized compression behaviour of structured soils

From the review of literature it was found that the compression behaviour of structured soils is significantly different from the behaviour of corresponding reconstituted soils. The compression behaviour of structured soils is characterized by two distinct mechanisms in different stress ranges. While the compression of structured soils immediately after yield is governed by the progressive collapse of the inter-aggregate cementitious bridges, the behaviour at large stress is controlled by the mineralogy of the soil aggregates. A new virgin compression model for structured soils was developed based on the experimentally observed compression behaviour of a large number of naturally structured soils. The model development was based on the data of naturally structured soils instead of artificially cemented soil mainly due to lack of sufficient literature data on the compression behaviour of artificially cemented soils at de-structured states. The newly developed compression model is based on a hyperbolic variation of a newly proposed secant compression index with pressure. The proposed compression model contains two parameters a and b which can independently characterize the two distinct mode of compression operating in two different stress ranges. The values of the parameters can easily be determined from routine laboratory compression test.

8.1.3 Compression behaviour of artificially cemented soils

The compressibility behaviour of lime-slag treated CIS was investigated with the help of comprehensive series of 1-D compression tests. It was found that slag content has the highest degree of influence on the compressibility behaviour of lime-slag treated CIS. The effect of curing was found to be most prominent from one month to three months and after that the influence of curing decreased considerably. On the other hand, lime content was found to have comparative much less influence on the observed compressibility behaviour. It was found that increasing degree of cementation increases both the 1-D yield strength and post-yield compressibility. It was suggested that increasing degree of cementation resulting from either increasing slag content or increasing curing period results in a more uniform distribution of smaller sized pores and this alteration of pore size distribution is likely to be responsible for the increase in yield strength and post-yield compressibility.

The proposed compression model was validated against the compressibility data of lime-slag treated CIS and its performance to simulate the compressibility behaviour of lime-slag treated CIS was found to be excellent. A clear correspondence was established between the experimentally observed post-yield de-structuration behaviour and the model parameter a. It was found that the model parameter a decreases with an increasing degree of cementation. On the other hand, the model parameter b was found to vary within a narrow range possibly due to limited impact of the variation of additive content and curing period on the mineralogical alteration of the treated soil.

8.1.4 Shear behaviour of lime-slag treated CIS

A comprehensive series of Isotropically Consolidated Undrained (CIU) triaxial testing program with pore pressure measurement was undertaken to understand the undrained shearing behaviour of lime-slag treated CIS. The main variables investigated in this part were the effect of degree of cementation and pre-shear consolidation pressure on the shearing response. It was found that both slag content and pre-shear consolidation pressure affect the stress-strain behaviour. Increasing slag content was found to increase both the undrained strength and stiffness magnitudes. The effect of consolidation pressure on the stresses-train behaviour was found to be dependent on the level of consolidation pressure. At low level of consolidation pressure, the effect of pre-shear confinement was found to be negligible whereas its influence on the shearing response became increasingly prominent at increasing level of consolidation pressure. It was found that at low level of consolidation pressure, the stress-strain response is characterized by brittleness in the vicinity of the peak stress and the stress value was found to remain almost constant in the post-peak strain range possibly due to the ongoing dilation within the fractures planes. The presence of post-peak dilation at low level of consolidation pressure was also confirmed by the observed pore pressure response. At higher level of consolidation pressures, the stress-train behaviour was found to be progressively strain-softening in nature. Based on the analysis of the stress-strain data and pore-pressure data it was suggested that with an increase in consolidation pressure, brittleness at peak decreases.

It was found that at low level of consolidation pressure, peak positive excess pore water pressure takes place near the vicinity of the peak stress value and thereafter the excess pore pressure gradually becomes negative at higher level of shear strain. At higher consolidation pressure, the excess pore water pressure increased to a maximum value at a strain level larger than the strain at which peak stress takes place. The excess pore pressure becomes almost stationary after reaching the peak value implying that fracture may have been initiated in the sample after the peak excess pore pressure is reached.

It was found that a unique set of strength parameter can be derived corresponding to peak stress condition and it was found that cohesion intercept increases noticeably with an increase in slag content but the peak friction angle was found to vary within a relatively narrow range. It was not possible to derive a set of strength parameters corresponding to de-structured condition because the combinations of mean effective stress and deviatoric stress corresponding to large strain condition did not fall on a unique straight line possibly due to the non-homogeneous deformations of the samples in the post-peak strain ranges.

8.2 Recommendations for further research

- 1-D compression tests should be carried out up to very large pressure and the associated pore collapse mechanism at different stress ranges should be studied with the help of advanced laboratory techniques to better understand the mechanisms controlling the volumetric compression behaviour of treated CIS.

- Although there have been several advanced experimental studies on the progressive pore-collapse mechanism under 1-D compressive loading, it appears to the author that study of such mechanism under shear loading has not been carried out as comprehensively. Study of the gradual pore collapse mechanism under shear loading can be studied at real time with the help of advanced imaging facilities such as X-Ray CT. Such studies can be invaluable in explaining observed shearing behaviour reported in this thesis.

- Progressive mineralogical development in CIS treated with different amounts of additives and cured for different periods should be investigated in quantitative terms. Results of such investigations will be helpful in explaining the influences of different experimental variables on various mechanical responses of treated CIS observed in this study.

- The unloading-reloading behaviour of treated CIS should be experimentally investigated to understand the effects of de-structuration on the unloading-reloading behaviour.

- Secondary compression behaviour of treated CIS should be studied with particular focus placed on how different degrees of de-structuration influence the secondary compression behaviour.

- The mechanical behaviour of lime-slag treated CIS in field applications may be somewhat different from the laboratory behaviour reported in this study. It is recommended that further field study be undertaken to investigate the influence of different field variables on the compression and shearing behaviour of lime-slag treated CIS.