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Performance of expansive soils stabilised be cementitious binders and inclusion of a nanotechnology-based additive

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Performance of expansive soils stabilised by cementitious binders and inclusion of a nanotechnology-based additive

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A Study to Develop a Solution to Predict the Behaviour of Expansive Soils Improved by the Soil Mixing Technology

This is to certify that the above named applicant has completed the Coventry University Ethical Approval process and their project has been confirmed and approved as Low Risk

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JOURNAL PUBLICATIONS, CONFERENCE AND WORKSHOPS

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5. Eyo E. U., Ng’ambi S., Abbey S. J., Eshmaiel Ganjian, E. Coakley. “Incorporating a nanotechnology-based product in cementitious binders for sustainable mitigation of sulphate-induced heaving of stabilised soils”. Engineering Science and Technology Journal.
6. Abbey, S.J. Eyo, E.U. Ng'ambi, S. “Swell and Micro-structural Characteristics of High Plasticity Clay Blended with Cement”. Bulletin of Engineering Geology and Environment, Vol 79: No 4. 2020.
7. Abbey, S.J. Olubanwo, Adegoke. Ngambi, S. Eyo, E.U. Adeleke, B. “Effect of Organic Matter on Swell and Undrained Shear Strength of Treated Soils”. Journal of Civil, Construction and Environmental Engineering 2019; 4(2): 48-58.
8. Eyo, E.U. Ng'ambi, S. Abbey, S.J. “Incorporation of a nanotechnology-based additive in cementitious products for clay stabilisation”. Journal of Rock Mechanics and Geotechnical Engineering. Vol. 12(5). Oct 2020.
9. Eyo E. U., Ng’ambi S., Abbey S. J., Eshmaiel Ganjian, E. Coakley. “Swell-strain responses and compressibility characteristics of cement stabilised low-to high plasticity soils”. Geotechnical and Geological Engineering.
10. Abbey S. J., Jonathan E. Oti., Eyo E. U. Ng’ambi S. “Mechanical Performance of Fibre-Reinforced Cement-Clay Mixture Blended with Industrial By-Product Materials”. Geosciences (MPDI). Vol 10 No 6.

11. Abbey, S. J., Eyo E. U., Okeke C. A, Ng’ambi, S. “Mechanical and Microstructural Evaluation of Nano- additive Treated Soils Blended with By-product Cementitious Materials”. Construction and Building Materials Journal.

Conferences and Workshop Presentations and Publications

1. Eyo, E.U. Ng'ambi, S. Abbey, S.J. “Investigative Study of Behaviour of Treated Expansive Soil Using Empirical Correlations”. American Society of Engineers (ASCE) – International Foundation Congress and Equipment Expo (IFCEE) 2018. March 5–10, 2018 Orlando, Florida.
2. Eyo, E.U. Ng'ambi, S. Abbey, S.J. “A Study to develop a solution to predict the volume change behaviour of weak clayey soils improved by the soil mixing technique”. EEC Postgraduate Research Symposium, Coventry University. 5 June 2019
3. Eyo, E.U. “Developing a Solution for the Prediction of the Behaviour of Expansive Soils”. Built and Natural Environment Lunchtime Research Seminar, Coventry University. 24, April 2019
4. “Urbanization and Infrastructure Development-Future Challenges” Workshop. DFI-EFFC International Conference: Deep Foundations and Ground Improvement, Rome Italy. 6 – 8 June 2018.
5. Eyo, E.U. “Improvement of Weak Soils by The Soil Mixing Technique”. EEC Faculty Research Seminar, Coventry University. 22 February 2017.

ABSTRACT

Traditionally, Ordinary Portland Cement (OPC) has been adopted as a binding agent over the past six decades to improve the engineering qualities of soft soils. However, the significant negative environmental impacts associated with their usage besides their potential to cause sulphate heaving of stabilised soils is a global concern. Therefore, this research was concerned with expansive clay stabilisation by partial substitution of OPC and incorporation of minimal quantities of a nanotechnology-produced additive called RoadCEM.

Industrial mix specification guidelines recommend usage of very minimal quantities (1-2% by weight of cement) of RoadCEM to improve the hydration property of OPC in soil stabilisation. However, in order to further reduce the larger OPC proportion that can be potentially used, this research utilised GGBS as a complementary cementitious and environmentally-friendly material while maintaining the industry recommendation of 1% of RoadCEM in the binder mixture. Hence, an investigation into the firmly established sustainability credentials of RC vis-à-vis its potential impact on mostly the volume change and hydromechanical behaviour of stabilised soils formed the major thrust of this study.

Preliminary studies carried out on five expansive soils (having differing Na-montmorillonite contents), enabled two model soils of extreme plastic properties to be adopted for the application of the stabilising agents with the incorporated RoadCEM. OPC proportion in the binder mixtures was reduced by up to 50% and evaluation of the effect of RoadCEM on the geotechnical properties of the stabilised soils was performed. Index property and engineering tests (swelling, consolidation, strength and suction) were employed to study the geotechnical characteristics of both the natural and stabilised soils. Microscopic examination adopting techniques such as SEM, EDS and XRD were also carried out to observe the mechanisms of change in the stabilised samples.

Results of preliminary studies involving an application of the particle size analyser (Mastersizer) and a grain size statistics programme (GRADISTAT) confirmed the presence of clay-sized samples as the major cause of expansive soil swelling within a 95% confidence interval. However, the variability in the engineering properties of the clays when stabilised by OPC was minimal and did not correspond to the percentage proportion of the clay-sized Na-montmorillonite particles present in the clays. Addition of RC to the soil-binder mixture reduced soil swelling to zero with 50% OPC replacement. Furthermore, 50% of OPC substituted by the by-product possessing RC led to more decreased settlement as compared to the stabilised soil without RC inclusion. RC additive caused a reduction in compression index, Cc of approximately 80% in 7 days but with an even greater reduction of about 90% occurring at 28 days of curing. Results of higher water soil moisture retention capacity where exhibited by RC-modified soils and are promising for contaminant encapsulation especially in dredging activities. The most short-term strength development observed with the use of RC was attributed to 40% of the OPC reduced in the soil-binder mixture over 28 days curing duration. Results also indicated the efficacy of RC in reversing sulphate heaving as compared to the stabilised mix containing only the cementitious products. The evolution of extended crystallisation in the hydration mechanism of the stabilised products with the incorporated RC as revealed by SEM indicated a “wrapping” (matrix of interlocking filaments) structure – a phenomenon which is responsible for the improved mechanical properties of the stabilised soils.

Keywords: RoadCEM, cement, ground granulated blast furnace slag, soil stabilisation, sulphate-bearing soil, swelling, unconfined compressive strength, compression index, soil water characteristic curve.

DEDICATION

I hereby dedicate this thesis most especially to God almighty who granted me the grace through His son Jesus Christ to carry out the research contained herein. I also specially dedicate this thesis to my dear wife, Teim and my lovely son, Jovan for their love and patience throughout the period of my PhD.

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LIST OF ACRONYMS, SYMBOLS AND ABBREVIATIONS

A	Activity (or Modified Activity)
ACI	American Concrete Institute
AEV	Air Entry Value
AFm	Alumina-ferric oxide-monosulphate
AFt	Alumina-ferric oxide-trisulphate
Al_2O_3	Aluminium Oxide
ASTM	American Society for Testing and Materials
BPA	By-product additives
BS	British Standard
C_2S	Di-calcium silicate
C_3A	Tri-calcium aluminate
C_3S	Tri-calcium silicate
C_4AF	Tetra-calcium Ferro-aluminate
C-A-H	Calcium Aluminate Hydrate
CaO	Calcium Oxide
C-A-S-H	Calcium Aluminosilicate
Cc	Compression index
C-S-H	Calcium-Silicate-Hydrate
DDL	Diffuse Double Layer
EDS	Energy Dispersive Spectroscopy
EGME	Ethylene Glycol Monoethyl Ether
EHT	Extra High Tension
Fe_2O_3	Iron Oxide
FX	Fredlund and Xing model
GGBS	Ground Granulated Blast Furnace Slag
GRADISTAT	Grain Size Distribution and Statistics
HS2	High Speed 2 rail project
LL	Liquid Limit
LOI	Loss on Ignition
LVDT	Linear Variable Differential Transducer
MCR	Mean Clay Ratio

MDD	Maximum Dry Density
MgO	Magnesium Oxide
MIP	Mercury Intrusion Porosimetry
N_2BET	Nitrogen-based Brunauer-Emmett Teller
OCMA	Oil Companies Material Association
OMC	Optimum Moisture Content
OPC	Ordinary Portland Cement
PFA	Pulverised Fuel Ash
pH	Measure of Hydrogen or Hydroxyl ion concentration of a solution
PI	Plasticity Index
PL	Plasticity limit
PVR	Potential Vertical Rise
RC	RoadCEM
RSV	Residual Suction Value
SEM	Scanning Electron Micrograph
SiO_2	Silicon Oxide
SMT	Soil Mixing Technology
SO_3	Sulphur Trioxide
SSA	Specific Surface Area
SWCC	Soil Water Characteristic Curve
SWRC	Soil Water retention Curve
TPA	Total Pore Surface Area
TSA	Total Surface Area
TSAR	Total Surface Area Ratio
UCS	Unconfined Compressive Strength
USCS	Universal Soil Classification System
VG	Van Genutchen model
WD	Working Distance
WEV	Water Entry Value
XRD	X-Ray Diffraction
XRF	X-ray Fluorescence

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Expansive soils have continued to throw up vital challenge to geotechnical engineers and soil mechanics practitioners due to their undesirable volume change (swelling and shrinkage) characteristic upon exposure to frequent climatic variations and subsequent groundwater fluctuation (Sharma and Sivapullaiah 2016b). The global spread of these soils has made them practically unavoidable especially in areas where their deposits are very extensive (Moses and Saminu 2012). The swell – shrink behaviour of these soils is often experienced in the active zone near the ground surface above the groundwater level. Consequently, these soils tend to lift structural loads or make infrastructures undergo differential settlements causing severe damage in the process with attendant economic implications (Punthutaecha et al. 2006).

1.2 RESEARCH MOTIVATION

1.2.1 Expansive Soils

The attendant disasters and resultant estimated cost of repair and rehabilitation of structures due to construction on expansive soils is indeed a global problem. Financial losses and insurance cover for the problems caused by expansive soils have been reported in literature (Nelson and Miller 1992). In the UK for instance, following the recognition of the problems of expansive soils in the dry summer of 1947, the cost of damage in the UK has risen continuously up to the present times. The cost due to the adverse effects of shrink-swell behaviour of expansive soils has been reported to amount to about £3 billion, thus, rendering it the most damaging geo-hazard in Britain today (Jones and Jefferson 2015). It has been estimated by The

Association of British Insurers that the average cost of expansive soils – related subsidence to the insurance industry amounts to over £400 million yearly (Driscoll & Crilly, 2000). Motivated by these unfortunate antecedents and circumstances, this research aims to provide and recommend a solution through the soil mixing technology by developing and using novel binder proportions to improve the mechanical properties of expansive soils.. Moreover, the outcome of this study shall also be of great interest to the proposed multi-billion sterling pound High Speed (HS2) rail project in the United Kingdom whose intended route shall pass through areas of the country with challenging expansive soil geology.

1.2.2 Soil Mixing Technology

The Soil Mixing technology (SMT) or the chemical treatment of soils introduced several decades ago has proven to be a very cost-effective technique amongst the potentially available methods used to improve the engineering performance of weak soils (Petry and Armstrong 1989; Ahnberg et al. 1995; Uddin et al. 1997; Bergado et al. 1999; Nalbantoglu and Tuncer 2001; Horpibulsuk et al. 2004; Al-Rawas et al. 2005; Seco et al. 2011b; Tran et al. 2014; Khemissa and Mahamedi 2014; Abbey et al. 2017; Eyo et al. 2017, 2018). SMT has gained much interest and acceptance in recent years particularly due to its wide applications in construction projects e.g. road subgrades, embankments, liquefaction mitigation, foundation structures, bridge abutments, cut slope stability, exclusion barriers, backfills (Shakri et al. 2014). In the UK/EU for instance where the uptake of the technology has been on the rise in the past two decades, the Landfill Directive, Environmental Laws and policies, taxes and the ever-increasing charges of moving excavated poor soils and/or treating them has made this research project even more imminent.

Stabilizing agents such as lime and cement have been used traditionally over the years as binders to improve the engineering qualities of soft soils. The temptation to rely solely on these energy intensive and environmentally unsustainable hydraulic binding agents to stabilised soils seems almost unavoidable owing to their good binding qualities and ease of accessibility. It is estimated that 1 tonne of cement produced could lead to about 5,000 MJ of energy consumed, 1.5 tonnes of non-renewable resources consumed and about 1 tonne of CO_2 emission (i.e. 8% of the total global CO_2 emissions) (Higgins 2007; European Commission 2010; Olivier and Peters 2018). Apart from the above-mentioned health and environmental concerns, soil-cement stabilisation could in some cases cause the growth of ettringite which is a deleterious expansive mineral (Rao et al. 2008; Verástegui-Flores and Di Emidio 2014).

Developments in knowledge and research are currently shifting from an over-dependence on cement and lime to the production and usage of waste materials, industrial by-products, organics, polymers, etc. in engineering applications (Obuzor et al. 2011; Celik and Nalbantoglu 2013; Ganjian et al. 2015; Al-Swaidani et al. 2016; Sharma and Sivapullaiah 2016b; Behnood 2018). One example of industrial by-products considered in ground improvement works is ground granulated blast furnace slag (GGBS). GGBS is desirable in soil stabilisation projects not only because of its pozzolanic effects but also because they are cost effective, energy saving and environmentally friendly (Wild et al. 1999; Higgins 2005, 2007; Mohamad et al. 2016; Ghadir and Ranjbar 2018).

1.3 STATEMENT OF NOVELTY

Substitute material such as ground granulated blast furnace slag (GGBS) (a by-product of steel industry) can only partially reduce the need for Portland cement (about 30% reduction) in soil stabilisation (Allwood et al. 2019).

This research proposes that the engineering properties of stabilised soils achieved by the partial replacement of cement with an industrial by-product such as GGBS could be further enhanced by incorporating minimal quantities of a nanotechnology-based additive called “RoadCem

(RC)". RC is a fine-grained industrial additive blend that is composed of synthetic zeolites, alkali earth metals and complementary complex activator that enhances its unique properties. RC has been tested and found to possess excellent environmental credentials and great economic prospects (Montero et al. 2012; Blass 2017). RC has also been utilised worldwide but mostly on small to medium-scale industrial projects. For instance, in the UK, RC has found applications in stabilising topsoil meant for parking lots, temporary low volume access roads, working platforms for cranes, pilling mats in projects such as:

i. University Science and Enterprise Park, Loughborough
ii. A244 Walton Road Bridge, Surrey
iii. Maritime Headquarters, Felixstowe, Suffolk
iv. Anglian Water Wolfterton Pumping station, Norfolk.
v. Claymills Severn Trent Sewage Treatment Works (STW), Burton-on-Trent

The above-mentioned projects are those in which volume change deformations and certainly an understanding of long-term effects was not an issue. Moreover, documented studies on the application of RC have only focused on the improvement of mainly mechanical properties such as the strength and stiffness of the stabilised soil (Ventura and Koloane 2005; Marjanovic et al. 2009; Mutepfa 2010; Ouf 2012; Faux 2015; Wu 2015). In response to the gap in research, the present study therefore intends to concentrate more on the volume change (swell and consolidation) properties and the moisture retention characteristics of sensitive expansive clays stabilised by inclusion of RC in traditional cementitious binders. Furthermore, this research is of great interest and importance to large scale projects such as the recently proposed multi-billion sterling pound High Speed (HS2) rail and certainly the expansion of the Heathrow airport runway whose intended routes shall pass through areas of the United Kingdom that possess challenging expansive soil geology.

1.4 RESEARCH AIM

This research aims to investigate and establish the credentials of RoadCem (RC) by an assessment of the effects of its inclusion on the engineering characteristics and hydromechanical properties of expansive soils stabilised by cementitious materials.

1.5 RESEARCH OBJECTIVES

In order to meet the aim and purpose of this study the following objectives shall be followed:

i. Identification and evaluation of natural expansive soil behaviour by developing a comprehensive volume change concept that incorporates particle size distribution and pore morphology as intrinsic properties in swell determination.

ii. Establishment of the engineering properties of expansive soils stabilised by using cement alone as a traditional binding agent which would serve as a frame of reference for further stabilisation with the cement substituted.

iii. Assessment and evaluation of the volume change, strength and water retention characteristics of expansive soils stabilised by a novel binder mixture and proportion that incorporates RC in cementitious products.

iv. Performance of a validation study on the optimal novel binder mixture by investigating its effect on an expansive soil exposed to sulphate attack.

1.6 RESEARCH SCOPE

This study is concerned mainly with the assessment of the geotechnical characteristics and performance of an expansive soil stabilised using the Soil Mixing Technology (SMT). In this regard, emphasis will be laid on the development of binders and proportions including an extensive comparative analysis of these binders used in soil stabilisation. The expansive soils used in this research are laboratory blended and remoulded natural clays. The binder selection intended for this project for the expansive soil improvement are sourced from the market and relevant industries. Standard and novel laboratory procedures and methods of mixing both soil and binders, curing under different conditions and testing shall be followed for the determination of the volume change movement, strength and hydromechanical behaviour of both the weak soil and the treated soil samples. Even though this research recognizes the need for mixing equipment capability, process of mixing, skills used by contractors and accurate geometric layout used in sound in – situ soil stability scheme (Madhyannapu et al. 2010), it may not however provide some of the detailed specifications for field logistics and implementation. Nevertheless, this research shall provide an investigation which will explain most of the important factors affecting the stabilised soil, establish reasonable mixing proportions and suggest ideas and direction for site application.

1.7 THESIS STRUCTURE

This research project is organised into the following proposed chapters:

- **Chapter 1** introduces the research with a background to aid an understanding of the subject of expansive soils and the need for their improvement. The motivation and problem statement as well as some key questions underpinning the challenges with expansive soils and the use of SMT in research are also mentioned followed by the aims and objectives of the study. Next, the scope and delimitation of the research with regards the concept of SMT are covered in this chapter.
- **Chapter 2** discusses the concept surrounding expansive soils including, its behaviour, mineralogy structure while also providing the historical background of this soil type. Next, the solution to the issues and problems of expansive soils by stabilisation are reviewed. Soil stabilisation by using cement as a traditional binder are then mentioned

in detail and includes first, a discussion on cement production, its composition and secondly, some of the engineering properties of expansive soils stabilised using cement. The section on cement soil stabilisation then concludes with a discussion on the negative impacts of its usage in soil stabilisation. A brief review of the advances made in developing environmentally friendly binders for soil stabilisation are carried out in the sections following. Next, a consideration of RC as a partial cement replacement in soil stabilisation are discussed and this includes the origin of RC as an additive, application of RC, chemistry and mechanism of RC in stabilised soils and the economic benefits of using RC in civil engineering construction. This chapter then ends with an overview of GGBS as another environmentally friendly binder for soil stabilisation.

- **Chapter 3** proposes to deal with the methodology of this research. Here, different methods and procedures such as Identification Tests, swelling, Consolidation, strength and suction tests shall be investigated and conducted to collect data for the natural soils and the soils mixed with different binders in various proportions. Microscopic examination adopting techniques such as SEM, EDS and XRD are also performed to observe the mechanisms of change in the samples.
- **Chapters 4** provides the discussion of experimental results with focus on the classification of the natural soil’s properties as measured in the laboratory. An evaluation and prediction of the swelling behaviour of soils are also carried out followed by an analysis of the consolidation and strength characteristics of the natural soils.
- **Chapter 5** is dedicated to the performance evaluation of engineering properties of the clays stabilised by OPC used as a sole binder. An identification of the properties of the OPC used are first carried out followed by a discussion to establish the quantity of OPC required to stabilise the clays according to standard recommended practices.

- **Chapter 6** provides analyses and discussions based of the engineering performance of the clays by incorporation of RC in soil stabilisation. Finally, several microstructural features derived from the scanning electron micrographs (SEM) are further used to explain the mechanism of reactions of the stabilised soils.
- **Chapter 7** presents further investigations and discussions to authenticate and validate the use of RC in the treatment of soils exposed to harsh chemical conditions. By so doing, an application of the most effective binder mixes with RC inclusion is used for the stabilisation of a soil that is prone to attacks by sulphates.
- **Chapter 8** concludes the study by providing a summary of major findings and highlights of some of the important achievements realised on the engineering properties of soils stabilised by using RC. Finally, suggestions and recommendations for future work are made on soil stabilisation using RC.

1.8 SUMMARY

An overview of the problems of using expansive soils as foundation of structures and a correspond solution to these challenges by soil stabilisation are made in this chapter. This chapter also provides highlights of some of the attendant problems surrounding the use of cement as a traditional binder in soil stabilisation. The novelty of this study lies in providing a solution to the use of cement alone as a binder in soil stabilisation by proposing its partial replacement with a nanotechnology-based industrially-developed additive blend called “RoadCem (RC)”. In order to meet this aim of the research, an outline of the objectives to be followed were provided and involved first, a classification, evaluation and prediction of natural expansive soil behaviour by providing a comprehensive volume change notion based on intrinsic properties. Next, an evaluation of the engineering characteristics of expansive soils stabilised by a novel binder mixture and proportion that incorporates RC in cementitious products are mentioned followed by a validation study on the optimal novel binder mixture. Finally, the scope of the research was stated, and the chapter arrangement of the thesis are also included.

CHAPTER 2

LITERATURE REVIEW

2.1 EXPANSIVE SOIL BEHAVIOURAL CHARACTERISTICS AND CHEMISTRY

Expansive soils are regarded as soils that are capable of experiencing considerable volume change (swelling, shrinkage and settlement). This behaviour is mostly associated with changes in water contents or moisture variation (Jones and Jefferson 2015). The nature of these soils can be examined at the microscale, macroscale and external levels. This is because the mechanism of expansion or swelling in clays is complex and is influenced by both the soil property factors and environmental conditions. Expansion is the disturbance in the internal stress equilibrium made possible by changes in the soil water system. Clay particles generally are considered as platelets possessing negative electrical charges on their surfaces and positively charged edges as shown in Figure. 2.1.

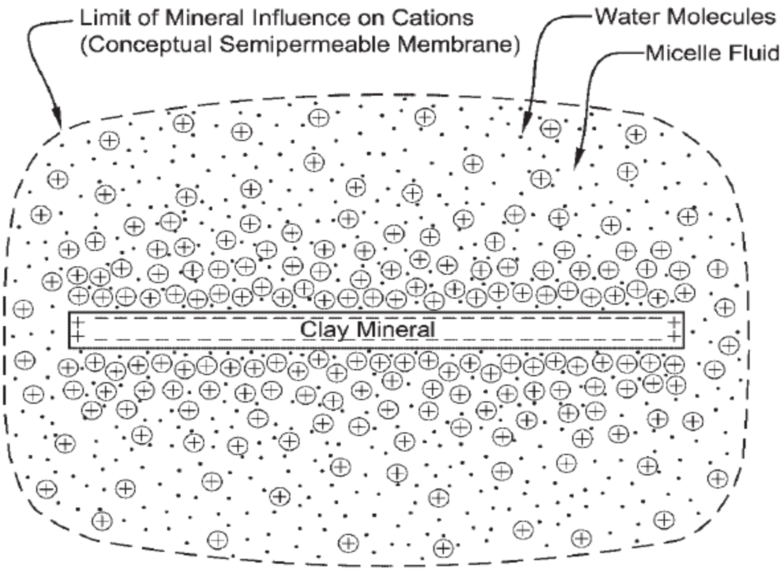


Figure 2.1 Clay soil platelet (modified after Nelson et al. (2015))

The negative charges are balanced by cations in the soil water system that become attracted to the surfaces of the platelets by electrical forces such as van der Waals and adsorptive forces.

These forces also have an influence on the inter-particle force field. The internal electrochemical attractive force fields must be maintained in equilibrium with the externally applied stresses and capillary force or tension (matric suction). When this equilibrium is disturbed due perhaps to changes in the water chemistry, a change in the inter-particle force field will also be triggered. If the resulting change in internal electrochemical forces is not properly balanced or counteracted by a corresponding change in the externally applied stress, the particle packing or spacing will tend to change so as to adjust the inter-particle forces until an equilibrium is established. This change in particle geometry or spacing is what then manifests as shrinkage or swelling (Nelson and Miller 1992).

It is very clear at this point to suggest that in discussing the nature of clays, three components must be considered – the cations, the minerals and the associated water or water of hydration. Nelson et al. (2015) used the term *clay micelle* to refer to these three components acting together as a unit. The *micelle* model for a clay mineral can be viewed as negatively charged internal mineral core that is surrounded by positively charged cations acting to neutralize the mineral charge as already depicted in Figure 2.1.

2.2 EXPANSIVE CLAY MINERALOGY

Different types of clay minerals usually exhibit different capacities to swell apparently due to the variations in the electrical force field system associated with each mineral. The swelling of the entire mass of clay soil is affected particularly by the type and amount of clay minerals in the soil as well as the specific surface area and arrangement of particles that make up the clay soil (Nelson and Miller 1992). The majority of clay minerals do have some forms of an orderly arrangement of the atoms that ultimately form crystal lattices, and these can be identified using X-ray diffraction techniques, electron microscopy and other methods of chemical analyses. Even though several recognizable minerals in existence are mentioned in Mitchell and Soga

(2005), the three important structural groups of clay minerals (Figure 2.2) that are normally considered for engineering are as follows (Nelson et al. 2015):

2.2.1 Kaolinite

This mineral group is composed of an alternative set of silica and octahedral sheets of aluminium as seen in (Figure 2.2a) having a structural formula of $(OH)_8 Si_4 Al_4 O_{10}$ (Mitchell and Soga 2005). The van der Waals forces and hydrogen bonds are responsible for the bonding that exists between each successive layer. The variations in the mineral structure due to the stacking of the layers and the subsequent positioning of the aluminium ions in the octahedral sheet can result in the production of other members of the subgroup kaolinite such as halloysite, dickite, nacrite, etc.Generally, the particles of kaolinite are positively charged on their edges when in a low acidic environment but charged negatively in high basic environment.

2.2.2 Illite

This is the most common clay mica-like mineral in soils that is often encountered in practice. It possesses a similar structure as that those of muscovite and may be referred to sometimes as hydrous silica. It can be represented structurally as a three-layer silica-gibbsite-silica with the tips of the tetrahedra oriented towards the centre and also having octahedral sheet like-ions (Figure 2.2b) (Mitchell and Soga 2005). The electrically neutral unit cell of illite has a formula of $(OH)_4K_2 (Si_6 Al_2) Al_4 O_2$. Other examples of the mica group are vermiculite, pyrophyllite, phlogopite or mica, biotite, etc. Illite is usually known to occur as very small, flaky particles and are capable of being mixed with other non-clay materials.

2.2.3 Smectite

This mineral group consist of an octahedral sheet that is sandwiched between two pairs of silica sheets as indicated in Figure 2.2c and have a general structural formula of $(OH)_4Si_8 Al_4O_2$

n(interlayer)H_2O (Mitchell and Soga 2005). The smectite group has a relatively large specific surface area and the layers are held up by van der Waals forces and the cationic balance within its structure. Bentonite which is a clay of very high plasticity and known to contain mostly a type of smectite – montmorillonite is an obvious example of the clay in the smectite group. Hence, montmorillonite or bentonite is regarded as the most expansive and problematic of the three commonest groups of clay minerals especially when found in an environment of wide moisture variation.

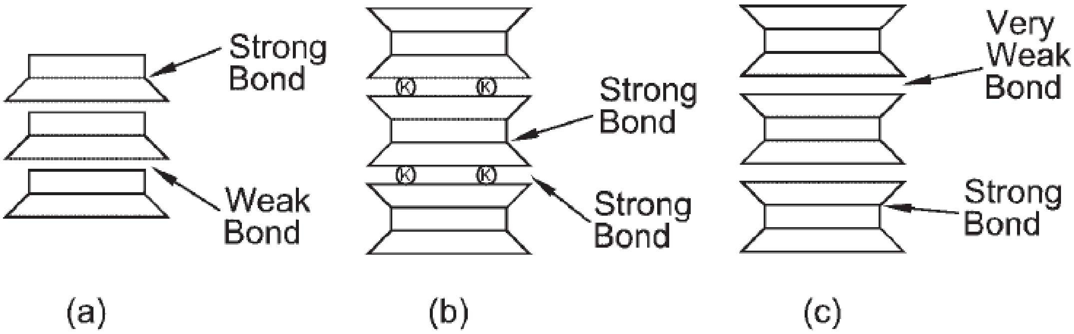


Figure 2.2 Expansive soil mineral structure: (a) Kaolinite; (b) Illite (c) Smectite (modified after Nelson et al. (2015))

2.3 STRUCTURE

The manner of soil particle interactions is influenced by spacing between particles and their orientations. The soil particles can assume various degrees of orientation depending on the history and conditions of deposition of the actual soil mass. Hence, the particles of the soil could become either dispersed or flocculated as shown in Figure 2.3. It is important to note that the structures depicted in Figure 2.3 are the extremes from wholly flocculated to wholly dispersed.

However, in most of the soils, the particle orientations would lie somewhere between the two extremes shown. For a flocculated structure, it is obvious that the interaction occurring between the micelles of the flocculated structure is influenced basically by the faces of adjacent soil particles and the end-to-end contacts between them. Moreover, the amount of spacing is much greater in the flocculated structure than in the dispersed structure. Hence, it is evident that both the crystalline and osmotic swelling (as would be noticed later) will be more effective in the dispersed structure than for the flocculated structure. Highly overconsolidated clays experiencing high amounts of overburden stresses would tend to display more dispersed structure and will swell more when inundated with water.

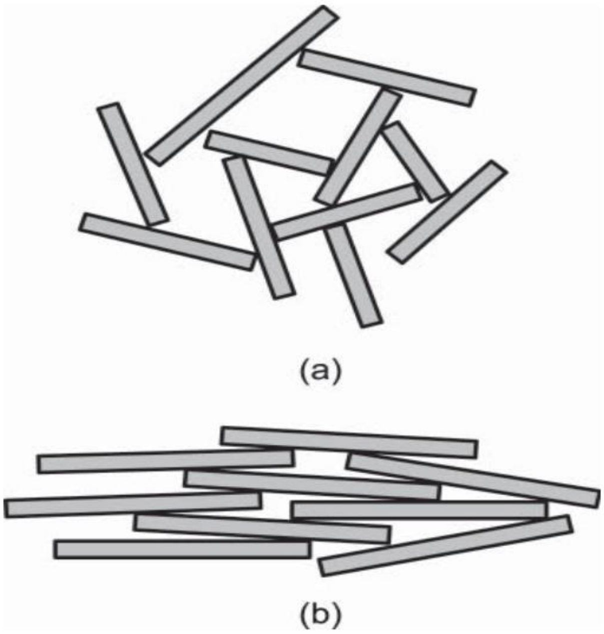


Figure 2.3 Soil sediment structure (a) flocculated structure (b) dispersed structure (modified after Lambe and Whitman (1969)).

2.4 CATIONIC HYDRATION

The clay soil water system and chemistry are very crucial as far as the determination of the potential for clays to swell is concerned. Exchangeable cations such as sodium, magnesium, calcium and potassium present on the clay surfaces are required to balance the negative electrical surface charges in the clay – water system. In case of dry soils, the adherence of the cations to the clay crystal surfaces is due to strong electrostatic forces. When water becomes available, the energies of cation hydration are adequately large to overcome the inter-particle forces of attraction. This is indicated by the tendency of the ions to diffuse away from the particle surfaces. The electrostatic forces then tend to counter the diffusion process to some degree, and this results in a large concentration of the cations close to the particle surface. A diffuse double layer (DDL) then forms comprising the concentration of positive ions in solution near the particle and negatively charged clay particle surface. As the thickness of the overlapping DDL increases, swelling tends to increase as well. According to Mitchell (1976), a thicker DDL and hence greater swelling are associated with cations with low valence and or lower cation concentrations. Thus, more swelling will likely take place in a sample having monovalent exchangeable ions like sodium (Na^+) than in those with divalent ions like magnesium (Mg^{2+}) or calcium (Ca^{2+}) in soils of same mineralogy. Also, the phenomenon of leaching of salt from the clay pore fluid will encourage more swelling (Nelson and Miller 1992).

2.5 CRYSTALLINE AND OSMOTIC EXPANSION

The expansion of the montmorillonite clays can be used to explain the nature and concept of swelling in most expansive clays (Nelson et al. 2015). It is suggested that the swelling in montmorillonite -rich clays occurs in two prominent manners (Norrish 1954). At the spacing of about 22 Å, the swelling could be regarded as being dependent on the cationic exchanges and the amount of water that is taken up in relation to the cationic energy of hydration. Whereas at a spacing that is greater than about 35 Å, the montmorillonite is thought to form the rest of the clay micelle fluid and the expansion becomes essentially an osmotic swelling. Given that the hydration energy is high, the initial penetration of water into the soil will be used for hydration and swelling at this stage is referred to as ‘crystalline’ swelling. Beyond this phase, the cationic concentration and movements between the particles of the soils will allow more water to be sucked into the interparticle spaces to force field differentials resulting in what is known as ‘osmotic’ swelling.

2.6 EXPANSIVE SOILS IN UNSATURATED SOIL MECHANICS FRAMEWORK

Several unsaturated soil theories and models do consider as their basis the Terzaghi's principle especially as applied to the saturated soils (Alonso et al. 1999; Wheeler et al. 2003; Sheng et al. 2008; Fityus and Buzzi 2009). Hence, provided the effective stress principle does hold true for expansive clays, the Terzaghi’s principle could be relied upon to describe the behaviour in the saturation range by a transference of the suction to an equivalent net stress when the degree of saturation does reach the value of one. However, literatures have regarded the electrochemical forces (diffuse double layer mentioned previously) existing in expansive clays as being very significant components of the stress equilibrium such that the effective stress principle which does not consider them or account for them may not always remain true for expansive soils (Sridharan and Venkatappa Rao 1973; Hueckel 1992; Fityus and Buzzi 2009). This also invariably means that the translation of the suction change within the range of saturation to an equivalent net stress or change in mean stress cannot be simulated given that the soil response could be different.

Even though the mechanisms of soil expansion attributed to the double layer theory has been sufficiently established in describing the mechanism of volume change in expansive soils, most researchers have tended to introduce counter evidences through experiments and relevant empirical correlations by incorporating other variables which are considered to have significant bearing on the expansive soil swelling (Alonso et al. 1999; Punthutaecha et al. 2006; Erzin and Erol 2007; Castellanos et al. 2008; Villar and Lloret 2008; Likos and Wayllace 2010; Chittoori and Puppala 2011; Pedarla et al. 2016; Puppala et al. 2016). For instance, Gens and Alonso (1992) have included factors such as the effect of ion size, anion adsorption, interlayer water potential modification, etc which could be regarded as having a direct influence on soil suction. It is pertinent to mention that most of these studies carried out to evaluate the volume change mechanisms of expansive soil (either directly or indirectly), involves very time consuming, expensive and cumbersome experimental procedures and techniques such as mercury intrusion porosimetry (MIP), suction measurements, cation exchange capacity (CEC), chemical ethylene glycol monoethyl ether (EGME), scanning electron micrograph (SEM) and other chemical analyses to quantify expansive soil swelling. For instance, Erzin and Erol (2007) attempted the prediction of swelling of statically compacted mixtures of Na-bentonite and kaolinite from measurements of suction using the thermocouple psychrometer technique. Multiple regression approach used to correlate soil suction with soil properties such as plasticity index, water content, dry density, specific surface and cation exchange capacity yielded a high coefficient of determination. Puppala et al. (2006) developed a soil-suction framework for the determination of swell properties of expansive soils based on matric suction and clay mineralogy. Their proposed framework for the tested soils highlighted swell behaviour in terms of mechanical, hydro- and chemical response of the soils and depended on methods such as one-dimensions and three-dimensional swell-strain tests, the filter paper method and the pressure cell using axis translation technique for suction measurement, CEC and total potassium tests for measurements of chemical and particulate properties. In an attempt to to correlate pore size in combination with other relevant soil properties for an evaluation of expansive clays swell properties, Pedarla et al. (2016b) developed a model by consideration of soil-pore void distribution and surface area properties (mineralogy). The Internal soil pores and their distribution were determined from studies of mercury intrusion porosimetry (MIP) while the Specific surface area (SSA) information was obtained from the method utilizing chemical ethylene glycol monoethyl ether (EGME). Their proposed model and predictions illustrated the importance of mineralogy and pore sizes. An interesting conclusion from the forgoing suggests that the proposed theories incorporating some of the factors mentioned tend to render the micro- structural description of the volume change behavioural framework of expansive soils open-ended and somewhat irresolute. Secondly, methods and approaches to rationally analyse and evaluate volume changes in expansive soils that in most parts can circumvent some of the stressful experimental work proposed in literature are needed. Lastly, it is also evident from previous researches that a comprehensive volume change concept to enable further understanding of swell behaviour of expansive soils by incorporating the effect of clay particle sizes in relation to specific surface area (SSA) soil activity and pore voids has not been considered.

2.7 EXPANSIVE CLAY STABILISATION

In developing solutions for the problem of expansive soils, novelty alternatives have been proposed over the past four decades to minimize the potential distress that can be caused to infrastructures by these weak soils (Das and Parhi 2013). One of such method of improvement is the soil stabilisation by the use of hydraulic binders, novel by- products and wastes (Zha et al. 2008). Chemical stabilisation of expansive soil as a method of improvement has been a subject of interest and discussion for many decades due to its advantages and potential for reducing construction expenses and maintenance costs of building on difficult soils (Shakri et al. 2014). More so, the reduction of the swelling potential of expansive soils by using chemical additives or binders either in isolation or in combination have been extensively covered in literature (Rao et al. 2001; Al-Rawas et al. 2005; Seco et al. 2011b; Khemissa and Mahamedi 2014; Abbey et al. 2016; Kumar and Devendra 2016; Pujar and Sudeep 2016; Wang 2016; Dayioglu et al. 2017).

2.8 HISTORICAL BACKGROUND

Engineer’s successes in the modification of the behaviour of clay were recorded in the mid-1950s. The control of heavy clays by the compaction method was described by Dubose (1955) (quoted in Petry and Little (2002)) while Jones (1958) studied the stabilisation of expansive clay by lime and cement. In the late 1950s, the mechanism of ion exchange in clays was explained by Taylor (1959). McDowell (1959) (quoted in (Petry and Little 2002) explained the methodology behind the stabilisation of soils from Texas, US using lime and fly ash. A period of advancement in the technology of soil mixing was heralded when Hilt and Davidson (1960) described the lime fixation point in clays. The influence of mellowing time on the properties of clays modified by lime and how this affected workability and compaction response was described by Mitchell and Hooper (1961). Eades and Grim (1960) carried out a quick test (1 – hour) to determine the optimum lime content needed for stabilisation based on pH. They also identified the products of soil – lime reaction and presented compelling evidence showing how soil mineralogy relates to amounts of lime needed for stabilisation. An important evidence indicating the curing of soils stabilised by lime to allow for the proper development of pozzolanic reaction which is crucial for strength gain and long-term stability was studied by Anday (1961).

Several other published works and technical literature covering important developments in the stabilisation of expansive clays from the 1970s until the turn of the millennium are outlined in Petry and Little (2002); Ikeagwuani and Nwonu (2019).

2.9 STABILISATION WITH CEMENT

Cement has been one of the most frequently used chemical hydraulic binders after lime in soil stabilisation. Several kinds of cement have been adopted for the stabilisation of soft soils however, the Ordinary Portland Cement (OPC) or Silicate cement remains the most common cement type that is widely utilised for the purpose of improvement of various aspects of the soft soil. The stabilisation of expansive soils by OPC with the aim of evaluation of its engineering behaviour is documented in literature (Erdal 2001; Consoli et al. 2010; Sivapullaiah and Lakshmikantha 2010; Abdelkader et al. 2013; Raftari et al. 2014; Iravanian and Bilsel 2016; Jamsawang et al. 2017; Por et al. 2017).

2.9.1 OPC Production and Composition

OPC is generally produced by combining gypsum with cement clinker and grinding the product to powder (Bitir and Muşat 2014). The clinker is produced from the constant firing of clay and mixture of limestone at a very high temperature (up to 1,500^{0}C). This process is then followed by the addition of gypsum to the clinker and the mixture ground until a very fine powder is obtained. In general, the chemical composition of OPC consists essentially of alumina, silica, calcium aluminoferrite along with several other oxides. Moreover, the standard of production and the corresponding properties of OPC are given by the British standard BS EN 197-1:2000 and specifies that at least one-third of the mass of OPC clinker shall consist of calcium silicate (C_3S and C_2S) including some other oxides such as alumina (Al_2O_3) and iron oxide (Fe_2O_3). Other specifications of the British standard BS EN 197-1:2000 are that the ratio of calcium oxide (CaO) and silicates (SiO_2) shall be equal to but not greater than 2; and that the magnesium oxide (MgO) shall not be more than 5%. Table 2.1 presents some of the typical compounds of OPC and their ranges in percentage.

Table 2.1 Typical chemical composition of OPC

Oxide	Percent (%)	Mean (%)
Lime (CaO)	59 – 67	64
Silica (SiO_2)	17 – 25	21
Alumina (Al_2O_3)	3 – 9	7

Iron Oxide (Fe_2O_3)	5 – 6	3
Magnesia (MgO)	0.1 – 4	2
Sulphur trioxide (SO_3)	1 – 3	2

2.9.2 Aspects of the Mechanism of OPC Stabilised Clays

The addition of cement to a clay soil system that contains some sufficient amount of water will result in complex sets of reactions and the formation of compounds in a process that includes fundamentally, exchange of cations, flocculation/agglomeration, hydration and pozzolanic reactions. In some instance, carbonation reactions may occur in the process depending on the nature of the soil used. The exchange of ions and flocculation/agglomeration reaction occur just after mixing the soil and cement and can take a few minutes or hours to complete. The hydration reaction occurs within a few weeks up to about one month after mixing meanwhile pozzolanic reactions can take place over a very long duration extending from one month to years.

2.9.2.1 Cation exchange

This process occurs immediately after the mixing of the cement and soil materials and involves the exchange of the released divalent ions (Ca^{2+}) from the cement and the metallic ions (Na^+ and Al^{3+}) from the clay soil lattice (Rogers et al. 1996). Some of the initial physical changes that occur in the soil-binder are as a result of the ionic exchanges. One such change is the decrease in the electrical double diffused layer of adsorbed water that surrounds the particles of the clay. The corresponding orientation of the clay particles dictates the size of the double layer, for instance, a parallel arrangement will encourage the formation of larger envelops as compared to the edge-to-face orientation. A decrease in the thickness of the double layer suggests that the particle of the clay does prevents penetration by water. The next stage that involves flocculation then leads to a reduced thickness of the layer of adsorbed water (Figure 2.4).

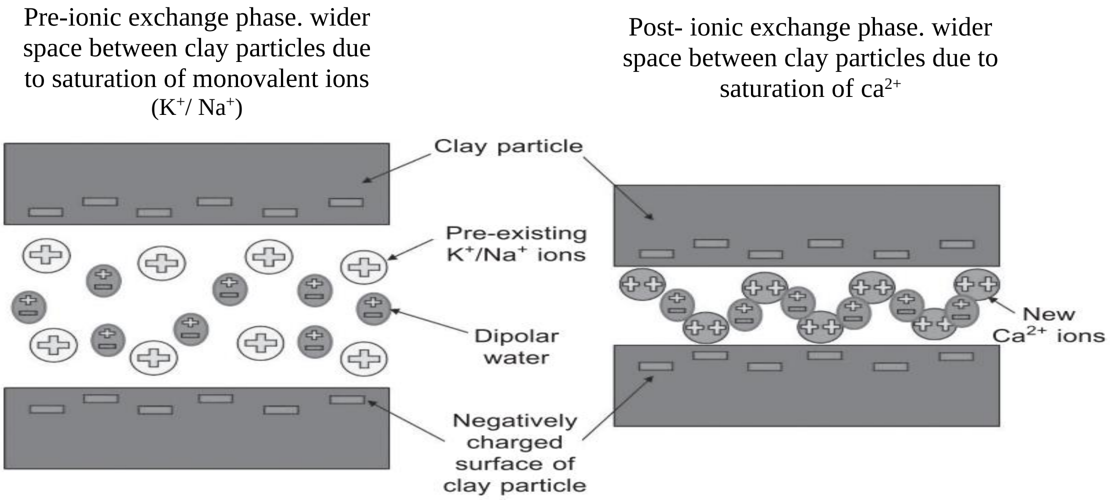


Figure 2.4 A demonstration of cation exchange in soils due to addition of cement (modified after Sargent (2015))

2.9.2.2 Flocculation/agglomeration

This process occurs post-mixing and involves a restructuring of most of the negatively charged particle of the clay which are themselves surrounded by a shell of positively charged particles which could increase in size depending on the level or amount of the charges. Initially, the particles of the clay are dispersed and appear in parallel arrangements as a result of the repulsion between the negatively charged surface and the surrounding positively charged shells (Figure 2.5a). This force of repulsion may be broken by van der Waals forces and cause the charged particle to move towards each other or flocculate to produce a new edge-to-face arrangement (Figure 2.5b). These changes would lead to numerous strength and textural alterations in the soil namely; a change from a highly plastic clay to clay of lower plasticity that is characterised by a more granular and friable soil. The angle of friction will also increase between the flocs/agglomerate particles leading to the increase in shear strength property (Rogers et al. 1996).

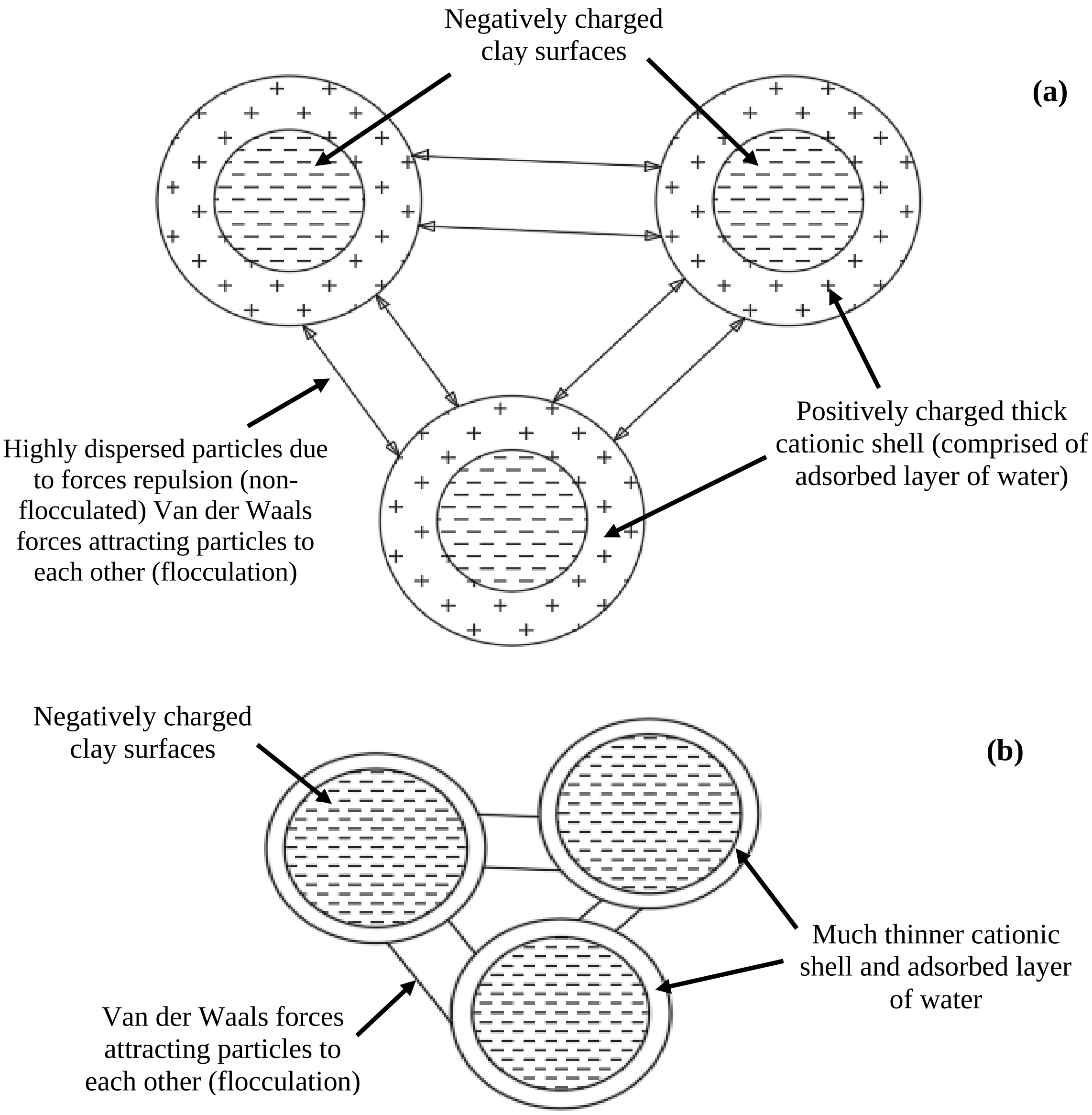


Figure 2.5 A demonstration of flocculation due to treatment by cement (a) before flocculation (b) flocculation (modified after Sargent (2015))

2.9.2.3 Hydration

This stage of soil-cement reaction proceeds with the evolution of large heat evolution in an exothermic reaction as time progresses. Mixing of the soil with the cement causes the water in the soil to reduce since the water is being consumed during the hydration process and this occurs in several key phases. Firstly, the cement is dissolved and is followed by an increase in the level of ion concentrations. Once the concentration is sufficiently high, compounds will develop from within the solution and precipitate as solid products of hydration. Some of the major strength-enhancing compounds in OPC (tricalcium silicate (C_3S), dicalcium silicate (C_2S) and tetra calcium aluminoferrite (C_4AF)) would hydrate and produce cementitious products namely; aluminates (C_3AHx and C_4AHx), hydrated calcium silicates (C_2SHx and C_3S_2Hx) and hydrated lime. It should be noted that the dissolution of hydrated lime in the existing pore water results in an increase in the pH of the soil. The particle of cement creates some bonding between themselves and soil particles resulting in the hardening of the soil-cement system.

2.9.2.4 Pozzolanic reactions

This phase in the soil-cement system can take a very long time to complete (several months and years) depending partly on the environmental condition at which the stabilised soil is left to cure. The mechanism of pozzolanic reaction involves the transfer of calcium hydroxide through the soil-water system to combine with silicate or aluminate minerals or both all occurring above a pH value of 10.5. Dissolved Ca^{2+} produced from the hydrated lime within the soil-water would react with dissolved Al_2O_3 and SiO_2 present on the surfaces of clay particles to produce calcium silicate hydrates (C-S-H), calcium aluminate hydrates (C-A-H) and calcium aluminosilicate hydrates (C-A-S-H) gels (Equations 2.1 and 2.2) the soil-cement mixtures all go on to cure and produce strong cementitious matrices of soil called ‘geopolymers’ (Sherwood 1993). It has been suggested that if the stabilised soil samples are made to cure at higher temperatures especially within 5 hours of mixing, then the rates of reaction will increase and result in further strength and volume change improvements (Palomo et al. 1999).

$Ca^{2+} + 2(OH)^{-1} + SiO_2$ = calcium silicate hydrate (CSH) (2.1)

$Ca^{2+} + 2(OH)^{-1} + Al_2O_3$ = calcium aluminate hydrate (CAH) (2.2)

2.9.3 Engineering Characteristics of OPC-Stabilised Soils

Published research literatures, manuals and handbooks have provided recommended guidelines as to the optimum quantity of cement content needed in soil stabilisation. Albeit, it has been suggested that cement addition up to about 2 % can modify soil properties, while much larger quantities could have more considerable effect (Bell 1993). Also, cement content may range from 3 - 16 % of the soil's dry weight and depends on soil type and required properties. It was also stated elsewhere that the quantity of cement needed to stabilise expansive soils could range from 2-6% by dry weight of the soil (Chen 1975). According to the Portland Cement Association, the higher the soil plasticity, the greater the quantity of cement to be used (PCA 1992). American association of state highway transport official (AASHTO) cement requirements by dry weight of soils of high plasticity ranges from 9-15%. The US army Corp of engineers have recommended a range of 7-20% of cement by dry weight of a silty or clayey soil. The seeming lack of a unified standard as to the quantity of cement required demonstrates that stabilisation with cement depends on the aim of application.

The following sections presents an overview of some recent studies conducted to evaluate the basic characteristics and engineering behaviour of stabilised expansive soils by utilising different cement proportions.

2.9.3.1 Atterberg limits

Depending on the condition of testing and the operator involved, slightly varying outcomes in the consistency of soils modified by OPC have been reported. For instance, the plastic limit and liquid limit have been found to increase with OPC even though the greater increase was attributed to the plastic limit and which ultimately led to a reduction in the plasticity index of the soil. The plasticity index of clays is also expected to reduce as the amount of OPC and sometimes the period of short-term curing observed increases as reported in Sariosseiri and Muhunthan (2009); Goodarzi and Salimi (2015). A typical variation in the Atterberg limits of an expansive clay that is modified by various quantities of OPC is given in Figure 2.6 (Abbey et al. 2016).

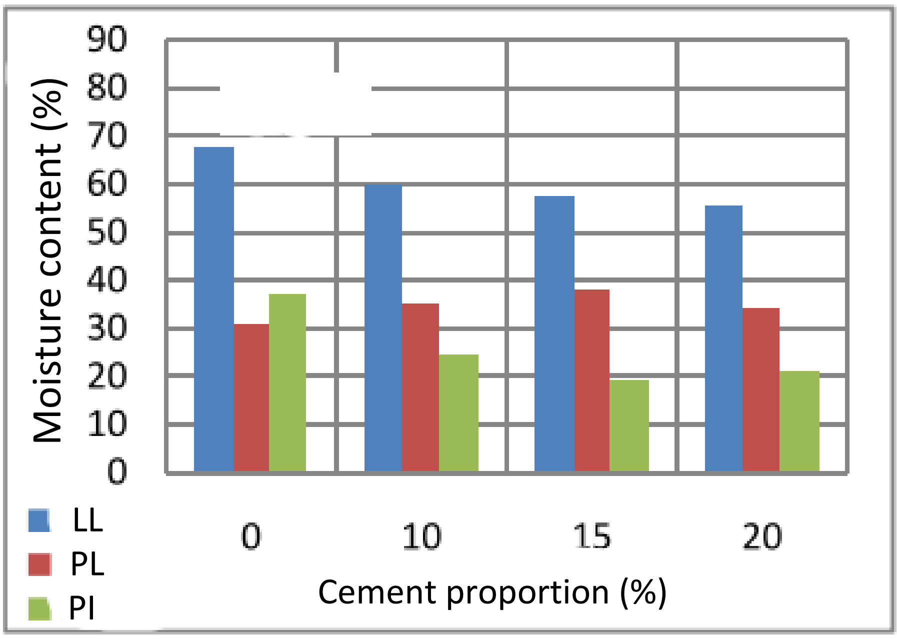


Figure 2.6 Consistency of expansive soil treated by cement (modified after Abbey et al. (2016))

As mentioned previously, the shorter-term reactions, especially during the initial processes of OPC hydration, cationic exchange and possible flocculation/agglomeration reactions are what enables a change in the soil's consistency thus making it more workable.

Nevertheless, some researchers have observed a decrease in liquid limit just immediately after the addition of OPC to the clay. They attributed this phenomenon to the depression or suppression of the electrically charged double diffuse layer of the modified soil system due to the rise in the concentrations of Ca^{2+} near the clay mineral surfaces (Kamruzzaman 2002; Goodarzi and Salimi 2015). It was noticed by Sariosseiri and Muhunthan (2009) that the liquid limit of an expansive clay had increased while the plastic limit reduced when 2.5% of OPC was added and which resulted in an increase in the plasticity index of the modified clay. However, with the increase of OPC content to 10%, they observed a significant reduction in the plasticity index due to a corresponding decrease in the liquid limit and a rise in the plastic limit.

2.9.3.2 Swelling

Swelling is a constant feature for soils of an expansive nature, especially when exposed to sufficient moisture or inundated. Studies on the use of OPC as a remedy to reduce the expansion in soils are plentiful (Al-Rawas et al. 2005; Hashim and Muntohar 2006; Estabragh et al. 2013; Phanikumar et al. 2014; Goodarzi et al. 2016; Jamsawang et al. 2017; Por et al. 2017). In evaluating the swelling potential of stabilised expansive soils, most authors have adopted some conceptual models as well as using well-established practices to determine heaving. For instance, the linear swell test (with the samples confined laterally in a mould) recommended in (BS1924-2) and the accelerated European volumetric test (with the samples unconfined) (EN13286-49) have been adopted under conditions defined in the relevant standard. On the other hand, standard methods of swelling could also involve a linear one-dimensional expansion whereby the soil is made to swell freely in the vertical direction by restraining it laterally in an oedometer (Holtz and Kovacs 1989; Sridharan and Prakash 2000). In order to ensure that more accurate rates are obtained from all the afore-mentioned swelling tests, average values of swelling are calculated for at least 2 similar samples subjected to free swelling under water. Even though each of these methods have their respective uses especially for the stabilised soil, it is pertinent to mention that these tests may not necessarily represent exact field expansion. It has been suggested that the procedure based on laboratory oedometer tests may over-estimate swelling compared to that obtained from the field (Dhowian and Al-Saadan 2010). However, the one-dimensional free swelling test are mostly favoured because it is believed it still does closely mirror field swelling given that some measure of lateral restraint exist in the field, although not completely. Jamsawang et al. (2017) utilised the free swell method by using the oedometer in accordance with ASTM D 4546-14 to investigate the swelling of OPC stabilised three artificially synthesized bentonite-kaolinite (B: K) mixtures. The bentonite was varied in the natural soil mixtures from 2 – 25% in order to provide various degrees of swelling. Three different OPC proportions (1, 3 and 5% by dry mass of the soil) were used to stabilise the soil. The stabilised samples were prepared and cured for 28 days with the water content determined at optimum compaction conditions established from the standard proctor test. Figure 2.7 shows the variation of swelling (%) plotted against the log of time in hours for both the unstabilised and stabilised soils. Swelling increased initially but slowly for all the mixtures and then progressed in a rapid manner before finally reaching a plateau or equilibrium the behaviour of which follows closely the three typical swelling stages reported in Sridharan and Gurtug (2004). The highest amount of swelling is noticed in the unstabilised mixture having the highest quantity of the bentonite (Figure 2.7a). Hence, 25% of the bentonite added to the soil produced the highest swelling potential. Furthermore, the maximum values of swelling for the soil having the highest potential to swell and stabilised by 1, 3, and 5% of the OPC were approximately 13.8, 4.2, and 2.7%. The reduction in the swelling is attributed to the time-dependent pozzolanic reactions occurring under the investigated curing period. Goodarzi et al. (2016) confirmed the influence of both the curing period and binder content on montmorillonite clays stabilised by cement. The montmorillonite was stabilised by 2.5, 5, 10, 15 and 20% (by dry mass of soil) and subjected to progressive periods of curing (1, 7, 14 and 28 days). Figure 2.8 shows the variation of the swelling potential of the stabilised soil with time and indicates that the swelling reduced with curing duration.

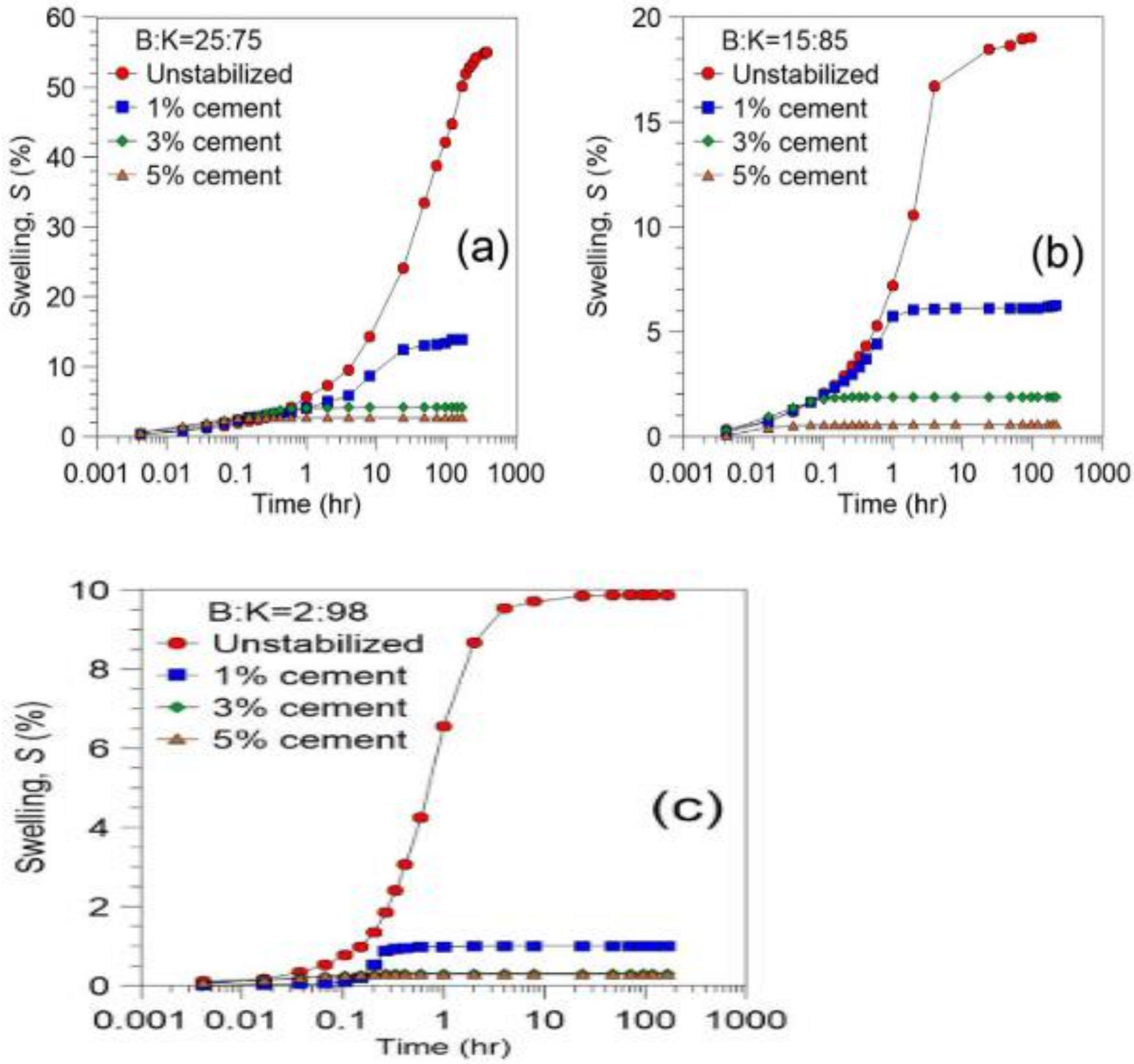


Figure 2.7 Swell percent of different expansive soils stabilised by different cement proportions (a) 25 % bentonite content (b) 15% bentonite content (c) 2% bentonite content (modified after Jamsawang et al. (2017))

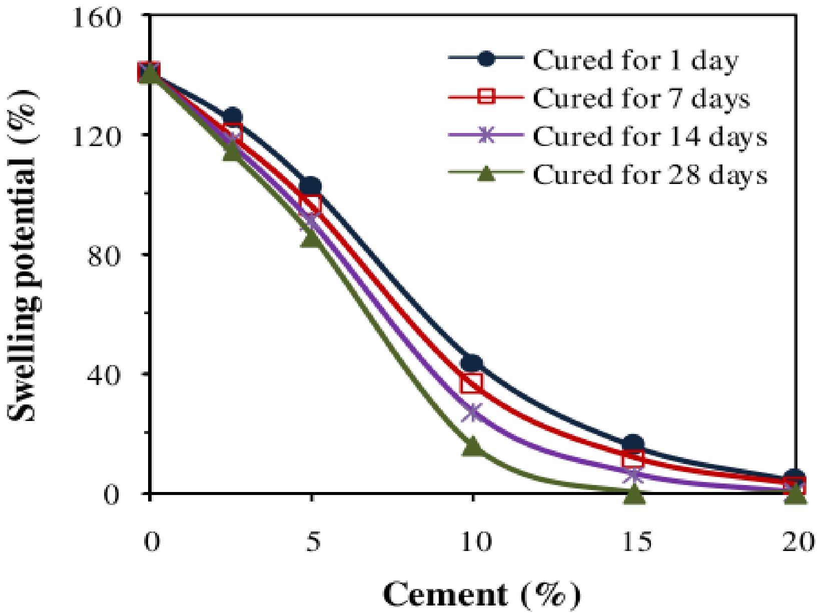


Figure 2.8 Swell potential of cement-stabilised expansive soil showing the effect of curing duration (modified after Goodarzi et al. (2016))

From the foregoing, it can be concluded that stabilisation with OPC does generally lead to a reduction in swelling and that the expansion seem to decrease with increased binder quantity and curing time. This phenomenon is confirmed in most literature (Phanikumar et al. 2014; Mollamahmutoğlu et al. 2017; Por et al. 2017; Mahedi et al. 2018). It is also important to bear in mind that OPC may have little effects on the swelling potential of soils especially when the mineral montmorillonite is in abundant quantities.

2.9.3.3 Settlement/consolidation

Soil consolidation can be relied on to determine the settlement properties of OPC-stabilised pavement works. Consequently, a sufficiently reduced rate of consolidation would be desirable to prevent undue settlement and failure of the road structure. Again, the standard Terzaghi's one-dimensional oedometer testing can be used to study or simulate compacted OPC-stabilised soil consolidation or settlement behaviour in the laboratory (Chan and Abdullah 2006; Pakbaz and Alipour 2012; Phanikumar et al. 2014; Mansour et al. 2015; Goodarzi et al. 2016; Mengue et al. 2018). The nature and rate of consolidation can be examined by the relationship between the void ratio of the soil and the applied sequential loading or effective stress and plotted on a graph as an e-log p curve. A further description of the compression behaviour of the OPC-stabilised soil composites in terms of compression index is shown plotted in Figure 2.9. Compression index (Cc) is defined as the gradient of the linear portion of the e-log p curve. This is an important index in soil consolidation because it indicates the extent of compression that is undergone by soils as the consolidation pressure increases. Just as in the case of swelling as mentioned previously, Goodarzi et al. (2016) indicated that the Cc of cement-stabilised highly expansive montmorillonite clays had reduced with the increase in the cement content and curing period as noticed in Figure. 2.9. Hence, it can also be deduced that stabilisation with cement and increase in the same tend to lead to a reduction in settlement capacity of a soil as the period of curing increases.

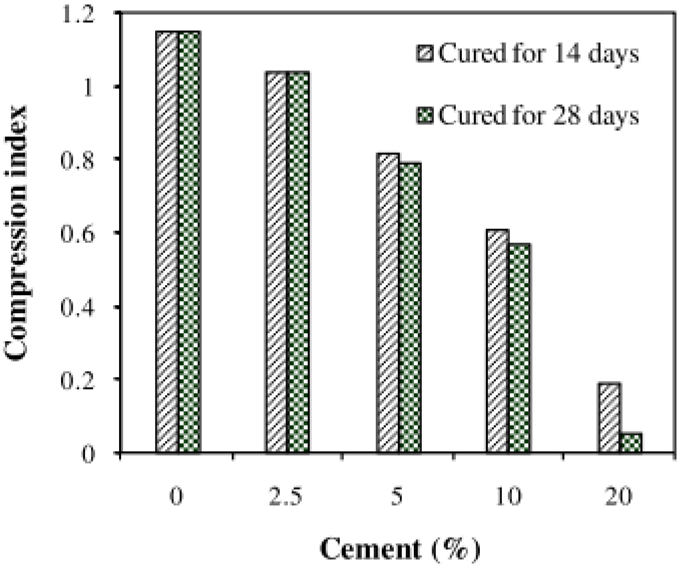


Figure 2.9 Compression index of stabilised expansive clay showing effect of cement proportion and curing duration (modified after Goodarzi et al. (2016))

2.9.3.4 Unconfined compressive Strength

Soil stabilisation by OPC to improve the unconfined compressive strength of weak soils has been well researched (Bahar et al. 2004; Muhunthan and Sariosseiri 2008; Sarkar and Islam

2012; Estabragh et al. 2013; Consoli et al. 2015; Goodarzi et al. 2016; Keramatikerman et al. 2016; Ribeiro et al. 2016; Abbey et al. 2017; Por et al. 2017; Mengue et al. 2018). The UCS is arguably the most widely used and referenced property of OPC improved soils (ACI 1990). It gives an indication of both the soil-cement-water mixture and the rate of solidification or hardening. The unconfined compressive strength is often relied on to determine the minimum cement requirements for the purpose of proportioning the soil-cement. Essentially, the UCS of OPC-stabilised soils should increase with an increase in both the proportion of the OPC used and the period of curing investigated. Figure 2.10 shows the values of UCS for weak samples of soils stabilised by OPC in different proportions and curing periods (Bahar et al. 2004). The UCS was determined at the unsoaked and soaked in water for 48 hours. It could be observed that the increase in the content of the cement in the soil and the period of curing increases the UCS. The soil mineral content also has an important role to play in affecting the UCS of the stabilised soils. Por et al. (2017) simulated this behaviour by stabilising Bangkok clays (BKK) samples possessing a range of bentonite content by using OPC as the binding agent. Figure 2.11 shows the UCS plotted as a stress-strain curve of both the natural and treated soils used after 28 days of curing. As could be observed, all the natural soil samples exhibited a ductile behaviour with constant strain-hardening until a steady-state is reached before failure as the amount of bentonite increased in the soil from 0 – 40%. Stabilisation with the 5 and 10% of OPC changes the failure pattern of the soils from ductile to brittle. However, apart from the proportion of OPC used in the stabilisation and the duration of curing adopted, several other variables (both intrinsic to the stabilised soils or externally applied) are also reported as having an influence on the UCS performance of the stabilised soils. Some of these factors are the type of soil used, cement type, freezing & thawing, wetting & drying conditions, water-cement ratio, compaction conditions, etc

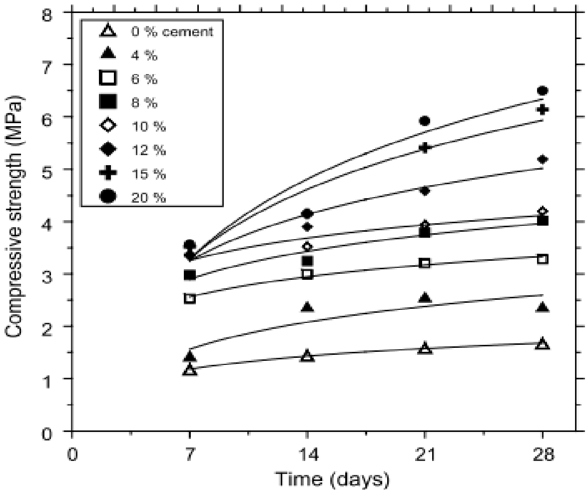


Figure 2.10 Effect of curing duration and proportion on the compressive strength of cement-stabilised soil (modified after Bahar et al. (2004))

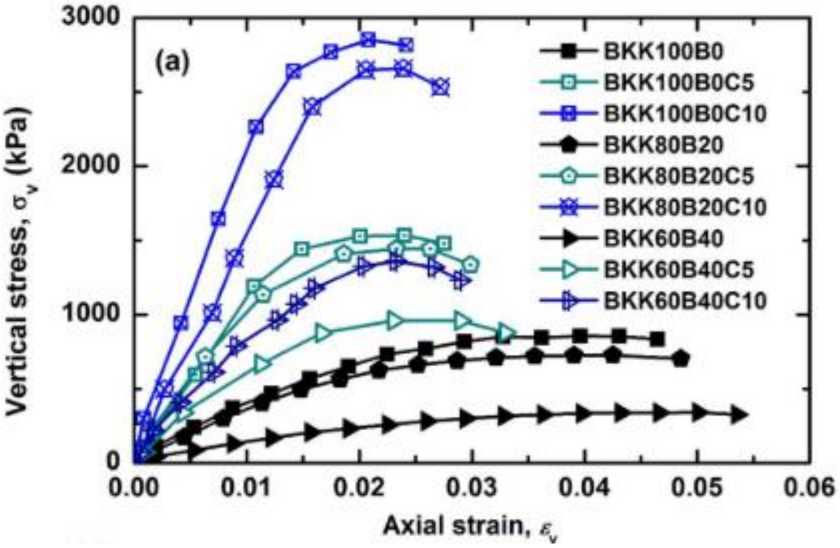


Figure 2.11 Effect of variation of bentonite content and cement proportion on stress-strain curve of stabilised soils (modified after Por et al. (2017))

2.9.3.5 Hydromechanical properties

In recent years, the engineering behaviour of expansive soils existing under unsaturated conditions for any given period has been effectively interpreted by considering the impact of suction as an independent stress state variable. The techniques and methods that have been developed to aid an understanding, estimation and description of the hydro-mechanical behaviour of the unsaturated soil have been well researched (Josa et al. 1990; Gens and Alonso 1992; Fredlund and Xing 1994; Fredlund et al. 1996; Vanapalli et al. 1996, 2014; Delage et al. 1998; Alonso et al. 1999; Yanful and Simms 2002; Eyo et al. 2019). Moreover, since unsaturated soil conditions are normally experienced above the groundwater table, most treated or stabilised soils for roadworks, earth dams' embankments, landfill sites, hydraulic barriers etc could be regarded as existing in this region. The soil-water characteristic (or retention) curve (SWCC) is one of the useful concepts by which an evaluation of the unsaturated property functions and corresponding macro-scale behaviour (strength, volume change, hydraulic conductivity, fluid flow, diffusivity, etc) can be carried out. The SWCC is defined uniquely by the relationship between the mass of moisture present in soil and the corresponding energy state or suction within the pore water. The SWCC is typically sigmoidal in shape as shown in Figure 2.12.

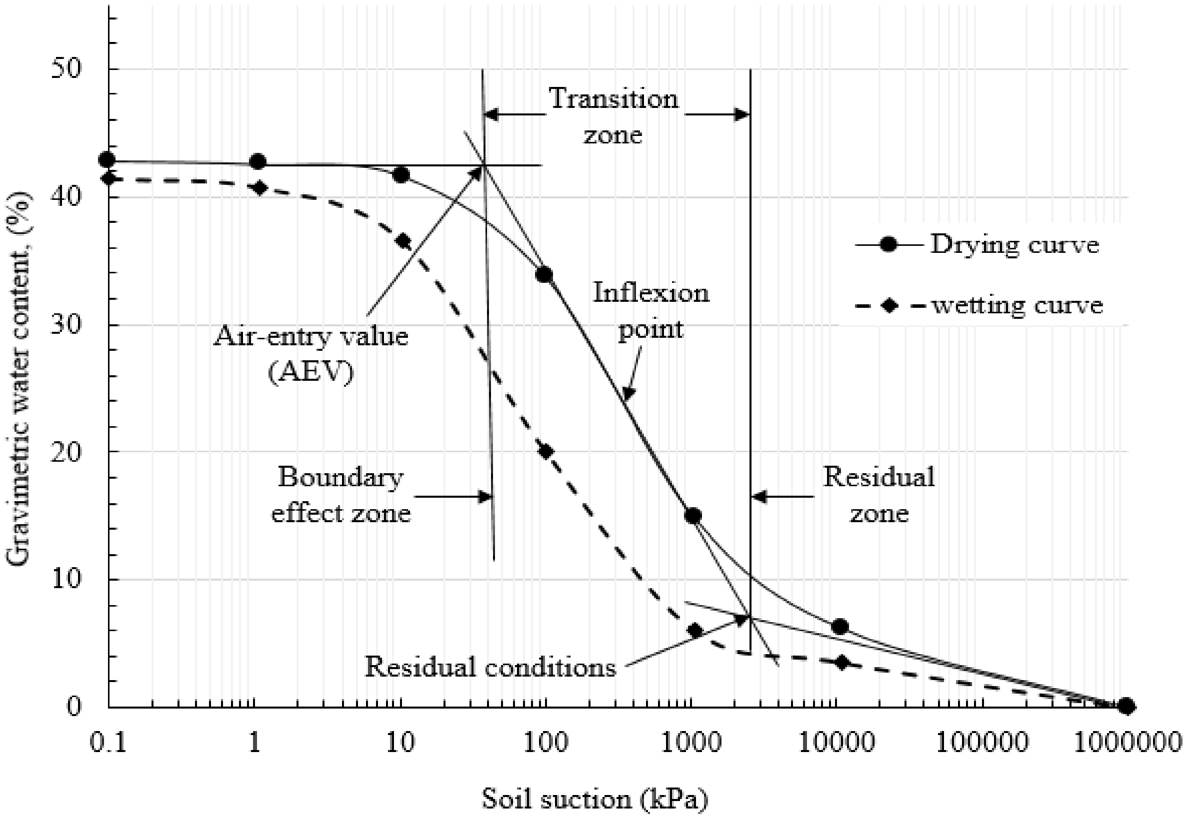


Figure 2.12 Typical soil water retention curve showing the drying and wetting branches (modified after Fredlund et al. (2012b))

The stress state variable of greatest significance to the mechanics of unsaturated soils is soil suction. Three components of soil suction are usually determined or measured namely: matric suction, osmotic suction and total suction. Matric suction is defined simply as that suction component which relates to the height to which water can be drawn or sucked up (i.e. capillary rise) into an unsaturated soil. The matric suction can also be thought of as the difference in pressure between the air pressure and the water pressure which causes a 'contractile skin' (surface tension) at the surface of an exposed body of water. Osmotic suction is that component resulting from the differences in the concentration of salts at different locations in the soil water. Total suction is mathematically the sum of matric and osmotic suction and can simply be quantified as the relative humidity just immediately adjacent to the surface of water. The slope of the curve on which is found the inflection point separates two key components namely: the air entry value (AEV) suction and residual conditions (residual suction or residual water content). Matric suction (capillary effects) governs the portion (up to about 1500 kPa) while osmotic suction (salt concentration) occupies the high suction values (1,500 kPa-10,000 kPa) (Fredlund and Rahardjo 1993). The AEV (or bubbling pressure) represents the suction value at which air begins to enter the soil's largest voids. The suction at the residual condition is termed the residual suction value (RSV) or residual soil suction and signifies the suction corresponding to the residual moisture content. The residual moisture content is the minimum moisture content beyond which there is no appreciable change in moisture with suction. The gravimetric moisture content (or volumetric moisture content) at complete saturation depicts the starting point of the drying path of the SWCC. It should be noted that if the wetting curve is considered then the point referred to as the water-entry value (WEV) is defined as the suction at which there is a significant increase in the water content as the wetting progresses. It is also important to bear in mind that some of the other terms used to refer to the water content-suction relationship are moisture retention curves or retention curves, soil moisture retention curves, soil suction curve and water retention curves (Kovacs 1981; Aubertin et al. 2003; Malaya and Sreedeep 2011).

A large body of research has been devoted to the development and the application of techniques to obtain the moisture content-suction relationship of soils as reported in several textbooks, conference proceedings etc (Murray 1949; Schanz 2003; Fredlund et al. 2012b; Mancuso et al. 2012; Briaud 2013). Some of the more relevant and commonly used mathematical functions are presented in Table 2.2.

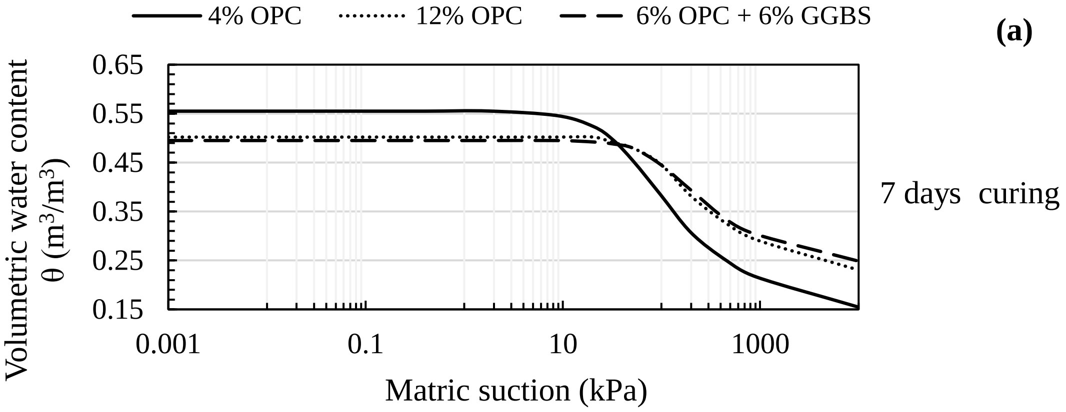
Table 2.2 Commonly used fitting mathematical models of SWCC

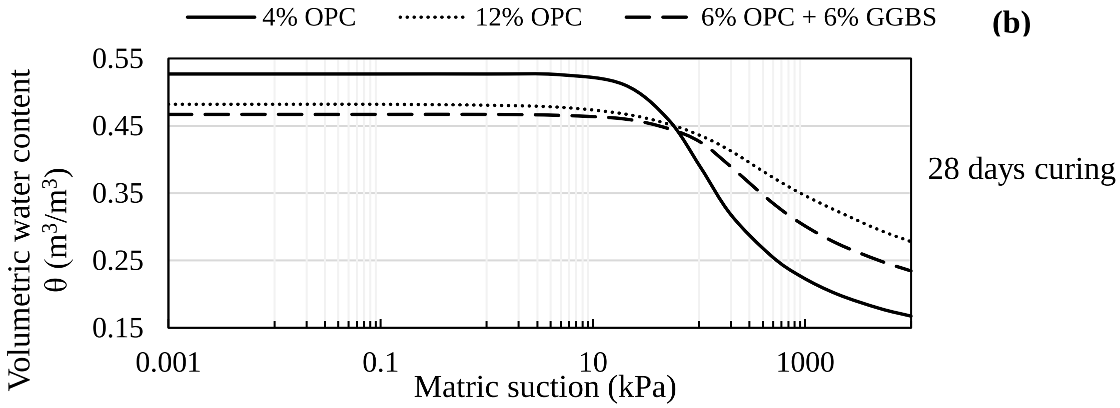
Reference	Mathematical fitting model	Parameter description
(Fredlund and Xing 1994)	$\frac{\theta_w}{\theta_s} = \left[1 - \frac{\text{In}\left(1+\frac{\psi}{h_r}\right)}{\text{In}\left(1+\frac{10^6}{h_r}\right)}\right]\left[\frac{1}{\text{In}\left\{e+\left(\frac{\psi}{a}\right)^n\right\}^m}\right]$	Where: θ_w = gravimetric water content (%) θ_s = saturated volumetric water content (volumetric water content at suction ψ=0) φ = soil suction (kPa) h_r = fitting parameter, which is a function of the suction at the residual water content e = exp (1), base of natural logarithm a = fitting parameter, which relates to the air entry value of the soil (kPa) n = fitting parameter, being a function of the slope of the SWCC m = fitting parameter, being a function of the residual water content λ = pore size distribution index
(van Genuchten 1980)	$\frac{\theta_w}{\theta_s} = \left[\frac{1}{\left(1+\left(\frac{\psi}{a}\right)^n\right)^m}\right]$	
(Gardner 1958)	$\frac{\theta_w}{\theta_s} = \frac{1}{\left(1+\left(\frac{\psi}{a}\right)^n\right)}$	
(Brooks and Corey 1964)	$\frac{\theta-\theta_r}{\theta_s-\theta_r} = \frac{\psi_a}{\left(\frac{\psi}{a}\right)^\lambda}$	
(Mualem 1976)	$\frac{\theta_w}{\theta_s} = \left[\frac{1}{\left(1+\left(\frac{\psi}{a}\right)^n\right)^{\left(\frac{1}{1-n}\right)}}\right]$	
(Burdine 1953)	$\frac{\theta_w}{\theta_s} = \left[\frac{1}{\left(1+\left(\frac{\psi}{a}\right)^n\right)^{\left(1-\frac{2}{n}\right)}}\right]$	

The calcium-based traditional stabilizing agents such as cement, when added to the natural soil would cause initial hydration, (and probably carbonation) leading ultimately to the formation of pozzolanic compounds which produces a cementation effect as time progress. This phenomenon is expected to influence the soil water retention not least, its mass-volume properties (Croce and Russo 2003). Thudi (2006) compared the effect of cement (type I/II) and hydrated lime on the volumetric water content of the stabilised soil. The volumetric moisture was found to have decreased as the proportion of the cement and lime increased. However, the volumetric moisture content for the cement-stabilised soil was observed to be slightly greater than that for the lime-treated soil. This seeming difference was attributed to the texture of the stabilisers used hence; the cement used was observed to have been finer than the lime. Therefore, the cement was more effective in filling the pore spaces and thereby decreasing the pore size distribution of the soil by occupying original voids and causing stronger bond formation with the soil aggregates at contact points. Hoyos et al. (2007) investigated the SWCC of a stabilised expansive clay from Texas by adding 2, 5 and 10% of cement (by dry weight of soil) to the soil. The test results indicated an increase in the AEV ascribed to the greater bonding effect and pore reduction caused by the treatment hence, requiring a very high suction at the beginning of desaturation or transition stage (at AEV suction) of air into the largest voids.

Zhang et al. (2017) studied the impact of curing method by subjected subjecting 4% lime stabilised samples under water and in the air. Results indicated that although SWCC of the samples cured in water began with higher saturation degrees (having more water retention capacity) however, both the air and water-cured samples coincided at suction levels higher than about 1000 kPa (Figure 2.13). This seemed to have occurred in spite of the higher (full) saturation and lower void ratios achieved before the start of the desorption process.

A comprehensive overview of the effect of OPC and other binder agents on SWCC has been given in a literature (Eyo et al. 2020). It is believed that treatment or stabilisation can cause a reduction in the soil's initial mass-volume properties (volumetric moisture content, gravimetric moisture content, degree of saturation or void ratio) at low suction (typically for a drying SWCC). Increased moisture retention occurs generally with increase in stabiliser proportion and its fineness. Moreover, some of the main suction parameters such as air entry value (AEV) and residual suction value (RSV) can increase when the soil is stabilised.





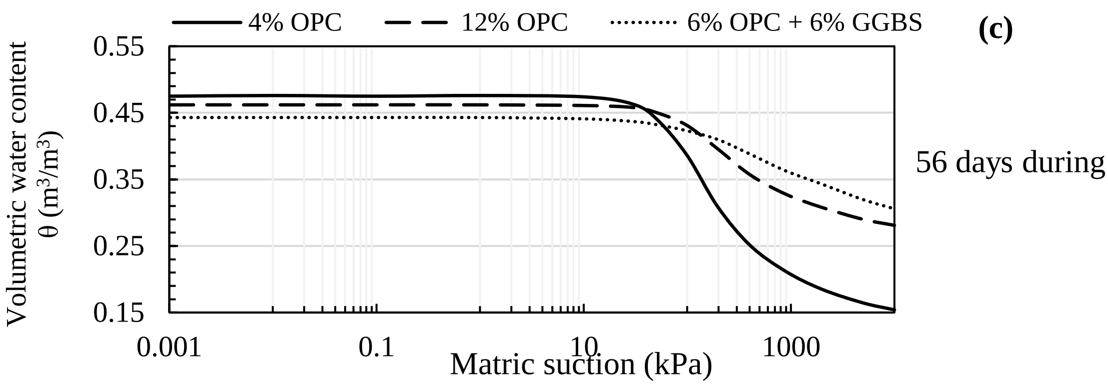


Figure 2.13 SWCC of stabilised samples at different curing duration (a) 7 days (b) 28 days (c) 56 days (modified after Zhang et al., (2018))

2.9.4 Negative Effects of Using OPC In Soil Stabilisation

Concrete which is a direct result of the use of mostly OPC has now been known globally as the second most used or consumed product after water. Apart from concrete, OPC has many other diverse uses as a construction material in several areas of civil engineering not least in soil stabilisation. Hence, the use of OPC in soil stabilisation has promoted immediate concerns given its various disadvantages globally. Some of the critical effects of the use of OPC in recent discussions include the following:

2.9.4.1 Environmental concerns

The global production of OPC has been on a continuous rise given the rapid developments and growth in the construction industry. This expanding industry has brought with it a corresponding rise in the amount of OPC produced worldwide. It has been indicated in a study carried out by CEM-Bureau (Figure 2.14) that in the year 2017, the global cement produced reached about 4.1 billion tonnes with an annual increase of approximately 6.3% from previous years. Van Ruijven et al. (2016) have demonstrated the trends in cement production both in the past and present with a further prediction of further consumption with a conclusion that the global market for cement is estimated to rise at about 5% annually. In relation to the environment, research has shown that about 1 tonne of cement produced could lead to 5,000 MJ of energy consumed, 1.5 tonnes of non-renewable resources released and 1 tonne of CO_2 emission (i.e. 8% of the total global CO_2 emissions) (Higgins 2007; European Commission 2010; Olivier and Peters 2018). Moreover, the British Geological Survey (2005) has reported that 1 tonne of cement produced would require approximately 1.5 – 1.8 tonnes of limestone with an additional 0.4 tonnes of clays. These critical carbon figures are certainly very ominous and threatening because of the enormous greenhouse gasses released and the resulting global warming vis-à-vis climate change currently being experienced globally. Hence, at the front burner of almost all international organisations, climate change activists, environmental campaigners, think tanks, etc are the issues of how to tackle and mitigate global carbon production partly from OPC production.

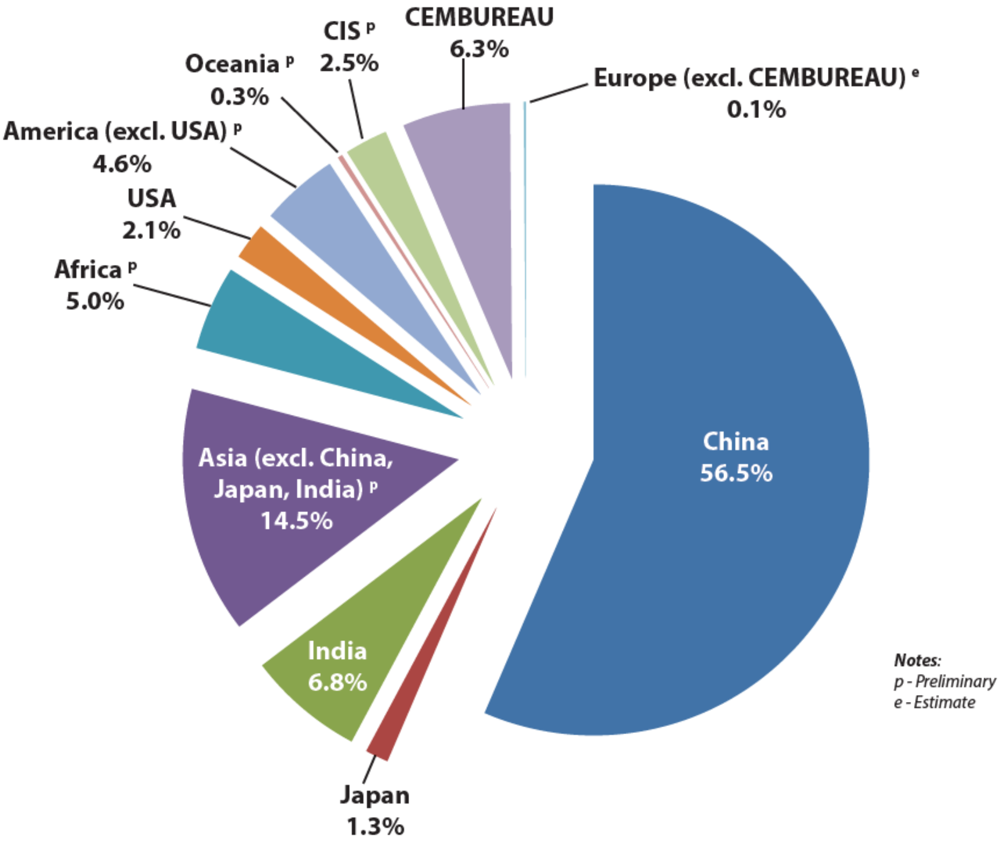


Figure 2.14 Worldwide percentage cement production in year 2017 by country and region (modified after CEM-Bureau (2017))

2.9.4.2 Sulphate-bearing expansive soils

Research has proven that the use of stabilisers rich in calcium for instance OPC for the treatment of sulphate-rich expansive clays could result to unwanted heaving (otherwise known as sulphate-induced heaving) instead of reducing it (Mitchell 1986; Hunter 1988; Kota et al. 1996). As the name implies, this phenomenon of heaving is known to occur in the presence of sulphates in the natural expansive clay and is due to the utilisation of stabilising agent such as cement or lime in the soil improvement activity. Within the mechanism of the soil-stabiliser reaction, a pH (between 11 and 13) environment does develop that leads to the formation of "ettringite" - an expansive mineral in the hydrated system. Ettringite is a hydrous calcium alumino-sulphate mineral compound that tends to precipitate in very high alkaline conditions in concrete and soil systems where high sulphate activities occur. The ettringite mineral could contribute to the sulphate heaving distress in the soil-binder reaction system either by hydration or by a progressive growth & development of itself or through both (Puppala et al. 2005). The destructive effects of sulphate heaving caused to structures have been variously reported in the literature (Mehra et al. 1955; Sherwood 1962). Of particular note are some of the case histories of failure as presented in Table 2.3.

Table 2.3 Case studies of heaving due to sulphate soil stabilisation (adapted from Puppala et al. (2005))

Location	Soil type	Sulphate amount (ppm)	Post-construction heave appearance	Reference
Las Vegas, Nev.	Silty clay	Up to 15,000	2 years	(Mitchell 1986)
Stewart Avenue, Las Vegas, Nev.	Silty clay	43,500	6 months	(Hunter 1988)
Lloyd Park, Joe Pool Lake, Dallas, Tex.	OC Clays	2,000–9,000	Immediately	(Perrin 1992)
Auxiliary Runway, Laughlin AFB, Spofford, Tex.	Clays	14,000–25,000	2 months	(Perrin 1992)
Denver International Airport, Denver, Colo.	Expansive clays	2,775	NA	(McCallister and Tidwell 1994)
SH-118, Alpine and SH-161, Dallas, Tex.	Clayey subgrades	>12,000	6 to 18 months	(Kota et al. 1996)
Localities in Dallas–Fort Worth Region, Tex.	Clays	233–18,000	Varies	(Burke et al. 1999)
Dallas–Fort Worth International Airport, Irving, Tex.	Clay	320–13,000	3 months	(Puppala et al. 1999)

Another type of mineral that could be found to exist in the presence of the expansive ettringite is “thaumasite”. The formation of the mineral thaumasite does occur due to low-temperature sulphate reaction and very intensive carbonation (Bensted 2000). It is important to note that while the formation of the ettringite mineral is accompanied by expansive reaction mechanisms, the formation of thaumasite on the other hand, could result in a rather softening of a previously stabilised soil matrix (Warren and Reardon 1994). It has been hypothesized that if conditions are favourable, ettringite minerals will first form and thaumasite then uses the ettringite as the basis or template for its initial development before going on to form directly from solution (Köhler et al. 2006). However, the degree of expansion due to ettringite is considered more than that of the thaumasite mineral. The thaumasite is presumed to be made up of about 45% of the ettringite volume which it is derived from. Therefore, thaumasite formation is believed to act to reduce the overall net volume of the stabilised matrix and cause its “crumbling” or deterioration. This phenomenon does seem to correlate with the field observation of a degenerating, powdery and wet stabilised matrix. There seems to be some controversies regarding the time of formation of thaumasite mineral due to the stabilisation of a sulphate bearing soil. Some literature seem to suggest that thaumasite mineral may form simultaneously with ettringite or separately from it (Crammond 2003; Köhler et al. 2006).

Hunter (1988) could be regarded as one of the pioneer researchers that provided a categorical description of the formation mechanism of the expansive mineral from the reasons, he gave for its precipitation based on origin, geochemistry and mineralogy. Two proposed mechanisms have been advanced for the formation of ettringite in cement chemistry namely: topochemical reaction and reactions from solution (Jan et al. 2002; Little and Syam 2009). Simply stated, topochemical reaction involves the growth of ettringite crystals at the interface of the solid solution (Ogawa and Roy 1981; Ivan and Michael 1988). When the soil is treated by cement or lime, the pH of the soil tends to increase leading to an enhancement in the solubility of silica and alumina. Mono-sulphates of aluminium will be formed when the alumina content becomes high with subsequent precipitation of ettringites ($Ca_6[Al(OH)_6]_{20}24H_2O\}(SO_4)_2\ H_2O$) on the surface of the alumina-laden phase in what is called the topochemical reaction (Min and Mingshu 1994). Scanning electron micrograph showing the growth of needle-shaped crystals occurring perpendicular to the surface of alumina-bearing phase during the hydration of cement is provided in Figure 2.15 to further support the reaction at the topochemical level. As the release rate of the alumina into the solution wanes, the ettringite crystals become more stable. However, if there are no sufficient soluble alumina and silica in the soil, then the calcium from the stabilisers would react with other minerals present to form gypsum, compounds of calcite and other pozzolanic products. It is then important to note that the quantity and rate of sulphate released into the solution would depend largely on the sulphate salt available in the soil. Ettringite mineral once formed, will grow continuously until the temperature reduces to below 15^0C. Provided that adequate moisture and carbonates are available, ettringite could be transformed into thaumasite (Equations 2.3 and 2.4) below 15^0C as mentioned previously.

$$3(CaSO_4*2H_20) + 4CaO*Al_2O_3*19H_2O + 17H_2O =3CaO*Al_2O_3*3CaSO_4*32H_2O \qquad (2.3)$$

$$SO_4^{2-} + 3Ca^{2+} + CO3^{2-} + SiO3^{4-} + 15H_2O = 3CaO*SiO_2*CO_2*SO_3*15H_2O \qquad (2.4)$$

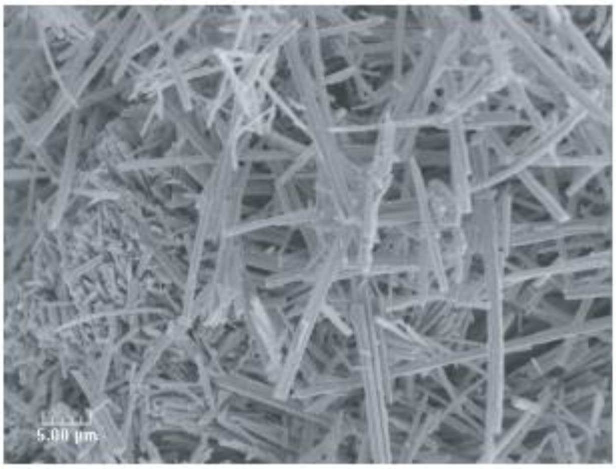


Figure 2.15 SEM showing the growth of needle-shaped crystals (modified after Nair and Little (2011)

Apart from the topochemical reaction, ettringite can also be formed by a through-solution mechanism (or what is also known as the "hydration theory"). In this case, the products would precipitate in a random manner from the liquid phase on the attainment of a state of supersaturation. If the cement or lime concentration is low, the aluminium ions would migrate freely in solution causing ettringite to precipitate from the solution provided by favourable condition (Min and Mingshu 1994).

Even though it is beyond the scope of this research, the rate or kinetics of reaction during ettringite formation have been variously studied (Plowman and Cabrera 1984; Grusczscinski et al. 1993; Ouhadi and Yong 2008).

2.9.4.3 Cost

Apart from the above-mentioned concerns, the cost of producing and using cement in soil stabilisation could be relatively higher as compared to the use of non-traditional alternative, by-products and wastes. However, for any conclusion to be made regarding the comparative cost advantage of using cement to other additives several factors would have to be taken into consideration such as transportation of the products to site, amount of binders required per cubic area to be constructed, actual technical operation of the stabilisation process, amount of binder required and other site logistical issues. In regard to this research, the cost comparison of constructing with cement used as a binder and using the other non-traditional additive are presented in section 2.12.3.

2.10 EMERGING TRENDS IN SOIL ADDITIVES USED FOR STABILISATION

As a result of the negative effects of the use of OPC in civil construction, developments in knowledge and research has constantly and persistently shifted from an over-dependence on OPC as a traditional binding agent to the production and usage of non-traditional ones such as waste materials, industrial by-products, organics, polymers, etc. in engineering applications (Erdal 2001; Poh et al. 2006; Khoury and Zaman 2007; Peethamparan et al. 2008; Seco et al. 2011a; Tastan et al. 2011; Obuzor et al. 2011; Celik and Nalbantoglu 2013; Onyejekwe and Ghataora 2015, 2014; Ganjian et al. 2015; Abbey et al. 2016; Al-Swaidani et al. 2016; Puppala 2016; Eyo et al. 2017; Behnood 2018; Zhao et al. 2019; Zhou et al. 2019). Even though some of the non-traditional type additives may sometimes be used in conjunction with cement either in a binary or ternary combination to improve the engineering properties of the soil, however, the use of the non-traditional binders has been proven to result in more sustainable and cost-effective construction in the long run (Modarres and Nosoudy 2015).

2.10.1 Consideration of RoadCEM in Soil Treatment

RoadCEM is a relatively uncommon additive and is produced based on nanotechnology. RoadCem can be regarded as a fine-grained additive that is composed mainly of synthetic zeolites and alkali earth metals and complemented with a complex activator to enhance its unique properties. This additive can also be utilised in other various kinds of soil stabilisation works including landfill lining or dyke construction and similar applications. As mentioned in the previous chapter, the use of RoadCem as an additive has been on demonstration and in some full-scale projects in several countries of the world(Holmes 2015).

RoadCem is manufactured in large quantities majorly by a company known as PowerCem Technologies in Moerdijk, The Netherlands. RoadCEM is produced in accordance to the ISO 9001:2008/14001:2004 requirement. The mix design involved in the production of RoadCem and its sister products (ImmoCem, ConcreCem, NucliCem) are also largely patented to PowerCem Technologies. RoadCem is mainly intended as an additive to improve and enhance the properties of cement, various other pozzolanic or cementitious products and recycled wastes materials. This additive does modify the chemical behaviour and mineralogical structure of the materials that it is utilised with. For instance, RoadCem is known to modify and extend the chemistry involved during the cement hydration process and to extend the process of crystallization. The major compounds of hydration of the cement will still remain the most important products of hydration however, the minor products will change together with the rate of hydration as RoadCem is added. Furthermore, the crystallization of calcium hydroxide will occur at different rates and the rate of the heat of hydration will be reduced. In addition, more crystals will be formed in the reaction processes with more extensive crystalline matrices.

2.10.1.1 Application of RoadCEM

The RoadCEM additive is designed primarily for applications during road subgrade construction and other related stabilisation activities. This material is majorly used in small quantities to improve the mechanical properties and performance of OPC (Ventura and Koloane 2005; Marjanovic et al. 2009). There are also documented cases of the use of RoadCEM in combination with other cementitious materials and by-products such as lime, PFA, GGBS, etc (Ventura and Koloane 2005; Ouf 2012). In fact, RoadCem has been thought to result in a well-bound product when there is a sufficiently high amount of PFA in the OPC (Marjanovic et al. 2009).

2.10.1.2 Chemistry and soil improvement mechanism using RoadCEM

When RoadCem is added to cement, the quantity of water that is trapped as free water becomes reduced with the crystals growing into the empty spaces. This will result in the mixed product being impermeable to water as well as becoming more resistant to chemical attacks. A larger proportion of the water is then converted to crystalline water than would be the case if the RoadCem were absent. The decrease in the porosity and an increase in the structural crystalline matrices then leads to an increase in compressive, breaking and flexural strength of the product mix.

The evolution of heat of hydration decreases in phases III and IV as shown in Figure 2.16 thus indicating that some of the reactions in phases III and IV may not be occurring or probably that different reactions are taking place.

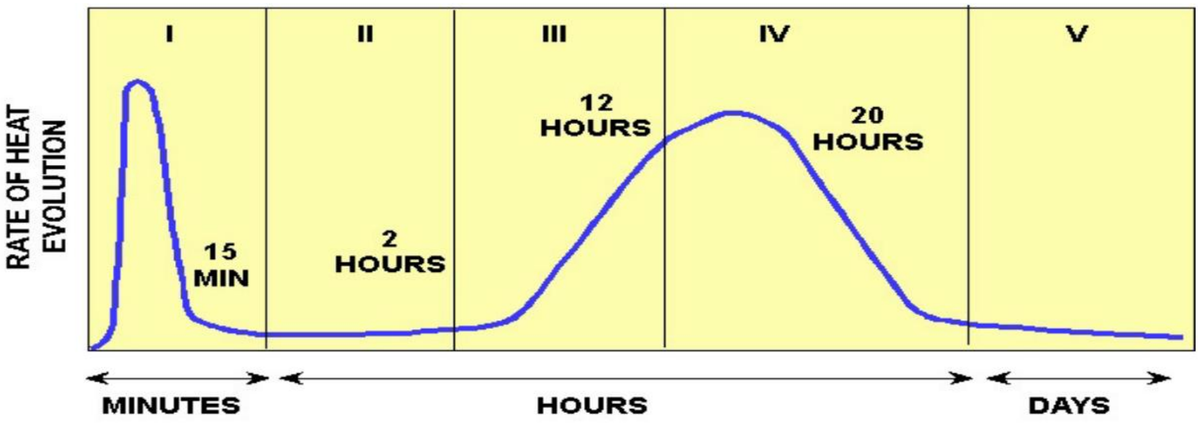


Figure 2.16 Heat evolution rate during Portland cement hydration (modified after Marjanovic et al. (2009))

The type and amount of water that is utilised in mixtures that involve the use of cement are very crucial in the determination of the final strength envisaged. With the addition of RoadCem however, the achievement of very satisfactory end results is feasible even when saltwater is used.

As mentioned previously, the extended process of crystallization does change in a significant manner when RoadCem is added. There is a change in the binding mechanism from ‘glue’ to ‘wrapping’ as depicted in Figure 2.17.

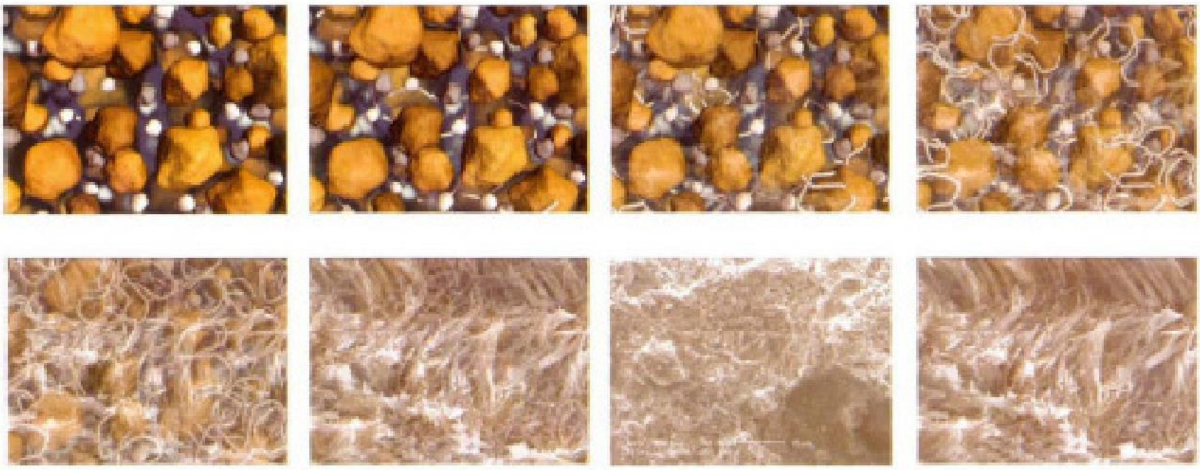


Figure 2.17 A demonstration of hydration reactions and “wrapping effect due to addition of RoadCEM (modified after Marjanovic et al. (2009))

The 'wrapping' effect and encapsulation that are associated with the formation of the crystalline reaction product in the hydration process are hitherto also responsible for the cement-modified product to bind very heavy clays together. This resulting phenomenon is nearly impossible with cement used alone. This is also true for the binding of non-bondable materials such as sludge and various wastes streams.

The composition of the RoadCem also enables other processes to occur simultaneously in the clays and other similar materials through ionic exchanges, modifications and charge neutralization and replacements (Figure 2.18).

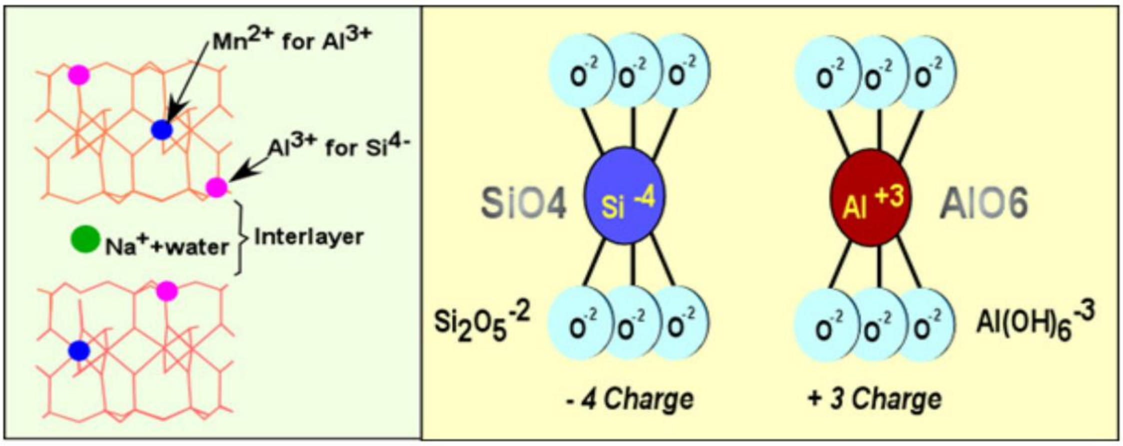


Figure 2.18 Ionic replacements and charge neutralization due to addition of RoadCEM (modified after Marjanovic et al. (2009))

The change in the electrochemistry of the mixed product due to RoadCem addition enables the binding together of different toxic materials and substances such as those found in acidic environments.

With the aid of an electron micrograph, it is easy to visualize the structure of the mixed products once the physicochemical and mineralogical processes have ended. For instance, Figure 2.19 shows the resulting end products of soil material stabilised with cement alone and with the addition of RoadCem to the cement. As can be seen, the addition of RoadCem allows a different kind of crystalline structure to be created.

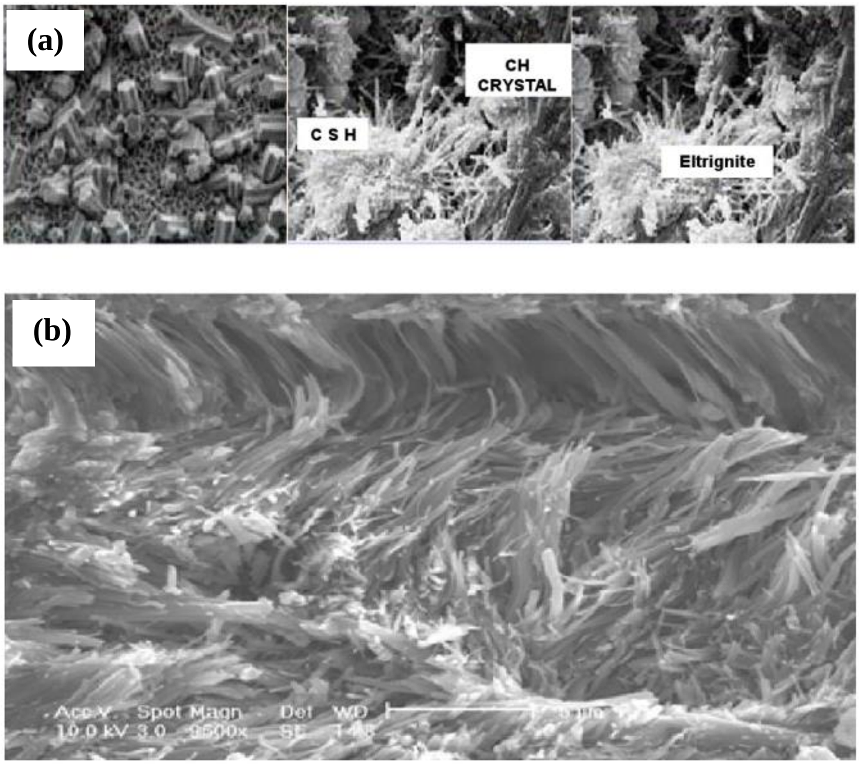


Figure 2.19 Hydration reactions occurring due to (a) cement used alone (b) cement combined with RoadCEM (modified after Marjanovic et al. (2009))

Adding the RoadCem enables a high resistance against extreme temperatures, attacks from de-icing salts and other external forces. The well-compacted soil-cement-RoadCem mix becomes impenetrable to water and prevents damage resulting from climatic influence like the cycles of freezing & thawing. Figure 2.20 depicts the impenetrable qualities of a compacted soil with the RoadCem added.

Figure 2.20 Impenetrable soil resulting from modification by RoadCEM (modified after Marjanovic et al. (2009))

2.10.1.3 Economic benefit of RoadCem use

It has been proven that RoadCem provides the highest cost-effective solution especially when the overall cost of construction is considered. The comparative cost advantage that can be realised between traditional construction and that in which RC is used in road constriction is demonstrated in Figure 2.21 and Table 2.4.

Table 2.4 Comparative cost analysis of using RoadCem and other methods in pavement construction

Traditional construction		Stabilised formation construction		RoadCEM construction	
Layer	Cost (£/m^2)	Layer	Cost (£/m^2)	Layer	Cost (£/m^2)
Wearing course	12.09	Wearing course	12.09	Wearing course	12.09
Binder course	13.21	Binder course	13.21	Binder course	13.21
Base course	18.76	Base course	18.76	RoadCEM stabilised Topsoil & Subsoil	15.00
Subbase (type 1)	7.50	Subbase (type 1)	5.00	Topsoil & subsoil (deposited)	1.25
Capping	10.50	In situ stabilised Subbase	5.10	-	-
Topsoil & subsoil (deposited)	9.30	Topsoil & subsoil (deposited)	3.75	-	-
Overall Cost	**71.36**	Overall Cost	**57.91**	Overall cost	**41.55**

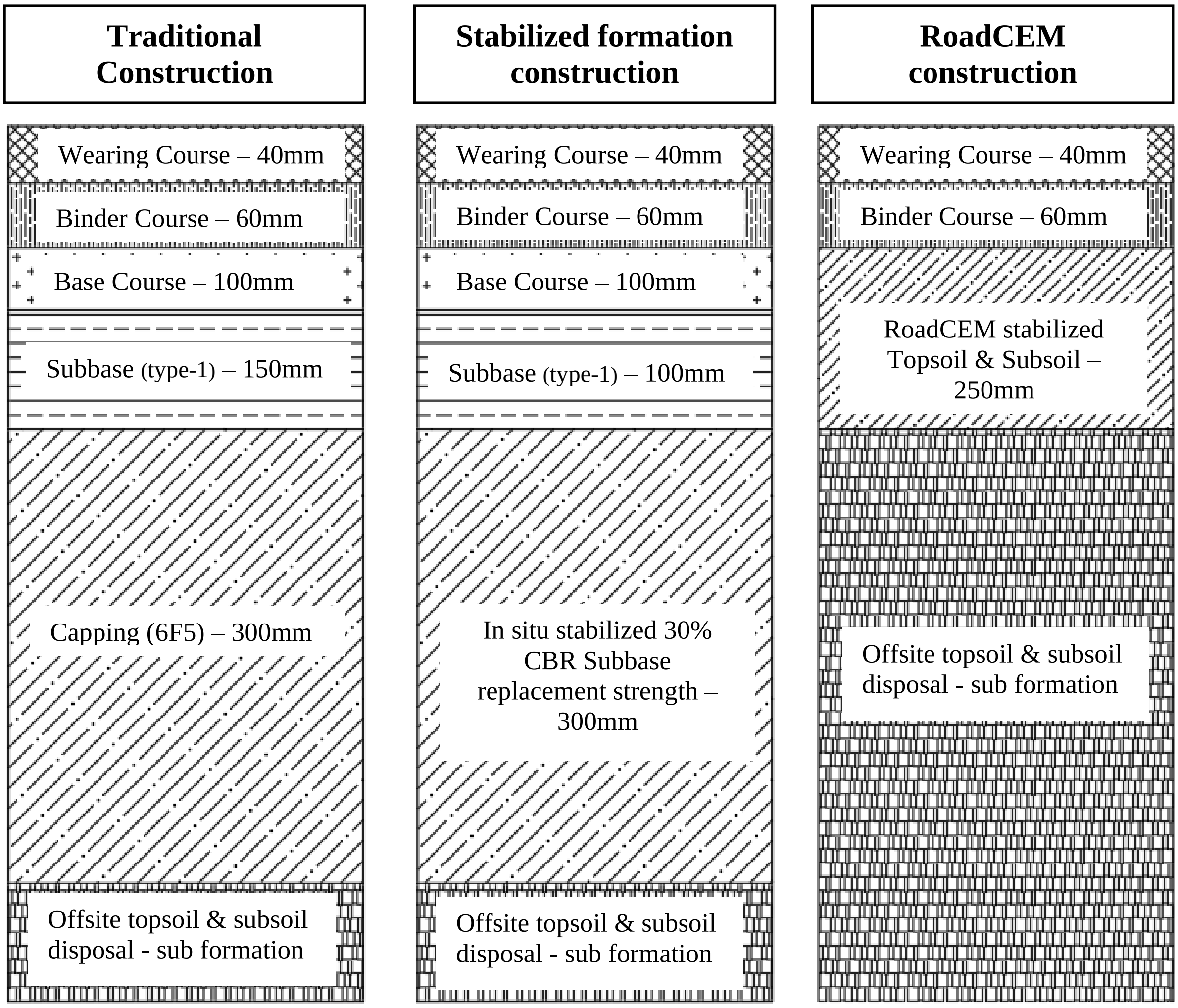


Figure 2.21 Depth of layers in typical pavement construction (modified after Marjanovic et al. (2009))

2.10.1.4 Review of studies on soil stabilised by RoadCEM inclusion

A few previous documented studies of the use of RC in soil stabilisation focused mainly on the enhancement of the strength, flexibility and stiffness as mechanical properties of soils (Ventura and Koloane 2005; Marjanovic et al. 2009; Mutepfa 2010; Ouf 2012; Faux 2015; Wu 2015). Ventura and Koloane (2005) examined the addition 1% of RC to cement replaced by fly ash in both fine-grained sand and fine-grained clayey sand. The soil material used in their study were fine-grained sandy material of low plasticity and a fine-grained clayey soil having high plasticity and composed of 70% of the fine-grained sand and 30% of the clay. The binder blends consisted of 59% fly ash, 40% cement (CEM-1 grade 42.5) and 1% RoadCEM. The total make-up of the binder blends expressed by dry weight of the soil material were 8, 12 and 18%. The soil-binder materials were compacted statically at optimum conditions as would be required in road construction and then subjected to curing for up to 28 days before the unconfined compressive strength testing. Results indicated a significant increase in strength as the period of curing increased for all the binder contents used. The UCS at 7 days of curing was observed to have met the minimum specification of 750kPa according to the Committee of Land Transport Officials (COLTO), Pretoria, South Africa with 8% of the binder used on the chosen fine-grained sandy soil and the mixture of clay and sandy materials. The other studied engineering properties (California bearing ratio, durability, erodibility and flexibility/stiffness) showed satisfactory performance thus complying with the standards used.

Wu (2015), incorporated variable quantities of RC in cement-stabilised sand to study the unconfined compressive strength of the samples. The amount of RC in the stabilised soil varied between approximately 0.2 – 2.0% by dry weight of the cement used. The highest unconfined compressive strength was exhibited by the stabilised soil mixture with the cement and RC applied at the rate of 270 kg/m^3 and 1.9kg/m^3 (or 0.7% by weight of the cement) respectively under 3- and 7-days periods of curing (Figure 2.22). As the period of curing increased, the stabilised soil containing the higher quantities of RC increased. However, as indicated in Figure 2.22 the effect of RC in the stabilised soil seem to be slightly minimal as the period of curing increases beyond 28 days. It is interesting to note that a higher quantity of RC in the mixture does not necessarily suggest that the strength will be higher than the mixture that contains a lower RC quantity or no RC at all. For instance, the stabilised soil with the dosages of RC and cement at 3.2 kg/m^3 (i.e. approximately 2% by dry weight of cement) and 159 kg/m^3, has lower strength as compared to the mixture that contains zero RC for all the periods of curing considered. This phenomenon suggests that the right balance must be sought for the amount of cement and RC to be used in the soil stabilisation to ensure improved strength.

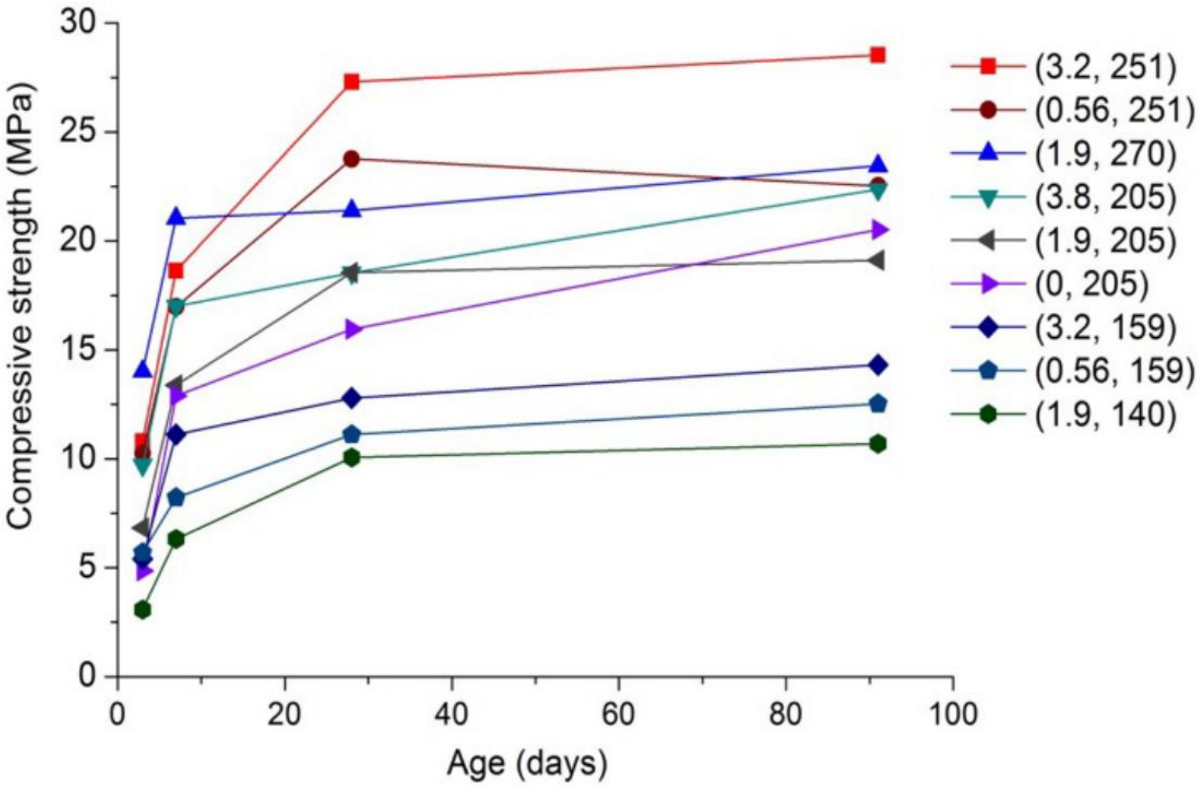


Figure 2.22 Unconfined compressive strength of soil stabilised by cement and RC

Faux (2015) proposed a design method for working platforms by comparing the influence of using cement bound material (CBM) and cement-RC combination in the stabilised soil. The use of cement-RC ensured a satisfactory reduction in the platform thickness occasioned by an increase in unconfined compressive strength (UCS) and elastic modulus (E_{mod}) as compared to the design based on CBM. Ouf (2012) experimentally assessed the strength and free swell index of a soil stabilised by cement/RC and cement-RC-lime- GGBS combinations in different mix proportions. They concluded that while the UCS and E_{mod} increased, the free swell index reduced with increases in the total binder content and the curing duration.

2.10.2 Ground Granulated Blast Furnace Slag (GGBS)

GBBS is a by-product material that is produced when iron is being manufactured in the blast furnace. It is the result of the fusion of limestone with ash from coke and the residue of alumina that remains after iron is reduced and separated from its ore. In its molten state, slag has a temperature of between 1,300-1,600^{0}C and can be made to cool rapidly in order to circumvent crystallisation. The resulting ground materials that is produced is then referred to as ground granulated blast furnace slag (GGBS). GGBS can possess cementitious properties on its own. Though mildly hydraulic, GGBS may require no additives except water to become hydrated and hardened other than water especially at very temperatures and long curing times (Song et al. 2000). The relatively high latent hydraulic property of GGBS means that it can be activated by Portland cement, lime alkalis or other activators in order to accord it some hydraulic properties (Gupta and Seehra 1989; Wild et al. 1998). Some of the concerns mentioned in section 2.10.4 regarding the use of OPC has instigated research into how GGBS's cementing properties could be further enhanced and be used as a replacement or partial substitute for cement in concrete construction, soil stabilisation (Wild and Tasong 1999). The potential environmental, economic and technical benefits of this gesture has been reported (Yong and Ouhadi 2007; Modarres and Nosoudy 2015; Seco et al. 2017). Notwithstanding, GGBS has been well-established in numerous civil engineering construction where it has been noted to provide improvement in the strength, swelling and consolidation properties of soil, resistance to chloride and sulphate attacks, good durability characteristics, etc (Wild 1996; Wild et al. 1999; Ouf 2012; Abbey et al. 2016; Keramatikerman et al. 2016; Sharma and Sivapullaiah 2016a; Zhang et al. 2018).

2.11 SUMMARY

This chapter has provided relevant reviews on the concept of expansive soils including their historical background, behavioural characteristics, expansion chemistry upon moisture ingress and other issues surrounding their mineralogy and structure. Soil stabilisation has been proposed as a method to solve some of the challenges due to construction on expansive soils. Consequently, the use of cement as a traditional binder in the stabilisation of expansive soils was discussed in detail and included subjects such as cement production and the engineering properties of expansive soils stabilised using cement. Treatment with cement can be very effective in the improvement of workability through a reduction in the plasticity index of soils. This phenomenon could mostly be attributed to the initial ionic exchange and flocculation reactions that take place between the monovalent ions from the soil and the divalent ions released from cement when in contact with water. It was also revealed in literature that stabilisation by cement can reduce the swelling potential of soils, reduce settlement and increase the compressive strength of expansive soils. Moreover, improvements in the geotechnical properties of expansive soils stabilised by cement can be enhanced by increasing the proportion of cement and the curing time in order to encourage a more stiffened soil-cement matrix as a result of the formation of pozzolanic compounds. Also, cement used alone to stabilise expansive soils can influence soils’ suction behaviour by causing a reduction in the initial volumetric water content and an increase in the moisture retention capacity as the amount and duration of curing increase. However, some negative impacts of cement usage were also considered from both the technical point of view in soil stabilisation and an environmental standpoint. Critical carbon figures linked to the utilisation of cement in civil engineering construction are certainly very gloomy because of the enormous greenhouse gasses released and the resulting global warming vis-à-vis climate change currently being experienced. Literature has also proven that the use of cement for the treatment of sulphate-rich expansive clays could result in unwanted heaving (otherwise known as sulphate-induced heaving) instead of reducing it. Nevertheless, it has been suggested that cement could be partially substituted by a nanotechnology-based additive called RoadCEM in order to curb the harmful effects of cement. Further background into the RoadCEM product was provided and this included its manufacture, application, mechanism of reactions with cementitious products in soil stabilisation and its general economic benefits in construction. Finally, though there are only a few studies on the utilisation of RoadCEM in the improvement of mainly soil strength as a mechanical property, other soil behaviours such as swelling, and soil-moisture retention capacity could also be a motivation for future studies.

CHAPTER 3

MATERIALS AND METHODS

3.1 GENERAL

This chapter sets out the methods of design and principles adopted for the realisation of the research goals and objectives which involves the determination of the engineering performance of natural and stabilised soils including the hydro-mechanical description and prediction of such behaviour. The methodology is divided into three phases. Phase I focuses on the determination of the materials used and includes soil and binder selection, sampling procedures and methods of preparation. Phase II deals mainly with index property and engineering testing of the macro-mechanical aspects of the prepared samples. Phase III involves an analysis of microstructure and is carried out in order to aid a description of the mechanisms that control the mechanical behaviour of both the natural and stabilised soil samples. The experiments are designed primarily for quantitative data collection and to enable necessary observations to be made.

3.2 PHASE I

3.2.1 Selection of materials

3.2.1.1 Expansive clays and composition

In order to meet most of the objectives of this research, expansive clays were utilised since it is known to exhibit much obvious but undesirable swelling upon moisture ingress, compressibility and low strength capacity typical of weak soils (Nelson and Miller 1992). It has been established that the volume change behaviour of clays (especially most expansive soils) is essentially influenced by two mechanisms namely: the diffused double-layer (DDL)-induced forces of repulsion as is the case in expanding lattice-type montmorillonite clay mineral and shearing resistance as is the case in non-expanding lattice-type minerals such as kaolinite at the inter-particulate level (Sridharan and Venkatappa Rao 1973). Hence, in this research clays rich in the mineral kaolinite and bentonite (Na-montmorillonite) are used. These clays have also been known to be capable of exhibiting behaviours that are markedly different in regards to both chemical and macro-mechanical characteristics especially when stabilised by binding agents (Mitchell and Dimitris 1992).

The kaolinite and the Na-montmorillonite clays were available and used in their processed forms. These industrially manufactured clays were selected for the following reasons; the distinctive spatial non-homogeneous features of naturally occurring clays make studying, analysing and comparing their behaviours difficult and complex especially when stabilised by binders. Hence, the industrial processing of the materials ensures that they are “pure” and of known composition, because some organics, sulphates or sulphides and other trace compounds would have been removed. This enables the micro-structural, physico-chemical features and behaviour of the stabilised clay to be ascribed to the true composition of the mixture and the testing conditions adopted. Another reason for the use of processed soil materials are that other factors such as self-expansion or relaxation, autotrophic bacterial action and pyritic oxidation could impose changes to the naturally occurring clay with the passage of time (James 2013).

The kaolinite and bentonite used were sourced commercially from Mistral Industrial Chemicals Company in Northern Ireland, United Kingdom. The kaolinite referred industrially as China Clay is a white mineral powder that is also suitable as a filler material whereas the bentonite, also known commercially as (Bentonex SB) is an OCMA (Oil Companies Material Association) grade sodium activated bentonite milled to produce a very consistent form of fine powder. This brand of mineral is ideal for geotechnical and civil engineering according to OCMA. The chemical tests from X-ray fluorescence (XRF) to derive the oxide compositions of the bentonite and kaolinite as performed and obtained from their suppliers, are presented in

Table 3.1. The major chemical components of these minerals as observed in Table 3.1 are the silicates and aluminates whereas the relatively trace compounds are oxides of iron, calcium, titanium, potassium, magnesium and manganese. The loss on ignition (LOI) which gives an indication of the level of organic matter in a soil is 12 and 7.85 % respectively for the Kaolinite and Na-bentonite. According to the British standard BS EN ISO 14688-2:2004, these clays could be considered as having medium level of organic matter.

Table 3.1 Oxide composition of minerals (% by mass)

Oxide	Kaolinite	Na-Bentonite
SiO_2	49	57.1
Al_2O_3	36	17.79
Fe_2O_3	0.75	4.64
CaO	0.06	3.98
MgO	0.3	3.68
K_2O	1.85	0.9
TiO_2	0.02	0.77
Na_2O	0.1	3.27
SO_3	-	-
Mn_2O_3	-	0.06
LOI	12	7.85

3.2.1.2 Sampling of expansive clays

The relatively low swelling kaolinite is used as the parent (control) or base clay. In order to simulate a wide range of expansivity, bentonite was added systematically in predetermined proportions to the base clay. This decision is in line with laboratory studies based on the use of remoulded and synthetically engineered clay samples as frame of reference for the properties and behaviour of naturally occurring expansive clays as presented in a large body of literature (Burland 1990; Erzin and Erol 2007; Guney et al. 2007; Horpibulsuk et al. 2007, 2011; Hotineanu et al. 2015; Jamsawang et al. 2017; Por et al. 2017). The limit of bentonite proportion in these previous researches are about 40% of the entire clay mixtures. However, this study has extended the amount of the bentonite in the mix to 75% in order to produce and assess the performance of very high plasticity clay in swell, compression and water retention. The proportion of bentonite blended with the kaolin clay in dry form was gradually increased until five different kaolin-bentonite (K: SB) mixes was achieved– 100:0, 90:10, 75:25, 50:50, 25:75% as also outlined in Eyo et al. (2019). Table 3.2 gives the designation and proportions of the representative expansive soil prepared and used in this study.

Table 3.2 Modelled expansive clays and designation

Clay soil	Soil A	Soil B	Soil C	Soil D	Soil E
% by dry wt. of soil Kaolinite: Bentonite	100:0	90:10	75:25	50:50	25:75
Designation	K100SB0	K90SB10	K75SB25	K50SB50	K25SB75

SB: Sodium Bentonite; K: Kaolinite

3.2.2 Soil Binders

The binders utilised for stabilisation of the prepared expansive clay specimens in this research were also commercially sourced. This research made use of OPC (CEM I with relatively high early strength), ground granulated blast furnace slag (GGBS), RoadCEM (RC) and their various combinations for the stabilisation of the expansive clays.

3.2.2.1 OPC

The OPC binder was supplied by the Hanson Heidelberg group in the UK. These cement properties comply with the requirements of BS EN 197-1:2011 CEM I Portland cement but with strength class of 52.5N making it an ordinary high strength cement that ensures rapid setting and rapid hardening for urgent works in cold weather conditions. This high strength OPC is a low CO_2 cement as it profits from the use of sustainable fuels in its production and it contains sufficiently highly recycled content. The OPC physical properties and oxide compositions as received from the manufacturer are outlined in Table 3.3 with calcium oxide and silicates having to be the most dominant compounds. The oxide composition of this OPC is similar to the chemical analysis of that used by Jamsawang et al. (2017).

Table 3.3 Chemical composition of cement (OPC)

Property	Value	Unit
LOI	5	%
Insoluble residue	5	%
Initial setting time	45	%
Soundness (expansion)	1	%
Chloride content	0.10	%
Early Strength (2 days)	20	MPa
Standard 28-day strength	52.5	MPa
Oxide composition		
SiO_2	20.7	%
Al_2O_3	4.6	%
Fe_2O_3	2.6	%
CaO	65	%
MgO	1.7	%
K_2O	0.4	%
TiO_2	0.3	%
Na_2O	0.1	%
SO_3	2.9	%
Mn_2O_3	0.1	%

3.2.2.2 GGBS

The ground granulated blast furnace slag (GGBS) used was produced and tested following the methods outlined in BS EN 196-2:2013 by the Hanson Heidelberg cement group in the UK. The results of chemical analysis are given in Table 3.4. Again, just as the OPC, this GGBS contains the most amounts of calcium oxides and silicates compounds. Hence, the GGBS could be regarded as being potentially cementitious.

Table 3.4 Chemical composition of GGBS.

Oxide	Value (%)
SiO_2	34.1
Al_2O_3	13.0
Fe_2O_3	0.5
CaO	39.0
MgO	9.0
MnO	0.4
Mn_2O_3	0.4
TiO2	1.2
S^{2-}	0.8
SO_3	0.3
LOI	1.86
I.R	0.5
C	0.2

3.2.2.3 RoadCEM

RoadCEM (RC) additive was supplied by PowerCem Technologies in Moerdijk, The Netherlands. As was stated in chapter 2, the RoadCem is the most relatively recent soil additive of the other two traditional binders or additives (OPC and GGBS) and is specifically produced for use in stabilisation and road construction applications (Marjanovic et al. 2009). The RoadCem additive is based on oxides and synthetic zeolites (5-10% by mass), alkali earth metals (60-80% by mass) that includes salts of sodium, potassium, calcium and magnesium. RC is also composed of complementary activators (5-10% by mass) that gives its unique properties. Some of the basic properties of the used RoadCem are summarised in Table 3.6 the main chemical compounds contained in the RC are those of calcium and silicon oxides as could be observed.

Table 3.5 Chemical composition of RoadCEM (Ouf 2012)

Oxide	Value
SiO_2	21.2
Al_2O_3	1.7
Fe_2O_3	0.63
CaO	47.1
MgO	4
K_2O	7.46
TiO_2	-
Na_2O	-
SO_3	-
Mn_2O_3	-
LOI	

1. The oxide component not included in the table is H_2O which is 17.9 for RC

3.2.2.4 Gypsum

Gypsum superfine white or calcium sulphate dihydrate ($CaSO_4.2H_2O$) was used to simulate the presence of sulphates in the soil. The gypsum was sourced from APC pure located in Cheshire, United Kingdom. This brand of process gypsum has a purity of 99.0%.

3.2.3 Soil-binder preparation

Both the clays and binders were sampled in dry form and then mixed thoroughly (mechanically and manually) using a mixing bucket to ensure adequate blending and homogenization of the mixtures. The amount of water added to the soil-stabiliser mixture was determined based on their optimum conditions through the standard proctor compaction testing as described in section 3.3.1.5. Apart from allowing for the homogeneous and uniform mixtures, the utilisation of optimum water content also guarantees an outcome that represents closely what would be obtainable in the field for instance during the construction of road pavement subgrade. In keeping with part of the objectives of this research, OPC is used herein as the stabiliser of reference. Due to cost, economic and technical implications as previously alluded to in chapter 2 (see section 2.10.4) the OPC would be subsequently substituted by inclusion of RC and GGBS in various proportions and combinations as would be noticed in the chapters following. Hence, further reference to the mix design and proportion of these binders as used in the stabilisation of the expansive clays are outlined in subsequent chapters of this research.

3.3 PHASE II

3.3.1 Engineering and Physical Property Testing

The performance of material property testing on both the blended natural and stabilised clay samples is very imperative for this study. These tests are carried out to appropriately characterize, assess and determine the nature of the natural and stabilised soil samples. Except otherwise stated, the basic properties and engineering characteristics of both the natural and stabilised clay samples were established by a series of standard tests by adopting the American standard for testing of materials (ASTM) in the laboratory. Even though some of the materials used e.g. OPC and GGBS were produced in the UK using the BS EN standard, it is pertinent to state that most of the type of apparatuses that were available to perform the actual engineering testing suggested that ASTM be followed.

The index property and engineering tests are soil classification tests, particle size distribution (laser diffractometry) and analysis, Atterberg limit tests, compaction tests, oedometer swell and consolidation tests, unconfined compression tests and suction tests. The ultimate goal of these tests was to collect data for use in the analysis of the engineering, volume change (swell and compressibility), strength and hydro-mechanical performance of the natural and stabilised soils.

3.3.1.1 Moisture content test

The moisture content test was necessary in order to establish a correlation between the behaviour of the materials and their index properties. This test was also carried out to determine the initial void ratio of the cured samples of the stabilised soils before the oedometer tests were performed. Moisture content test needed not to be determined on the materials especially in their dry forms. However, this test was conducted on the natural and stabilised soils after compaction and curing by adopting the standard procedures in ASTM D2216-19.

3.3.1.2 Specific Gravity, G_s

Specific gravity is regarded as the density of solid material makeup of a substance. With regards to soil analysis, the specific gravity test is very important because the result obtained is used for the determination of the other geotechnical properties (especially the oedometer consolidation test) in this research. The ASTM D 854-10 standard was adopted for the specific gravity testing of the natural and stabilised soils.

3.3.1.3 Particle size distribution

Particle size distribution tests are necessary to establish the gradation and textural characteristics of the materials used in this study. Moreover, the distribution of the solid particles bears a relationship to the distribution of pore sizes or void. Hence, information on the distribution of the particles can be used to derive the pore-sizes and the estimation of the unsaturated behaviour in terms of the moisture content–soil suction relationship for the soil (i.e., SWCC) (Perera et al. 2005).

- *Procedure*

Tests to enable an analysis of particle size and distribution of the expansive clay minerals were carried out by the laser diffractometry method utilizing the Malvern Mastersizer 2000 equipped with the sample dispersion Hydro 2000G module (Figure 3.1b). It has been proven that the

Mastersizer 2000 can analyse the particles of materials in the range between 0.02µm and 2000µm (Jewell and Rathbone 2009; Horpibulsuk et al. 2010; Keramatikerman et al. 2016). The use of this laser technology is also known to adequately and directly address the problem of soil dispersion which is typical of the traditional method using sedimentation analysis based on Stoke's law. The internal mechanism of the technology involves a focussed laser beam which scatters light at a specific angle that is inversely proportional to the particle sizes passing through the medium following which a series of light-sensitive detectors then allows the scattered light's angular intensity to be measured as could be seen from Figures 3.1a (Malvern 2007). A map depicting the intensity of light scattering versus the angle then becomes the primary source of information for calculating and analysing the particle sizes. This study was based on the wet method of dispersion in the preparation of the samples. The expansive clays in their dry and powdery form and respective combinations were first pre-mixed, dispersed in water (used here as a non-reactive liquid), sonicated for about 1 minute so as to ensure complete particle-cluster disaggregation before being fed into the Mastersizer 2000 granulometer bins for the analysis to be performed. Relevant parameters such as the statistics of the particle D-sizes were determined from a grain size distribution and statistics (GRADISTAT) version 4 spreadsheet package developed by Blott and Pye (2000).

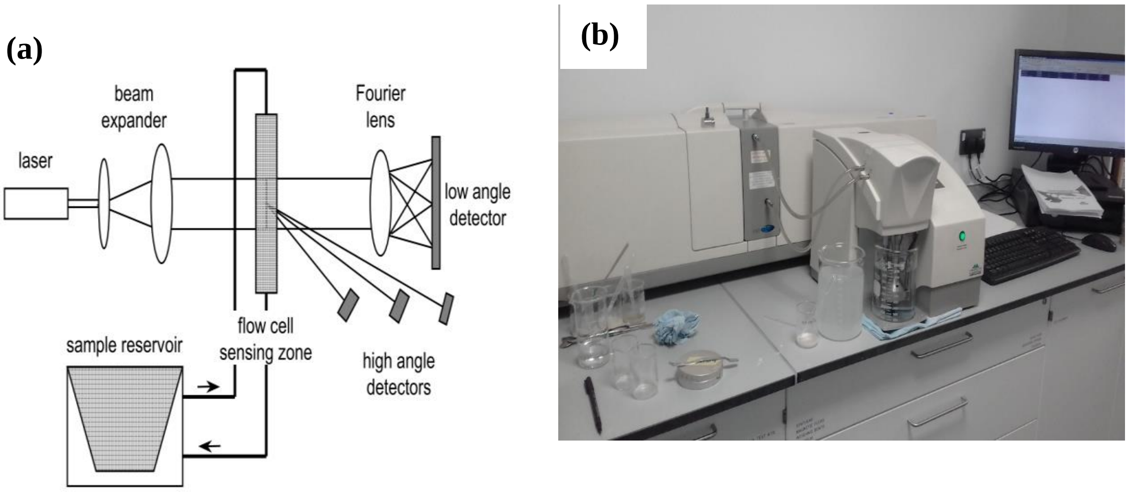


Figure 3.1 Laser diffractometry (a) description of light-sensitive detectors (modified after Hackley et al. (2006)) (a) Malvern Mastersizer 2000

3.3.1.4 Atterberg Limits Test

The Atterberg limits test to determine the natural and stabilised soil's consistency (liquid Limit, plastic limit and plasticity index) were performed in accordance to the ASTM D 4318-17 test standard. This test was important, and the parameters were used to mainly classify the swell potential of the soil samples by an assessment of their mouldability (Figure 3.2). Generally, soils exhibiting plastic behaviours over a wide range of moisture contents and having high liquid limits tend to experience a larger amount of swelling (Puppala et al. 2013). Moreover, the soil plasticity index (PI) could indirectly be related to the expansive behaviour of a particular soil.

Figure 3.2 Demonstration of laboratory consistency limit tests

3.3.1.5 Compaction Test

This test was carried out to obtain the relationship between the moisture content and dry density of the soil sample used. Standard proctor soil compaction test performed according to ASTM D 1557 was very necessary for this research to ensure homogeneity of the mixtures and uniformity of the other macro-mechanical properties considered at the optimum conditions (moisture content and maximum dry density) of both the natural and stabilised soil samples (Figure 3.3). Hence, subsequent engineering property tests relied on the standard proctor compaction test. The stabilised samples were compacted and the optimum moisture content (OMC) and maximum dry density (MDD) determined in accordance to (ASTM D1557-12e1). The amount of water added to the soil-binder mixture was determined based on the optimum moisture content of the natural soil derived from the standard proctor compaction test but with an additional 2% of water to accommodate for the binders used based on research and experience (Wild et al. 1998; Sarkar and Islam 2012).

3.3.1.6 Sample preservation and curing

Chronologically, there is usually a paramount need to preserve the compacted samples (especially the stabilised soils) immediately after the compaction tests before the performance of the engineering tests. Hence, depending on the type of geotechnical engineering test to be conducted on the already compacted samples following the actual standard proctor tests, multiple samples were appropriately extruded from the compaction moulds using the extractor jack, wrapped in a cling film and further sealed in zip-lock type of bags and preserved in the bucket to cure for a period of up to 28 days at a temperature of 20 ±2^{0}C.

Figure 3.3 Laboratory standard proctor compaction and sample extraction from mould

3.3.1.7 Oedometer Swelling

This test was performed to determine the amount of heave that can occur in the field by considering the worst possible scenario of infiltration of moisture due to precipitation or natural groundwater flow. Linear swell test (with the samples confined laterally in a mould) recommended in (BS1924-2) and the accelerated European volumetric test (with the samples unconfined) (EN13286-49) have been adopted under conditions defined in the relevant standard. On the other hand, standard methods of swelling could also involve a linear one-dimensional expansion whereby the soil is made to swell freely in the vertical direction by restraining it laterally in an oedometer (Holtz and Kovacs 1989; Sridharan and Prakash 2000). In order to ensure that more accurate rates are obtained from all the afore-mentioned swelling tests, average values of swelling are calculated for at least 2 similar samples subjected to free swelling under water. It has been suggested that the procedure based on laboratory oedometer tests may only slightly over-estimate swelling compared to that obtained from the field (Dhowian and Al-Saadan 2010). However, the one-dimensional free swelling test is still mostly favoured because it does closely mirror field swelling given that some measure of lateral restraint exists in the field, although not completely. Moreover, in the case of the stabilised sulphate-bearing soil, using the one-dimensional swelling methods ensures that field conditions are closely simulated whereby water replenishment from ground water flow and precipitation can result in constant dissolution of the SO_4 thus, providing a suitable environment for the formation and growth of ettringite Hence, one – dimensional wetting-induced swell strain test was performed in this research in accordance to ASTM D4546 – 14e1 (method A) but with some slight modifications in the testing procedure.

Figure 3.4 Automated oedometer used in the laboratory

- *Procedure*

Grease-coated standard oedometer rings measuring 70 mm in diameter and 20 mm in height were inserted into the soil and soil-binder materials in the standard proctor compaction mould and compaction carried out as described in section 3.3.1.5 with the rings in the mould. This was done in other to eliminate the problem of breakage or cracking upon coring after compaction and extraction of the samples from the mould. The extracted material was carefully trimmed to remove the soil or soil-binder-laden oedometer ring and the smaller chunks of the trimmed materials preserved as mentioned in section 3.3.1.6. At the completion of the designated periods of curing, the ringed-samples were transferred to the consolidation apparatus (Figure 3.4) and placed between porous stones lined by filter papers while the moisture content tests using the oven-drying method (ASTM D2216 – 19) and microscopic observations were performed on the smaller trimmed samples. For the measurement of free swell, the loaded-swell method was used; a seating pressure of 5 kPa was applied to the oedometer carrying the sample (Horpibulsuk et al. 2011; Jafer 2017). Water was then gradually fed into the oedometer cell and complete inundation of the sample ensured. A minimum duration of between 2 to 21 days was allowed for the samples to undergo vertical swelling. However, preliminary studies indicated that for some of the samples (especially those with **Soil A** used) more than 95% of the final vertical swelling had occurred within 48 hours hence, very long periods were not investigated on such samples. Sample free-swell deformations were recorded at regular intervals using an automated linear variable differential transducer (LVDT) until the completion of the free swell. The swell per cent was determined as the ratio of increase in the height (Δh) to the original height (H) of the sample at equilibrium under the seating load expressed as a percentage.

3.3.1.8 Consolidation testing

Consolidation is considered as one of the most vital geotechnical engineering properties and provides an indication of the amount of settlement in the field either during construction or at the end of a project or both. The load-induced consolidation after wetting performed in accordance to ASTM D4546-14e1 (Method C) was used in order to simulate the worst field condition of stress-induced settlement.

- *Procedure*

Following on from the free swell tests and upon the attainment of equilibrium swell, continuous consolidation was ensured on the samples by successively applying the vertical pressure of 5, 10, 20, 40, 100, 300, 500 and 1000 kPa stress with each loading sustained in 24hrs cycles to ensure full primary consolidation. The compression index (Cc) was determined from the collected data while the square root of time fitting method was used to determine the coefficient of consolidation whereas saturated permeability, k of the samples was calculated using the Terzaghi's one-dimensional consolidation equation (Equation 3.1).

$$k = c_v \, m_v \gamma_w \tag{3.1}$$

Where:

m_v = *coefficient of volume compressibility*

γ_w = *density of water*

c_v = *coefficient of consolidation*

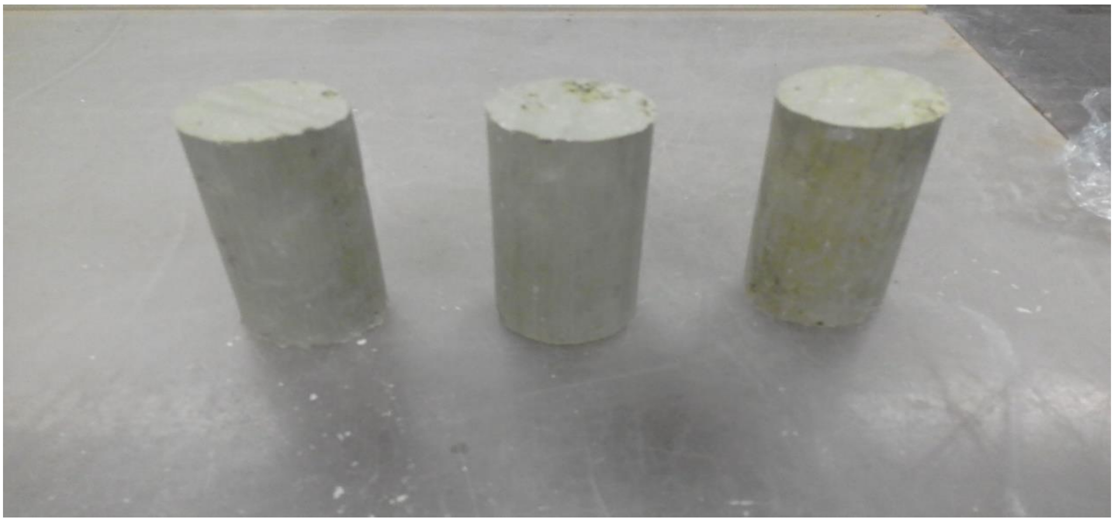
k = *coefficient of permeability*

3.3.1.9 Unconfined compressive strength (UCS) test

On the basis of practical engineering applications and research, the unconfined compressive strength is one of the mechanical properties that is most commonly used for determination and evaluation of cemented or stabilised soils (EuroSoilStab 1997). This UCS test is performed to obtain the stress-axial strain relationship, and consequently, the maximum or peak strength and corresponding strain at failure. The UCS test was conducted as per ASTM D2166-00 on the natural and stabilised soil samples at the end of the designated curing periods. The results of the UCS were also intended in this research to be used as a frame of reference to aid an interpretation of some of the volume change results as would be described in the later sections and chapters of this study.

- *Procedure*

Grease-coated standard cylindrical steel of dimensions 76 mm height and 38 mm diameter were cored through the samples compacted using the standard proctor mould as described in section 3.3.1.5 and the cored samples (Figure 3.3) were then extracted and preserved. The UCS test was conducted using an automated triaxial machine with the horizontal or lateral stress maintained at zero (Figure 3.5) through the elimination of the cell pressure ($\sigma_3 = 0$) which is normally provided by water. According to the test standard, the rate of strain may not exceed 2.0%/min of the original height of the test sample. In adherence to rule, constant rate of axial displacement of 1.0mm/min was followed in this study. In order to properly establish the maximum UCS at failure, at least two samples were tested and the average UCS values determined.



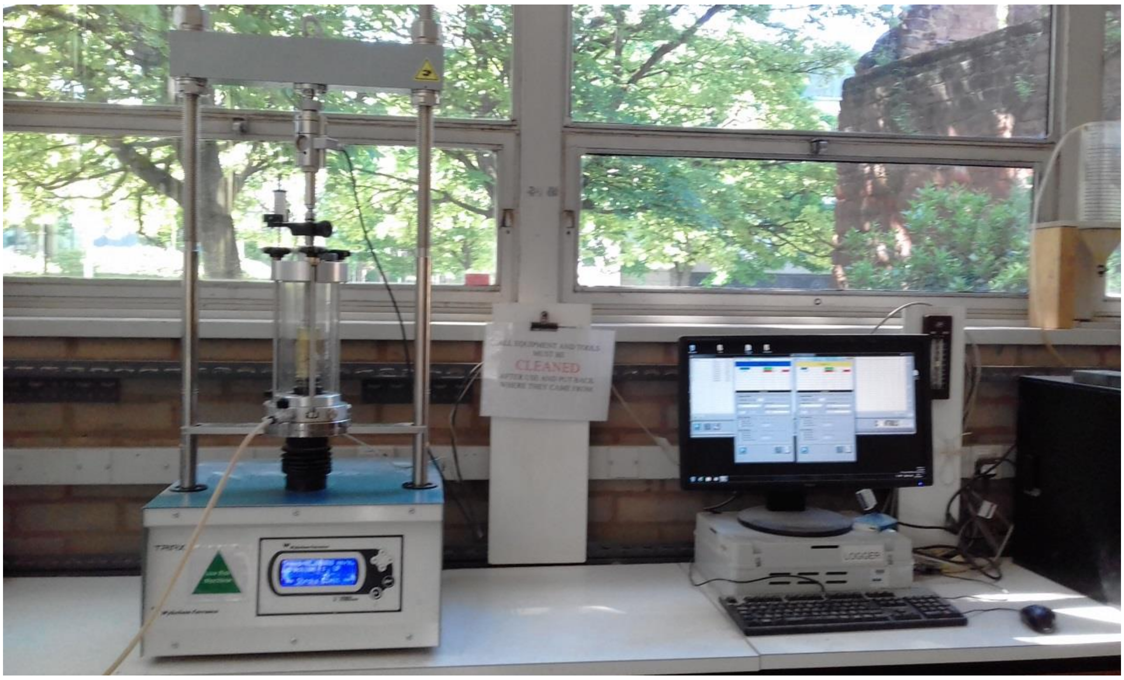


Figure 3.5 Samples for unconfined testing and laboratory triaxial testing machine

3.3.1.10 Soil suction measurement

The treated or stabilised soils in this research are intended mostly for road and railway track subgrade applications. These applications suggest that the stabilised and compacted soil would remain mostly above the groundwater table (nearer to the ground surface or the the active zone) and as such could be regarded as being unsaturated. Hence, suction measurements of the stabilised soil are necessary for the determination of unsaturated hydraulic characteristics of the stabilised soil through the moisture retention curve. The methods utilised to measure or apply suction can be generally grouped into two major categories – direct and indirect methods. The direct method essentially measures matric suction or negative pore water pressure, since it requires that the equipment's sensor must be in direct contact with the soil. The indirect method (which could be applicable for both matric and total suction measurement) requires that measurement be taken from other parameters like relative humidity (or water content), conductivity, resistivity, etc for deriving the measured total suction. Several research have established the variabilities and errors that can be generated in the measurements of natural soil's suction by comparing the equipment used in such measurements (Agus and Schanz 2007; Nam et al. 2010; Tarantino et al. 2011; Zhai et al. 2020). Moreover, a summary of the equipment that can be used to measure or apply suction are provided in literature (Agus and Schanz 2007; Fredlund et al. 2012a) In the case of stabilised soils, it has been discovered that the resulting variabilities in the suction testing or application seem less significant compared to the actual differences in the results of some of the factors that have a bearing on the retention property of the stabilised soils (Eyo et al. 2020). Both direct and indirect methods can be utilised reasonably well and in combination with each other to obtain SWCC having good fits for the respective suction ranges considered or measured. The subtle differences observed in the utilisation of the techniques could be mostly down to the procedure employed and the operator's skills and competence. Suction measurement using the filter paper is the only method that could simultaneously provide two components of suction (matric and total suction) (Nam et al. 2010). Besides, it is simple, low cost and does not require special equipment but can measure a very wide range of suction. Suction measurement utilizing the filter paper method was applied in this research to measure matric suctions of the specimens of the as-compacted samples in accordance to (ASTM D5298-16). The suction testing was carried out to determine subsequently, the wetting curve of the water retention curve.

- *Principles of the Filter paper method.*

Moisture will be transferred unto an absorbent material (Filter paper) that is positioned either in direct contact or within the vicinity of a soil until both the soil and the material show or possess a common attraction for the moisture available. On attainment of equilibrium, both the soil and the absorbent material will be the same. Provided that one of the soil materials of the absorbent medium is homogeneous, then its moisture content could be related to an equivalent therefore, establishing the suction of the soil. Hence, the method using the filter paper tends to rely on the equilibrium by either vapour or liquid moisture exchange between the filter paper and the soil. The equilibrating period has been shown to be largely dependent on many factors including filter paper position in relation to the soil (contact or non-contact), the initial water content of the soil sample, number of the filter paper used, type of material, relative air humidity, the mass of soil and the containing space. However, the ASTM D5298-16 procedure recommends a minimum of 7 days equilibrium time period for direct contact method. At equilibrium the state of stress of the moisture in the filter paper will correspond to the total suction of the soil if the filter paper is not in direct contact with the soil meanwhile, the matric suction will be recorded if the paper is in direct contact with the soil. It has been widely agreed that the filter paper is capable of measuring soil suctions up to 30 MPa (Leong et al. 2002).

- *Filter Paper Calibration*

Total suctions where ultimately used in this research as a surrogate for the matric suction for some of the reasons suggested in addition to the challenges in measuring matric suction which are further enumerated later. Since the filter paper must of necessity be ash free, the Whatman Grade No. 42 qualitative type filter paper with 55mm diameter was used. Most of the established bi-linear calibration curves seem to rely on an initially dry filter paper with a change of gradient of the suction-filter paper moisture content relationship occurring at a moisture content of about 47% provided measurements are taken at a minimum equilibrium period of 7 days. However, for an initially wet filter paper, a similar change in linearity occurs at approximately 15.5%. The calibration method used in the present research for matric suction measurement is that proposed by Leong et al. (2002) Equations 3.2 and 3.3 for the initially dry Whatman 42 filter paper.

$$\varphi = 10^{2.909-0.0229\mathrm{w_f}} \qquad \mathrm{w_f} \geq 47 \tag{3.2}$$

$$\varphi = 10^{4.945-0.0673\mathrm{w_f}} \qquad \mathrm{w_f} < 47 \tag{3.3}$$

Where:

φ = suction
w_f = filter paper water content

- *Saturation procedure and suction measurement*

As-compacted samples each having the same dimensions prepared as described in section 3.3.1.9 for the unconfined compression test were used for the filter paper experiment. In order to obtain the wetting curve, multiple identical compacted samples were allowed to absorb controlled quantities of water using a syringe. The water added was in increasing degree of saturation by ensuring that the moisture increments were in multiples of 2 but beginning initially at 1g of added water. The saturated samples were then wrapped in transparent cellophane bags and a time duration of about 1 hour allowed to ensure adequate penetration and absorption of the moisture. After this period, the samples were then ready for the introduction of the filter papers. Non-contact filter paper was placed sufficiently close to the compacted samples and resting on a spacer thus preventing some form of hydraulic continuity but ensuring vapour flow between the soil and the filter paper. All the jars holding the specimens and the filter papers were carefully wrapped in layers of cling film and left inside a temperature – controlled room at 20^0C ± 22^0C and relative humidity of ± 50% for a minimum period of 7 days to ensure equilibrium was attained. Following the equilibrium period, the jars were opened carefully, and the water contents of the filter paper measured on a weighing balance with sensitivity in the order of 0.001g. The suction data obtained from this were then subjected to a fitting process through a nonlinear regression procedure using the SoilVision ® software package version 5.3.03 which is a program designed for saturated and unsaturated soils.

- *Challenges with the suction measurements*

Matric suction measurements (following ASTM D5298-16) of the samples using filter papers were initially taken and repeated for about 3 times for each of the prepared as-compacted samples. However, the results of the matric suction components seeming not to agree even though the equilibrium time was extended beyond 7 days. It was observed that the errors and inaccuracies arising from the matric suction measurements were due to small pieces of the soil particles sticking to the sides of the filter papers in contact with them irrespective of the “sacrificial” filter papers sandwiching the middle one. Secondly, due to the rough surfaces of the compacted samples, good contact could not be ensured between these samples and the filter papers for accurate measurements. Thirdly, difficulty also arose in measuring the matric suction given that the highly plastic samples compacted at relatively much higher optimum moisture gave up some patches of moisture to the filter papers in contact hence interfering in the measurements. These errors are besides the fact that the suction of soils is a particularly difficult parameter to measure (Fredlund et al. 2012a). Since none of the above challenges was faced in measuring the total suction of the samples, the total suction values were thus used to represent the matric suction as is justified in literature (Fredlund et al. 2012a; Nelson et al. 2015; Zhang et al. 2018). The total suction measurements have proven to be more accurate than the matric suction measurements using the filter paper techniques, and this applies to practical geotechnical applications.

3.3.1.11 Soil water retention property

- *Mathematical models for soil water retention curve (SWRC)*

Stabilised soils used as materials in roadworks are intended to be above the groundwater table or near the surface of the ground (active zone) and as such, they are considered to exist essentially in an unsaturated state. Hence, their hydraulic characteristics interpreted through the SWRC does enable a description and understanding of the corresponding mechanical behaviour under unsaturated conditions. The SWRC describes the relationship between the mass of moisture present in a soil and the corresponding energy state or suction within the pore water. The behaviour of the SWRC is herein used to forge an understanding of the effect of stabilisation on the two model soils used.

Laboratory suction data were subjected to a nonlinear regression fitting process to obtain the SWRC by using the models proposed by Fredlund and Xing (1994) and van Genuchten (1980) both which are widely used in engineering practice and presented in Table 3.7. The soil module function of SoilVision program (version 5.4.08) was utilised to enable an effective non-linear fit of the suction data using the in-built fitting models.

Table 3.6 SWRC fitting models

Reference	Notation	Mathematical model
(Fredlund and Xing 1994)	FX	$\frac{w}{w_{sat}} = \left[1 - \frac{\mathrm{In}\left(1+\frac{\psi}{h_r}\right)}{\mathrm{In}\left(1+\frac{10^6}{h_r}\right)}\right]\left[\frac{1}{\mathrm{In}\left\{e+\left(\frac{\psi}{a}\right)^n\right\}^m}\right]$
(van Genuchten 1980)	vG	$\frac{w}{w_{sat}} = \left[\frac{1}{\left(1+\left(\frac{\psi}{a}\right)^n\right)^m}\right]$

Where:
w = gravimetric water content (%)
w_{sat} = saturated water content (gravimetric water content at suction ψ=0)
φ = soil suction (kPa)
h_r = fitting parameter, which is a function of the suction at the residual water content
e = exp (1), base of natural logarithm
a = fitting parameter, which relates to the air entry value of the soil (kPa)
n = fitting parameter, being a function of the slope of the SWRC
m = fitting parameter, being a function of the residual water content

3.4 PHASE III

3.4.1 Microfabric Examination

Microscopic examination was carried out to explain the mechanism of change in the compacted treated and non-treated samples as well as to support the results and outcome of the analyses of the engineering properties of the tested samples. Soil properties at the macroscopic level are often explained in terms of microstructural behaviour—particle size, distribution and connectivity of pores, shape and distribution in conjunction with the arrangement of the grains and the contact between them (Romero and Simms 2008). Microscopic features of the natural and stabilised clays were studied and analysed using the scanning electron micrograph (SEM), energy dispersive spectroscopy (EDS) and X-ray diffractometry (XRD) tests. Electron microscopes such as the SEM can provide magnification powers compared to standard visible light microscopes because electrons possess relatively shorter wavelengths than light waves (Mirzababaei and Yasrobi 2007) . For instance, the highest magnification achievable with modern electron microscopes can be up to 1,000,000X whereas light microscopes can only achieve magnifications of about 2,000X. Besides the SEM, transmission electron microscope (TEM) has also been used for image analysis. However, SEM is very useful for detailed three-dimensional analysis compared to TEM which only produces images in two-dimension. Also, even though scanned specimens for SEM analysis would have to be thoroughly dry, they do not need thin slicing as do TEM specimens. On the other hand, different kinds of XRD exist for the determination of the oxide component of soil samples. The Panalytical Empyrean is used in this research for the measurement and analysis of the hydration components of the stabilised soil. This equipment is equipped with a cobalt (Co) target and has a longer wavelength than the other copper (Cu)-based systems such as the Panalytical X-Pert Pro MPD $K_{\alpha 1}$ thus making it possible to avoid any unnecessary iron (Fe) fluorescence.

3.4.1.1 Scanning electron micrograph (SEM)

The SEM operates by scanning an electron beam that is tightly focused over a sample (mostly in the dry state). Electrons in the beam are scattered off from the sample and onto a screen or a cathode ray tube. Each point on the sample then corresponds to a picture or pixel element on the screen. The resolution of the sample is determined by the number of electrons meeting the screen. Hence, the more electrons hitting a particular element of the screen means higher resolution or brighter image, pixel or picture and a complete image of the sample will be displayed on the monitor when the electron beam scans over the entire sample.

Small-sized chunks of soil (both natural and stabilised) derived from selected originally compacted and cured samples were used to obtain the micrographs. To ensure that the surfaces of the samples were sufficiently conductive electrically, for the derivation of the SEM images, a Polaron SC7640 sputter coater (Figure 3.6) was used to coat the samples with gold. The ZEISS EVO equipment (Figure 3.7) was utilised to collect the micrographs (SEM) of the cured, dry and completely vacuumed samples. A minimum working distance (WD) of 7.7mm using a minimum acceleration voltage (EHT) of 5.00kV and various degrees of magnifications to obtain clear pictures were observed.

Figure 3.6 Sample preparation for SEM analysis using the sputter coater

3.4.1.2 Energy dispersive spectroscopy (EDS)

Energy dispersive X-ray spectroscopy (EDS or EDS) is an analytical technique that can be used for the chemical characterisation or elemental analysis of samples. It could be regarded as one of the forms of an X-ray fluorescence spectroscopy that relies on sample investigations through some interactions between matter and electromagnetic radiation or an analysis of X-rays that are emitted by matter in response to the incidence of charged particles. Even though there a number of stand-alone EDS systems, most EDS systems are now produced in conjunction with scanning electron micrographs (SEM-EDS). Energy-dispersive X-ray spectroscopy was also employed in this research on selected samples taken immediately after the SEM exercise in order to determine the main chemical, elemental compositions and changes on the sample surfaces responsible for the possible fabric changes.

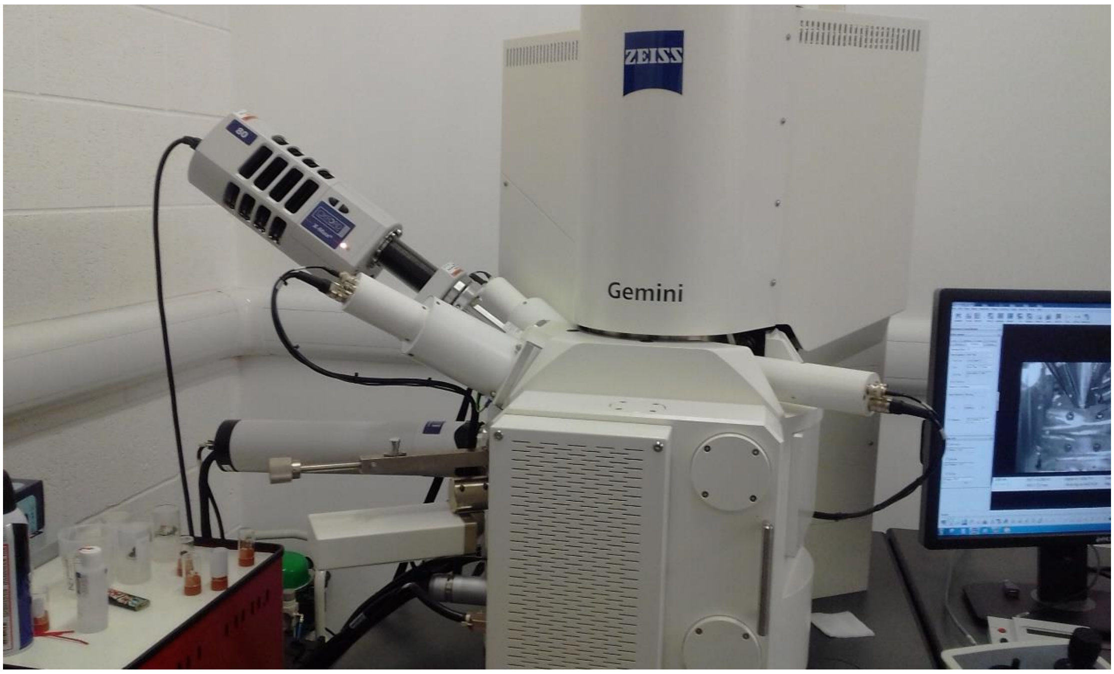


Figure 3.7 The ZEISS EVO Scanning electron microscope

3.4.1.3 X-ray diffraction (XRD)

X-ray diffraction is a very powerful technique that is often employed to study the powder materials, coatings, polycrystalline and nanomaterials. One of the simplest uses of XRD is for phase analysis and identification made against a database of already known patterns. XRD utilises photons that are produced from metallic targets in order to diffract off a crystalline lattice plane having similar spacings to the wavelengths of x-rays (approximately in the range of 10^{-10} m). A high-resolution X-ray diffraction analysis was conducted on representative samples of the investigated clay mixtures for the purpose of evaluation and identification of the phases of the natural and the stabilised clay minerals in this research. The samples were measured between 5-80° 2θ on a Panalytical Empyrean X-ray diffractometer equipped with Co Kalpha (1.790307Å) radiation (Figure 3.8). The phases were determined using the Panalytical Highscore Plus software and the latest ICDD PDF4+ database. Few challenges were encountered in the determination of some of the phases involved in the mixtures due to the unique structures, complexities and the various element compositions of natural and stabilised samples.

Figure 3.8 X-ray diffraction equipment.

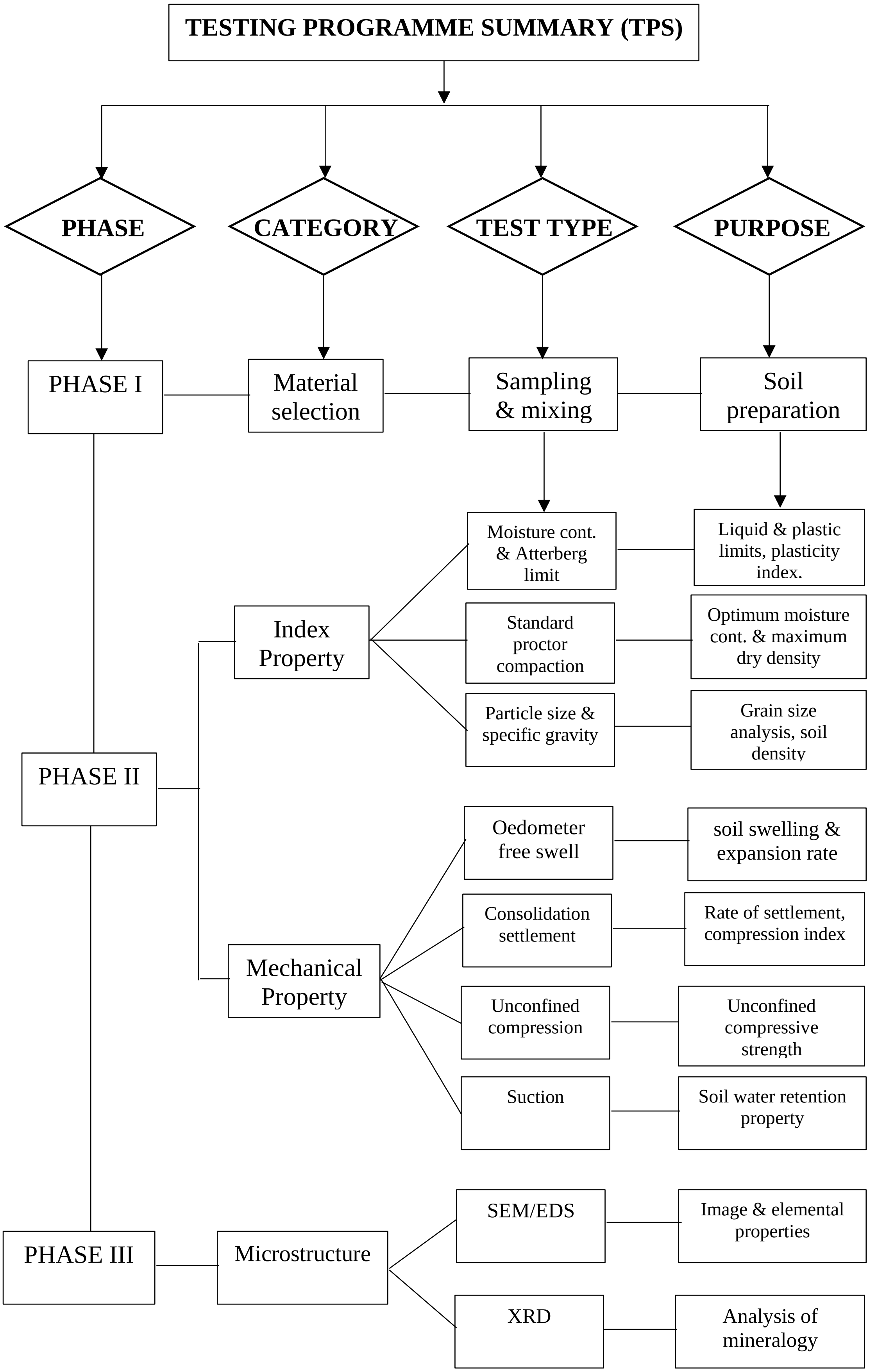


Figure 3.9 Testing programme flowchart

3.5 SUMMARY

A summary of the laboratory exercises embakrked upon to collect the necessary data for subsequent analyses in this research are presented in phases in the flow chart of Figure 3.9 and are further expalined as follows:

i. Phase I involved the selection and sampling of the materials to be used in this study. Clays that are rich in the mineral kaolinite and bentonite (Na-montmorillonite) were selected for this research. Kaolinite alone are used and considered solely as the parent (control) or base clay. In order to simulate a wide range of expansivity, bentonite was added systematically in predetermined proportions to the base clay until five different kaolin-bentonite (K:SB) mixes was achieved. The binders intended for use in the stabilisation of the clays are: OPC (CEM I with relatively high early strength), ground granulated blast furnace slag (GGBS), pulverised fuel ash (PFA), RoadCEM (RC) and their various combinations for the stabilisation of the expansive clays. Gypsum superfine white or calcium sulphate dihydrate ($CaSO_4.2H_2O$) was also selected to be used to simulate the presence of sulphates in the expansive soils during stabilisation.

ii. Phase II included the performance of material property testing on both the artificially synthesized clays and stabilised clay samples to appropriately characterize, assess and determine the nature of the natural and stabilised soil samples. The index property and engineering tests involved soil classification tests, particle size distribution (laser diffractometry) and analysis, Atterberg limit tests, compaction tests, oedometer swell and consolidation tests, unconfined compression tests and suction tests.

iii. Phase III involved microscopic examination carried out in order to describe the mechanism of change in the compacted treated and non-treated samples as well as to support the results and outcome of the analyses of the engineering properties of the tested samples. Microscopic features of the natural and the hydration mechanism of the stabilised clays were studied using the scanning electron micrograph (SEM), energy dispersive spectroscopy (EDS) and X-ray diffractometry (XRD) tests.

CHAPTER 4

IDENTIFICATION AND CLASSIFICATION OF NATURAL SOILS

4.1 GENERAL

This chapter discusses experimental results and provides analyses with a focus on the classification of the natural soils' properties as measured in the laboratory. The natural clays are first distinguished and classified according to the Universal Soil Classification System (USCS). Next, the nature of the clays is discussed, and this includes an identification, evaluation and prediction of their expansion properties. Given that the soils used could possess some measure of expansiveness, emphasis is laid on the assessment of their swelling potential. Finally, other characteristic behaviours such as the natural clay consolidation and strength are presented from the results derived from the experiments in the previous chapter.

4.2 CLASSIFICATION OF THE NATURAL SOILS

Some of the relevant and fundamental characteristics of the tested natural clay samples are presented in Table 4.1.

Table 4.1 Properties of the investigated soils

Property	**Soil A**	**Soil B**	**Soil C**	**Soil D**	**Soil E**
	K100SB0	K90SB10	K75SB25	K50SB50	K25SB75
Specific Gravity	2.60	2.65	2.69	2.70	2.76
Silt Content (%)	74	70	65	58	48
Clay Content (%)	26	30	35	42	52
MDD (kN/m^3)	15.0	13.9	13.5	13.2	12.9
OMC (%)	17	21	23	25	30
Modified activity	0.67	1.92	2.73	2.76	4.06
Specific surface area	1.70	1.75	6.03	10.35	14.68
USCS classification	CL	CH	CH	CH	CH

4.2.1 Natural Soil Consistency

Clay plasticity is a very important index to determine workability in the field of construction. Clays of high plasticity have a tendency of being difficult to mix and homogenize during compaction. However, insufficient workability does not preclude the utilisation of clay for lining systems but can be an issue in road-subgrade construction. A general increase in the Atterberg limit properties as the amount of bentonite increases is noticed in Figure 4.1. Bentonite generally contains more fines and are more clayey than the kaolin clay hence the increased Atterberg limit properties are expected because a relatively significant amount of water will be required to make the mixture more plastic.

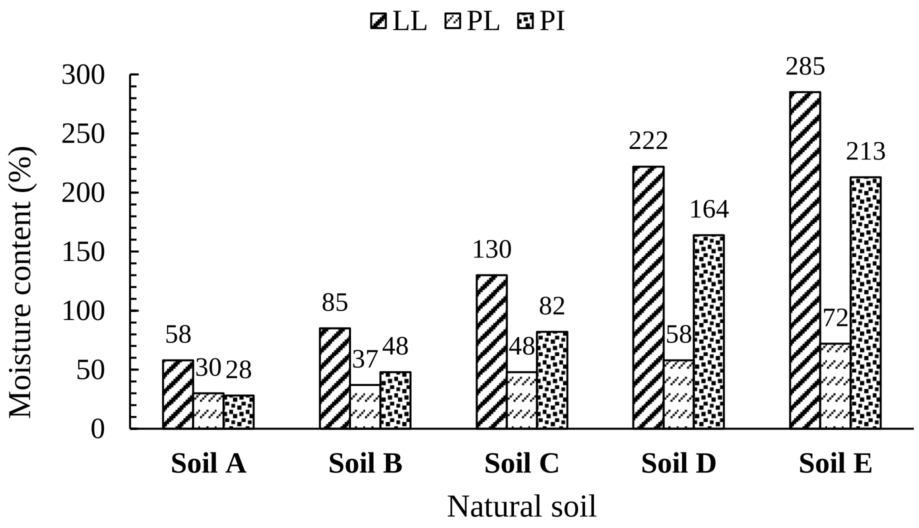


Figure 4.1 Consistency limits of natural soils

4.2.2 USCS Classification

Figure 4.2 shows the Casagrande’s soil plasticity chart which uses the Unified Soil Classification System (USCS) to identify natural soil such as those used in this study through group symbols. By superimposing the plasticity indices and liquid limit values on the Casagrande’s soil plasticity chart of Figure 4.2, the soils could be located on the chart as ranging from medium plastic to highly plasticity clays (Table 4.1).

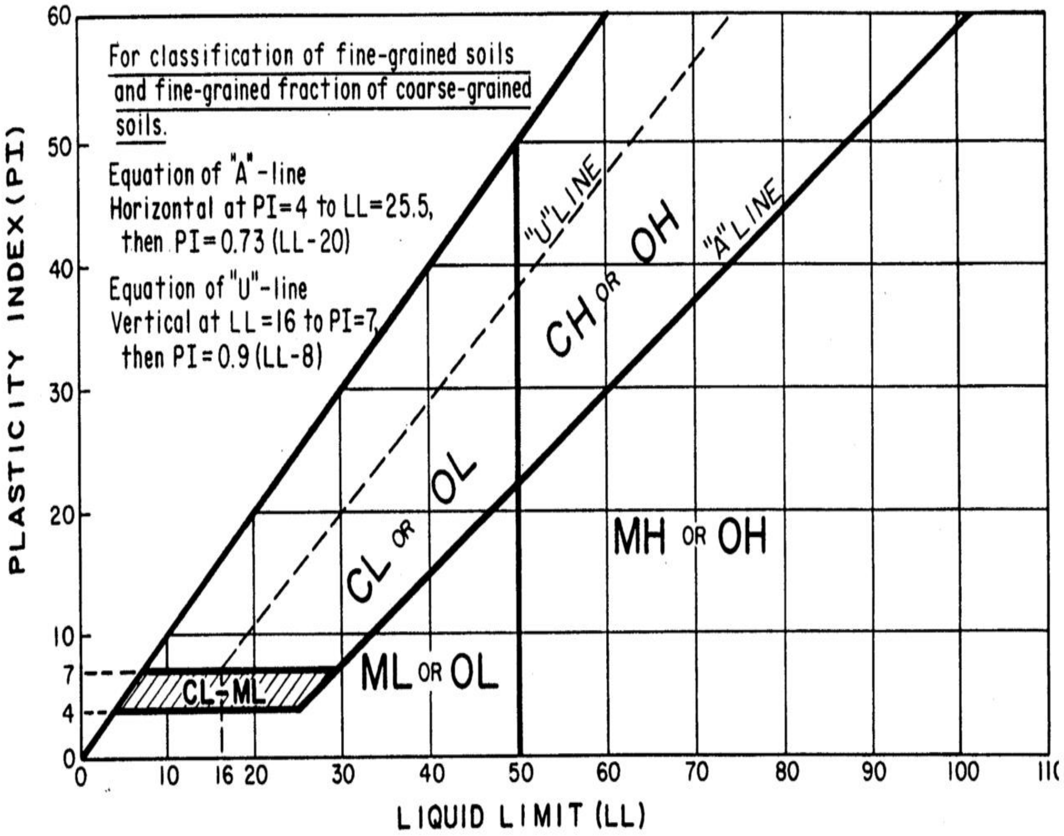


Figure 4.2 Casagrande plasticity chart for USCS classification of natural soils (adapted from Casagrande (1948))

4.2.3 Compaction Parameters of Soils

The compaction curve showing the relationship between the soils' dry density and moisture content indicates a decrease in the maximum dry density (MDD) and a corresponding increase in the optimum moisture content as the amount of bentonite or the percentage of clay fines increased. This phenomenon is much expected because the soils containing much of the bentonite which is a "heavier" clay, and which possess greater specific gravity compared to the soils with lower quantity of the bentonite tends to retain more water.

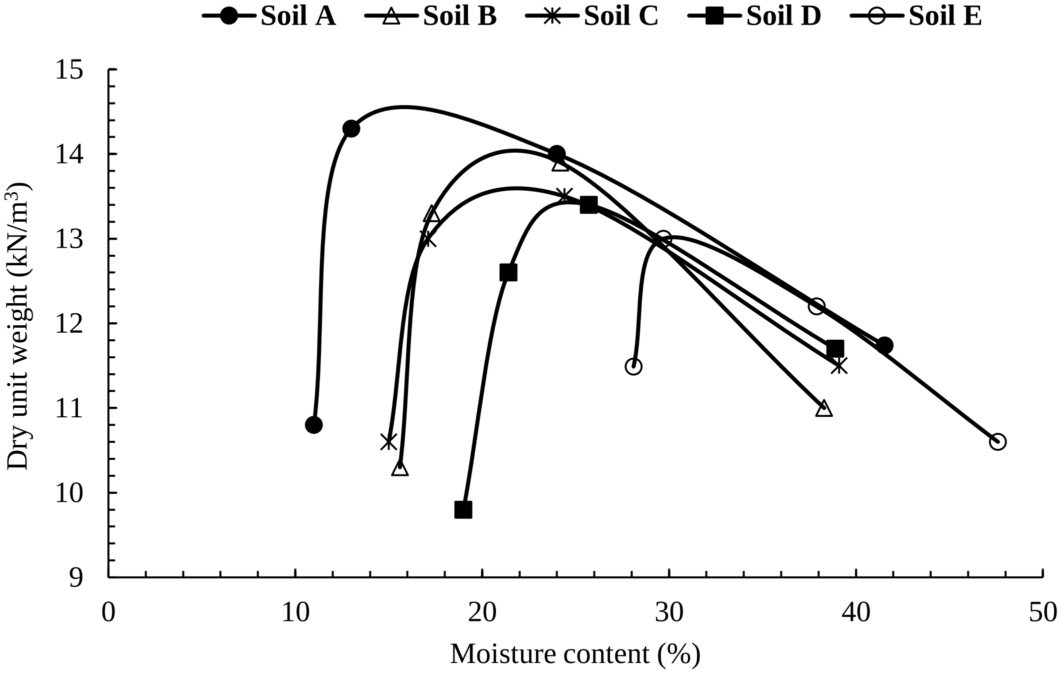


Figure 4.3 Relationship between dry unit weight and moisture content

4.3 CHARACTERISATION OF NATURAL SOIL SWELL POTENTIAL

From the foregoing, it could be observed that the soils chosen for this study are potentially expansive with the ability to undergo swelling when exposed to moisture. Hence, a determination or an identification of their degree of swelling or expansion is necessary. Even though expansive soils are known to be distributed widely across the globe with potential to cause distress to foundations of buildings and discomfort to users of road infrastructures, the characterization and classification of expansive soils are until now not clearly captured in national codes or standards used in practice (Sridharan and Prakash 2000). There are two categories of testing methods that have been suggested in a relatively recent study by Sridharan and Prakash (2016) that could aid the identification of the presence of expandable minerals such as montmorillonite in expansive soils and these are: inferential and mineralogical methods. The inferential testing is essentially composed of a direct method that is used in the calibration of soil's index properties and an indirect method such as the oedometer free swell tests and the free swell tests (free swell ratio, differential free swell and the free swell value). The identification of expansive soils using the mineralogical method consists of techniques such as differential thermal analysis, scanning electron microscopy (SEM), X-ray diffractometry, chemical analysis and the dye adsorption method. Even though there is a growing science concerning the use of the mineralogical methods to explain the mechanism of swelling, this technique of identification may be restricted to only the research laboratory rather than in the field of practice in characterising the expansiveness of soils. The other challenge of using the mineralogical methods relates to the cost of acquisition of the equipment or apparatus and technical know in their usage, operation and interpretation of the results (Sridharan and Prakash 2000, 2016).

4.3.1 Mineralogical Method of Soil Identification

4.3.1.1 SEM and XRD

Figure 4.4 shows the micrographs of the samples (at optimum moisture content) revealing their pore and aggregate structures. These micrographs confirm that the microfabric of Na-montmorillonite rich clays are likely to be characterised by dispersed and undulating filmy particles as compared to the low swelling kaolin rich china clay which possesses more of a leaf-like arrangement (Latifi et al. 2015). As the montmorillonite content increases, the compacted mixtures tend to exhibit more aggregated and concentrated clusters of clay particles. This behaviour could result in impervious layers thus giving rise to an initial low rate of swelling upon moisture ingress but with the ultimate free swelling taking a longer time to be completed. This phenomenon also invariably suggests that as the kaolin content increases, the pore structure becomes more interlinked resulting in high permeability rate at the initial and primary stages of expansion and reduced time of swelling when inundated.

Given that only two pure minerals were blended to achieve the natural soil samples, it was only necessary to take the XRD on representative soil samples, that is **Soil A** containing only the kaolinite and **Soil E** containing 75% of the bentonite. The justification provided for this option is that the expense on resources was further decreased by not carrying out the XRD on the remaining soils. As mentioned in the section 3.4.1.3 of the previous chapter, few challenges were encountered in the determination of some of the phases involved in the mixtures due to the unique structures, complexities and the various element compositions of the natural clay. Apart from the presence of a larger amount of kaolinite in S**oil A** used, the XRD also indicates traces of other minerals like quartz, muscovite (mica) and microcline (feldspar) present in the clay (Figure 4.5a). The XRD showing the mineral composition of **Soil E** seems slightly complex. Even though the mineral montmorillonite is present in abundance another mineral called the Loughlinite seems to be also available in larger quantities (Figure 4.5b). The Loughlinite is basically hydrous sodium magnesium silicate that is found mostly in oil shales (Fahney et al. 1960).

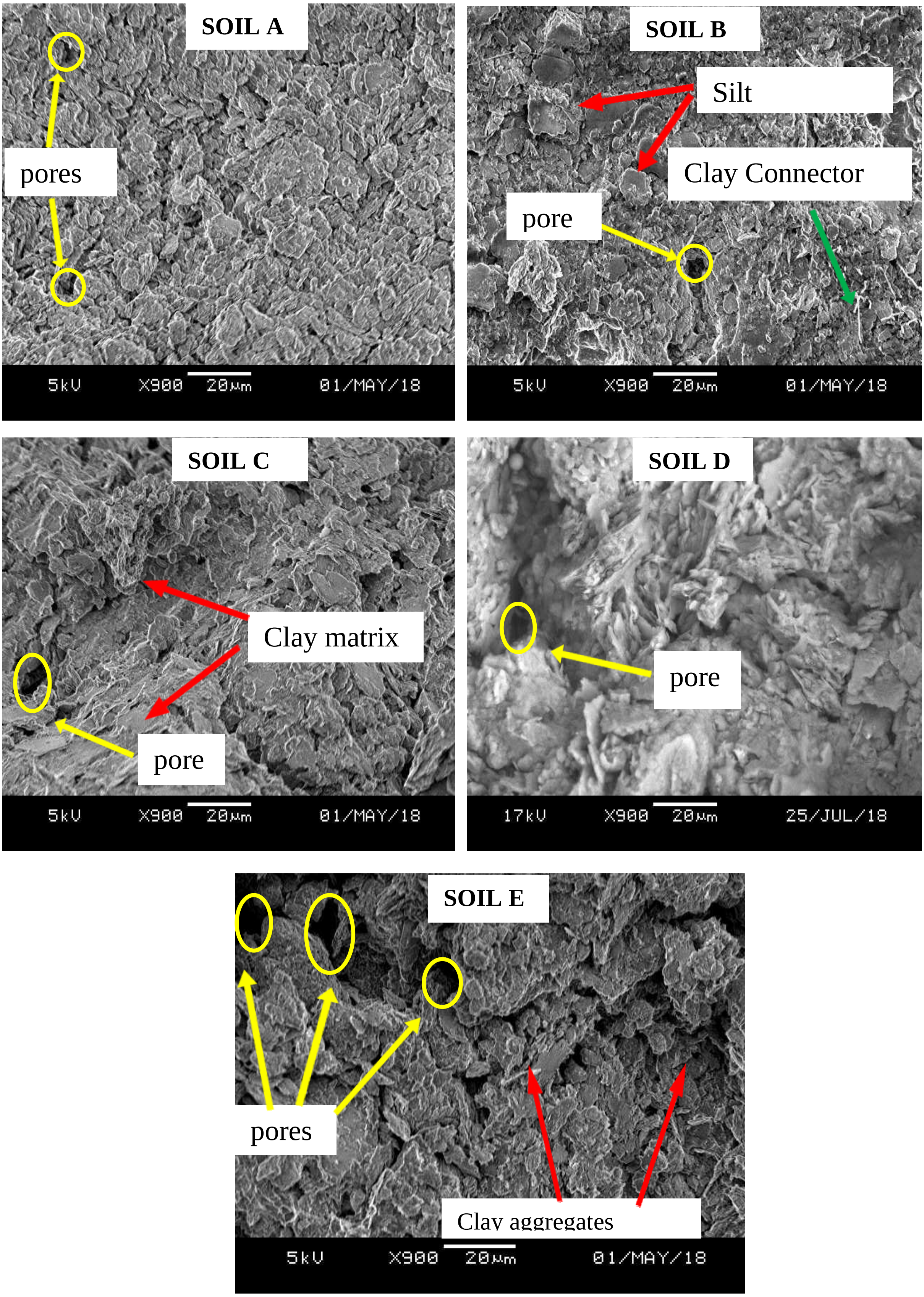


Figure 4.4 Microstructure of natural soils

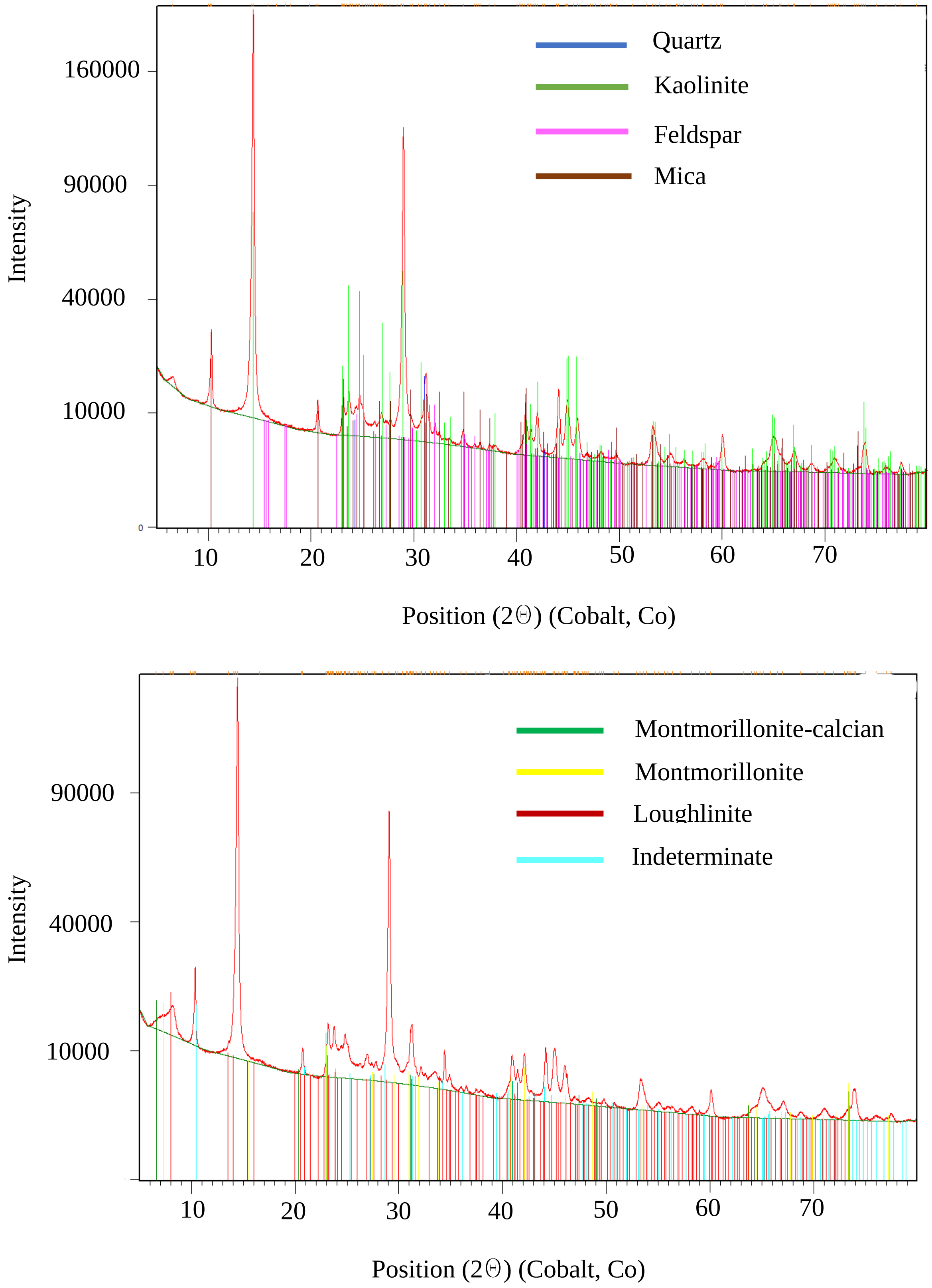


Figure 4.5 X-ray diffraction showing mineralogical properties of soil (a) **Soil A** (b) **Soil E**

4.3.2 Inferential Method of Soil Identification

Using inferential testing could prove to be rather helpful for the identification of the behaviours of clays in swelling given that the methods (direct and indirect) that are involved are relatively inexpensive and less complicated. Sridharan and Prakash (2000) proposed the free swell ratio which is an inferential method, for the classification of expansive soil swell potential. The free swell ratio method has been applied in recent studies by Wubshet and Tadesse (2014); Soltani et al. (2018) Free swell ratio can be defined as the ratio of equilibrium volume of sediment of 10g of soil passing the 425 mm sieve having been oven-dried before soaking in water to that which is soaked in kerosene/carbon tetrachloride (Sridharan and Prakash 2000).

The inferential method based on the results obtained from the index property tests and engineering tests are presented and discussed as follows:

4.3.2.1 Modified plasticity chart

The Unified Soil Classification System (USCS) of the natural soils according to the Casagrande Plasticity chart as seen previously in Figure 4.2 indicated that the clays may possess the propensity to swell as the amount of bentonite increases in the mixture. However, the modified plasticity chart shown in Figure 4.6 depicts the swelling potential of the clays in terms of their degree of expansiveness (Dakshanamurthy and Raman 1973; Chen 1988). Superimposing the values of plasticity index and liquid limits on Figure 4.6 indicates that **Soil A** has a ‘medium’ potential to swell whereas **Soil B** possess a ‘high’ potential to swell according to Dakshanamurthy and Raman (1973) but ‘very high’ swelling potential according to the description given by Chen (1988). Soils **C, D and E** are all ‘very high’ swelling soils.

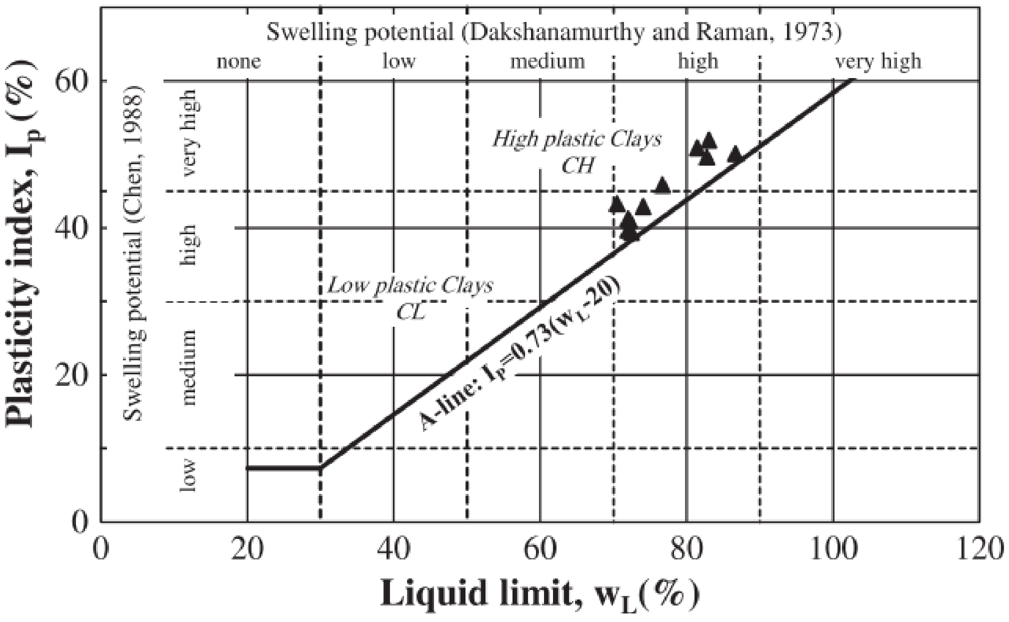


Figure 4.6 Modified plasticity chart for soil swelling (adapted from Dakshanamurthy and Raman (1973; Chen (1988))

4.3.2.2 Method based on plasticity index

The Plasticity Index of soils can be relied on as an indicator for the ability of clays to swell. Some of the various schemes proposed by authors to recognise clay soil expansion based on the PI is given in Table 4.2. As could be observed, the soils utilised in this study tend to range from ‘medium’ to ‘very high’ swelling potential.

Table 4.2 Degree of expansion based on PI

Soil	(PI) (%)	Expansion potential				
		(Chen 1975)	(Dakshanamurthy 1978)	(Peck et al. 1974)	(Holtz and Gibbs 1956)	(IS 1970)
Soil A	28	M	M	M	M	H
Soil B	48	H	H	H	H	VH
Soil C	82	VH	VH	VH	VH	VH
Soil D	164	VH	VH	VH	VH	VH
Soil E	213	VH	VH	VH	VH	VH

M = Medium; H = High; VH = Very high

4.3.2.3 Method based on liquid limit

The liquid limit is an upper bound plasticity limit or moisture content of a soil. It is regarded as the moisture-holding capacity of a soil which in turn could serve as a measure of the swell potential of soils (Sridharan and Prakash 2016). Some of the classification schemes propounded by authors and international standards for the recognition of the expansivity of soils based on liquid limit are shown in Table 4.3. According to Table 4.3, the degree of expansiveness of the soils would range from ‘medium’ to ‘very high’.

Table 4.3 Degree of expansion based on LL

Soil	(LL) (%)	Expansion Potential		
		(Chen 1975)	(IS 1970)	(Snethen et al. 1977)
Soil A	58	H	H	M
Soil B	85	VH	VH	H
Soil C	130	VH	VH	VH
Soil D	222	VH	VH	VH
Soil E	285	VH	VH	VH

M = Medium; H = High; VH = Very high

4.3.2.4 Oedometer method

The oedometer method is a direct method of assessment of the swell potential of soils and has been regarded as the most reliable and useful technique (Winterkorn and Fang 1986). Slightly different criteria for the degree of soil swelling using the oedometer have been suggested. For instance, by referring to the United States Bureau of Reclamation (Holtz and Gibbs 1956), the rationale for the classification of swelling soils is the total change in volume of a soil from the state of complete dryness to full saturation under a token load of 7kPa in an oedometer. The criterion given by Seed et al. (1962) is that per cent swell which is exhibited by a soil compacted at optimum conditions (MDD and OMC) under a token surcharge of 7kPa. Using these criteria, the degree of expansion of the soils from this research are given in Table 4.4. Utilizing the data of oedometer tests on the five soils it is clear from Table 4.4 that a somewhat upper- bound swelling potential is that proposed by Seed et al. (1962).

Table 4.4 Degree of expansion based on oedometer

Soil	Percent oedometer expansion (or free swell)	
	(Holtz and Gibbs 1956)	(Seed et al. 1962)
Soil A	M	H
Soil B	M	H
Soil C	H	VH
Soil D	VH	VH
Soil E	VH	VH

M = Medium; H = High; VH = Very high

It is also pertinent to state that some of the inferential methods of testing especially those relying on the Atterberg limits to identify the degree of swelling in expansive soils are fought with some drawbacks not least, a possible overestimation of the expansion potential (Sridharan and Prakash 2000). The reasons for this are that the mechanisms that influence the liquid limit of say kaolinites and montmorillonites minerals are different. While the liquid limit of clays with the mineral montmorillonite is controlled by the double layer thickness, those containing kaolinite seems to be influenced by the inter-particle attractive forces and the arrangement of particles within the clay matrix (Sridharan et al. 1986, 1988). Therefore, higher liquid limits do not necessarily suggest expansive soils having the montmorillonite mineral. Research has also indicated that two separate soils possessing different mineral characteristics and intra-aggregate pore distributions could have similar consistency limit properties such as the liquid limit and yet contrasting swell or shrinkage characteristics (Chittoori and Puppala 2011; Pedarla et al. 2013; Puppala et al. 2016). This is why the reliance on only the liquid limit and in effect some of the other related index properties, without any due consideration of the mineralogy of the clay, may be misleading in indicating the degree of expansion in a clay. On the other hand, even though it could be time-consuming depending on the nature of the expansive clay, most authors seem to agree that there is a very high level of consistency in the free swell methods which is carried out using the oedometer test for instance.

4.4 DESCRIPTION OF OEDOMETER SOIL SWELLING

Results of vertical swell strains showing the expansion path of the swelling soils are shown in Figure. 4.7. It can be seen that the expansive clays exhibit maximum swelling strains and reduced permeability as also clearly demonstrated in Figure. 4.8. This is due to the presence of high quantities of montmorillonite content in the expansive clays. Vertical swell strains reached their stable peak within approximately 2, 8, 24, 50 and 333hrs with the increase in montmorillonite content of 0%, 10%, 25%, 50% and 75% respectively. The increase in the amount of montmorillonite provides the impetus for the increase in the volume of the micropores during the swelling process. Dakshanamurthy (1978) and Komine and Ogata (1994) used an expansion-deformation model to describe the water uptake mechanism of non-stabilised swelling clays. Three phases of swelling are said to be prominent during clay soil swelling (Figure 4.9). The initial phase is usually less than 10% of the overall swelling. This stage involves inter-crystalline swelling where water is said to be adsorbed in monolayers on the surface of the mineral montmorillonite and immediately followed by swelling as a result of the double-layer force of repulsion. The initial phase in the case of this research would be due to the swelling of the bentonite clays within the voids of the low-swelling kaolin clay. The second phase is regarded as the primary phase whereby swelling tends to occur at a much faster rate when the voids can no longer hold further clay particle expansion. More than 75% of the total swelling is said to occur in the primary phase. For kaolin-bentonite mixtures, the primary phase of swell varies between 200 and 1000 minutes (Hashim and Muntohar 2006). It has been suggested that the rate of swelling in the primary stage (slope portion of the curves) could be a principal factor in the prediction of swell in relation to the permeability since most of the soil swelling (about 90%) occur in this region (Güneyli 2017). In addition, the time of completion of this process could be very critical in the design of engineering structures for which backfill applications are required such as railways, motorways, water and energy transmission lines (Güneyli 2017). The last phase referred to as the secondary (or plateau) phase occurs with slow but continued swelling.

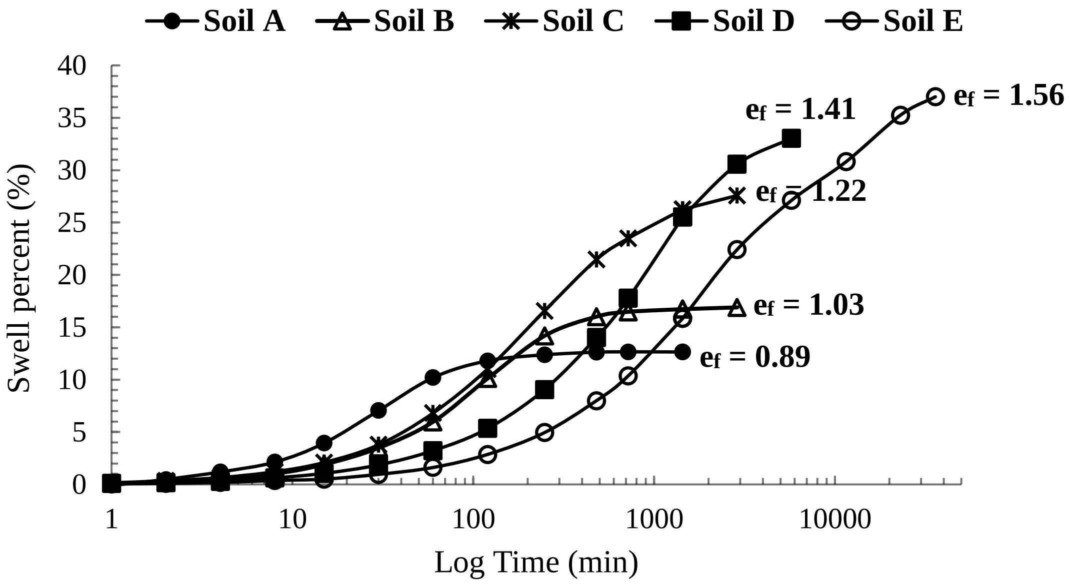


Figure 4.7 Vertical swell percent of natural soils

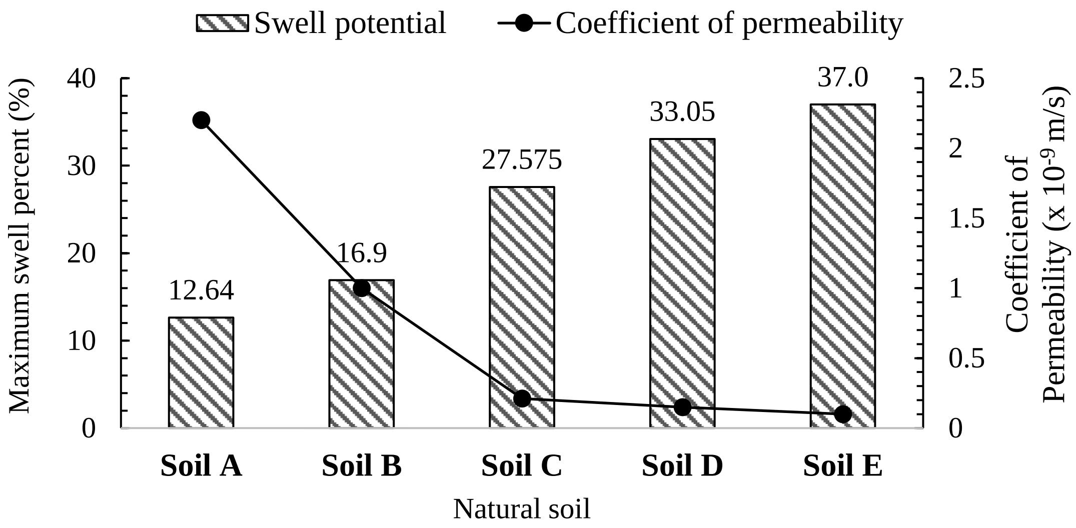


Figure 4.8 Maximum vertical swell percent and permeability of natural soils

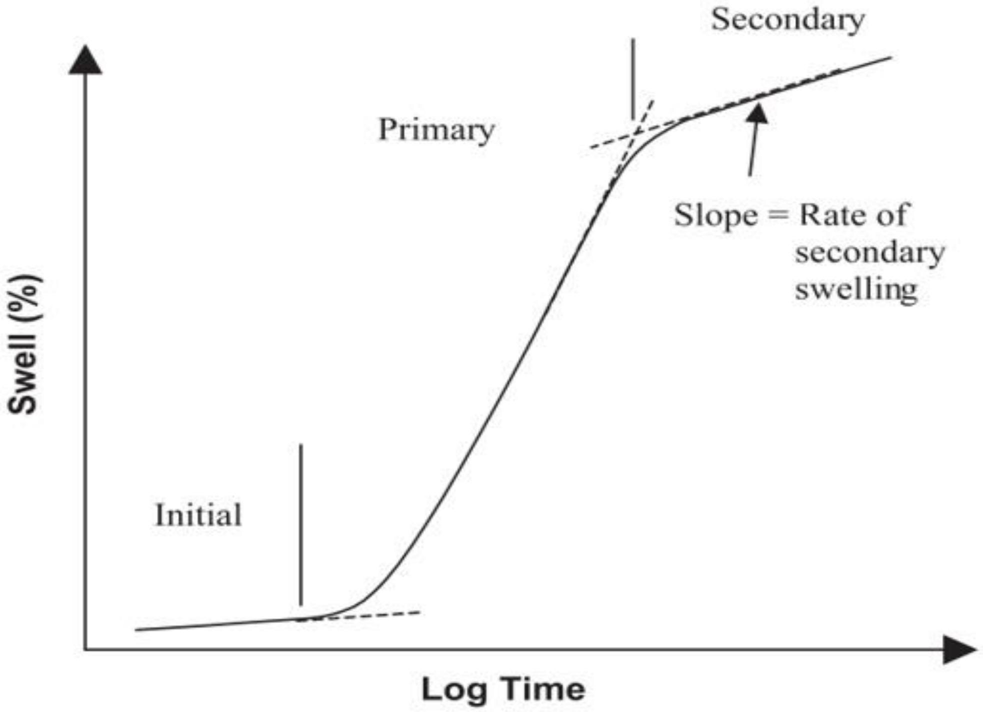


Figure 4.9 Description of soil swelling Phases (modified after Sridharan and Gurtug (2004))

4.5 NOVEL ASSESSMENT OF SWELL POTENTIAL USING THE MASTERSIZER

Particle size analysis relying on the particle size distribution is a resourceful tool that is more frequently used to classify and characterise soils. A novel application of this great tool is proposed in this research that allows it to be used for the identification and prediction of the expansion potential of clays in relation to both the physical and mineralogical properties as noted previously.

Following the laser diffractometry test, the clay particles were graded by Laser Granulometry and the percentage sizes calculated by GRADISTAT program. This program uses Method of Moments in Microsoft Visual Basic programming language to calculate the grain size parameters (Krumbein and Pettijohn 1938). Soil A (K100SB0) was determined as being uniformly graded while **Soils B-E** (K90SB90, K75SB25, K50SB50, K25SB75)) were adjudged as gap-graded as shown in Figure. 4.10. A precise fitting curve for particle-size distribution was achieved for the unimodal and bimodal particle size functions based on the equation proposed by Fredlund and Xing (1994) but modified as in Equations 4.1 and 4.2 respectively for particle-sizes. This was done to ensure a proper analysis of two predominant major size fractions namely clay and silt whose sizes are very small and sometimes invisible to the naked eyes. The fitting parameters from the unimodal and bimodal fits are presented in Table 3 and it could be observed that the coefficients of determination, R^2 resulting from the curve-fitting are very close to unity.

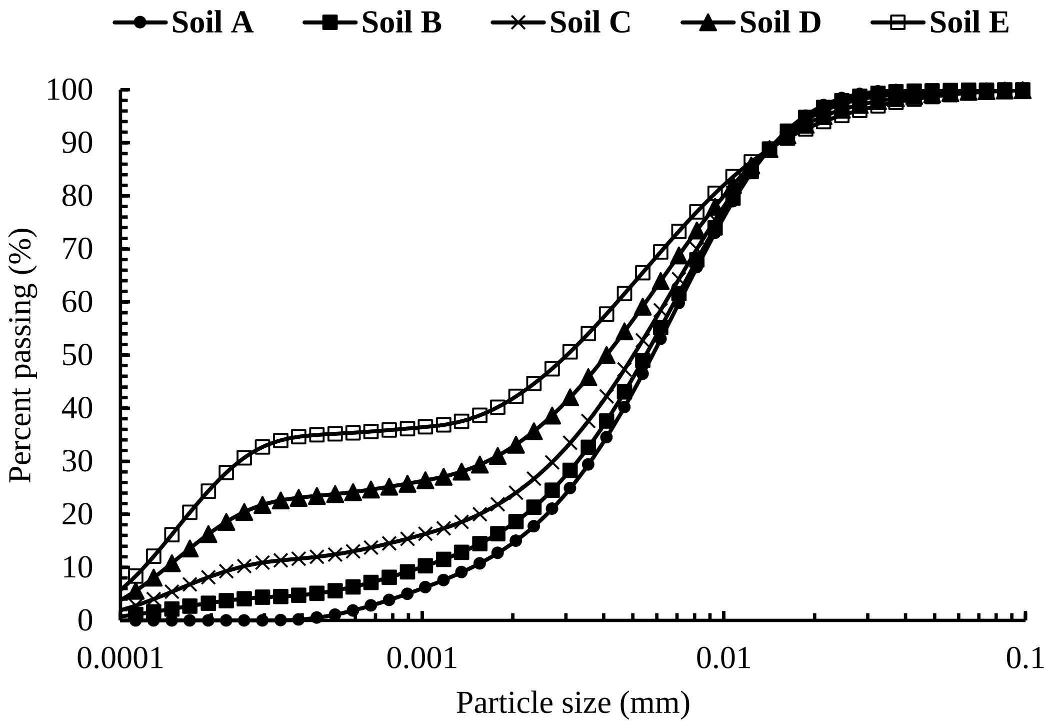


Figure 4.10 Particle size distribution curve of the expansive clay samples.

$$P_p(d) = \frac{1}{\left\{In\left[exp(1)+\left(\frac{a_{gr}}{d}\right)^{n_{gr}}\right]\right\}^{m_{gr}}} \times \left\{1-\left[\frac{In\left(1+\frac{h_{gr}}{d}\right)}{In\left(1+\frac{h_{gr}}{d_{min}}\right)}\right]^{7}\right\} \tag{4.1}$$

$$P_p(d) = \left\{w\left[\frac{1}{\left\{In\left[exp(1)+\left(\frac{a_{bi}}{d}\right)^{n_{bi}}\right]\right\}^{mbi}}\right] + (1+w)\times\left[\frac{1}{\left\{In\left[exp(1)+\left(\frac{j_{bi}}{d}\right)^{k_{bi}}\right]\right\}^{l_{bi}}}\right]\right\} \times \left[1- \left(\frac{In\left(1+\frac{h_{rbi}}{d}\right)}{In\left(1+\frac{h_{rbi}}{d_{min}}\right)}\right)^{7}\right] \tag{4.2}$$

Where:

P*p* (d) = percent by weight of particles passing

a_{gr} = parameter representing the inflection point on the particle-size distribution curve (mm).

n_{gr} = parameter that relates to the steepest slope on the particle-size distribution curve

m_{gr} = parameter designating the shape of the particle-size distribution curve as it approaches the fine-grained region

h_{gr} = parameter related to the particle size in the fine-grained region and is referred to as the residual particle size (mm)

d_{min} = diameter of the minimum allowable size particle (mm)

d = diameter of any particle size under consideration

a_{bi} = parameter related to the first breaking point along the coarse-grained potion of the particle-size distribution curve

n_{bi} = parameter related to the first steep slope along the coarse-grained portion of the curve

m_{bi} = parameter representing the first part of the particle-size distribution curve

j_{bi} = parameter that relates to the second breaking point pf the curve

k_{bi} = parameter that relates to the second steep slope along the curve

l_{bi} = parameter that relates to the second shape along the curve

h_{rbi} = parameter that relates to the amount of fines in the soil.

w = weighting factor for each of the sub-curves under consideration.

Table 4.5 Particle size analysis fitting parameters.

Clay sample	Unimodal fitting parameter					
	a_{gr}	n_{gr}	m_{gr}	h_{gr}	d_{min}	R^2
	mm			*mm*	*mm*	
Soil A	0.0065	2.1499	2.0629	0.00001	0.00001	0.9988

Clay sample	Bimodal fitting parameter									
	a_{bi}	n_{bi}	m_{bi}	j_{bi}	k_{bi}	l_{bi}	w	h_{rbi}	d_{min}	R^2
	mm			*mm*				*mm*	*mm*	
Soil B	0.0081	3.6763	9.4264	0.0049	2.7890	1.3529	0.3209	0.00001	0.00001	0.9984
Soil C	0.0062	2.0617	2.2362	0.0001	2.9332	4.0625	0.8756	0.00001	0.00001	0.9993
Soil D	0.0050	1.8230	3.0558	0.0001	2.6800	7.0146	0.7534	0.00001	0.00001	0.9997
Soil E	0.0033	1.5536	5.1377	0.0001	2.8983	6.2722	0.6414	0.00001	0.00001	0.9998

Grain size analysis undertaken with the Malvern Mastersizer 2000 also enabled the determination of the specific surface area (SSA) of the clay particles. The Mastersizer calculates the SSA as the total area divide by the total weight (Equation 4.3) based on the assumption that the particles are both non-porous and spherical (Malvern 2007). This method of analysis to derive the SSA using the Malvern Mastersizer is in agreement with that obtained by Latifi et al. (2016) using the Nitrogen-based Brunauer-Emmett-Teller (N_2-BET) surface area technique.

$$SSA = \frac{6\sum\frac{V_i}{d_i}}{p\sum V_i} = \frac{6}{pD} \tag{4.3}$$

Where:

V_i = relative volume in class *i*

d_i = mean class diameter *i*

p = particle density

D = surface weighted mean (or surface area moment mean diameter or sauter mean)

Figure. 4.11 presents some relevant statistics from the particle-size analysis. A gradual reduction in the median of particle sizes, mean particle sizes of each whole sample and the effective size particles (D_{10} sizes) towards the finer fractions as the amount of montmorillonite increases could be observed. The median of the particle distribution fluctuates around 8µm which corresponds to ‘medium silt’ size particles (6-20µm). The highest value of the mean particle size distribution as observed for Soil A is about 8.2µm which is to be expected given that kaolinite is siltier as compared to the more clayey bentonite. This value reduces by approximately 67% as the percentage of the sodium bentonite increases to 75%. Meanwhile, the effective size for all the clays remains within the clay region (<2µm). It is interesting to note as would also be shown subsequently that the clay-sized particles seem to have more influence on the swell behaviour of the expansive clays.

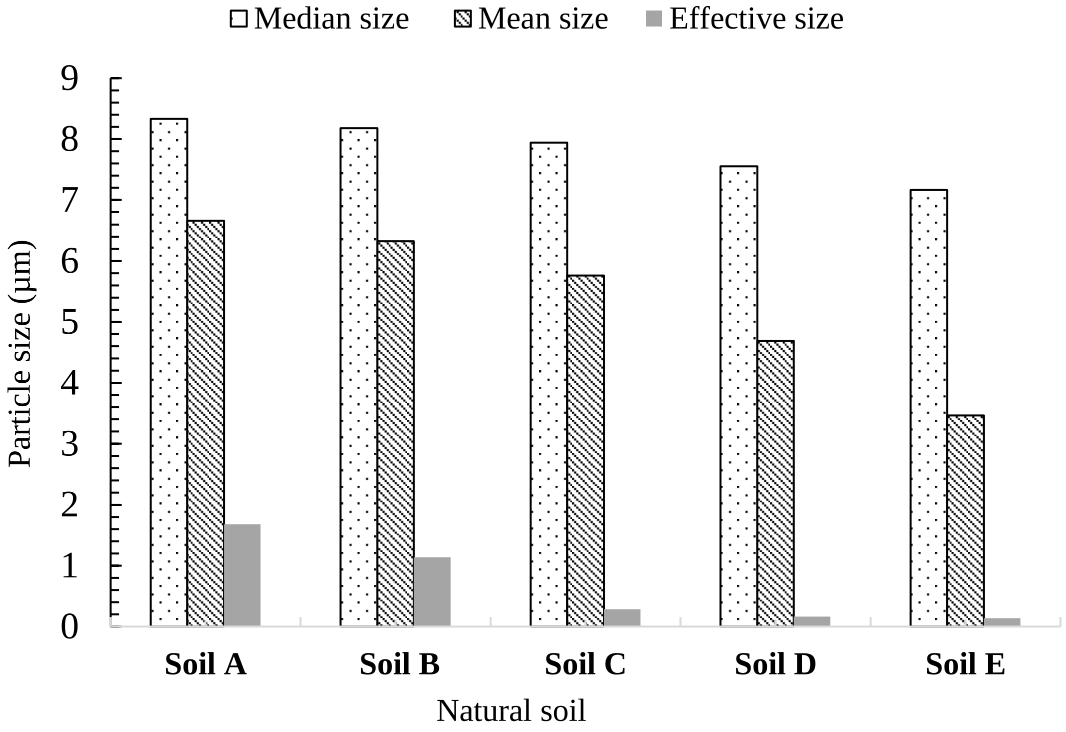


Figure 4.11 Expansive clay particle size statistics.

4.5.1 Correlation of Particle Size with Clay Properties

Basma (1993) in his study of 128 soils from Jordan concluded through a regression analysis that clay content and plasticity index have very high influence on the studied soils' swell potential. The surface area of expansive clay minerals is also regarded as an influential factor in the swelling of expansive clays (Mitchell and Soga 2005). Clay soils possessing very high percentage of montmorillonite mineral tend to have very reactive surface area per given weight of the soil specimen. It therefore follows that soils having high specific surface area (SSA) can retain moisture and hence will experience greater swell potential. Several researchers have attempted to correlate the clay-sized fraction (<2µm) of soils with swell potential and other soil properties (Seed et al. 1962; Ranganatham and Satyanarayana 1965; Basma 1993; Erguler and Ulusay 2003; Watabe et al. 2011). However, this method may not be entirely rational for predicting the swell potential of soils having predominantly clay and silt fraction distributed uniformly as is the case in this research. The use of correlations involving clay fraction may prove insufficient because just as the clay fraction, the silt fraction may also provide a strong correlation with the plasticity index (PI) suggesting that both clay and silt fractions bear similar influence on PI.

In this research, the particle distribution curve as previously shown in Figure 4.10 was used to critically examine the various size particles in a given clay soil sample and to demonstrate the effect of the particles on the physical and mineralogical factors that influence swell behaviour such as the SSA, PI and activity (A) of the expansive clays reported in literature (Seed et al. 1962; Ranganatham and Satyanarayana 1965; Basma 1993). In order to execute this, the mean sizes of each individual fractions contained in the samples and the mean particle size of each sample considered as a whole both determined by the Malvern Mastersizer were considered. The particle fractions derived from the GRADISTAT programme are categorized into size classes (Blott and Pye 2000) as shown in Table 4.6. Values of the mean sizes of the individual fractions and the mean size (henceforth referred to as d_p) of each whole sample are given in Table 4.7 which also includes, as well as Table 4.8, regression analyses performed to correlate the mean sizes (dependent variables) and the soil properties (PI, SSA and A).

Table 4.6 Expansive clay size class fractions.

Soil fraction	Designation	Size range (µm)
Clay	C	0-2
Very fine silt	VFS	2-4
Fine silt	FS	4-8
Medium silt	MS	8-16
Coarse silt	CS	16-31
Very coarse silt	VCS	31-63

Table 4.7 Expansive clay particle sizes and soil properties.

Clay	Mean sizes							Soil Properties		
	Dependent variable							Independent variable		
	C	VFS	FS	MS	CS	VCS	d_p	PI	SSA	A
				μm				(%)	$(m^2)/g$	-
Soil A	1.03	2.059	4.305	9.089	18.94	37.8	6.663	14	1.7	0.67
Soil B	0.936	3.084	6.09	11.44	21.42	44.49	6.324	48	1.75	1.92
Soil C	0.69	3.079	5.98	11.43	21.58	42.18	5.76	82	6.03	2.73
Soil D	0.475	3.07	5.951	11.43	21.89	43.74	4.691	102	10.35	2.76
Soil E	0.357	3.059	5.91	11.42	22.28	44.54	3.465	191	14.68	4.06

Table 4.8 Regression analysis result.

Dependent variable	Regression Equation	R^2	F	*p*-value
C	0.973PI – 1.233SSA – 0.754A	0.99	308.25	0.042
VFS	-1.829PI – 0.204SSA + 2.722A	0.92	3.80	0.357
FS	-1.772PI – 0.343SSA + 2.752A	0.90	3.00	0.396
MS	-1.800PI – 0.192SSA + 2.692A	0.92	3.76	0.358
CS	-1.249PI +0.011SSA + 2.102A	0.94	4.89	0.318
VCS	-0.419PI – 0.632SSA + 1.749A	0.73	0.921	0.626
d_p	-0.528PI – 0.622SSA + 0.153A	0.99	44.88	0.109

All the mean size classes considered in the analysis seem to give much higher values of the coefficient of determination R^2 when correlated with the PI, SSA and A (Table 4.8). However, the regression analyses result also show that the only statistically significant correlation (*p*-value) considered within 95% confidence interval occurs between the mean size class involving the clay fractions, C and the predictor or independent variables. Hence, it could be said that the presence of the clay-sized particles in a clay samples could potentially affect the behaviour of expansive clay samples in swell given their strong correlation with the PI, SSA and A. Also, the approach presented herein regarding the relationship between the mean sizes of the representative clay fractions and the investigated soil properties appears relatively reasonable in describing the effect of clay fractions on swell.

It should be noted that other physical properties such as moisture content and density could have a strong bearing on the nature of expansiveness of clay soils even though not analysed here. Hence, it is suggested that this proposed method relying on particle size analysis be extended to cover the water content and density of the clays with both used as independent variables in the regression analysis.

4.5.2 Empirical swell prediction incorporating particle sizes

Following the assessment of the natural clay properties based on the method of particle sizing as presented in section 4.5 using regression analysis for the clay soils, an attempt was therefore made to estimate swelling potential by incorporating the average pore sizes of the clays.

4.5.2.1 Clay pore characteristics

The pore sizes and their distribution in clays provide reliable basis for the description of the mechanism of expansive soil water transport and gaseous exchange. An approach adopted from published literatures is utilised to determine the average sizes of pores in the clay samples. The mean pore diameter (d_m) which is calculated for various particle geometries and fabrics in terms of the specific surface area (SSA) and void ratio e, is adopted for the determination of the average size pores in the clay samples (Phadnis and Santamarina 2011). By applying this theory, it is considered that the volume of voids is evenly distributed around the clay particles as a "void layer" of thickness, t. Then the distance between particles, d_m = 2t void is taken as an estimate of the mean or average pore diameter as follows:

$$d_m = \frac{2e}{SSA \times \rho_m} \tag{4.4}$$

Where:

e = void ratio,

SSA = specific surface area (m2/ g),

ρ_m = mass density of the clay mineral (kg/m3)

d_m = mean pore diameter

The following assumptions are made in using Equation 4.4:

- The clay particles are round in shape and with parallel stacking arrangement.
- The compacted samples consist of dispersed fabrics or structure

4.5.2.2 Pore and particle size effect on swell

This study accounts for factors which act to control the macro swelling properties of expansive clays which are the expansive clay particle sizes and pore structure. As such, the swell behaviour of the expansive clays in this research is related to the following terms: mean pore diameter (d_m), mean particle diameter (d_p), and mean clay diameter (d_{clay}).

- *Void ratio*

As previously stated, the particle sizes representing the clay fraction and the pore sizes expressed in terms of the void ratio, density and SSA could potentially influence the swelling of expansive clays. The final void ratio e_f that corresponds to the final swell values of the clays previously shown in Figure 4.7 is used to evaluate the mean diameter of the pores at maximum swell. According to Equation 4.4 therefore, the mean of the maximum size of pore (d_m) at the end of the swell process (i.e. at equilibrium) represents or describes the swell given that the final void ratio e_f at that point is used in the equation. The relationship between important clay soil swell properties (i.e. PI, SSA and A) and the particle sizes as demonstrated earlier revealed the potential significant contribution the mean particle fractions and the clay fraction could have on the overall swell behaviour of clays. Hence, the d_m and the d_{clay} are used to explain the swell characteristics of the expansive clays. The relationship presented in Figure. 4.12 and Figure. 4.13 depicting the variation of d_m with d_p and d_{clay} respectively, shows that despite an increase in the void ratio as the montmorillonite increases, the mean pore size tends to reduce. This explains why the permeability of the samples seems to reduce as the amount of montmorillonite increases as earlier observed (Figure. 4.8). This is also the reason clays with high amounts of montmorillonite take much longer to reach the equilibrium swell. A strong relationship between the d_m and d_p is also noticed even though the mean clay diameter (d_{clay}) seems to produce a higher correlation than the mean particle size (d_p) which represents the entire soil samples. This does clearly validates the dependence of swell on the clay fractions in the samples (Seed et al. 1962; Ranganatham and Satyanarayana 1965; Basma 1993). Furthermore, the model Equations 4.5 and 4.6 given in terms of the d_m, d_p and d_{clay} demonstrates that knowledge of either the average size of particle of the whole soil sample or the average size of the clay fraction (d_{clay}) in a soil sample can enable a direct determination of the average pore sizes (d_m) and an indirect evaluation of the final void ratio at maximum swell respectively.

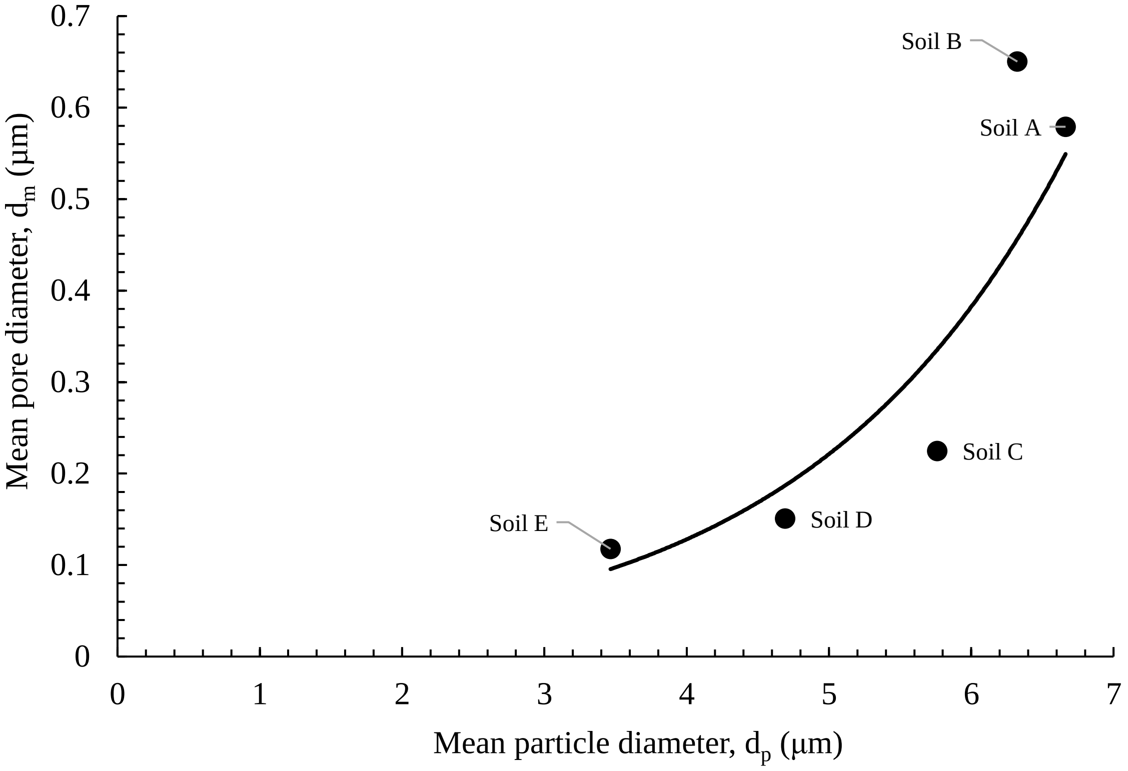
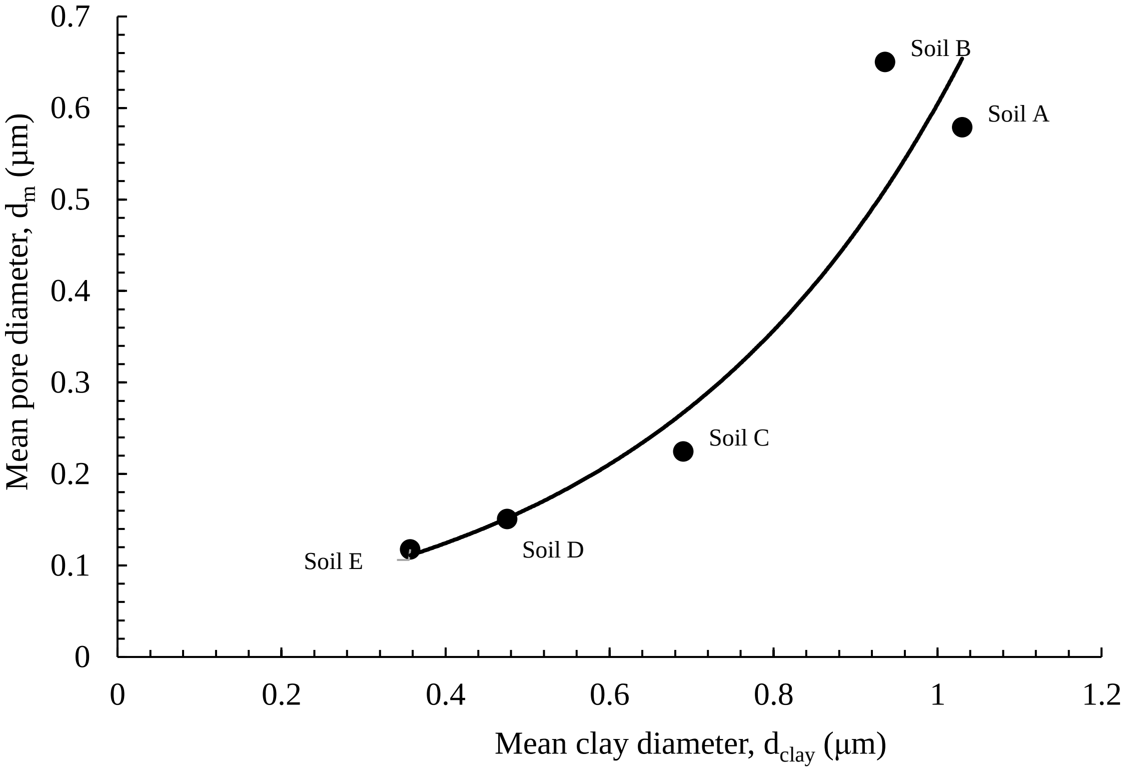


Figure 4.12 Variation of dm with dp for the clay samples at maximum swell



4.13 Variation of dm with d_{clay} for the clay samples at maximum swell.

$$d_m = 0.0144e^{0.55d_p} \qquad R = 0.85 \qquad (4.5)$$

$$d_m = 0.0434e^{2.63d_{clay}} \qquad R = 0.96 \qquad (4.6)$$

4.5.3 Verification of the pore and particle size approach

Pedarla et al. (2016) attempted an evaluation of the swell characteristics of eight natural expansive clays acquired from different parts of the United States by relying on the pore size distribution and the SSA of the soils. They used a hypothetical parameter known as the total surface area ratio (TSAR) in the estimation of swell. TSAR was defined as the ratio of the total surface area (TSA) derived from clay fraction in the soils to the total pore surface area (TPA). Chemical ethylene glycol monoethyl ether (EGME) method was employed to determine the SSA while the micro-internal pore distribution was measured using the mercury intrusion porosimetry technique (MIP). Both the TSA and the TPA were analytically derived from the measurements using the EGME and the MIP methods. Table 4.9 presents the engineering characteristics of the clays used by the authors. The one-dimensional swell strain of the samples compacted at optimum moisture content and maximum dry density was determined under a seating load of 7 kPa.

Table 4.9 Properties of the tested soils by Pedarla et al. (2016).

Soil	PI	Vertical swell strain (%)
Keller	11	7.9
Oklahoma	21	4.8
Anthem	27	5.8
San Diego	28	4.4
Burleson	37	8.8
Colorado	42	12.0
San Antonio	43	10.2

Variation of the TSAR with the swell potential of the specimens is shown in Figure 4.14 including the mean clay ratio (MCR) obtained using the concept from this research. The X-axis of Figure 4.14 plot combines the TSAR and the mean clay ratio (MCR) defined as the ratio of the mean clay diameter (d_{clay}) to the mean pore diameter (d_m) determined from this research. The TSAR and the MCR are evaluated at the initial soil placement conditions. As could be observed, the present concept from this research relying on the mean sizes of the particles and pores is in agreement with the model proposed by Pedarla et al. (2016) and does possess a good relationship with the swell potential. The slight variation in both curves could be attributed to the difference in the soil properties used for instance, the higher PI values of the samples from this research as compared to those of Pedarla et al. (2016). Also, it should be noted that the vertical swell strain of a laterally confined soil sample is dependent on the amount of the seating load which in the case of this research, 5 kPa was the seating pressure while 7kPa load was used by (Pedarla et al. 2016). Hence, irrespective of the contrast in methods used, it could be said that the concept presented in this research can be relied upon to describe swell properties of clays.

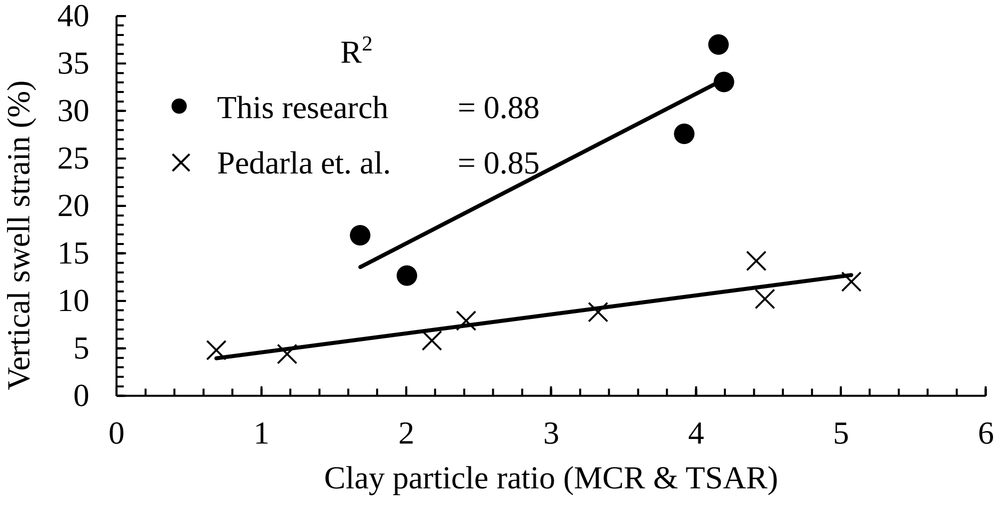


Figure 4.14 Validation of approach based on the dependence of predicted swell on pore and particle sizes.

4.6 NATURAL CLAY CONSOLIDATION

The e-log p curve of Figure 4.15 shows a typical pattern of reduced ratio of voids as the effective pressure increased during the stages of loading, due to the expelling of interstitial fluids and rearrangement of the soil particles into a much denser configuration resulting from the reduction of volume as a response to the applied stress. An obvious increase in both the initial and final void ratios of the samples as the proportion of the bentonite increased can also be observed.

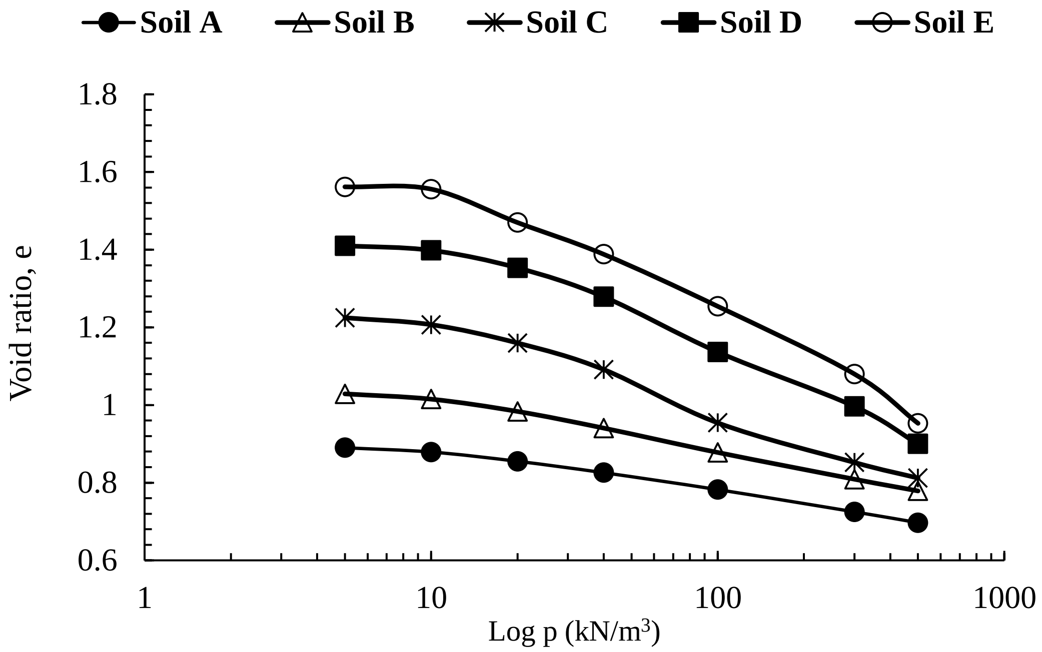


Figure 4.15 Soil consolidation showing variation of void ratio with effective stress

A further description of the compression behaviour of the natural soils in terms of compression index, Cc is shown plotted in Figure 4.16. Compression index (Cc) is defined as the gradient of the linear portion of the e-log p curve. This is an important index in soil consolidation because it indicates the extent of compression that is undergone by soils as the consolidation pressure increases. A steady rising trend in the compression index that corresponds to an increase in the bentonite content from 0 to 75% is noticed for the untreated mixes. This phenomenon is due to an increase in the bentonite fines in the mixture thus giving a great degree of compressibility. Factors such as mineralogy of clay, bentonite type, exchangeable cations and pore fluid chemistry are known to influence the diffuse double layer (DDL) thickness which can consequently generate a significant influence on the compressibility properties of clays (Nalbantoglu and Tuncer 2001; Jha and Sivapullaiah 2015a). Sreedharan and Puvvadi (2013) reported that the compressibility of any given clay having a given pore fluid is controlled generally by the net pressure of repulsion which is associated with the evolution of an electrical diffused double layer surrounding the particles of clay. As the value of effective stress increases and tends towards equilibrium at a reduced ratio of voids, the repulsive pressure would decrease as the bentonite proportion decreases.

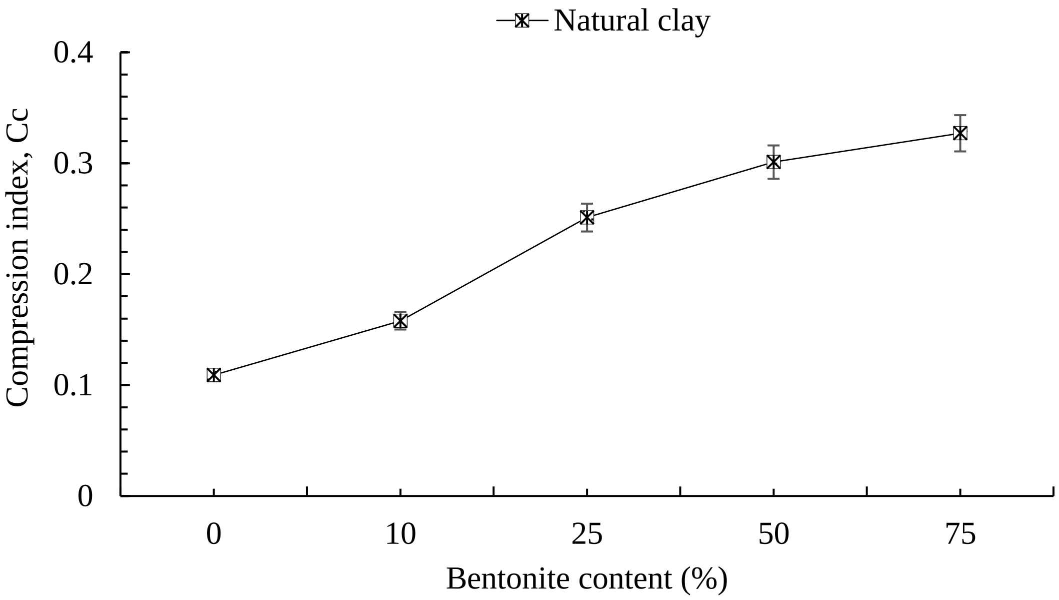


Figure 4.16 Variation of compression index with bentonite content

4.7 NATURAL CLAY STRENGTH

Bentonite addition to the kaolin produced variation in the unconfined compressive strength (UCS) of the compacted natural as the UCS seems to fluctuate between 190 and 270kPa as seen in Figure 4.17. However, of interest is the pattern of failure of the soils. There seems not to be any clear peak strength for the **Soils B-E** this is because of the very elastic or ductile nature of the soils resulting from the presence of bentonite. Whereas, **Soil A** experiences a brittle failure with clear points of failure at the strain of about 2%.

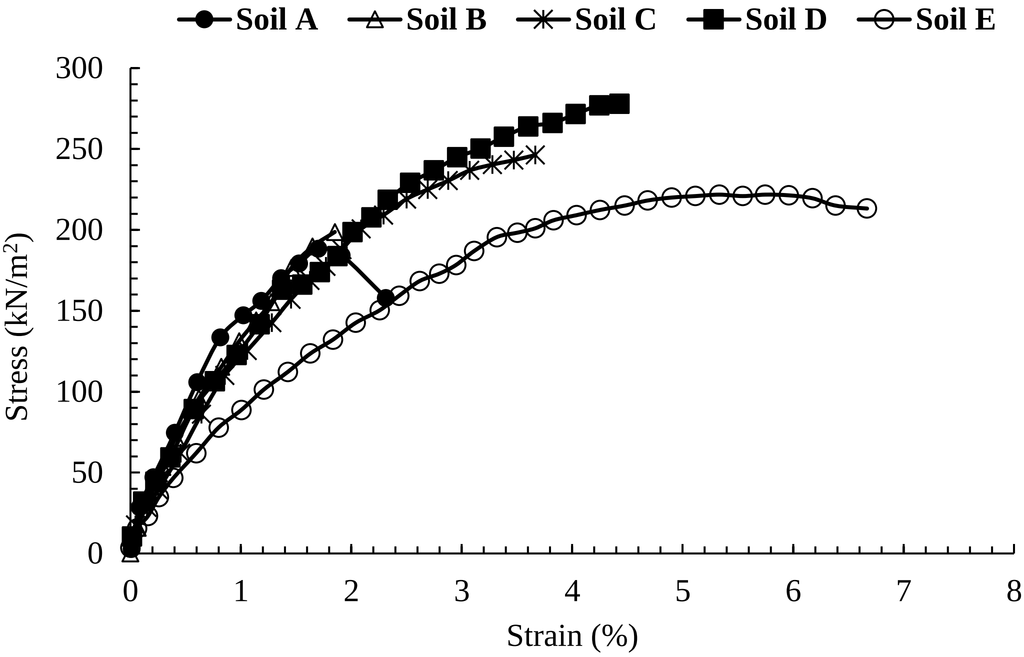


Figure 4.17 Stress-strain curve of natural soils

SUMMARY

This chapter's discussions involved the identification and classification of the physical, microstructural, geotechnical characteristics of the natural soils used in this study. The following are some of the major outputs of this chapter:

i. By adopting the Unified Soil Classification System, the test data obtained from the index property testing such as the Atterberg limit test an superimposed on the Casagrande plasticity chart revealed that the clays used varied from being inorganic clays of low plasticity (CL) to inorganic clays of high plasticity (CH) especially as the proportion of the bentonite increased in the clays. The standard proctor compaction test to determine the relationship between the soils' dry density and moisture content indicates a decrease in the maximum dry density (MDD) and a corresponding increase in the optimum moisture content as the amount of bentonite or the percentage of clay fines increased in the soils.

ii. The two categories of testing methods suggested to aid an identification of the presence of expandable minerals montmorillonite in expansive namely: inferential method and mineralogical methods were also employed to classify the soils according to their degree of swelling. A mineralogical method using the scanning electron micrograph (SEM) showed that the microfabric of the clays rich in Na-montmorillonite rich clays is characterised by dispersed and undulating filmy particles as compared to the low swelling kaolin rich china clay with more of a leaf-like arrangement. This suggests that as the kaolin content increases or with a decrease in the bentonite content of the clays, the pore structure become more interlinked and could result in high permeability rate at the initial and stages of expansion and reduced time of swelling when inundated with water. On the other hand, some of the inferential methods such as the Atterberg limits tests that rely on an indirect assessment of the degree of swelling suggest a variation from medium to very high swelling with an increase in the bentonite content.

iii. A novel application of particle size analysis using the Mastersizer and a visual basic programme (or GRADISTAT) enabled as identification and prediction of the expansion potential of the clays in relation to both the physical and mineralogical properties as noted previously. Regression analysis revealed that the mean size classes of the clays provided higher values of the coefficient of determination R^2 when correlated with properties such as plasticity index, specific surface area and soil activity. However, the regression analyses result also revealed that the only statistically significant correlation (*p*-value) considered within 95% confidence interval occurs between the mean size class involving the clay fractions, C and the predictor or independent variables. Hence, it could be said that the presence of the clay-sized particles in a clay sample can potentially affect the behaviour of expansive clay samples in swell given their strong correlation with the often-used swell indicators (plasticity index, specific surface area and soil activity).

CHAPTER 5

EVALUATION OF ENGINEERING PROPERTIES OF SOILS STABILISED BY OPC

5.1 GENERAL

Having established the properties of the natural clays relevant to this research in chapter 4, this chapter will be dedicated to the performance evaluation of engineering properties of the clays stabilised by OPC used as a sole binder. An identification of the properties of the OPC used are first carried out followed by a discussion to establish the quantity of OPC required to stabilise the clays according to standard recommended practices. The sections following will then involve analyses of the geotechnical engineering properties namely; consistency limit, swell, consolidation and strength of the stabilised soils. Subsequent comparison of these properties on the five stabilised clays will then inform the decision made as to how to proceed for further stabilisation of the clays by partial replacement with the environmentally friendly by-product additives (BPA) in the chapter following.

5.2 PARTICLE DISTRIBUTION OF OPC

The OPC used for the stabilisation of the expansive soils is a high strength (52,5N) cement and is also suitable for mortar, concrete and rendering and screeding where higher early strength is mostly required especially in a temperate climate. According to its manufacturers (Hanson Heidelberg group, UK), this OPC depends mostly for its production on the use of raw material resources with efficient dry-process kilns with the benefit of lower consumption of energy. Using this process, about 60% of recycled and non-fossil fuels are burnt up. Much like all the soils used, up to 100% of the OPC particles passes the 0.425mm sieve size (Figure 5.1). Hence, this OPC is seems to be slightly poorly graded and can potentially mix with the soils and fill up most of the voids thus enhancing their engineering properties upon stabilisation.

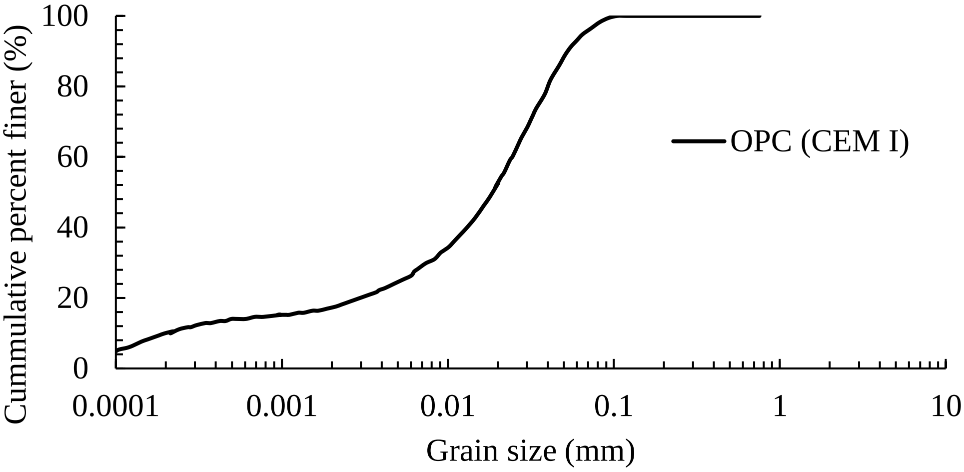


Figure 5.1 Particle size distribution of OPC

5.3 OPC-SOIL STABILISATION

In the consideration of the mix design for soil stabilisation by OPC, the content of OPC in the soil plays an important role in the determination of the properties of the resulting stabilised product. Depending on the proportion used, OPC has been known to alter the plasticity, strength, volume change, elasticity properties, resist the influence of freeze-thaw and wet-dry cycles, resist frost heaving and contribute to many other improved behaviours in various degrees and for different soils (Erdal Cokca 2001; Yilmaz and Civelekoglu 2009; Sivapullaiah and Lakshmikantha 2010; Consoli et al. 2010; Abdelkader et al. 2013; Asma Muhmed 2013; Raftari et al. 2014; Kechouane and Nechnech 2014; Kumar et al. 2014; Tilak B et al. 2015; Latifi et al. 2015; Iravanian and Bilsel 2016; Alrubaye et al. 2017; Solanki et al. 2017; Jamsawang et al. 2017). Hence, selecting the amount of OPC to be used in soil stabilisation does depend on the property of the soil and the type of improvement that is desired. For instance, the use of relatively small amounts of OPC can modify the properties of a soil by causing a reduction in its plasticity or by decreasing the rate of volume change with moisture variation. Using OPC in this manner to cause a change in basic soil properties is mostly referred to as “soil modification” (Bell 1993). On the other hand, when the requirement is to significantly improve the strength and probably durability properties, then very large amounts of the OPC will be necessary and this would be referred to as “soil stabilisation” (Bell 1993). Published research literatures, manuals and handbooks have provided recommended guidelines as to the optimum quantity of cement content needed in soil stabilisation (Maclean and Lewis 1945; Chen 1975; PCA 1992; Bell 1993; Ouhadi et al. 2014; Abbey et al. 2016). Depending on the type of soil, Table 5.1 indicates the amount of OPC proposed to be used in stabilisation according to the American Concrete Institute (ACI). It is expected that as the amount of OPC increases in the mix, the engineering properties should increase, however, care has to be taken because there could be a risk of shrinkage cracking (Wu 2015).

Table 5.1 Typical cement proportions required for treatment of various soils (modified after (ACI 1990))

USCS	% of cement required (by dry mass of soil)
GW, GP, GM, SW, SP, SM	3 – 5
GM, GP, SM, SP	5 – 8
GM, GC, SM, SC	5 – 9
SP	7 – 11
CL, ML	7 – 12
ML, MH, CH	8 – 13
CL, CH	9 – 15
MH, CH	10 – 16

5.3.1 Soil-OPC Mix Design

The five clays were sampled in their natural state and thoroughly mixed dry with the OPC. 8% of OPC calculated by dry weight of the clays was added to the clays. This quantity of OPC was chosen based on some already established procedures and recommendations in literature (Table 5.1) for the enhancement of the engineering qualities considered in this study. Figure 5.2 is a flow chart indicating the OPC proportion utilised.

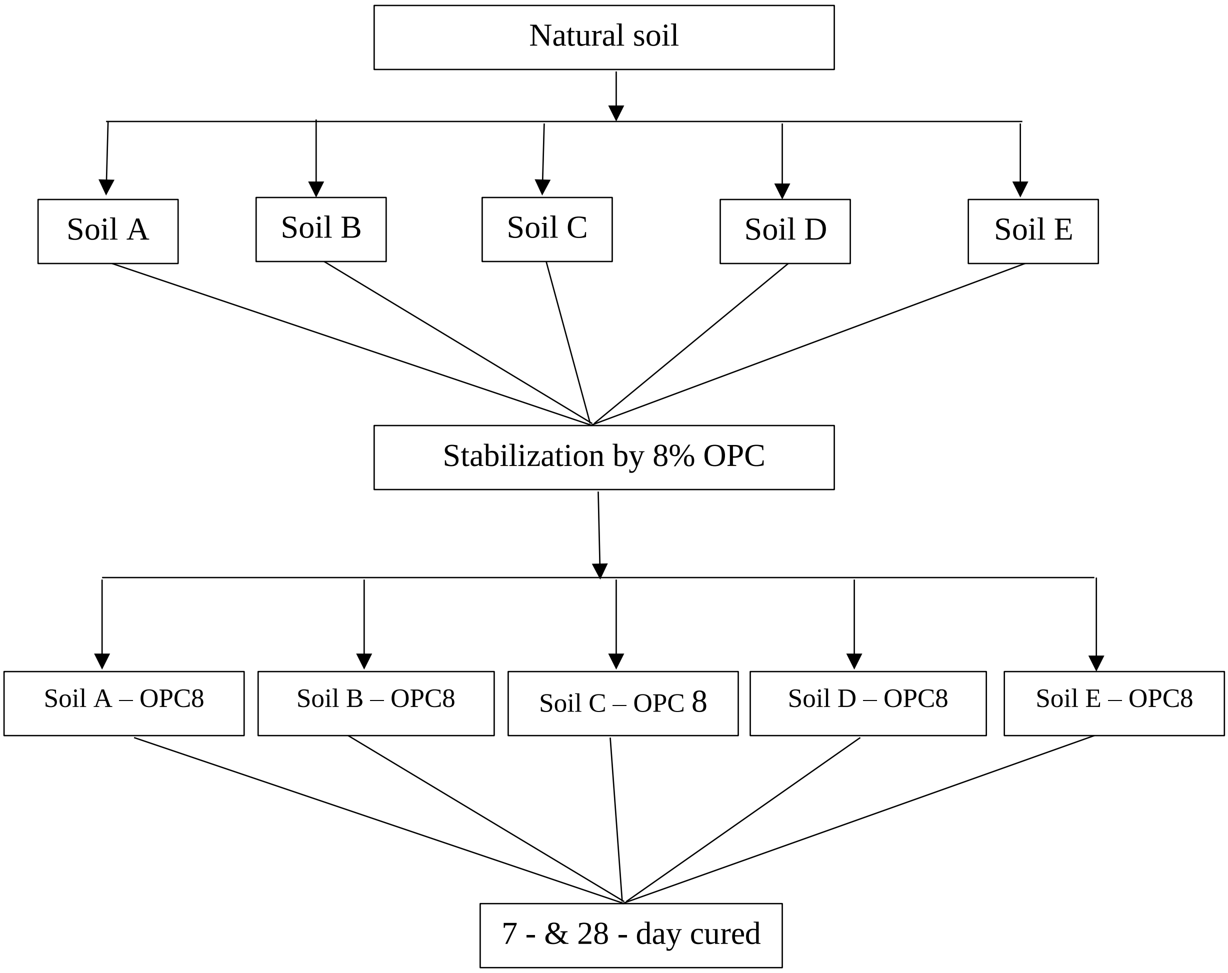


Figure 5.2 Soil stabilisation flowchart

5.3.2 Material Preparation and Testing

The soil-OPC binder mixture programme and testing were performed as already outlined in chapter 5 (see section 3.3). Hence, the following sections involve mostly the presentation and analyses of test data and results of analyses carried out on the stabilised soils.

5.4 PROPERTIES OF STABILISED CLAYS

5.4.1 Consistency Limits

The plasticity property of clays is a very important index and is used to determine workability in the field of construction. Clays of high plasticity do have a tendency of being difficult to mix and homogenize in field compaction. Even though insufficient workability does not preclude the utilisation of clay for lining systems for instance, but this can be an issue in road-subgrade construction. The soil consistency values of the OPC-treated soils along with the natural clays are presented in Figure 5.3 and Figure 5.4. As could be observed, there is a continuous rise in the liquid limits of the natural soils from 58% for **Soil A** to 285% for **Soil E** as the percentage of bentonite increases by 75% (Figure 5.3). This can be attributed to the greater affinity or demands for moisture in the soils as the clay content increases in order to make the soils more workable. The modification properties of the OPC seems to cause an increase in the liquid limit of OPC-stabilised **Soil A** but a general drop for the rest of the soils probably due to the presence of bentonite. On the other hand, Figure 5.4 indicates a general decrease in plasticity indices of the soils when the OPC is added. The percentage drop in the plasticity index decreases as the amount of bentonite increases in the mixture (except for **Soil B**). This phenomenon invariably signifies the reduced influence of the OPC on the soils as the amount of clay fines increases. Hence, an increase in the amount of the binder may be needed to cause an obvious decrease in the highly rich bentonite mixtures given their very high plasticity indices. This same trend of reduced plasticity index and liquid limit when clays are treated with binders was also observed by Jha and Sivapullaiah (2015a). Moreover, The index properties of both the non-stabilised and stabilised clays with bentonite content up to 10% does meet the general requirement for most clay liner applications (Sullivan and Quigley 2012). The changes occurring in soils due to the addition of OPC can be examined and explained based on the phenomenon of the evolution of electrical double diffused layer and modifications occurring in the soil particle fabric. When OPC is added to clayey soils, the changes that can possibly ensue are as follows: (i) a rise in the concentration of the divalent ions (Ca^{2+}) and a corresponding increase in the concentration of the electrolytic make-up of the pore fluid solution (ii) exchange of ions between the soil's monovalent ions and the OPC divalent ions resulting in the replacement of the monovalent ions and corresponding decrease in the thickness of the electrical double diffused layer (DDL). These factors are thus responsible for the reduction in the soil's liquid limit. Initially though, the soil fabric might change from a dispersed to a more flocculated arrangement leading to an increase in the liquid limit. However, the actual reduction in the liquid limit and plasticity index indicates that the changes in the soil fabrics dominate. The two factors (increase in Ca^{2+} and cationic exchange) resulting from OPC addition also occur when considering the plastic limit as this tends to increase in most cases. The plastic limit is that moisture content at which the soil approaches some shear resistance when remoulded. Modification by OPC tends to increase the viscosity and ionic concentration of the pore fluid solution hence causing an increase in the interparticle shear resisting forces and resulting in the rise of liquid limit of the soil (Dash and Monowar 2012).

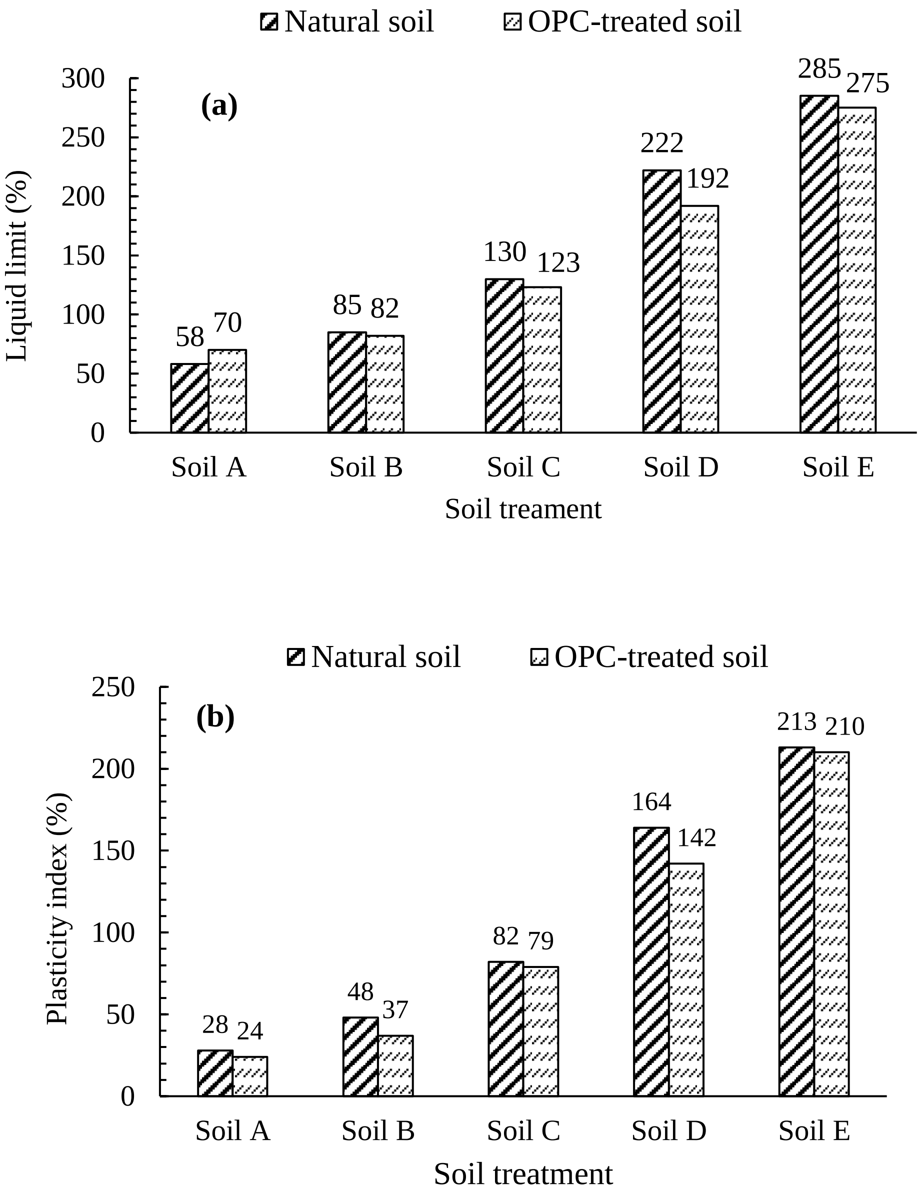


Figure 5.3 Natural and treated clays (a) liquid limit (b) plasticity index

5.4.2 Volume change properties

5.4.2.1 Swell potential

The free vertical or one-dimensional swell-strain as measured in this study is referred to as the percentage linear expansion of a sample that is laterally confined and completely submerged in water after standard proctor compaction at optimum conditions (OMC and MDD). Figure 5.4 shows the behaviour of the natural clays upon swelling alongside the influence of stabilisation of OPC on the linear expansion of the clays. The rise in the values of swelling of the natural clays at equilibrium can be mostly attributed to the presence of a greater percentage of the clay fines especially as the amount of bentonite increase as was observed in chapter 4 (see section 4.4). However, the addition of the OPC causes a general decrease in swelling for the curing durations investigated. Figure 5.5 presents the swelling in terms of their maximum values at equilibrium and enables a comparison of the effect of the bentonite fines in the clays. As could be observed, treatment by OPC with curing maintained for up to 7 days before inundation with water, causes a drastic reduction in the amount of swelling across the five soils (approximately 80% for **Soils A** & **D**, 90% for **Soils B**, **C** & **E**). There is clearly a further reduction in the maximum swell almost to zero per cent as the duration of curing is increased to 28 days. Figure 5.5 indicates at first glance that the amount of bentonite or clay fines does not seem to have an effect on the swelling of the OPC-treated soils as all the percentage swell values seem to be below 5%. However, the following section offers an explanation of the effects the bentonite addition might have on the progress of swelling of the OPC stabilised soils.

- *Swell description in terms of water absorption rate.*

Figure 5.6 enables a closer examination of the path of swelling (i.e. the progress of swelling) of the soils stabilised by the OPC under the durations of curing investigated. Figure 5.6 is plotted in terms of the water absorption rate which is empirically derived from basic soil parameters as given in Equation 5.1. This equation was utilised by Hashim and Muntohar (2006) to determine the quantity of water absorbed by kaolinite-bentonite mixtures. The usage of Equation 5.1 is only intended as a theoretical model and does not assume that the resulting linear expansion behaviour is obtainable in reality.

$$w_a = \varepsilon\,(1 + e_0)\frac{\gamma_s}{\gamma_w} \tag{5.1}$$

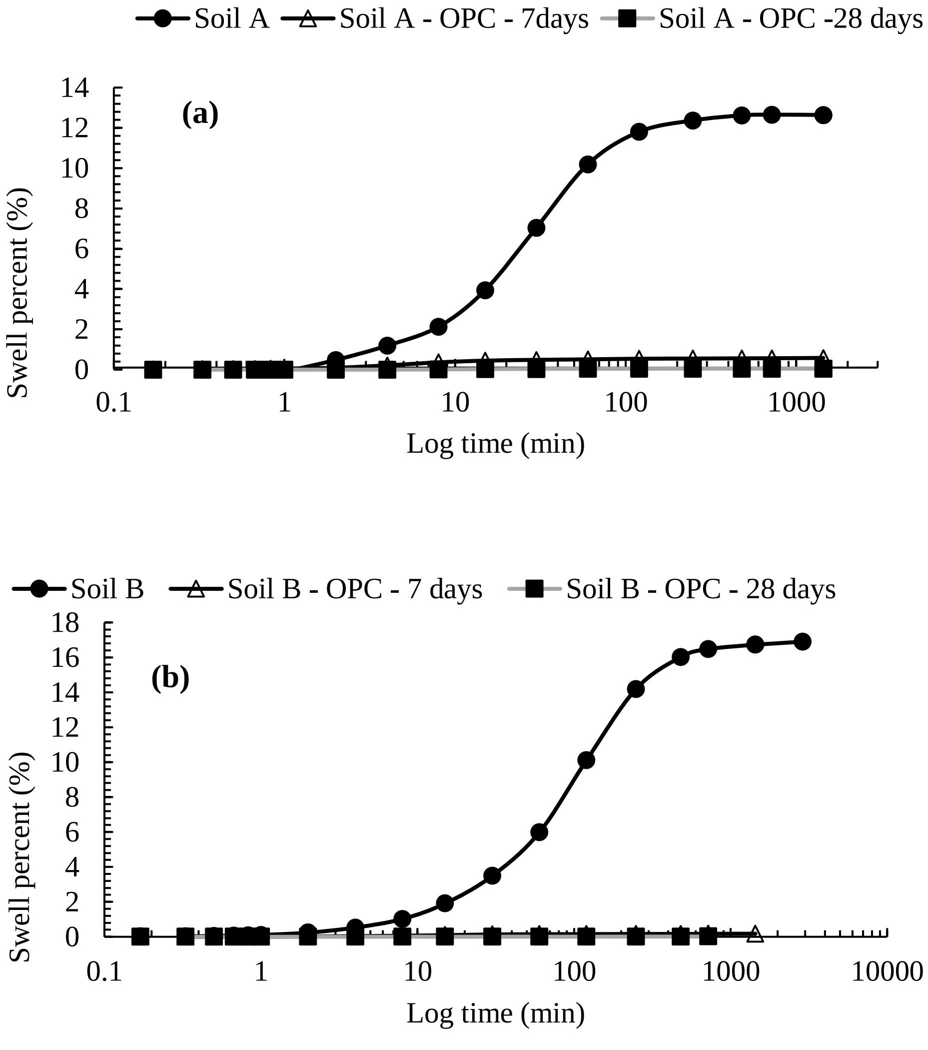
Where:

w_a = water adsorption (%)
ε = axial swell (%)
e_0 = initial void ratio of sample
γ_s = dry density of soil particles (Mg/m^3)
γ_w = density of water (Mg/m^3)

It was mentioned in chapter 4 (see section 4.4) that more than 75% of the total linear expansion occurs in the primary phase of soil swelling. Moreover, the rate of swelling in the primary stage could be a principal factor in the prediction of swell in relation to the permeability since most of the soil swelling occurs in this region (Güneyli 2017). For clays of pure kaolin-bentonite mixtures, the primary phase of swell tends to vary between 200 and 1000 minutes (Hashim and Muntohar 2006) using the standard oedometer apparatus in this study. After 7 days of curing, the rate of adsorption in the first 50 mins duration is higher for **Soils A** & **E** stabilised by the OPC with **Soil E** containing the most amount of the bentonite ultimately having the highest amount of water adsorbed after this time period. As the duration of curing is increased to 28 days, the amount of the adsorbed water seems to almost correspond with the quantity of bentonite in the OPC stabilised soils even though stabilised **Soil D** having 50% of the bentonite shows the highest amount of the adsorbed water followed by **Soil E**. It can also be observed that **Soil A** stabilised by OPC has the lowest absorption rate at 28 days period of curing.

- *Mechanism of swell reduction*

The adsorption of water (which possesses just a thin monolayer thickness) onto the negatively-charged surfaces of the clay soil mineral thus creates a concentration gradient between a double-diffused layer (consisting of the water molecules and exchangeable cations) and the existing bulk solution. On the other hand, stabilisation of the soil by OPC enables a modification of the created electrical double-diffused layer by causing a reduction of its thickness, hence leading to a reduction in expansion when inundated by water. Depending on the amount and type of OPC used, the decrease in swelling could be attributed to the reduced affinity of the soil to the adsorb water within a shorter period of curing of say 7 days due to the process of continuous agglomeration and flocculation. However, with the progress of time, the formation of crystalline cementitious or fibrous pozzolanic products of hydration (CASH, CSH or CAH) develops (see section 5.5) which aids a further reduction in the swell of the stabilised soil.



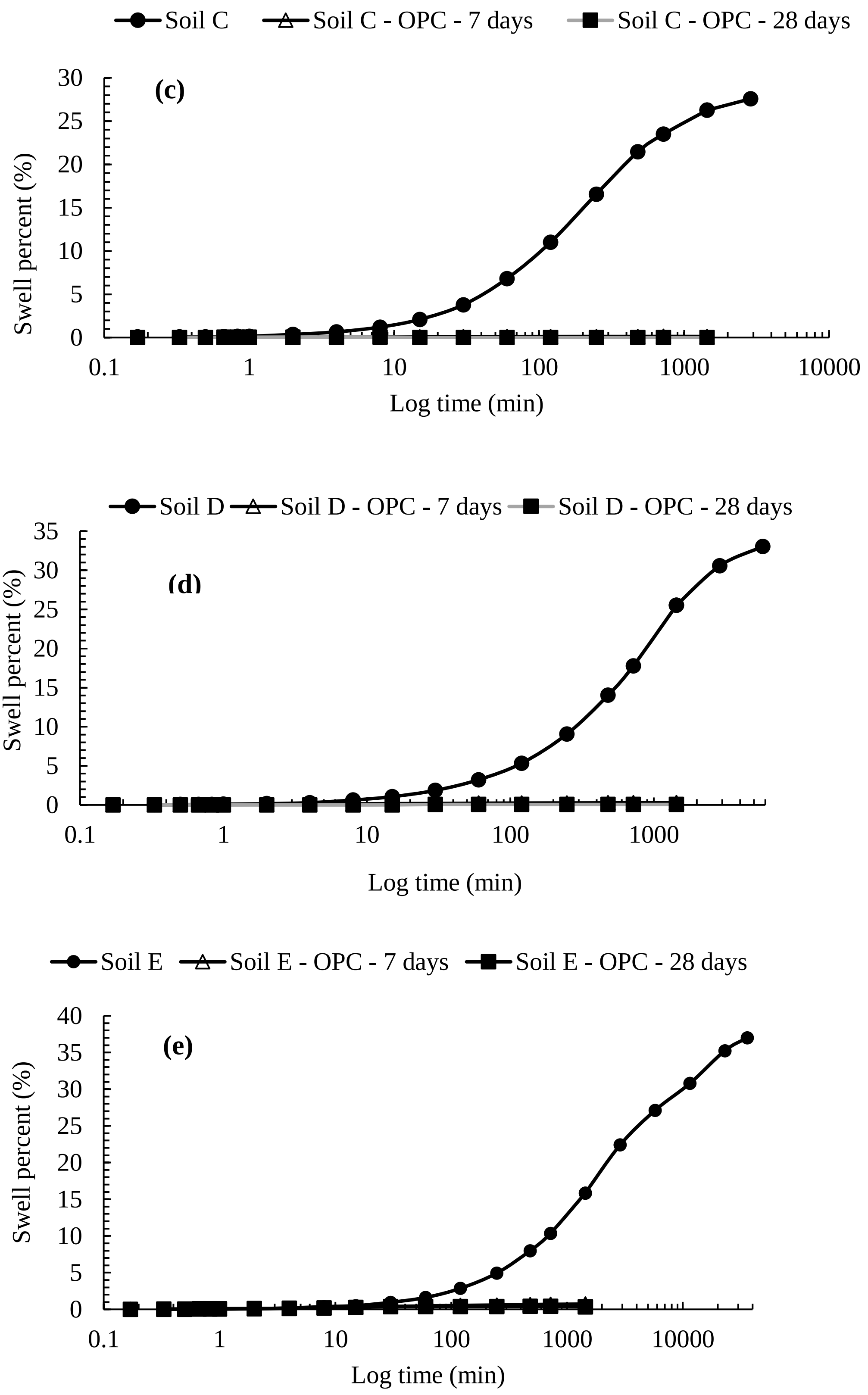


Figure 5.4 Vertical swell strain versus time for natural and stabilised soils at 7 and 28 days (a) Natural and stabilised Soil A (b) Natural and stabilised Soil B (c) Natural and stabilised Soil C (d) Natural and stabilised Soil D (e) Natural and stabilised Soil E

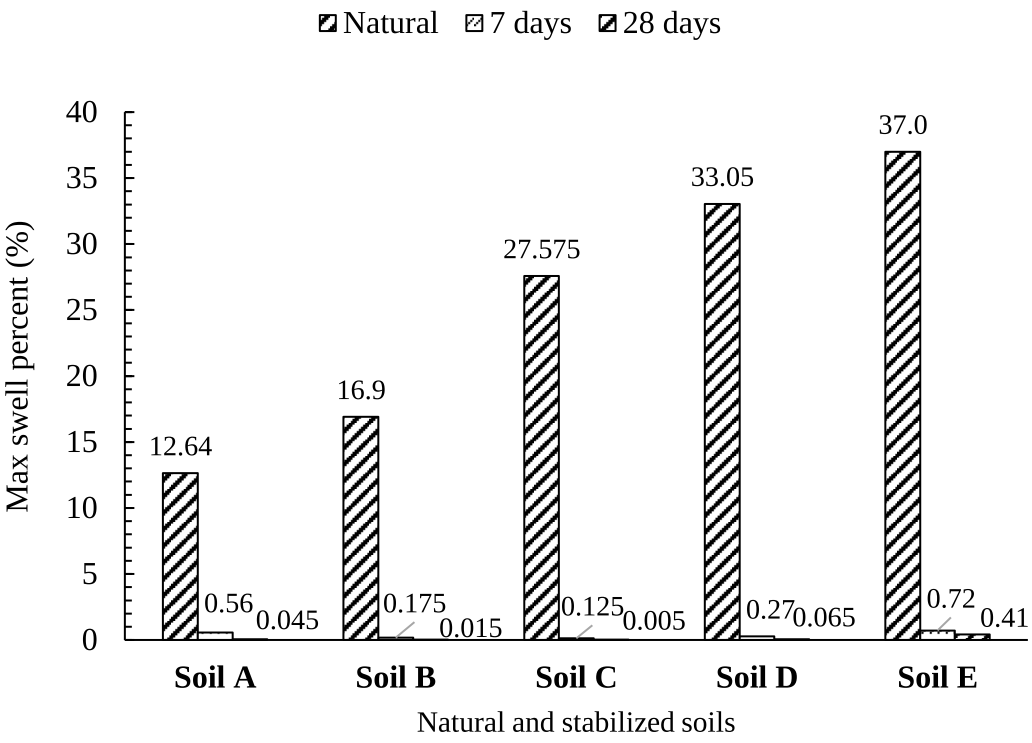


Figure 5.5 Maximum swell percent of natural and stabilised soils

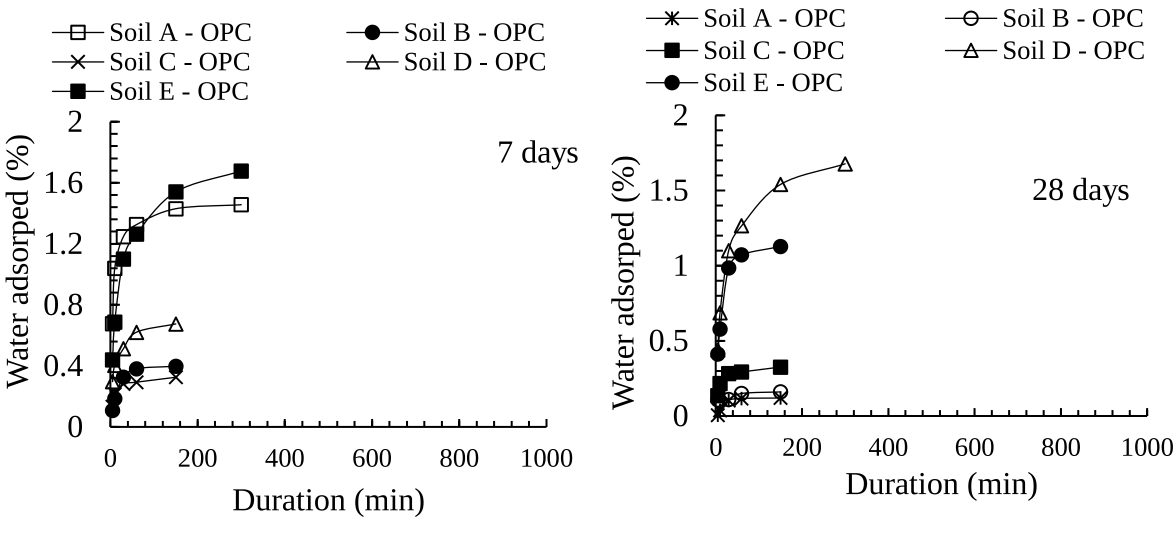
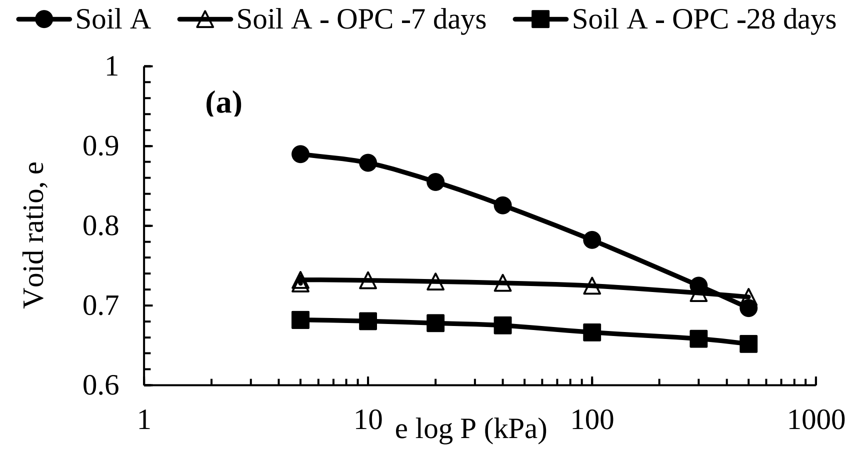


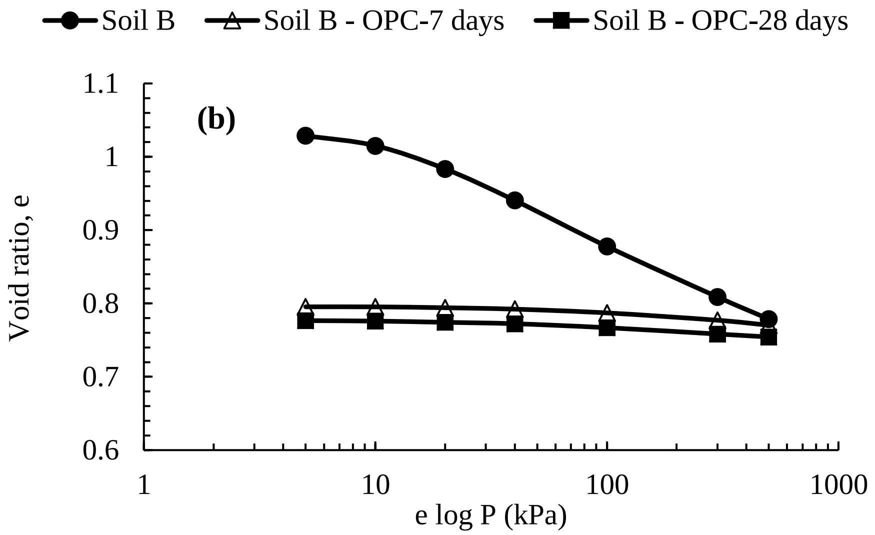
Figure 5.6 Water adsorption rate of stabilised soils at 7 and 28 days

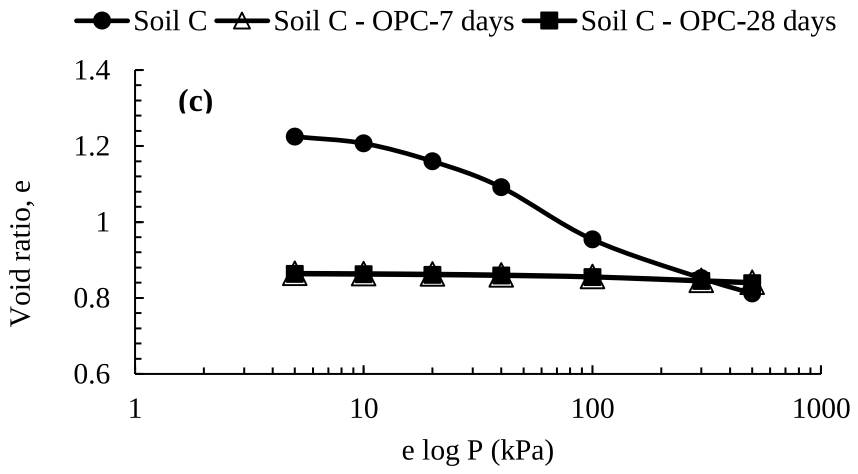
5.4.2.2 Consolidation characteristics

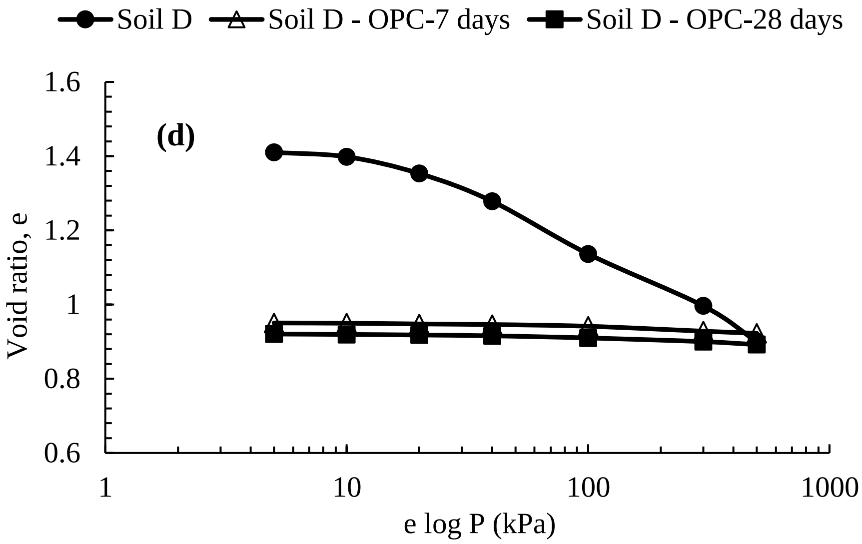
Figure 5.7 is the e-log p curves indicating the relationships of the stabilised soil samples after being compacted and submitted to curing durations of 7 and 28 days alongside the natural soils.

The effect of stabilisation with OPC and curing are easily noticeable for all the five stabilised soils. Firstly, stabilisation by OPC causes a drastic decrease in the initial void ratios and consolidation seen from the flat-shaped behaviour of the curves. A reduction in the void ratio is further observed across the five stabilised soils as the period of curing increases from 7 to 28 days. The larger reduction in the void ratios as the curing duration is increased is mostly felt in **Soil A.** On the other hand, irrespective of the quantities of the amount of bentonite in **Soils B-E,** the decrease in the ratio of voids tends to be minimal. An assessment of the change in volume ($\Delta H/H = \Delta e/1+e_0$) due to consolidation at an effective stress of 100kPa (which is the stress level that covers most subgrades (Ouhadi et al. 2014) indicates an increase from 0.057-0.120 as the amount of Na-montmorillonite increased in the natural clays. On the other hand, the range of volume change is from 0.013-0.0.009 for the OPC-stabilised soils at 100kPa effective stress. Hence, there seems to be only a slight difference in the amount of consolidation at this effective stress level for the OPC added. Overall, a reduction in the rate and amount of settlement due to stabilisation is in the range between 82-96%. The rate of settlement reduction of approximately 94% was observed by Ouhadi et al. (2014) who utilised 6% of cement in the treatment of soft clays. The reduction in void ratios is attributed firstly, to the well-known exchange of ions between the clay particles and OPC which results in a flocculated structure and secondly to the pozzolanic reaction that results in the formation of cementitious gels that connects and binds the clay particle together to resist further compression.


Soil A
Soil A - OPC -7 days
Soil A - OPC -28 days
(a)
Void ratio, e
1
0.9
0.8
0.7
0.6
1
10
100
1000
e log P (kPa)


Soil B
Soil B - OPC-7 days
Soil B - OPC-28 days
(b)
Void ratio, e
1.1
1
0.9
0.8
0.7
0.6
1
10
100
1000
e log P (kPa)


Soil C
Soil C - OPC-7 days
Soil C - OPC-28 days
(c)
Void ratio, e
1.4
1.2
1
0.8
0.6
1
10
100
1000
e log P (kPa)



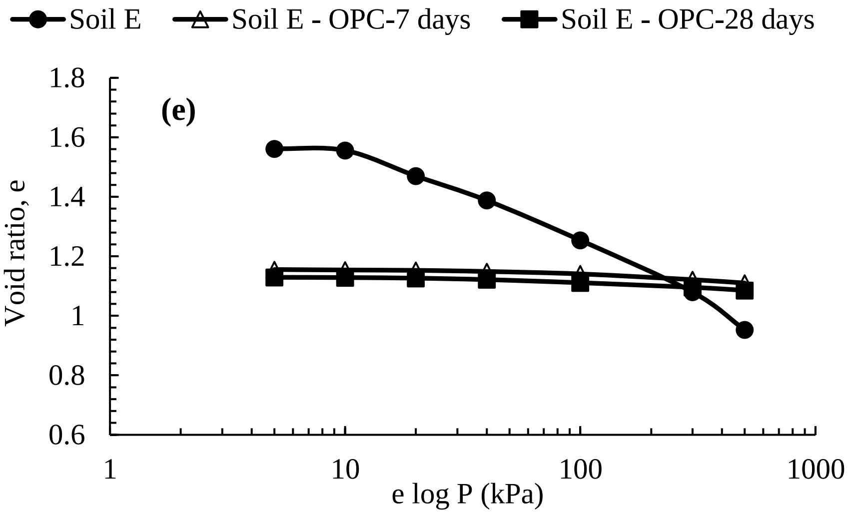


Figure 5.7 Consolidation of natural and stabilised soils at 7 and 28 days (a) Natural and stabilised Soil A (b) Natural and stabilised Soil B (c) Natural and stabilised Soil C (d) Natural and stabilised Soil D (e) Natural and stabilised Soil E

A further description of the compression behaviour of the OPC-stabilised soil composites in terms of compression index is shown plotted in Figure 5.8. Compression index (Cc) is defined as the gradient of the linear portion of the e-log p curve. This is an important index in soil consolidation because it indicates the extent of compression that is undergone by soils as the consolidation pressure increases.

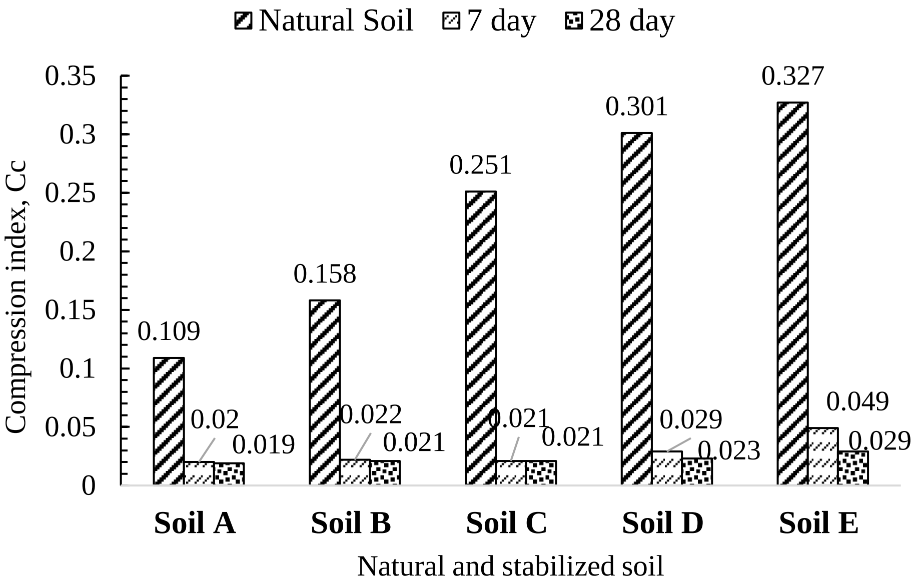


Figure 5.8 Compression index of natural and stabilised soils at 7 and 28 days

A steady rising trend in the compression index that corresponds to an increase in the bentonite content from 0 to 75% is noticed for the untreated mixes. This phenomenon is due to an increase in the bentonite fines in the mixture thus giving a great degree of compressibility. As described earlier in chapter 2 (see section 2.10.3.3), factors that includes the mineralogy of clay, exchangeable ions and pore fluid chemistry are known to influence the diffuse double layer (DDL) thickness which can consequently generate a significant influence on the compressibility properties of clays (Nalbantoglu and Tuncer 2001; Jha and Sivapullaiah 2015a). Figure 5.9 also indicates a significant reduction in Cc up to about 50-85% corresponding to 0-75% bentonite content when OPC was added to the mixture. The influence of curing duration seems very minimal on the five stabilised soils. A reduction in the consolidation characteristics is described by the filling of voids and the aggregation of stabilised soil particles in the mixes which then leads to stronger cement particle bonding thus, producing a greater resistance to compression loads (Nalbantoglu and Tuncer 2001).

5.4.3 Strength Properties

The strength properties of both the natural and stabilised soils as determined from the unconfined compression testing are given in Figure 5.9. The effect on the average UCS of a shorter curing period of 1 day was assessed in addition to the 7-and 28- day curing durations employed previously for the determination of swelling and consolidation. It is evident from Figure 5.9 that the UCS of the OPC-stabilised soil samples increased with the period of curing. Moreover, there is a significant development in the compressive strength of the stabilised soils as compared to the natural soils. Even though there appears to be a dramatic rise in the strength of **Soils A** & **B** over the 28 days of curing in contrast to the small variation in strength of **Soils C**, **D** & **E**, there seems to be slight stability for the same curing period of 28 days with the values of strength fluctuating around approximately 1200 and 1400kPa. Even at 1-day curing duration, the increase in the unconfined compressive strength was very impressive especially for the soils containing the various proportions of bentonite. Generally, as was described in chapter 2 (see section 2.10.2), the improvement in the compressive strength at a shorter duration of curing is primarily due to the short-term chemical reactions occurring between the soil particles and the divalent ions from the OPC (Jha and Sivapullaiah 2015a). During the mixing process, cation exchange involving the substitution of monovalent ions by the calcium ions ensues. At this stage, both particle flocculation and agglomeration may occur thus, making the soil more granular or friable. As the curing period increases, the OPC hydration is further enhanced through an increase in hydroxyl ions concentration in the soil-binder mixture which causes the pH level to rise resulting in the dissolution of both the monovalent ions from the alumina and silica compounds in the clay minerals (Jha and Sivapullaiah 2015a). The combined dissolved alumina and silica compounds together with the calcium ions from the additive in the ensuing highly alkaline environment then result in the formation of complex cementitious pozzolanic hydrates. The formation of pozzolanic reaction compounds enables an improvement in the strength of the stabilised soil during the stabilisation process. Even though the strength gain in the stabilised clays has been attributed to cementation by OPC, the presence of bentonite might also have a role to play in increasing the strength of the stabilised mixture in spite of the high plasticity of the bentonite-rich clays especially when in contact with water. The effect of bentonite used alone to improve the strength of kaolinite was reported in Muntohar and Yogyakarta (2010). The rise in strength of the OPC-stabilised **Soils B-E** could be partly attributed to a phenomenon called “thixotropy”. Thixotropy is essentially a reversible, isothermal and time-dependent process that occurs under certain compositional conditions and enables a soil material to harden upon remoulding and compaction (Mitchell and Soga 2005) just as is the case with the clays containing the montmorillonite in this study. Kaolinite has been known to display relatively very less amount of thixotropy as compared to montmorillonite (Zhang et al. 2017b). This could be attributed to the montmorillonite layered-structure which is 2:1 as compared to kaolinite with a layered-structure of 1:1 (Zhang et al. 2017b). Thus, the 2:1 mineral tends to possess a much larger specific surface area, higher ionic exchange capacity, higher activity and larger amounts of electrically charged particles. Moreover, montmorillonites minerals are capable of providing the forces of interaction between particles of soils as well as enhance the strength of contacts between the neighbouring particles. In agreement to this phenomenon, Mitchell (1960) opined that inter-particle forces of interaction are influential factors in the thixotropic clay behaviours which causes an increase in the strength of the bentonite-rich soils the mechanism of which can be elucidated further using the electrical double diffused layer theory.

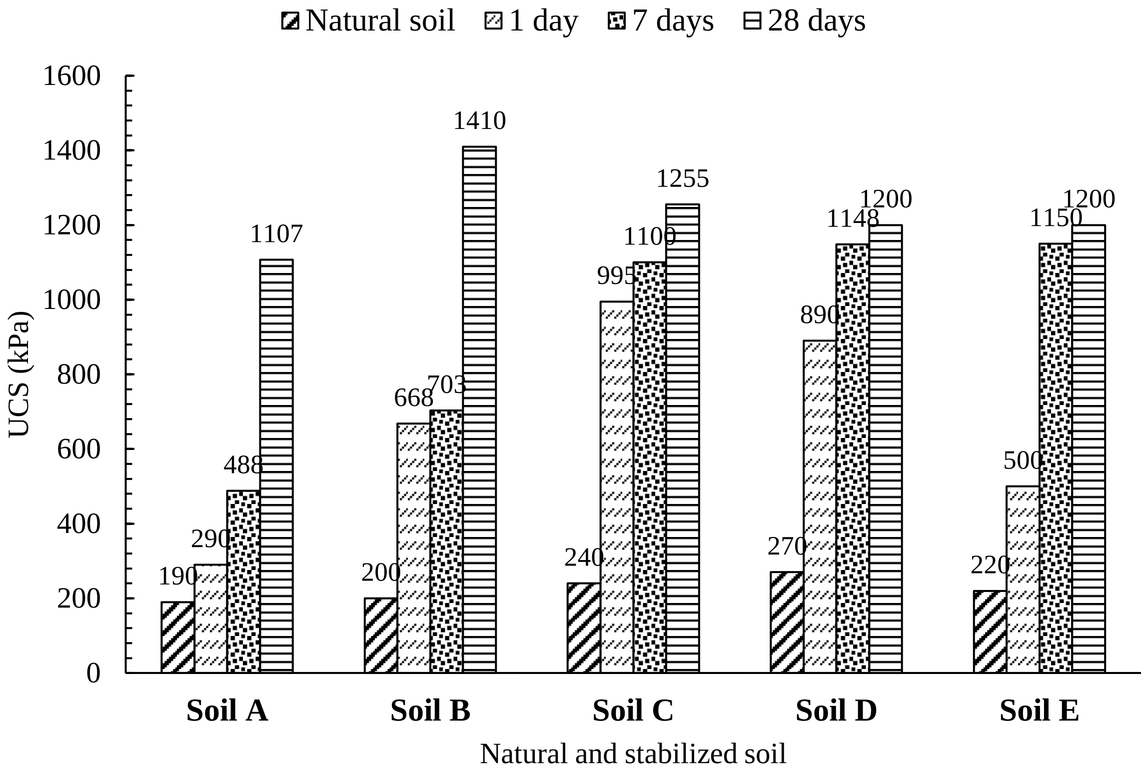


Figure 5.9 Unconfined compressive strength of natural and stabilised soils at 7 and 28 days

5.5 MICROSTRUCTURAL ANALYSIS OF STABILISED SOILS

SEM/EDS and XRD analyses were used to provide a perspective that supports the geotechnical behaviour and characteristic properties deduced from the tested mixes. For the sake of brevity, only the most prominent differences in the SEM and EDS images are chosen and provided for representative stabilised to explain the relevant variations in the geotechnical properties studied. SEM was applied in the previous chapter (see section 4.3.1) to explain the morphologies of the natural soils compacted at the optimum conditions. In this section, a comparison will be drawn on the major changes observed in some of the natural clays and the clays stabilised using OPC.

The process of stabilised soil hydration due to the addition of OPC essentially involves contribution from the following compounds: C_2S C_3S, C_3A and C_4AF. Marchon and Flatt (2016) summarized the stages of OPC hydration as follows:

i. The formation of crystalline and hexagonal crystals called Portlandite in the early phase of hydration reaction resulting from lime hydration of C_2S and C_3S.

ii. Formed ettringites compounds (calcium-sulpho-aluminates) Aft which are needle-like crystals and Monosulfo-alumino-ferrite (AFm) forms in platy crystals both growing into the pore spaces in the early hydration stage due to the reaction between calcium sulphates and C_3A and C_4AF.

iii. The formation of cementitious compounds or gels (C-S-H and C-A-H) which ultimately contributes to the improvement in the geotechnical properties of the stabilised soils. The cementitious gels are formed due to complex sets of pozzolanic reactions involving the silicates (C_2S and C_3S) with the Portlandite occurring within the first few hours during the hydration phase and continuing up to several weeks, months and even years. This stage of reaction does also depend on the quantity of water and silicates available.

These reaction phases were also observed in the stabilised microstructure with OPC addition as obtained from the SEM tests. The morphological complexities of the stabilised soils are immediately apparent as compared to the natural soil (Figures 5.10 to Figure 5.14). The formation of cementitious gels and ettringite are evident as noticed in Figure 5.10, Figure 5.12 and Figure 5.13. Moreover, it could also be observed that the spaces between the particles are much narrower with dense packing resulting from the binding of the soil particles by cement. The same changes in microstructure were observed by Horpibulsuk et al. (2010); Jha and Sivapullaiah (2016). The EDS analyses of **Soils A** and **D** stabilised by OPC are shown in Figure 5.11 and Figure 5.14. The principal peaks observed for the stabilised **Soils A** and **D** are composed of calcium, oxygen, aluminium and silicon and traces of sulphur, magnesium and sodium. These EDS spectrums confirm the possible elements which compose of the compounds of hydration and pozzolanic reaction product CSH formed when OPC is added. However, on average, the EDS showing the stabilised Soil E sample with 50% bentonite content gives reduced quantities of the elements which comprise the compounds of the products of hydration as compared to **Soil A** containing only the kaolinite. This further explains that little quantity of OPC was available to be used up by **Soil D** to enable reasonable cationic exchange and pozzolanic reactions. This was experienced previously by the small effect the cement addition had on the Atterberg limits of the mixture. Hence, an increase in the amount of the binder may be needed to cause an obvious decrease in the highly rich bentonite mixtures given their very high plasticity indices as brought about by the high montmorillonite content.

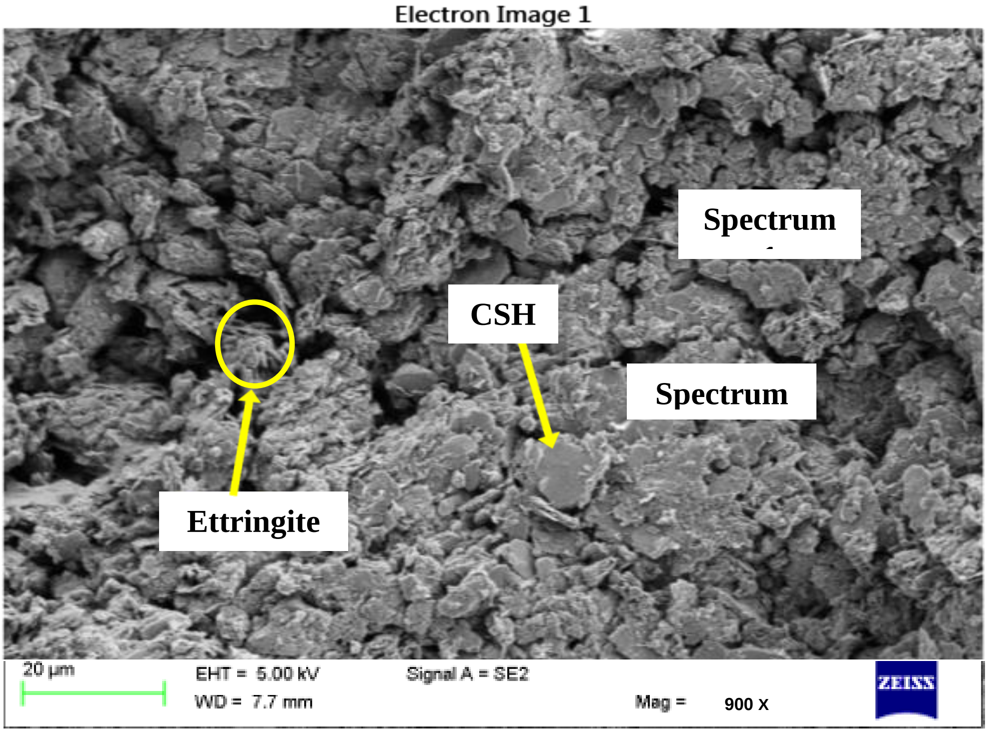
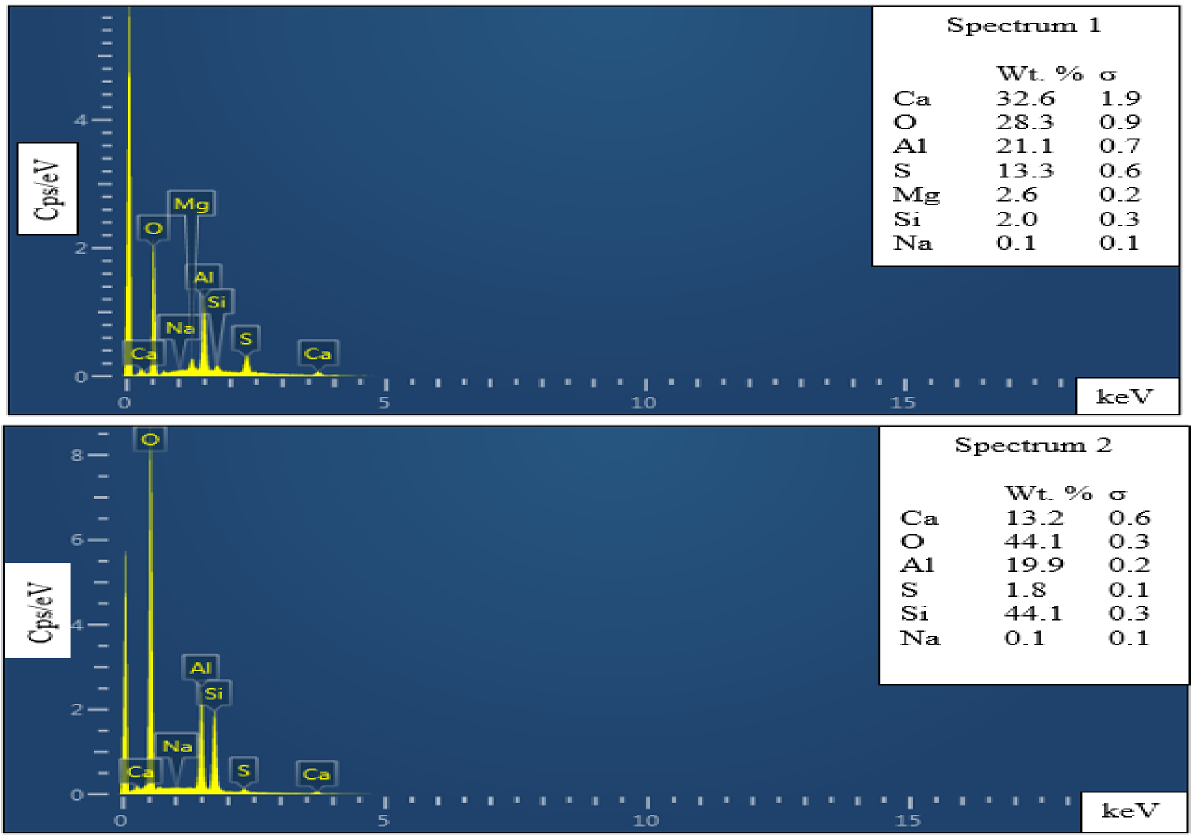


Figure 5.10 SEM of OPC- stabilised **Soil A**



Spectrum 1

	Wt. %	σ
Ca	32.6	1.9
O	28.3	0.9
Al	21.1	0.7
S	13.3	0.6
Mg	2.6	0.2
Si	2.0	0.3
Na	0.1	0.1

Spectrum 2

	Wt. %	σ
Ca	13.2	0.6
O	44.1	0.3
Al	19.9	0.2
S	1.8	0.1
Si	44.1	0.3
Na	0.1	0.1

Figure 5.11 EDS of OPC- stabilised **Soil A**

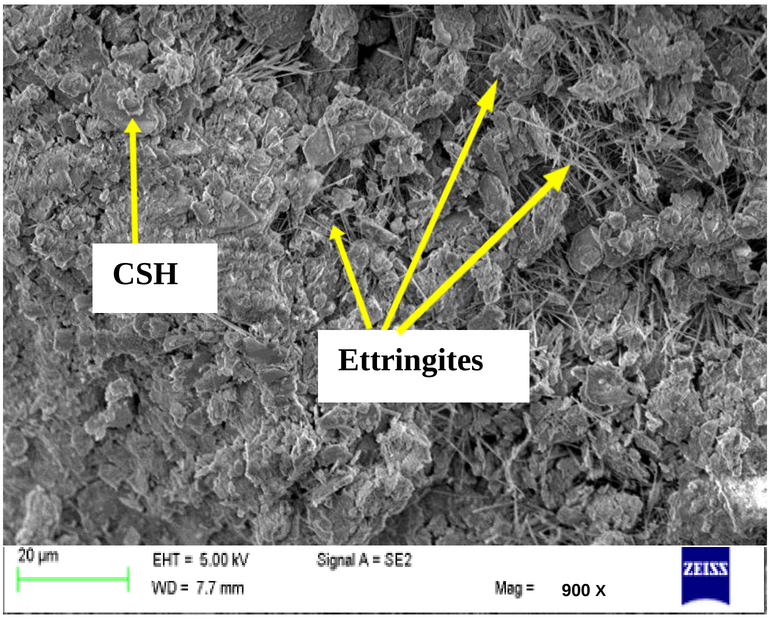


Figure 5.12 SEM of the stabilised **Soil B**

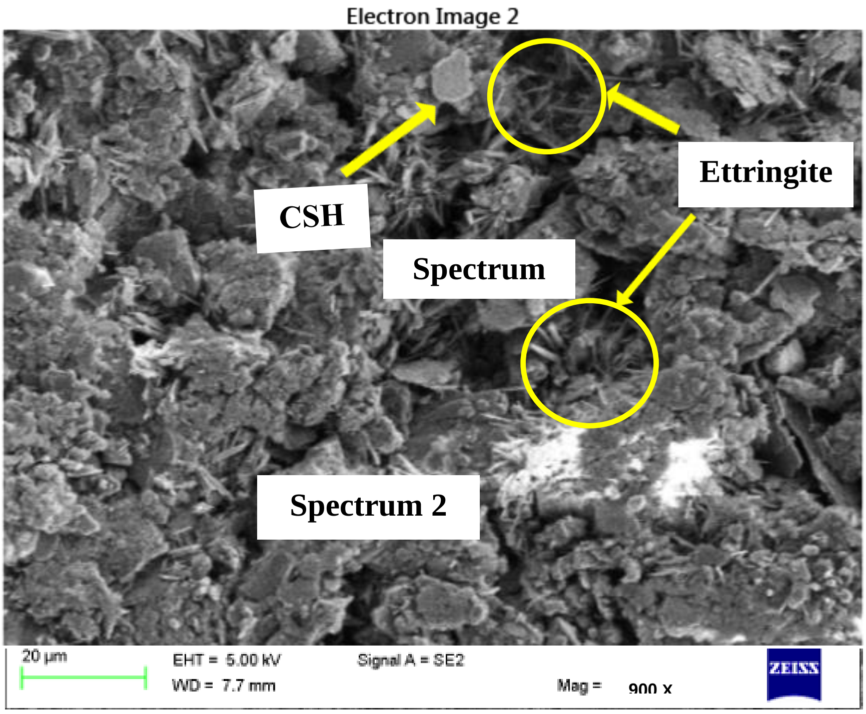
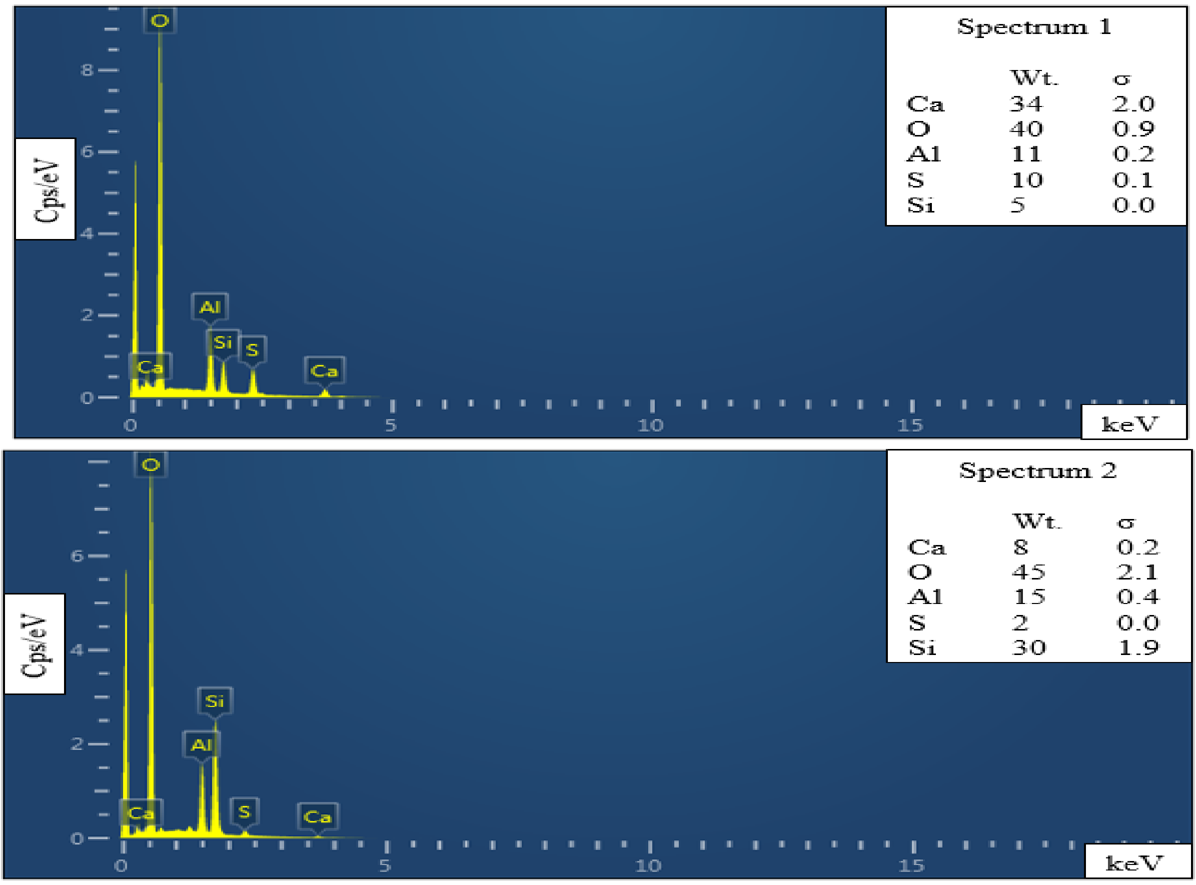


Figure 5.13 SEM of stabilised **Soil D**



Spectrum 1

	Wt.	σ
Ca	34	2.0
O	40	0.9
Al	11	0.2
S	10	0.1
Si	5	0.0

Spectrum 2

	Wt.	σ
Ca	8	0.2
O	45	2.1
Al	15	0.4
S	2	0.0
Si	30	1.9

Figure 5.14 EDS of stabilised **Soil D**

5.6 Discussions on effect of bentonite in the clay mixtures

From the foregoing and considering the behaviour of the natural clays without stabilisation (see chapter 4), it seemed that the variability in the engineering properties of the natural and stabilised bentonite-rich clays are very minimal and do not appear to correspond to the amount of bentonite added. Several authors have also reported on the lack of influence on the properties of bentonite-rich clays of increasing the amount of the bentonite beyond a certain limit. In their research on the use of a kaolinite-bentonite mix for clay liners with the amount of bentonite varied from 10% to 100% (in multiples of 10s), Karunaratne et al. (2001) attempted to determine the threshold mix ratio that could provide behaviours similar to the pure bentonite used alone. A comparison of the settlement properties of the soil mixtures revealed that a minimum quantity of 30% bentonite was needed to obtain a similar trend of reduction in the coefficient of consolidation as the pure bentonite. This same outcome of the decrease in the consolidation property with 30% bentonite in a kaolinite-bentonite mix as compared to the pure bentonite was noticed by Ahmad et al. (2000). Sinha (2000) in their studies of the mechanical behaviour of bentonite-kaolinite mixtures observed that the threshold strength was reached at 50% of the bentonite used. Moreover, Muntohar and Yogyakarta (2010) studied the strength behaviour of kaolinite modified by adding increasing amount of the bentonite and indicated that the unconfined compressive strength of the mixtures increased only slightly when the quantity of bentonite added was more than 30% while also concluding that the strength was not directly proportional to the amount of bentonite used. Given that the behaviours of the natural clays studied in this research are not markedly different from those of the above-mentioned previous studies, it was decided that the base or reference clay being **Soil A** and either one of the remaining four soils be chosen for further stabilisation in the succeeding chapters. Hence, after some detailed technical analyses and thoughtful consideration, it was decided that **Soil E** be utilised alongside **Soil A** for subsequent stabilisation in the next chapter of this research. First and foremost, **Soil A** and **Soil E** are comparatively different because they possess a very wide range of plasticity index values. Secondly, and from an economic perspective, the cost of acquisition of kaolinite alone is certainly double that for obtaining Na-montmorillonite. Hence, using **Soil E** which contains the highest proportion (75%) of the cheaper Na-montmorillonite as compared to using **Soils B**, **C** or **D** with relatively greater amounts of kaolinite content does make perfect sense.

SUMMARY

An evaluation of the engineering properties of the clays stabilised by OPC used as a sole binder was carried out in this chapter. The quantity of OPC required to stabilise the clays according to standard recommended practices especially as it relates to the types of clays used in this study was first established. A summary of the analysis of the level of improvement of the use of OPC in the soil stabilisation are as follows:

i. A continuous rise in the liquid limits of the natural soils from 58% for **Soil A** to 285% for **Soil E** as the percentage of bentonite increased in the clays was noticed. This was attributed to the greater affinity or demands for moisture in the soils as the much clay content increases in order to make the soils more workable. On the other hand, a general decrease in plasticity indices of the soils when the OPC is added was observed. The percentage drop in the plasticity index decreases as the amount of bentonite increases in the mixture (except for **Soil B**). These Atterberg limits changes from OPC treatment occurred due to a rise in the concentration of the divalent ions (Ca^{2+}) and a corresponding increase in the concentration of the electrolytic make-up of the pore fluid solution as well as exchange of ions between the soil's monovalent ions and the OPC divalent ions resulting in the replacement of the monovalent ions and corresponding decrease in the thickness of the electrical double diffused layer (DDL).

ii. The increase in the values of swelling of the natural clays at equilibrium was attributed to the presence of a greater percentage of the clay fines especially as the amount of bentonite increased. However, the stabilisation by OPC caused a general decrease in swelling for the curing durations of 7 and 28 days investigated. The adsorption of water onto the negatively charged surfaces of natural clay soil minerals can create a concentration gradient between a double-diffused layer and the existing bulk solution. On the other hand, stabilisation by OPC enabled a modification of the created electrical double-diffused layer by causing a reduction of its thickness, hence leading to a reduction in expansion when inundated by water. The decrement in swelling was attributed to the reduced affinity of the soil to the adsorb water within a shorter period of curing of say 7 days due to the process of continuous agglomeration and flocculation. However, with the progress of time, the developed crystalline cementitious or fibrous pozzolanic products of hydration aided a further reduction in the swell of the stabilised soil.

iii. Stabilisation by OPC caused a drastic decrease in the initial void ratios and consolidation. A reduction in the void ratio was further observed across the five stabilised soils as the period of curing increases from 7 to 28 days. The larger reduction in the void ratios as the curing duration is increased was mostly felt in **Soil A.** Moreover, a steady rising trend in the compression index that corresponded to an increase in the bentonite content from 0 to 75% was noticed for the natural soils. Also, a significant reduction in Cc up to about 85% when stabilised by OPC was noticed. However, the influence of curing duration seemed very minimal on the five stabilised soils. A reduction in Cc is due to the filling of voids and the aggregation formations in the mixes stabilised by OPC which then leads to stronger cement particle aggregates and produces a greater resistance to compression loads.

iv. It is evident that the UCS of the OPC-stabilised soil samples increased with the period of curing. Moreover, there was a significant development in the compressive strength of the stabilised soils as compared to the natural soils. At 1-day curing duration, the increase in the unconfined compressive strength was very impressive especially for the soils containing the various proportions of bentonite. Generally, the improvement in the compressive strength at a shorter duration of curing is primarily due to the short-term chemical reactions occurring between the soil particles and the divalent ions from the OPC. Meanwhile, in the longer term, the formation of pozzolanic reaction compounds enabled an improvement in the strength of the stabilised soil during the stabilisation process. Although the strength gain in the stabilised clays was attributed to cementation by OPC, the presence of bentonite also had a role to play in increasing the strength of the stabilised mixture by a phenomenon referred to as “thixotropy”.

CHAPTER 6

RC INCLUSION IN CEMENTITIOUS PRODUCTS FOR STABILISATION OF THE SOILS

6.1 GENERAL

In this chapter, analyses and discussions are carried out to evaluate the engineering performance of the clays by incorporation of RC in soil stabilisation. **Soils A** & **E** were subsequently chosen for further stabilisation in this chapter. The reasons for the choice of these two soils as stated in the previous chapter (see section 5.6) were both technical and economic. Nevertheless, the main objective of this chapter is to substitute most of the quantity of OPC that was used in the stabilisation of the soils from the previous phase. Hence, the engineering (swell, consolidation & strength) and water retention properties of the stabilised soils by a replacement of up to 50% of the OPC by the inclusion of RC are assessed. Finally, several microstructural features derived from the scanning electron micrographs (SEM) are further used to explain the mechanism of reactions of the stabilised soils.

6.2 SPECIFICATIONS FOR CHOSEN BINDERS

Until presently, the general recommendation for RC application in pavement subgrade construction requires that about 1-2% (by weight of OPC) of it be used in combination with OPC for the stabilisation of $1m^2$ area of soils of medium to high plasticity (Marjanovic et al. 2009; Holmes 2015; PowerCem Technologies 2015). Invariably, the challenge of using more quantities of OPC still remains not to mention the attendant concerns previously mentioned in chapter 2 (see section 2.10.4). Hence, GGBS being a more environmentally-friendly cementitious by-product is used to replace some of the OPC proportions while maintaining the recommended quantity of RC at 1% in the soil-binder mixtures. In this chapter, the OPC content in the stabilised soil in relation to GBBS and RC shall be varied from 30% to 50% (by dry weight of OPC). As could be noticed, the percentage terms used is in relation to the overall OPC weight (which is in effect 8% of the total dry weight of the soil as used in chapter 5). The justification given for the use of percentages in this manner is because most of the discussions in this chapter will include an analysis comparing the ternary (OPC-GGBS-RC) binder combinations to OPC used solely in the stabilisation of the soils with the OPC being the binder of reference.

6.3 PARTICLE ANALYSIS OF BINDERS

Figure 6.1 shows the particle distribution curves of OPC, GGBS and RC. All the three soil binders as could be seen are poorly graded. Both OPC and GGBS seem to cut across the silty and clay particle sizes ranges (0.0001 – 0.1mm). This also invariably means that 100 per cent of both OPC and GGBS are finer than the RC on a sieve of 0.425mm size (or sieve No. 40). However, below approximately 0.105mm (sieve No 140), most of the GGBS particles seem to be slightly finer than OPC up until approximately 0.074mm (sieve No 200) mm size of the particles. On the other hand, RC could be regarded as being somewhat gravelly sand considering the range of particle sizes it includes (i.e. approximately between 0.1 and 10mm size ranges).

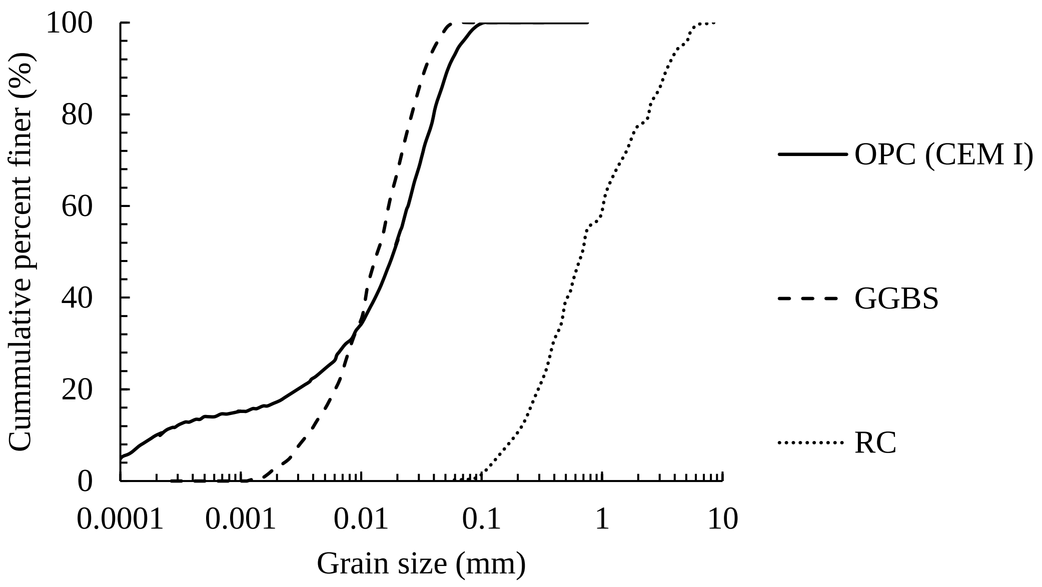


Figure 6.1 Particle size distribution of OPC, GGBS and RC

6.4 STABILISATION BY CHOSEN NOVEL BINDER MIX

6.4.1 Soil-Binder Mix Design

Soil A and **Soil E** were sampled in their natural states and thoroughly mixed dry with the additives. 8% OPC proportion (by dry weight of soil) are firstly replaced by 50%, 60% and 70% GGBS calculated by dry weight of OPC. In order to study the influence of RC, the clay-binder mixtures were prepared by substituting the GBBS in their respective mixes with 1% of the RC calculated by dry weight of the OPC. Hence, the total binder content does not exceed 8% of the clay soil mass in each of the soil-binder mixtures. For the sake of brevity during result presentation and discussion, the OPC-GGBS-RC proportion is represented in terms of the mixture ratio of their percentages by weight with their respective notations as presented in Table 6.1. Additionally, with regard to the phases of soil stabilisation, Table 6.1 is further elucidated in the flow chart given in Figure 6.2 where RC and GGBS are regarded both as by-product additives (BPA).

Table 6.1 Cement replacement mix proportion

Mix proportion	Phase 1 mix	Phase 2 mix	Phase 3 mix
	% by dry wt. of OPC		
OPC	100	-	-
Notation	OPC100	-	-
OPC: GGBS	50:50	40:60	30:70
Notation	OPC50-GGBS50	OPC40-GGBS60	OPC30-GGBS70
OPC: GGBS: RC	50:49:1	40:59:1	30:69:1
Notation	OPC50-GGBS49-RC1	OPC40-GGBS59-RC1	OPC30-GGBS69-RC1

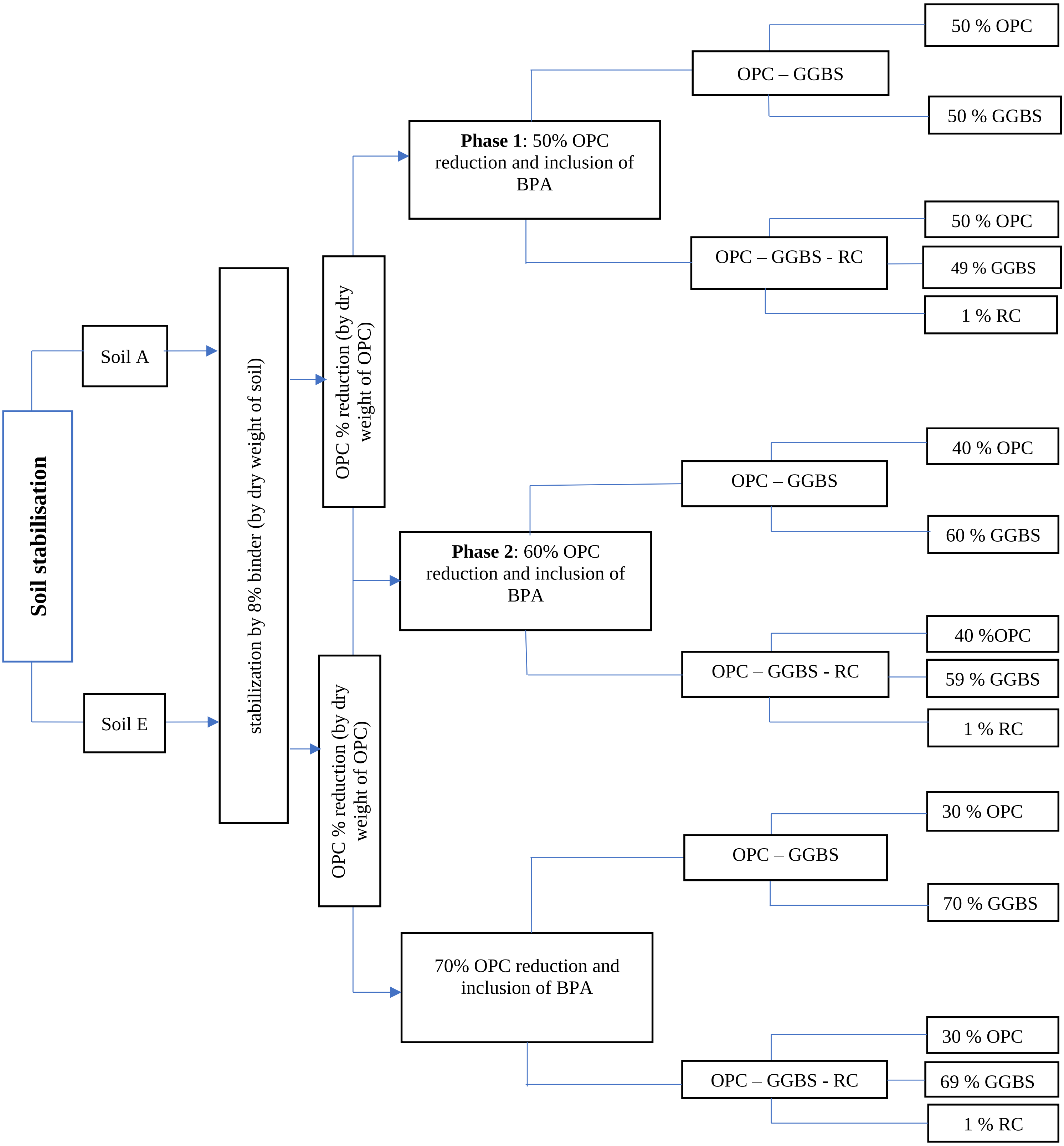


Figure 6.2 Soil stabilisation follow chart of the novel binder mixture

6.4.2 Soil-binder preparation and testing

The soil-binder mixture programme and testing were performed as already outlined in chapter 3 (see section 3.3). Hence, the sections following, include data presentation and analyses of results of the tests carried out involving the stabilisation of **Soil A** and **Soil E**.

As would be generally observed subsequently in this study, values of the engineering properties (swell, consolidation and unconfined compressive strength) of the natural clays as discussed in chapter 4 were much improved when treated with the different compositions and quantities of the binders used. However, in keeping with the primary objective of this chapter, a comparison shall only be drawn between the engineering behaviour of the clays stabilised with OPC alone and the clays stabilised by inclusion of the BPA (GGBS and RC). Most importantly, the resulting effect of using RC in the soil-binder combinations shall be highlighted.

6.5 STABILISED SOIL A

6.5.1 Swell

Figure 6.3 compares the rate of linear expansion of **Soil A** stabilised by the binding agents after 7 and 28 days of curing. There is a significant drop in the rate of linear expansion as the curing period increases. Generally, though, for the shorter curing duration, Figure 6.3a indicates a progressive increase in the expansion rate as the proportion of OPC in the soil reduces. However, a much closer investigation of the swell-strain path followed by the stabilised soil shows that within about 10-15mins after inundation with water, the lowest expansion does occur in the mixtures with 50 and 60% of the OPC replaced by the by-product additives at 7 days curing period. Nevertheless, at the same curing duration of 7 days, there seems to be only a slight difference in the final expansion (at equilibrium) between the mixture stabilised by OPC alone and that in which 50% of OPC is replaced by the by-product materials. The greatest linear swelling immediately after inundation until equilibrium was reached, is exhibited by the mixture having 30% of the OPC proportion at 7 days of curing. With the curing period extended to 28 days, it can be seen in Figure 6.3b that the soil stabilised by using only the OPC alone seems to show the highest expansion as compared to the mixtures having the by-products. There appears not to be any difference in the expansion rate with 30 and 40% of the OPC used in stabilisation of the soil with both swelling up to about 0.01% at equilibrium whereas the of expansion is reduced to almost zero with half of the OPC proportion substituted by the by-product additives.

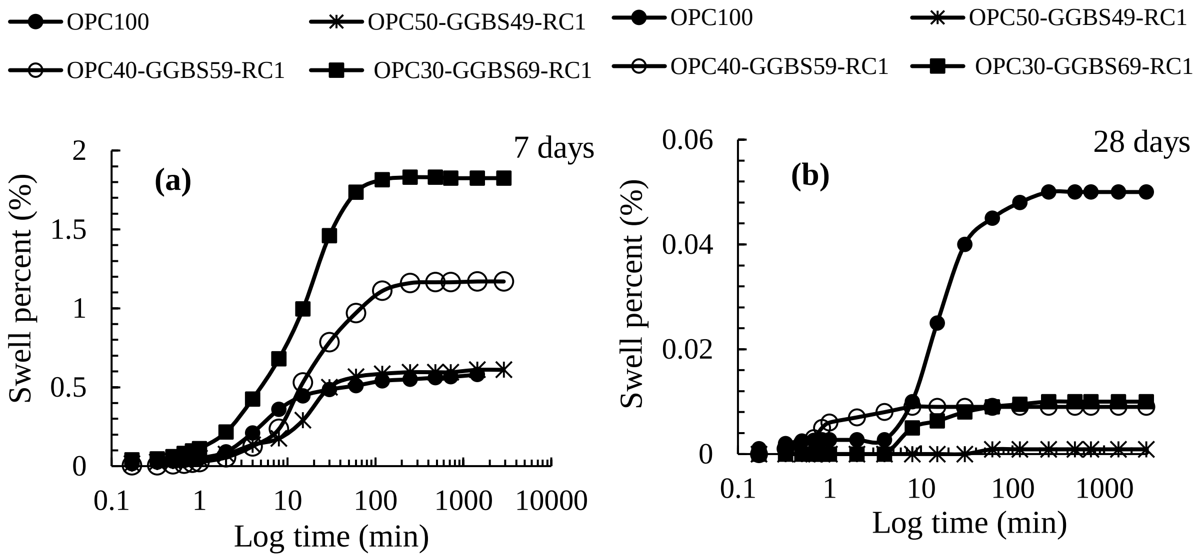


Figure 6.3 Swell potential of stabilised soil (a) 7-day cured stabilised soil for 50, 60 & 70% OPC replacement (b) 28-day cured stabilised soil for 50,60,70% OPC replacement.

An examination of the effect of the incorporation of RC in the stabilised soil with up to 50% of the OPC substituted can be readily observed in Figures 6.4. Addition of RC to the soil-binder mix reduces the expansion rate to almost zero as compared to the mixture without the RC as the curing was extended to 28 days. At 50% OPC replacement, the percentage difference between the samples that contained RC and those without the RC are 46% and 97% at 7- and 28-days period of curing respectively.

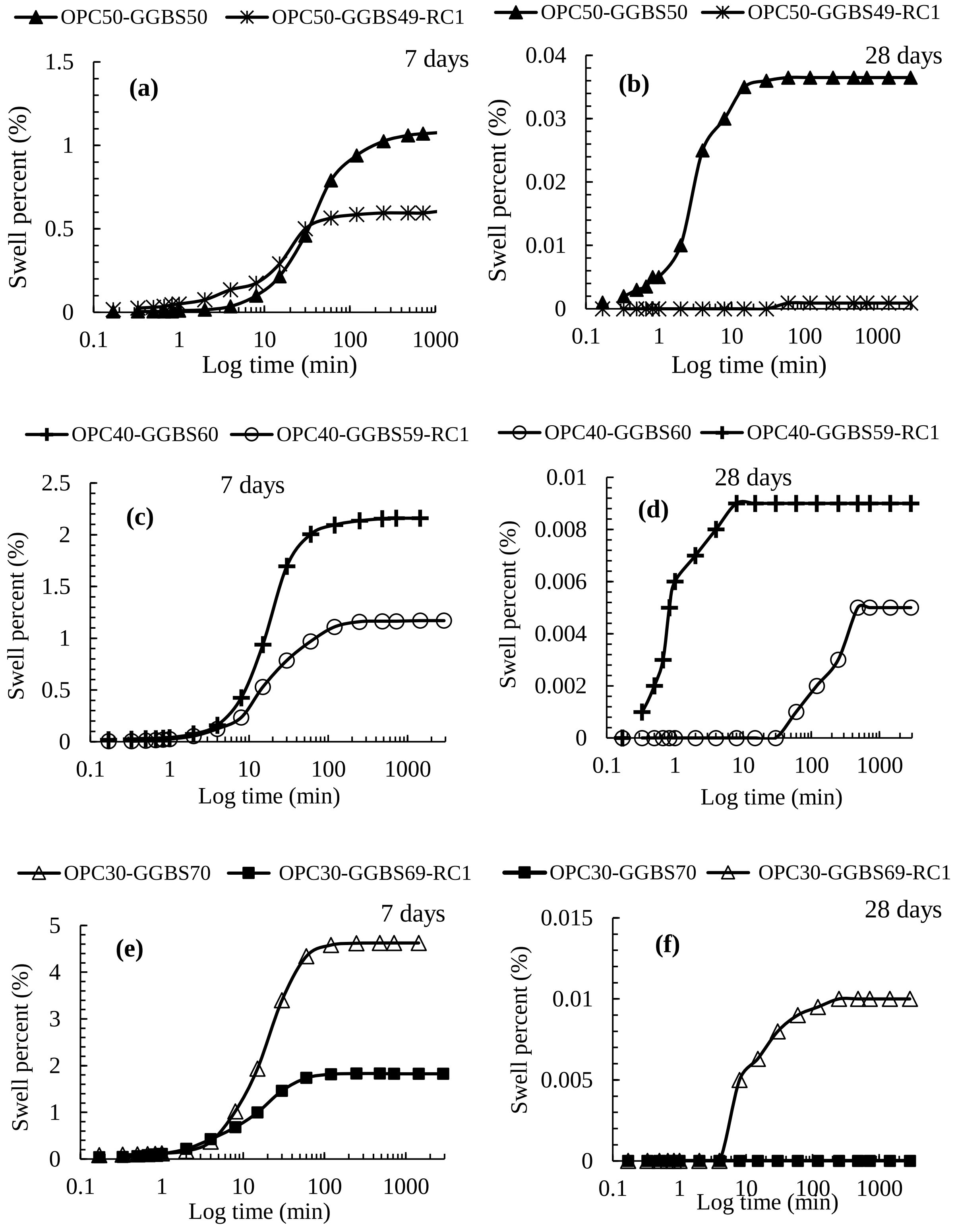


Figure 6.4 Stabilised soil swelling showing effect of RC (a) 7-day cured stabilised soil for 50% OPC replacement (b) 28-day cured stabilised soil for 50% OPC replacement (c) 7-day cured stabilised soil for 60% OPC replacement (d) 28-day cured stabilised soil for 60% OPC replacement (e) 7-day cured stabilised soil for 70% OPC replacement (f) 28-day cured stabilised soil for 70% OPC replacement

6.5.2 Compressibility

The behaviour of the stabilised **Soil A** under consolidation by comparing the mixtures with only the OPC used to those in which the by-product materials are added is shown plotted on the e-log p curve of Figures 6.5. Unlike the case of swell as indicated in the previous section, there appears not to be a huge difference in consolidation rates with the curing time. Nevertheless, as could be observed in Figure 6.5a, there is an increase in the initial void ratios of the stabilised soil as the percentage of the OPC binder reduces in the mixtures at 7 days curing duration. Moreover, at the same period of curing, Figure 6.5a indicates further that the initial void ratio and the compression path followed by **Soil A** stabilised with only OPC and the OPC replaced by 50% of the by-products are almost the same. The highest value of the initial void ratio is observed for the mixture with 30% of the OPC used over the 7-day duration of curing. But, as the period of curing increases to 28 days, **Soil A** stabilised by only OPC seems to bear the least initial void ratio (Figure 6.5b). However, examining the amount of compression induced interpreted in terms of the compression index (Cc) in the stabilised soil is shown in Figure 6.6. Much of the compression occurs in the soil-binder mixture with OPC replacement by the proportions of the by-product materials used especially over 28 days of curing. Hence, it could be observed that even though the OPC-only stabilised soil has the least initial void ratio at 28 days of curing, it tends to consolidate more than all the mixtures possessing the by-product additives but with 50% of the OPC replaced having the least compression index Cc, value

A comparison of the soil-binder mixture with and without the RC added indicates an increase in the initial void ratio for the mixture containing the RC for all the investigated proportions of the OPC replaced as seen on the e Log P curves of Figure 6.7. This phenomenon is expected given that the initial void ratio before the beginning of the consolidation loading corresponds to the end of swelling at equilibrium. Hence, mixtures without the RC should have higher initial void ratio since they swelled more than the mixtures containing the RC as seen in the previous section. However, from the point of view of degree of settlement using the compression index values to compare **Soil A** stabilised with 50% of OPC replaced with the by-product possessing RC and without RC reveals that the mixture having the RC seems to consolidate much less than that without the RC at 7- and 28-days curing with the differences being approximately 30% and 50% respectively (Figure 6.8).

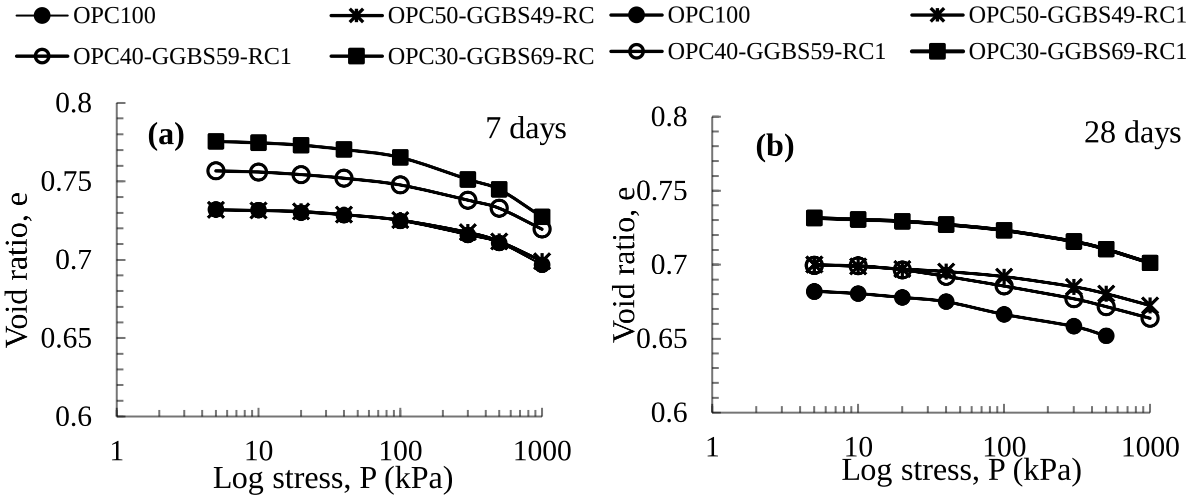


Figure 6.5 Stabilised soil consolidation (a) e vs. log P curve of 7-day cured stabilised soil for 50, 60 & 70% OPC replacement (b) e vs. log P curve of 28-day cured stabilised soil for 50, 60 & 70% OPC replacement.

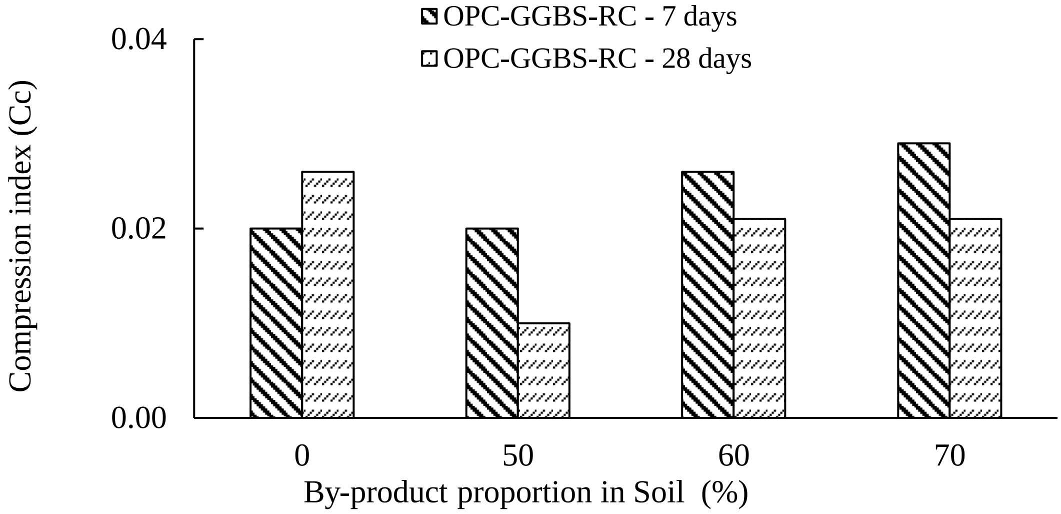
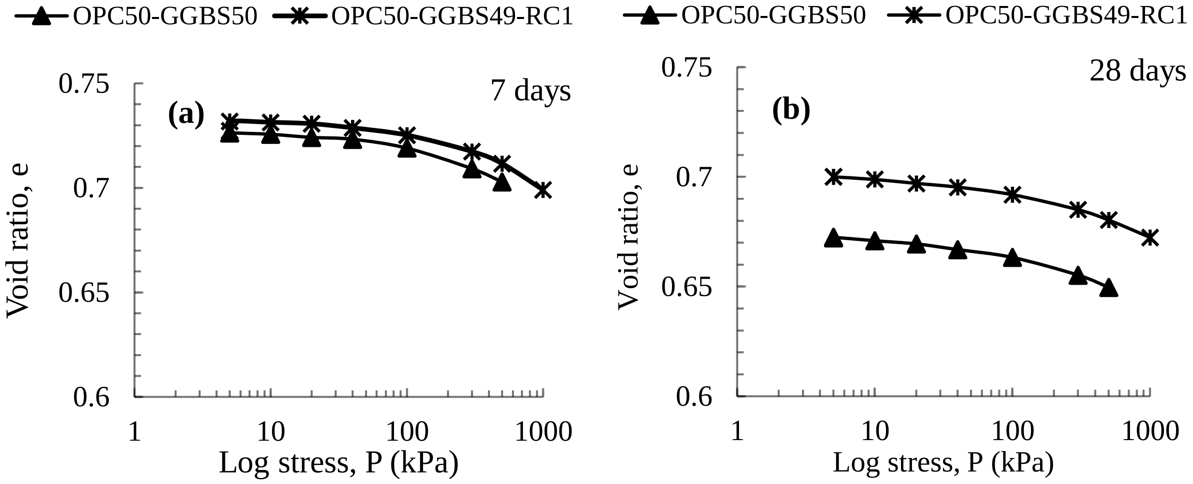


Figure 6.6 Stabilised soil compression indices of 7- & 28-day cured stabilised soil for 50, 60 & 70% OPC replacement.



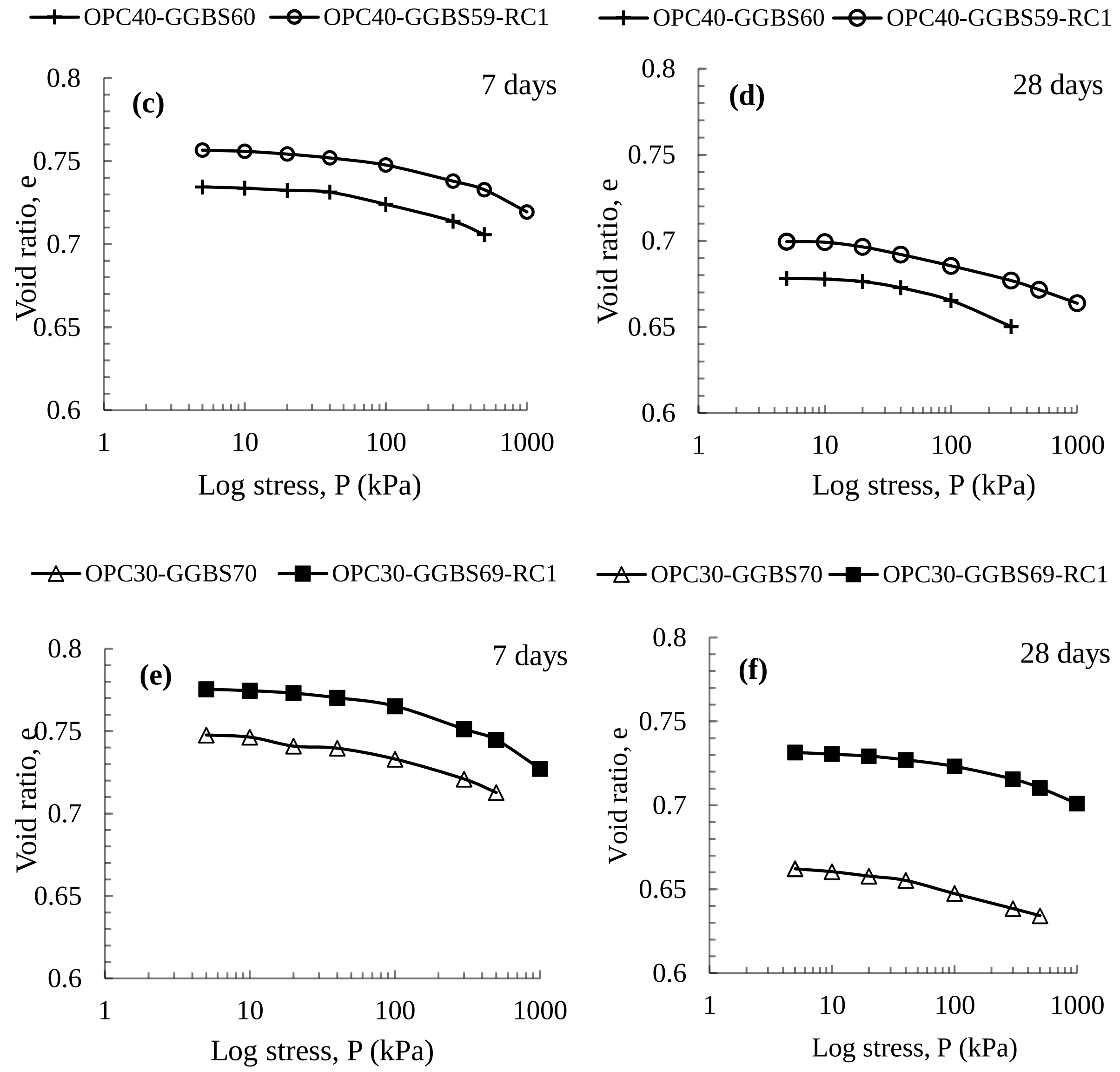


Figure 6.7 Stabilised soil e vs. log P consolidation curve showing effect of RC (a) 7-day cured stabilised soil for 50% OPC replacement (b) 28-day cured stabilised soil for 50% OPC replacement (c) 7-day cured stabilised soil for 60% OPC replacement (d) 28-day cured stabilised soil for 60% OPC replacement (e) 7-day cured stabilised soil 70% OPC replacement (f) 28-day cured stabilised soil for 70% OPC replacement.

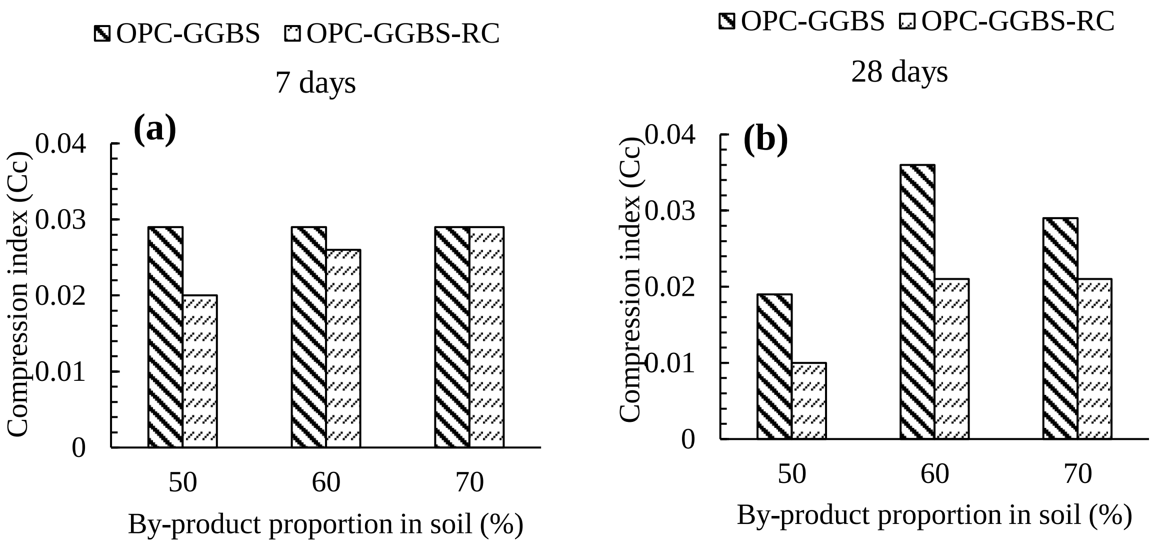


Figure 6.8 Stabilised soil compression index showing effect of RC (a) 7-day cured stabilised soil for 50, 60 & 70% OPC replacement (b) 28-day cured stabilised soil for 50, 60 & 70% OPC replacement.

6.5.3 Unconfined compressive strength (UCS)

The UCS of all the OPC-by-product combinations in the stabilised soil increases generally more than the OPC-only stabilised soil at 7 days of curing (Figure 6.9a). The percentage gain in strength (peak value) between the soil stabilised by OPC used alone and that in which half of the OPC proportion is replaced is about 80% at 7 days of curing. However, after 28 days of curing the samples, the strength gained in the soil-binder mixtures containing OPC alone seems to rise more remarkably to slightly more than twice of its maximum strength value at 7 days. There is also a noticeable but relatively slow increase in strength occurring in the OPC-replaced mixtures over the 28 days period of curing. Nevertheless, the strength of the stabilised soil with the OPC replaced by 50 and 60% of the by-products are both higher than that of the soil stabilised using only OPC (Figure 6.9b).

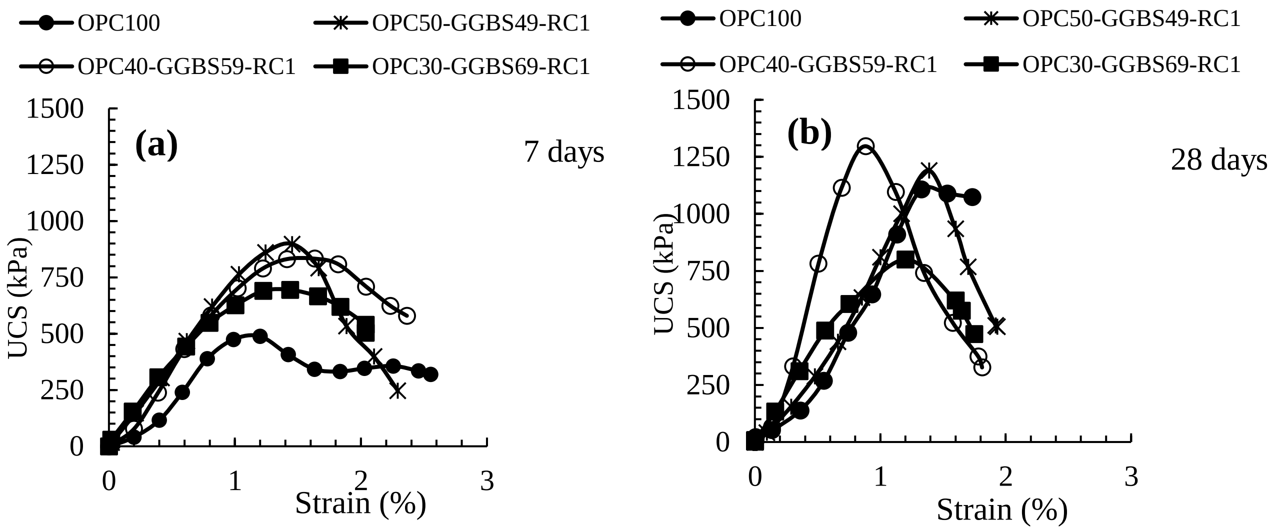


Figure 6.9 Unconfined compressive strength of stabilised soil (a) 7-day cured stabilised soil for 50, 60 & 70% OPC replacement (b) 28-day cured stabilised soil for 50,60,70% OPC replacement

When comparing the mixtures without RC and that in which RC is added, it could be seen in Figure 6.10 that at 7 days of curing, the peak value of the UCS of the mixture without RC is only slightly less than that containing the RC. But, with the curing extended to 28 days, the mixture without the RC tends to increase more than that having the RC. This interesting occurrence could be related to the e-log p curve (Figure 6.7) where the mixture without the RC had a slightly less initial void ratio at the start of consolidation as compared to the mixture that contains the RC. Hence, the rate of strength gain for the samples containing the RC seems to be slightly slower than the samples without the RC.

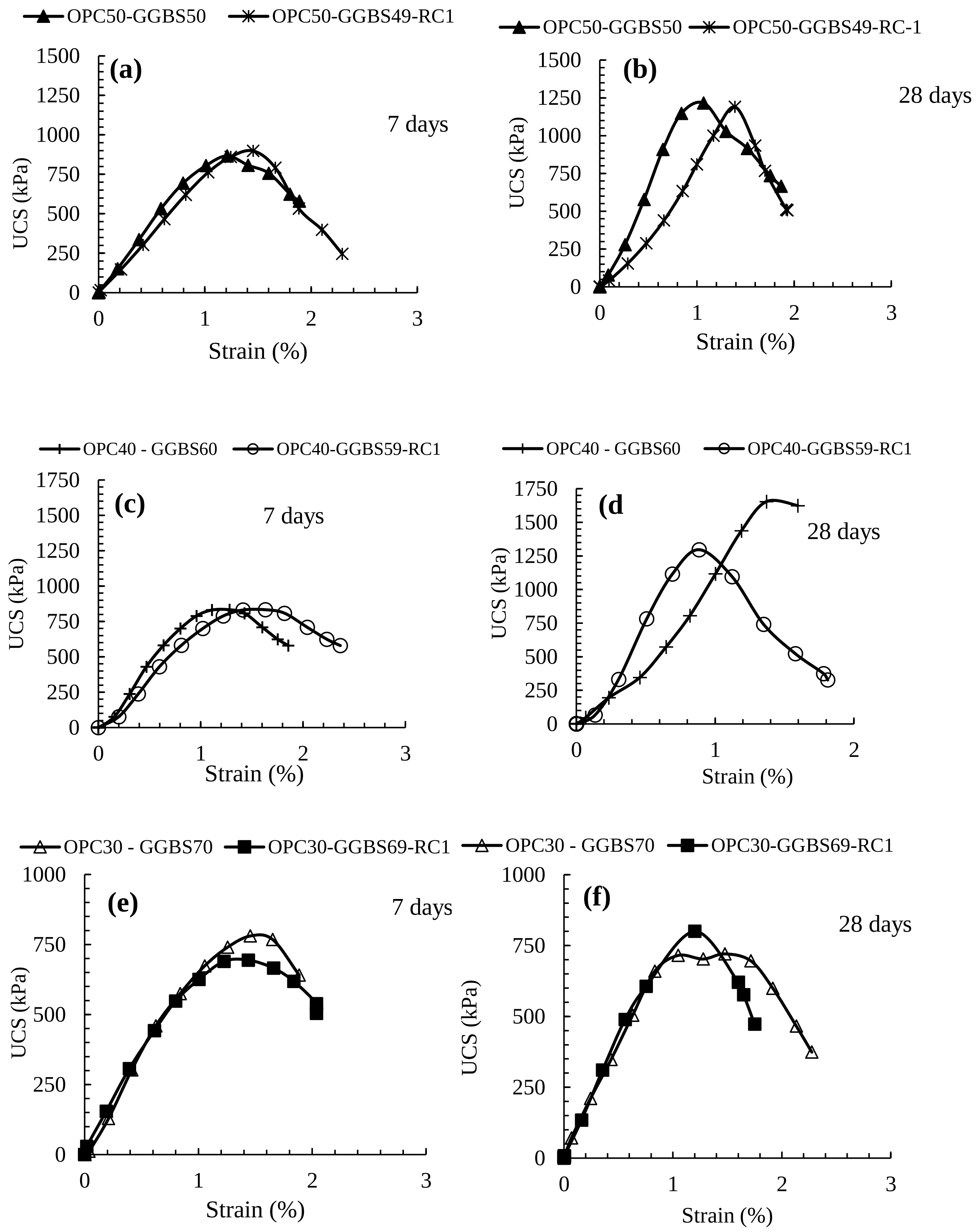


Figure 6.10 Unconfined compressive strength of stabilised soil indicating the effect of RC (a) 7-day cured stabilised soil for 50% OPC replacement (b) 28-day cured stabilised soil for 50% OPC replacement (c) 7-day cured stabilised soil for 60% OPC replacement (d) 28-day cured stabilised soil with and without RC for 60% OPC replacement (e) 7-day cured stabilised soil for 70% OPC replacement (f) 28-day cured stabilised soil for 70% OPC replacement

6.5.4 Moisture retention behaviour

The SWRC describes the relationship between the mass of moisture present in a soil and the corresponding energy state or suction within the pore water. The behaviour of the SWRC is herein used to forge an understanding of the effect of stabilisation on the two model soils used. The moisture retention behaviour of the samples stabilised with 50% replacement of the cement are studied in this section since these appear to provide the most acceptable performance in terms of the studied strength and swell properties above. Furthermore, the SWRC of the stabilised samples are analysed irrespective of the curing condition given that the relatively longer curing duration of 28 days adopted in this study has been proven to have very minimal and in most cases no effect on the stabilised curve (Stoltz et al. 2012; Elkady and Al-Mahbashi 2013; Zhang et al. 2014, 2017a).

6.5.4.1 SWRC models for natural and stabilised clays

Suction data for the derivation of the wetting branch of moisture retention curves were collected by the measurements of the natural and stabilised samples using filter papers according to the ASTM D5298-16 (2016) method as described in chapter 3 (see section 3.3.1.10). Laboratory suction data were subjected to a nonlinear regression fitting process to obtain the SWRC by using the models proposed by Fredlund and Xing (1994) and van Genuchten (1980) both which are widely used in engineering practice (see chapter 3, Table 3.7). The soil module function of SoilVision program (version 5.4.08) was utilised to enable an effective non-linear fit of the suction data using the in-built fitting models. Since the wetting method was adopted for the determination of the moisture retention curves, the description given here will be most appropriate for the swelling behaviour of the natural and stabilised soil. The variation of air entry value (AEV) of the natural and stabilised soils are shown plotted in Figure 6.11. As could be seen, the FX fitting model seems to provide lower-bound AEV as compared to the VG model. AEV is that value of suction at which air will begin to penetrate the largest void structure and this occurs at the transition zone from unsaturation to saturation or vice versa. Since the soil's treatment mechanism (mainly the production of hydration or pozzolanic products) with binders would ultimately lead to a closely-packed and well-bound treated soil particles, it therefore follows that the AEV should rise as in Figure 6.11 as compared to the natural soil due to the binding effect that is occasioned by the used stabilisers (Khattab and Al-Taie 2006; Puppala et al. 2006; Elkady et al. 2015). **Soil A** stabilised by OPC used alone seem to produce the largest AEV as compared to the natural soils and those stabilised by a combination of OPC and by-products. This indicates that greater suction (capillary behaviour) tends to occur in the soil-OPC samples (as compared to the samples having the by-products) due to a preponderance of smaller pore spaces as the wetting progresses.

On the other hand, an examination of Figure 6.12 indicates that both the vG and the FX models seem to predict almost identical SWRC with the only differences observed as the values of suction becomes higher. However, it could be said that a slightly better fit is generally obtained by using the FX model as seen from the coefficient of determination (R^2) for the SWRC and is thus recommended for the stabilised **Soil A** containing mostly kaolinite.

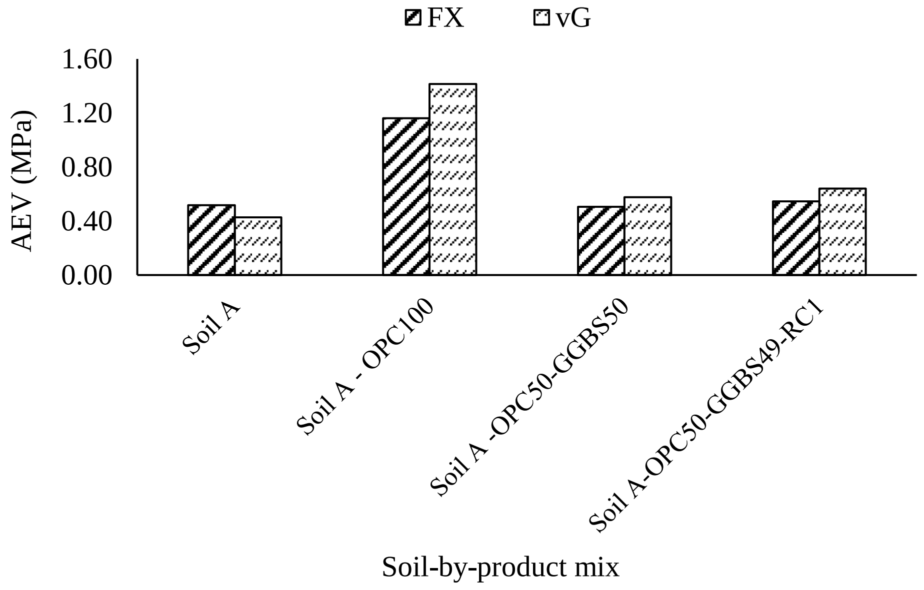
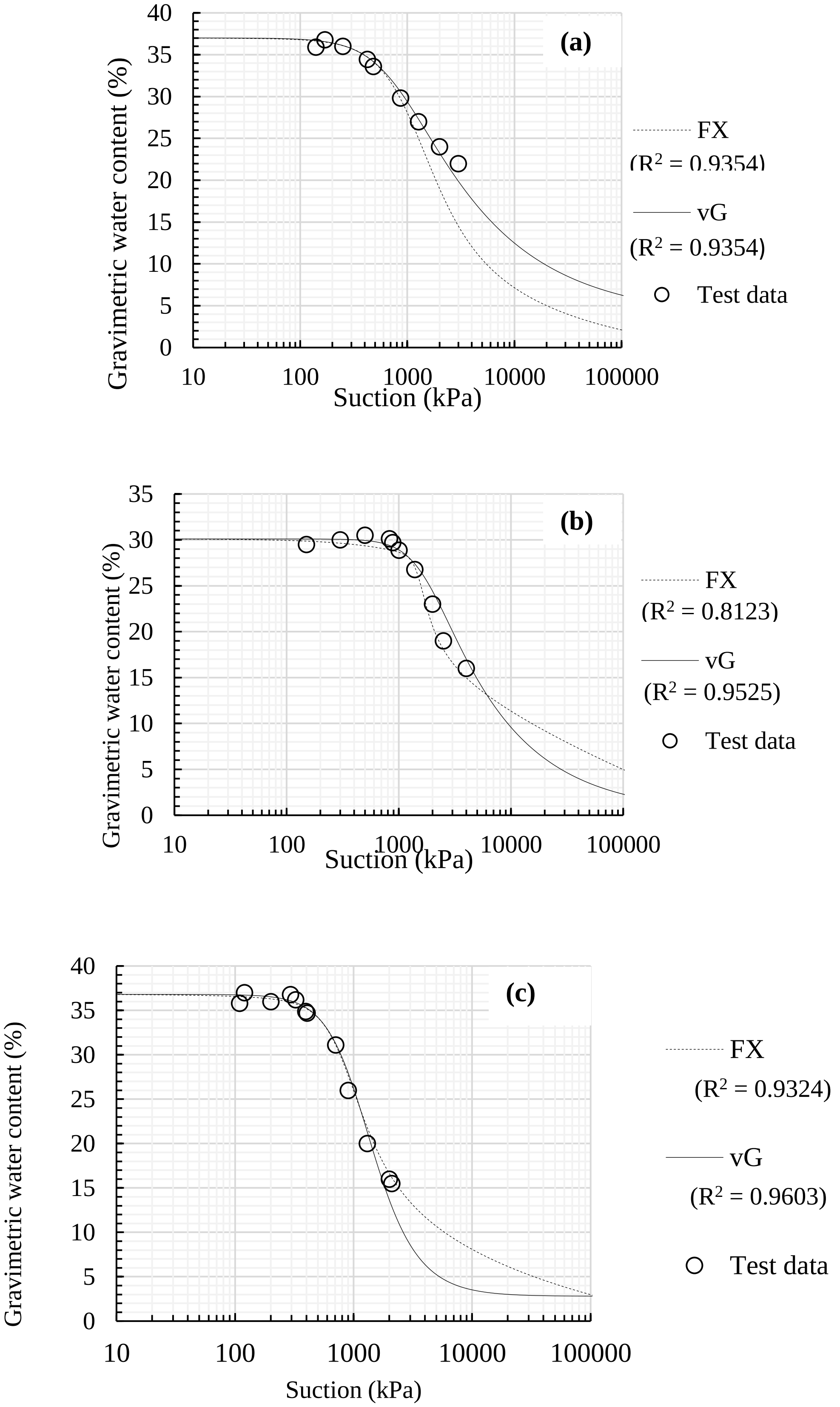


Figure 6.11 Air entry value (AEV) of natural and stabilised soil for van Genuchten (1980; Fredlund and Xing (1994) models



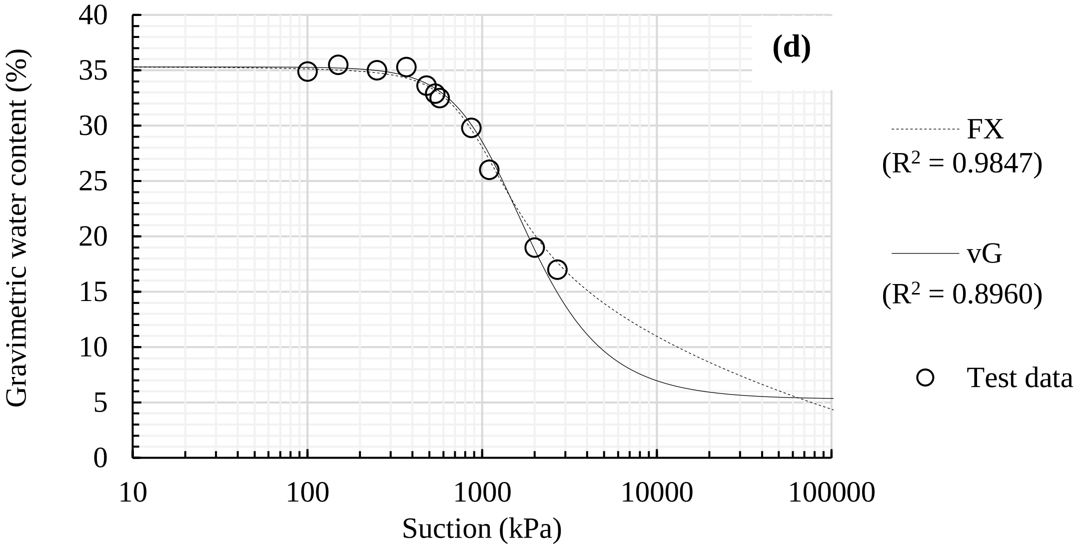


Figure 6.12 SWRC model comparisons for natural and stabilised clays (a) **Soil A** (b) Soil A - OPC100 (c) Soil A - OPC50-GGBS50 (d) Soil A - OPC50-GGBS50-RC1

Further comparison of the effects of the by-product addition in the stabilised samples is hereby carried out by relying on the FX model. As could be observed in Figure 6.13, the stabilised as-compacted soil samples tend to exhibit greater moisture retention capacity during the initial stages (water entry phase with suction approximately above 1000 kPa) of the wetting process as compared to the natural soil. This is to be expected given a modification of the physiochemistry and microstructure of the soil caused by treatment with binders. The exchangeable calcium ions from the binders alter the electrical charge (double diffused layer) that surrounds the clay enabling the formation of flocs (particles being attracted to one another) and an increase in the moisture content of the compacted mixed product (Bell 1996; Chew et al. 2002; Tedesco and Russo 2010). However, as the suction reduces further (especially below 1000 kPa) and as the saturation progresses, the stabilised soil using OPC alone tend to possess the lowest gravimetric moisture. It has been suggested that at reduced suction levels, the moisture storage mechanism is determined mostly by capillarity and the retention curve is thus influenced by soil fabric (Tedesco 2006). Accordingly, it is presumed that OPC replacement by either GGBS should lead to more of the pores being filled and a more reduced gravimetric moisture as compared to the OPC used alone (Keramatikerman et al. 2016; Zhang et al. 2018). However, it seems the presence of RC may have distorted this phenomenon slightly for the stabilised soil. It is interesting to also note the similar moisture retention behaviour of OPC-only stabilised and OPC-GGBS-RC stabilised at the higher suction range (above 1000 kPa). Overall, it can be inferred from Figure 6.11 and Figure 6.13 that much smaller void spaces are available for the penetration of the added water during the saturation process in the stabilised soil when only the OPC is utilised as compared to the combined OPC-by-product materials used especially at suctions below about 1000 kPa. In other words, the fast-reacting OPC used alone in the stabilisation of the soils does seem to thrive relatively more in the presence of sufficient hydration moisture. This further substantiates the slightly lower swelling potential value obtained (at zero suction) with the OPC only-stabilised clays as compared to the soil stabilised by a combination of OPC and the by-products (with RC included) at 7 days of curing.

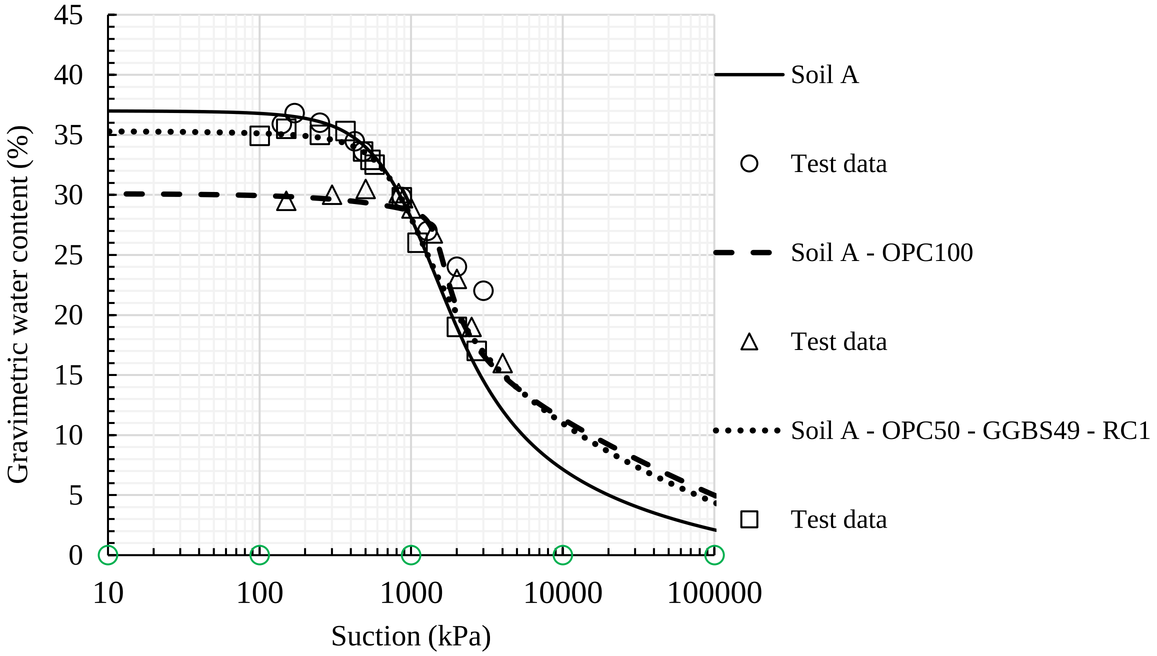


Figure 6.13 SWCC of stabilised soil

Table 6.2 FX fitting model parameters of stabilised **Soil A**

Samples	FX parameters		
	a	*n*	*m*
	(kPa)	-	-
Soil A	990	2.17	0.87
Soil A – OPC100	2322	12.80	1.74
Soil A – OPC50 – GGBS50	746	3.53	0.55
Soil A – OPC50 – GBBS49 – RC1	488	6.99	0.11

6.5.4.2 Effect of RC on SWRC

A comparison to depict the effect of the addition of RC to the stabilised mixes is shown plotted in Figure 6.14. The main observation is that the SWRCs of the stabilised soil samples with RC become relatively ‘flatter’ (demonstrated by the higher ‘*n’* values of Table 6.2.) which thus clearly demonstrates the effect of the RC in retaining more moisture as earlier claimed. Initially though, the water holding capacity of the stabilised soils having the RC is higher but tend to reduce as the saturation level increases with decreasing suction. Hence, further hydration may have possibly occurred with more saturation leading to the formation of a water-proof structure with reduced porosity at reduced suction. The greater moisture retention property is promising for contaminant encapsulation during dredging activities as suggested by Zhang et al. (2018) while the relatively reduced porosity (compared to the combination without RC) at low suctions is desirable for swell reduction in the subgrade of pavement structures. But it should be recalled that at reduced suction levels, the rapid hardening OPC used solely to stabilise the clays do possess slightly more reduced porosity and higher AEV as compared to the stabilised clays with the RC included.

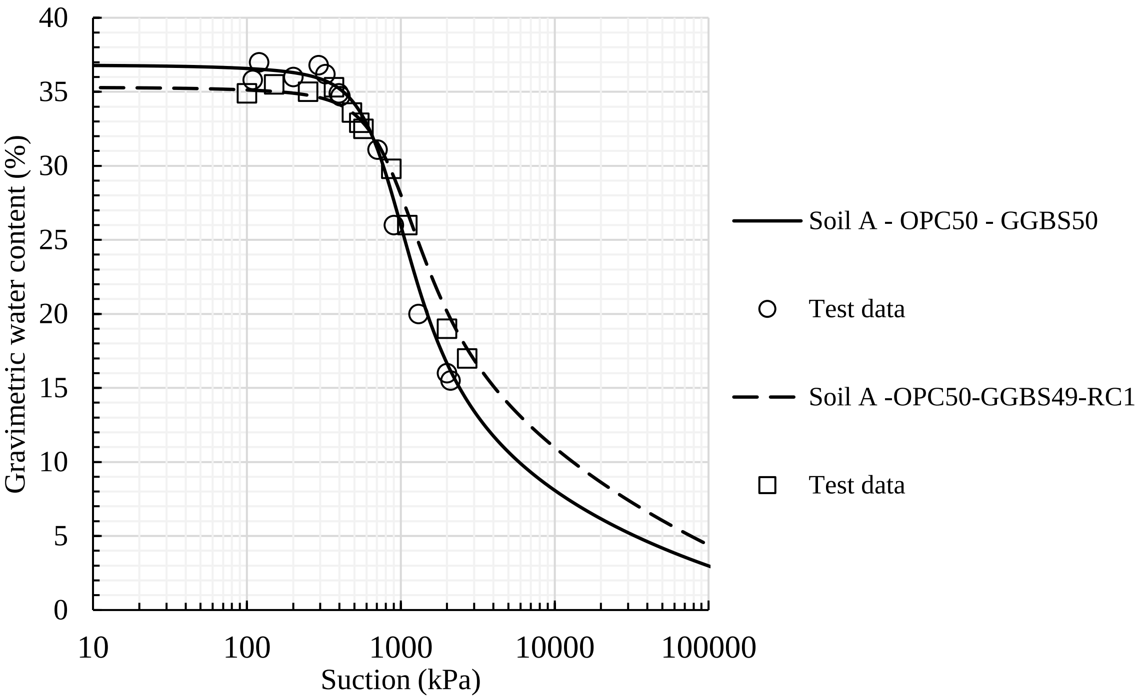


Figure 6.14 SWCC of stabilised soil showing effect of RC

6.6 STABILISED SOIL E

6.6.1 Swell

The behaviour of stabilised Soil E in swell using OPC alone and OPC combined with the by-products are shown plotted in Figures. 6.15a & b over the 7- and 28-day curing periods. There is a significant reduction in linear expansion as the curing period increases for the soil stabilised by OPC and all the binder combinations. Generally, though, for the shorter curing duration, a progressive increase in the expansion rate as the proportion of OPC in the soil reduces is noticeable as seen in Figure 6.15a. Nevertheless, at the same curing duration of 7 days, there seems to be only a slight increase in the final expansion (at equilibrium) between the mixture stabilised by OPC alone and that in which 50% of OPC is replaced by the by-product materials. The greatest linear swelling immediately after inundation until equilibrium was reached, is demonstrated by the stabilised soil containing 30% of the OPC proportion over 7 days of curing. With the curing period extended to 28 days, it can be seen in Figure 6.15 that the soil stabilised by using only the OPC seems to show the least expansion as compared to the mixtures with the by-products used. Hence, for the two investigated curing periods, OPC used alone appears to be more effective in stabilising **Soil E** compared to **Soil A** as discussed previously.

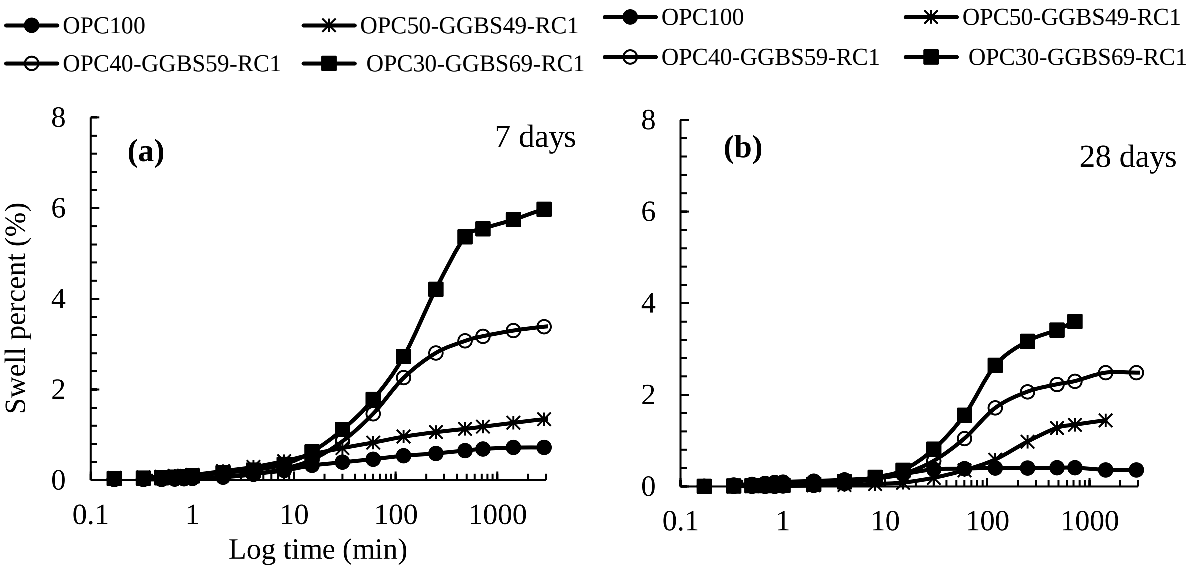


Figure 6.15 Swell potential of stabilised soil (a) 7-day cured stabilised soil for 50, 60 & 70% OPC replacement (b) 28-day cured stabilised soil for 50,60,70% OPC replacement.

The influence of RC on the swell behaviour of Soil E stabilised by the by-product combinations with the OPC replaced up to 50% in the mix are further investigated and presented in Figure 6.16.

Generally, the addition of RC reduces the swelling more than mixture without the RC used for both the 7 and 28 days investigated periods of curing. Using RC in the stabilised soil mixture decreases the linear expansion remarkably to almost zero over 28 days of curing for 50% of OPC replacement. Comparatively, only small decrements in the swelling are observed for OPC replaced by 60 and 70% of the by-products possessing the RC.

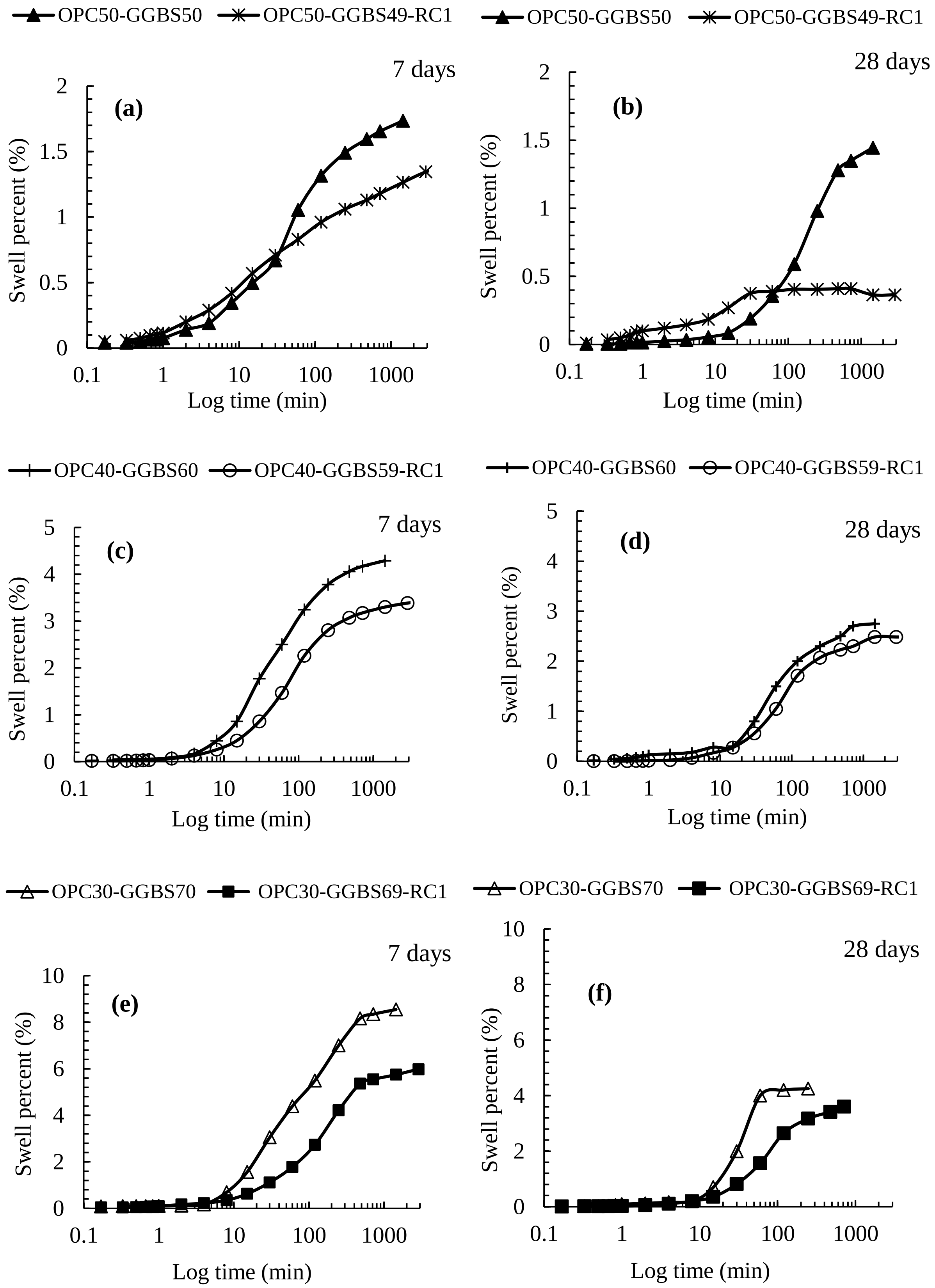


Figure 6.16 Stabilised soil swelling showing effect of RC (a) 7-day cured stabilised soil for 50% OPC replacement (b) 28-day cured stabilised soil for 50% OPC replacement (c) 7-day cured stabilised soil for 60% OPC replacement (d) 28-day cured stabilised soil for 60% OPC replacement (e) 7-day cured stabilised soil for 70% OPC replacement (f) 28-day cured stabilised soil for 70% OPC replacement

6.6.2 Compressibility

The behaviour of the stabilised **Soil E** under consolidation by examining the mixtures with only the OPC used to those in which the by-product materials are added is shown plotted on the e-log p curve of Figure 6.17. The effect of curing does not seem to bear much effect on the stabilised Soil E for all the binders and their various combinations and proportions used given that the initial ratio of voids is very minimal between 7 and 28 days of curing. However, it is interesting to notice at first glance that OPC replaced by 50% of the by-product materials in the stabilised soil seems to possess the greatest compressibility as seen from the e log p curves for 7 days and 28 days of curing. Nevertheless, as could also be observed there is a gradual decrease in the initial void ratios of the stabilised soil as the percentage of the by-products increases in the mixtures over 7 days period of curing. An investigation of the amount of compression induced in terms of the compression index (Cc) in the stabilised soil is shown in Figure 6.18. As was noticed in the e-log p curve especially over 7 days of curing, OPC replaced by 50% of the by-product produces the greatest compression. Furthermore, the least value of Cc is seen in the stabilised soil with OPC used alone at 7 days of curing. Meanwhile, there is only a slight variation in the Cc for 30% and 40% of OPC substituted by the by-products. However, as the period of curing is increased to 28 days, the Cc reduces more for the stabilised soil with OPC replaced by 50% of the by-product as compared to OPC substituted by 60 and 70% of the by-products. But the soil stabilised by using OPC alone reduces the Cc more than those of the by-products used in the soil stabilisation.

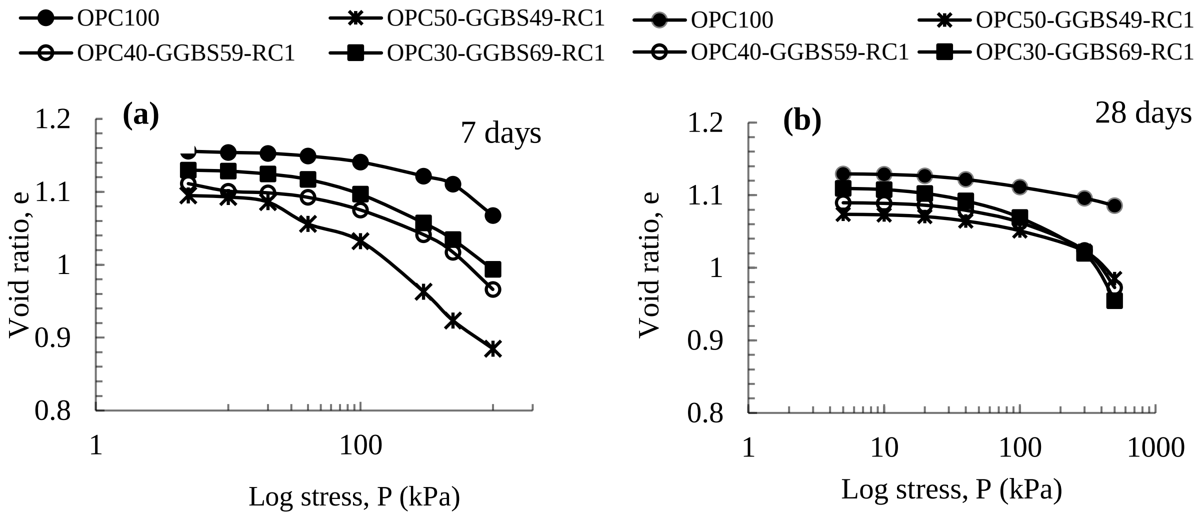


Figure 6.17 Stabilised soil consolidation (a) e vs. log P curve of 7-day cured stabilised soil for 50, 60 & 70% OPC replacement (b) e vs. log P curve of 28-day cured stabilised soil for 50, 60 & 70% OPC replacement.

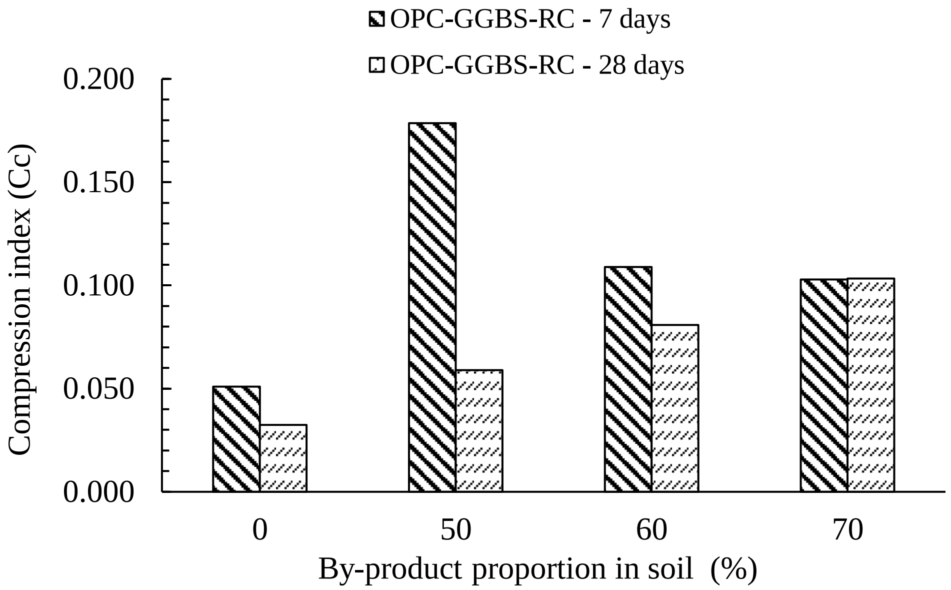
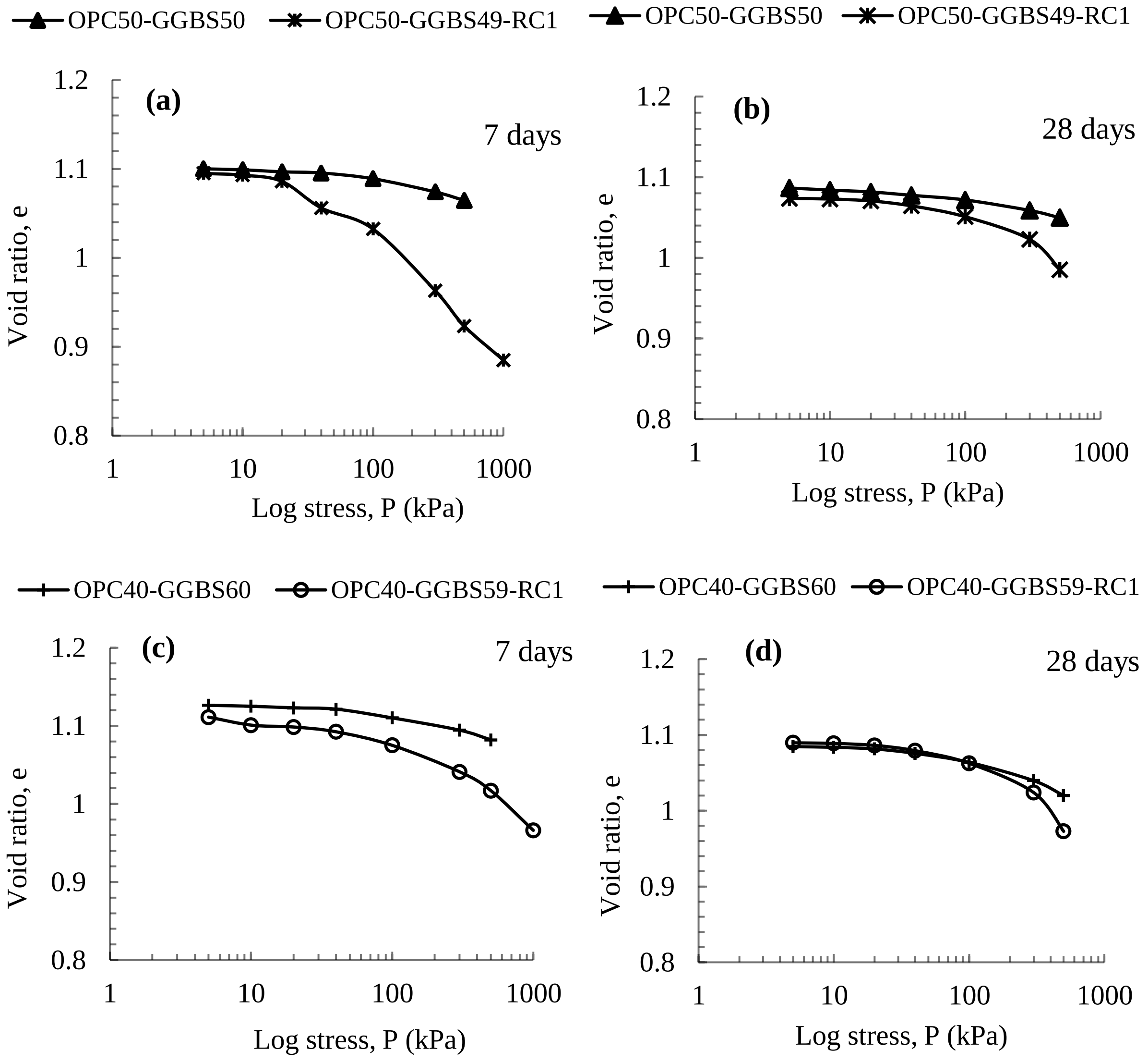


Figure 6.18 Stabilised soil consolidation and compression index (a) e vs. log P curve of 7-day cured stabilised soil for 50, 60 & 70% OPC replacement (b) e vs. log P curve of 28-day cured stabilised soil for 50, 60 & 70% OPC replacement (c) compression index of 7- & 28-day cured stabilised soil for 50, 60 & 70% OPC replacement

The effect of RC within the by-product mixture is further analysed and presented in Figure 6.19. Comparing the stabilised soil containing RC and those without the RC added indicates that the RC-laden mixtures have the greater compression for all the investigated proportions of the OPC substituted. This phenomenon is expected given that the initial void ratio before the beginning of the consolidation loading corresponds to the end of swelling at equilibrium. Hence, mixtures without the RC should have higher initial void ratio since they swelled more than the mixtures containing the RC as seen previously in Figure 6.16. However, from the perspective of the degree of settlement, it is noticed generally that the stabilised soil mix possessing RC seem to consolidate more because of the higher values of the Cc (Figure 6.20) calculated from the e-log p curves for both 7- and 28-days curing duration.



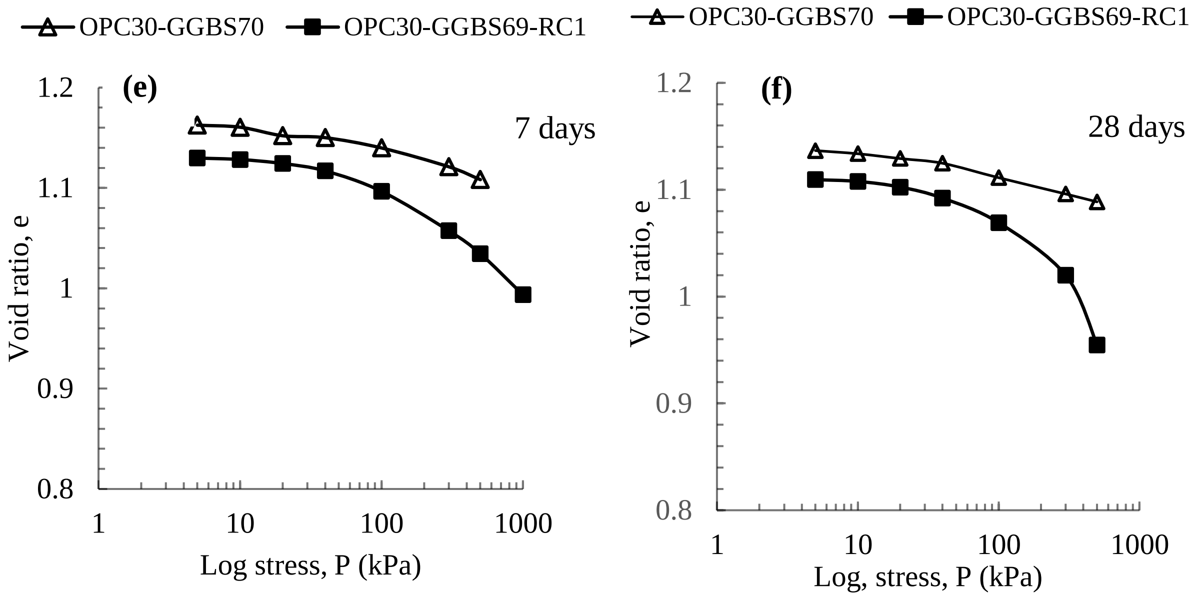


Figure 6.19 Stabilised soil e vs. log P consolidation curve showing effect of RC (a) 7-day cured stabilised soil for 50% OPC replacement (b) 28-day cured stabilised soil for 50% OPC replacement (c) 7-day cured stabilised soil for 60% OPC replacement (d) 28-day cured stabilised soil for 60% OPC replacement (e) 7-day cured stabilised soil 70% OPC replacement (f) 28-day cured stabilised soil for 70% OPC replacement.

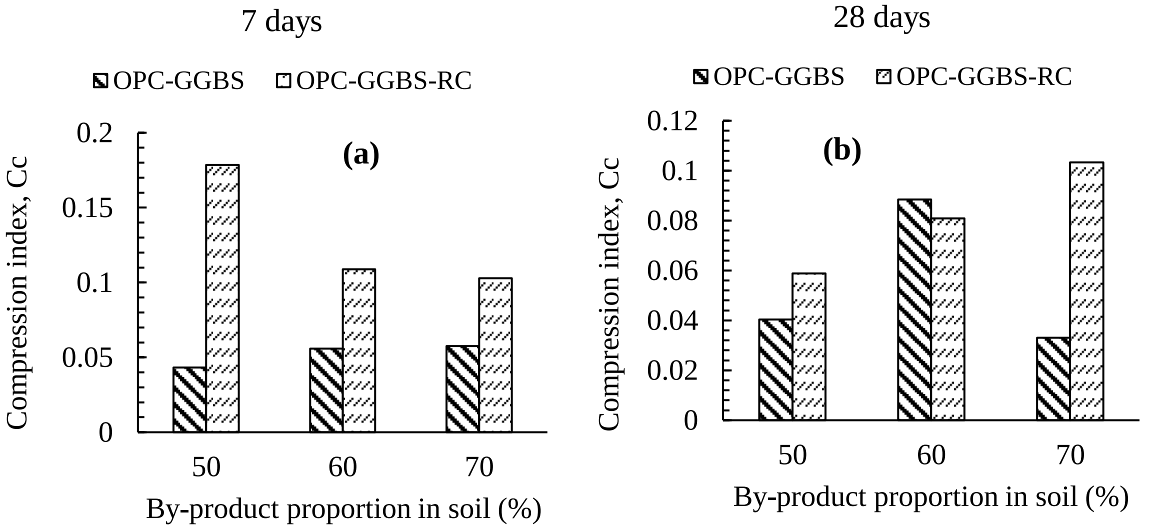


Figure 6.20 . Stabilised soil compression index showing effect of RC (a) 7-day cured stabilised soil for 50, 60 & 70% OPC replacement (b) 28-day cured stabilised soil for 50, 60 & 70% OPC replacement.

6.6.3 Unconfined Compressive Strength

Figure 6.21 shows the UCS of the stabilised soil and depicts the OPC-by-product combinations along with the OPC used alone in the stabilisation. As could be observed, the peak strength of **Soil E** stabilised by the OPC-by-product binder mixture with 50% of the OPC replaced seems to be equal (at about 1,300kPa) to that in which only the OPC is used over the 7 days period of curing. Meanwhile, the maximum strength of **Soil E** stabilised with the lower percentages of OPC in the mixture happen to fluctuate around 700kPa. With the period of curing increased to 28 days, the strength gained in the soil-binder mixtures containing OPC alone seems to rise more remarkably higher than the stabilised soils with the by-products added (Figure 6.21b). Moreover, as compared to the 7 days curing, a rather brittle failure of the stabilise samples are noticed at 28 days. Hence, there is a slow gain in strength of the soil stabilised by the OPC-by-product combinations as the curing period increases.

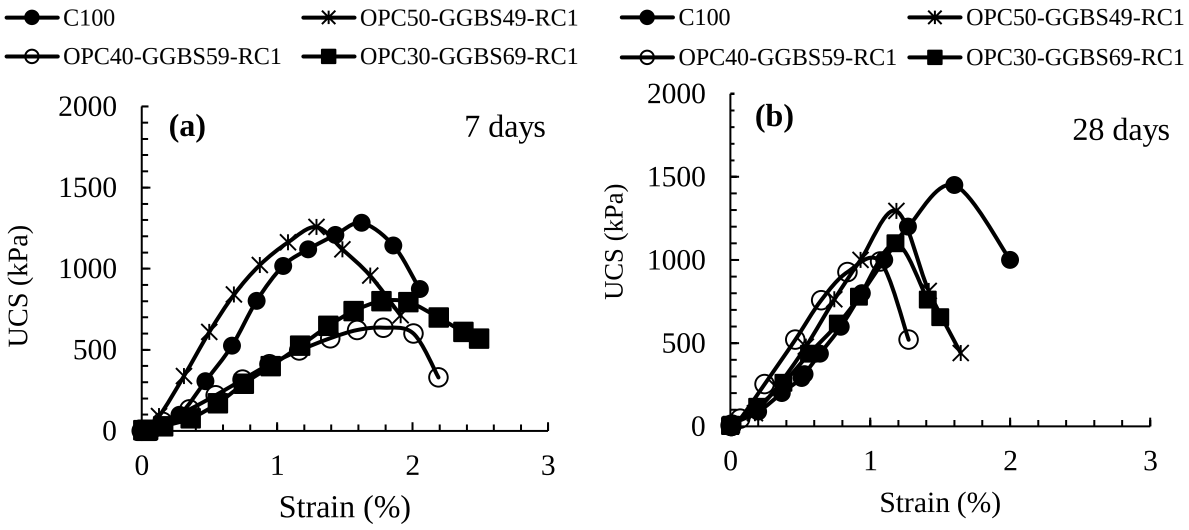
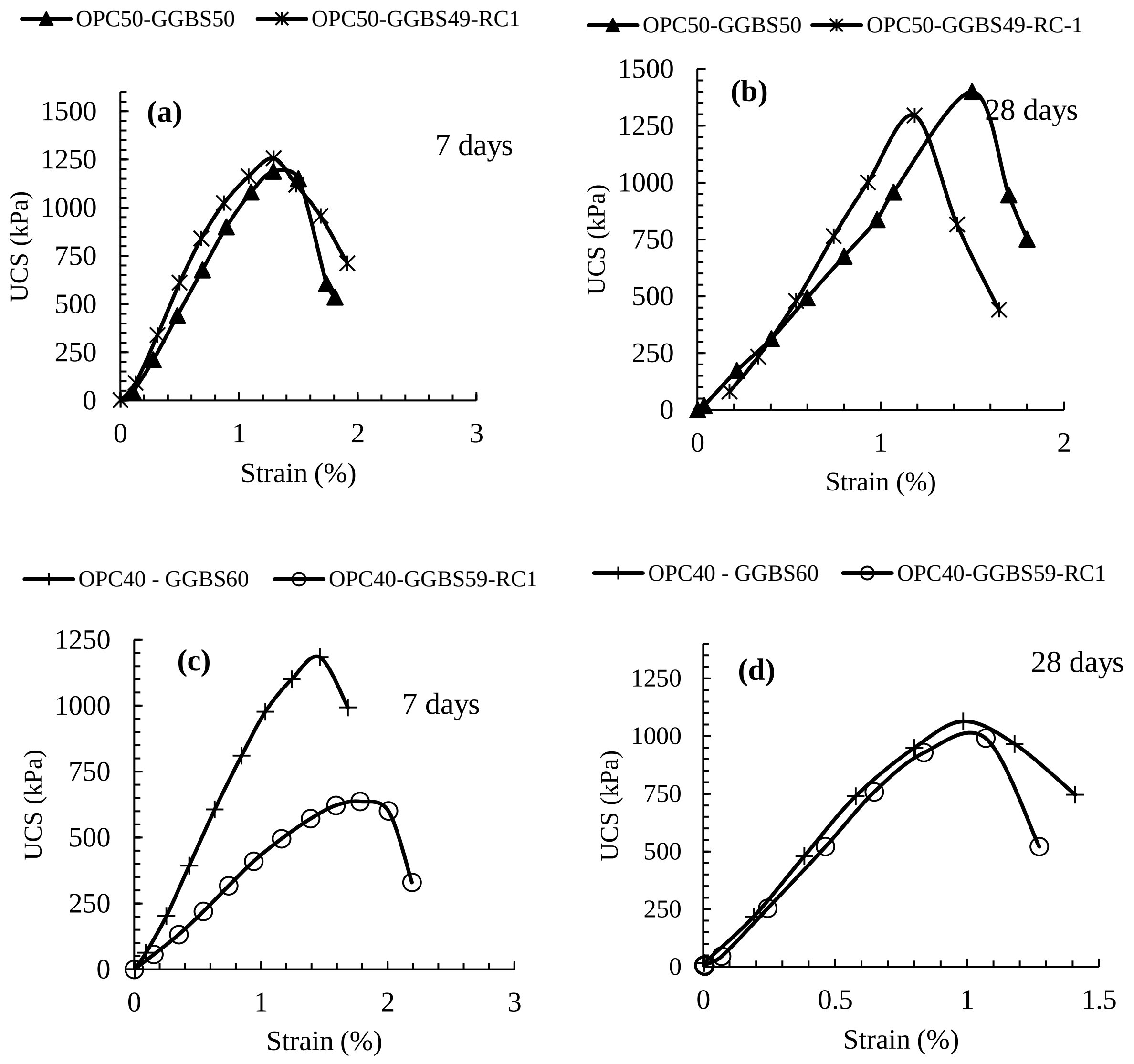


Figure 6.21 Unconfined compressive strength of stabilised soil (a) 7-day cured stabilised soil for 50, 60 & 70% OPC replacement (b) 28-day cured stabilised soil for 50,60,70% OPC replacement.

Figure 6.22 depicts the stress-strain curve of soil E stabilised by the OPC-by-product binders and indicates the influence of RC in the soil-binder mixtures. A comparison of the mixtures without RC and that in which RC is added, it could be seen that the peak value of the UCS with 50% replacement of and inclusion of RC is slightly higher than the stabilised soil without RC at 7 days of curing. However, as the curing increases to 28 days, **Soil E** stabilised by the OPC-by-product with 50% of the OPC substituted but without RC inclusion seems to have a higher peak UCS value (1400 kPa) than the stabilised soil containing the RC with a peak value of approximately 1350kPa (Figure 6.22b). For the stabilised **Soil E** with 60 and 70% replacement of OPC by the by-products, it is observed in Figures 6.22 (c-f) that the soil-binder mixture without the RC included are generally slightly higher than those with the inclusion of RC over the investigated 7 and 28 -days period of curing.



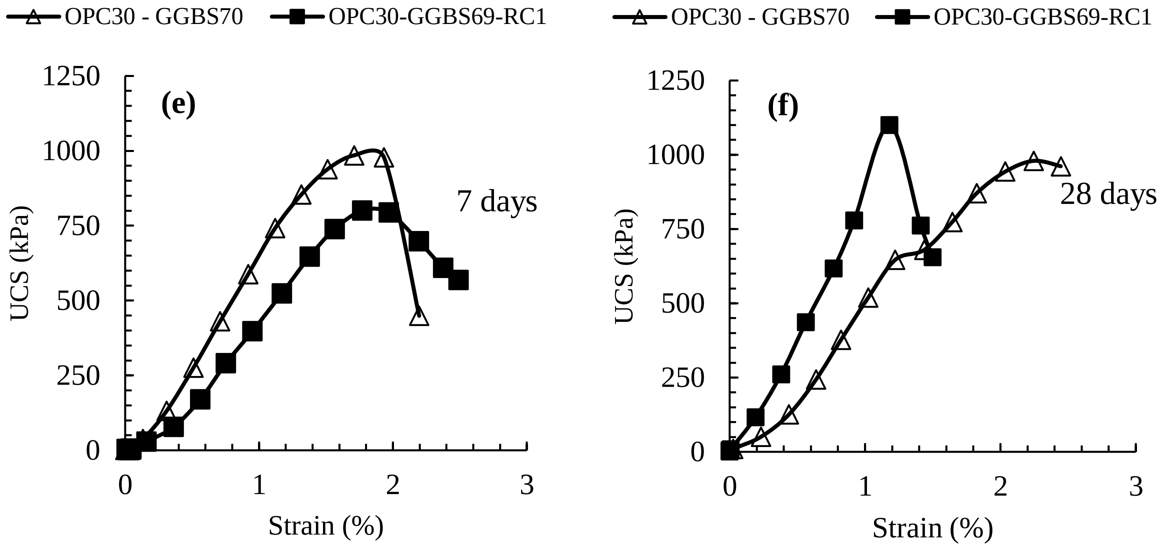


Figure 6.22 Unconfined compressive strength of stabilised soil indicating the effect of RC (a) 7-day cured stabilised soil for 50% OPC replacement (b) 28-day cured stabilised soil for 50% OPC replacement (c) 7-day cured stabilised soil for 60% OPC replacement (d) 28-day cured stabilised soil with and without RC for 60% OPC replacement (e) 7-day cured stabilised soil for 70% OPC replacement (f) 28-day cured stabilised soil for 70% OPC replacement

6.6.4 Water retention properties

The variation of air entry value (AEV) of the natural and stabilised soils are shown plotted in Figure 6.23. As could be seen, the FX fitting model appears to provide lower-bound AEV as compared to the VG model. As mentioned previously, given that soil stabilisation mechanism involves the production of hydration or pozzolanic products, it therefore, follows that the AEV should increase. As indicated in Figure 6.23 **Soil E** stabilised by OPC used alone seem to produce the largest AEV as compared to the natural soils and those stabilised by a combination of OPC and by-products. This indicates that greater suction (capillary behaviour) tends to occur in the soil stabilised by only the OPC used alone (as compared to the samples having the by-products) probably due to an increase in the number of smaller pore spaces.

An examination of Figure 6.24 indicates that both the vG and the FX models seem to predict almost the same pattern of the SWRC just as was the case in **Soil A**. However, it could be said that the best fit is generally obtained by using the FX model as seen from the coefficient of determination (R^2) for the SWRC and is thus recommended for the stabilised **Soil E** containing mostly the montmorillonite proportion.

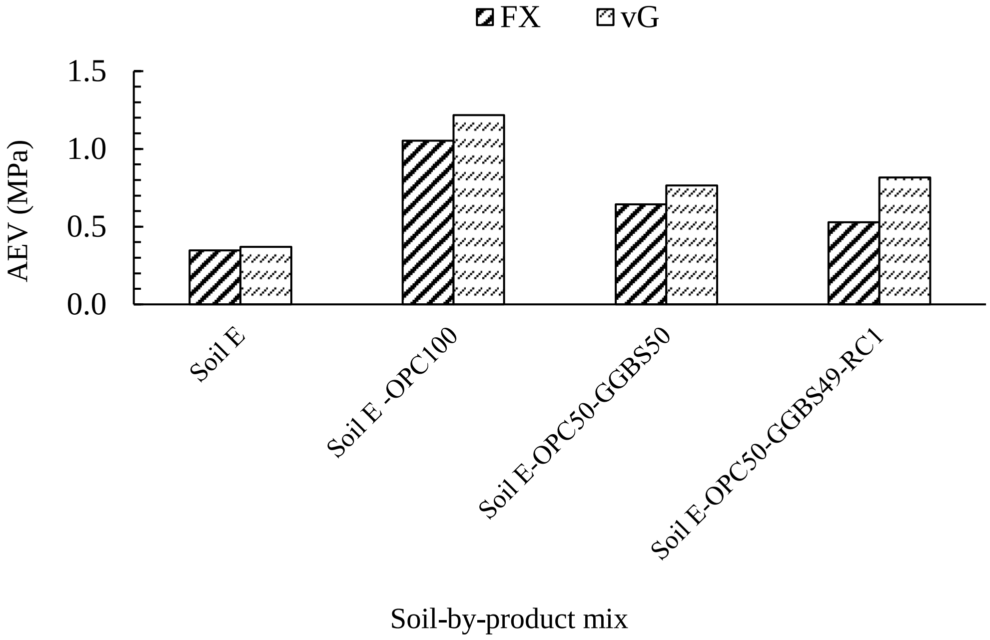
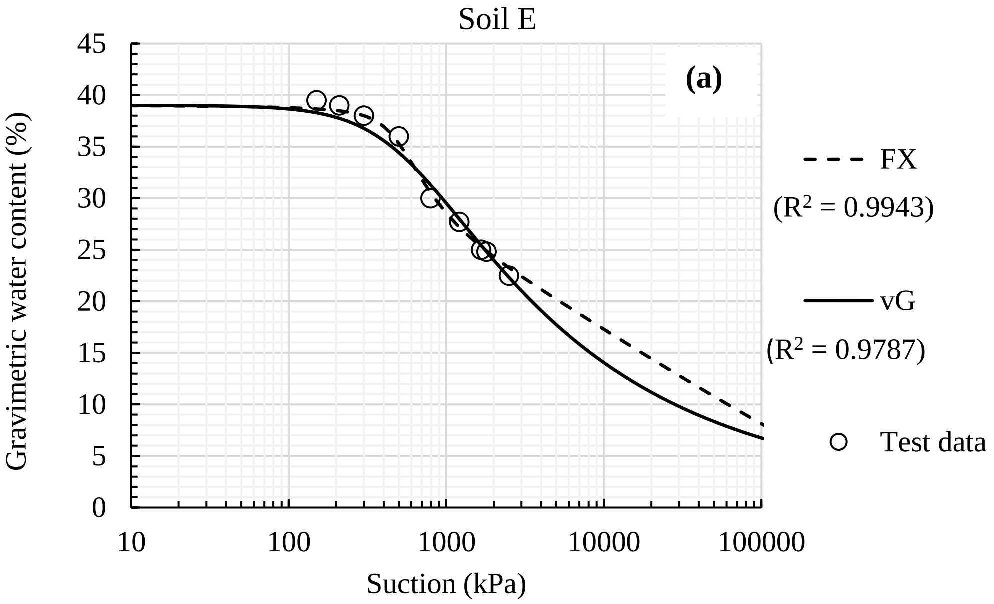
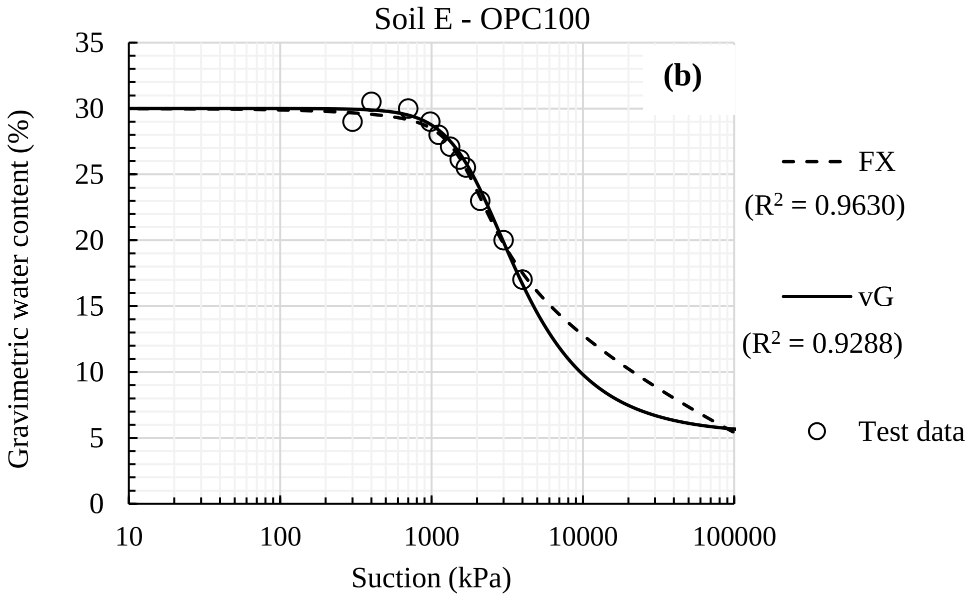
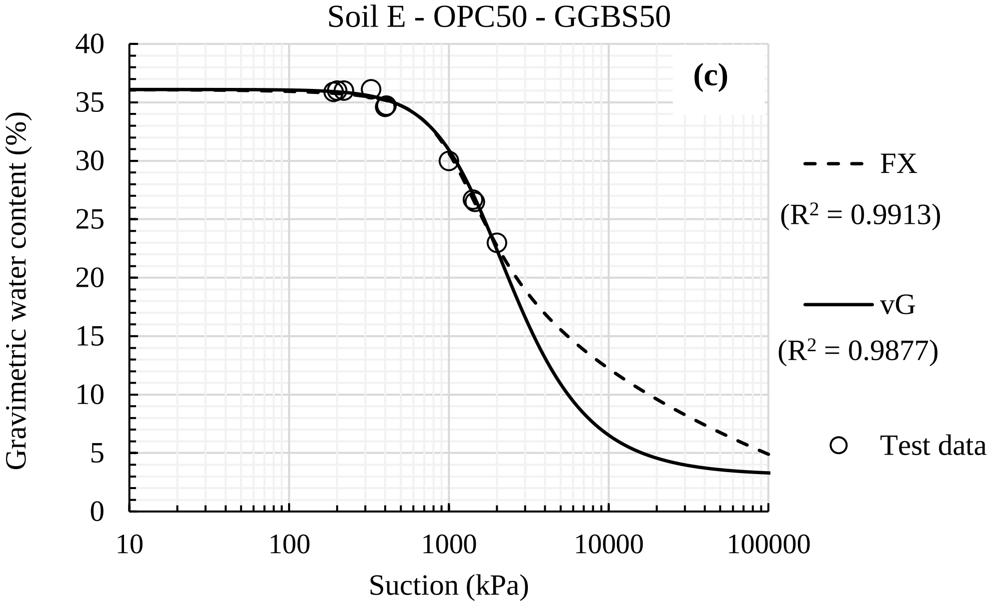


Figure 6.23 Air entry value (AEV) of natural and stabilised soil for van Genuchten (1980); Fredlund and Xing (1994) models


Soil E
(a)
Gravimetric water content (%)
0
5
10
15
20
25
30
35
40
45
10
100
1000
10000
100000
Suction (kPa)
FX
(R² = 0.9943)
vG
(R² = 0.9787)
Test data


Soil E - OPC100
(b)
Gravimetric water content (%)
0
5
10
15
20
25
30
35
10
100
1000
10000
100000
Suction (kPa)
FX
(R² = 0.9630)
vG
(R² = 0.9288)
Test data



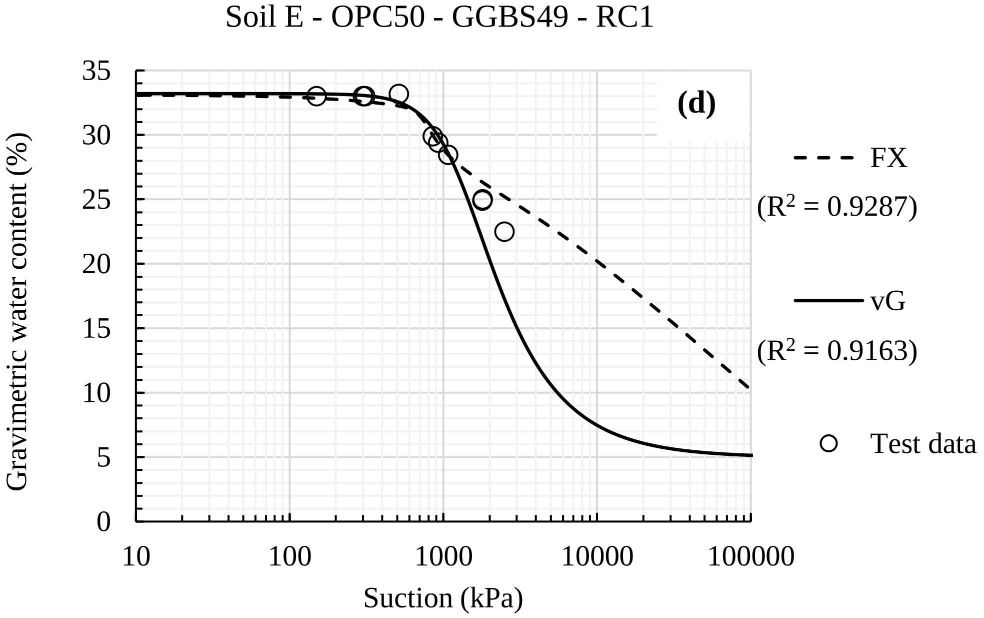


Figure 6.24 SWRC model comparisons for natural and stabilised clays (a) Soil B (b) Soil B - OPC100 (c) Soil B - OPC50-GGBS50 (d) Soil B - OPC50-GGBS50-RC1

As could be observed in Figure 6.25, there is a general reduction in the gravimetric moisture content with stabilisation by OPC alone and OPC partially replaced by the by-product with reduced suction. Also, the suction reduces due to wetting (below 1000 kPa) and as the saturation progresses, the stabilised soil using OPC alone tend to possess the lowest gravimetric moisture.

Furthermore, the stabilised as-compacted soil samples stabilised by using 50% of the OPC tend to exhibit greater moisture retention capacity observed as a flattening of the curve. As mentioned earlier, it has been suggested that at reduced suction levels, the moisture storage mechanism is determined mostly by capillarity and the retention curve is thus influenced by soil fabric (Tedesco 2006). It has been reported that OPC replacement by GGBS should reduce soil voids as compared to the OPC used alone (Keramatikerman et al. 2016; Zhang et al. 2018). The addition of RC tends to modify the OPC-GGBS reaction mechanism and enables an increase in the moisture retention behaviour of the stabilised soil as the suction decreases.

Overall, it can be inferred from Figure 6.23 and Figure 6.25 that much smaller void spaces are available for the penetration of the added water during the saturation process in the stabilised soil when only the OPC is utilised as compared to the combined OPC-by-product materials used especially at suctions below about 1000 kPa. In other words, the fast-reacting OPC used alone in the stabilisation of the soils does seem to thrive relatively more in the presence of high amounts of available hydration moisture. This further substantiates the slightly lower swelling potential value obtained (at zero suction) with the OPC only-stabilised clays as compared to the soil stabilised by a combination of OPC and the by-products (with RC included) at 7 days of curing.

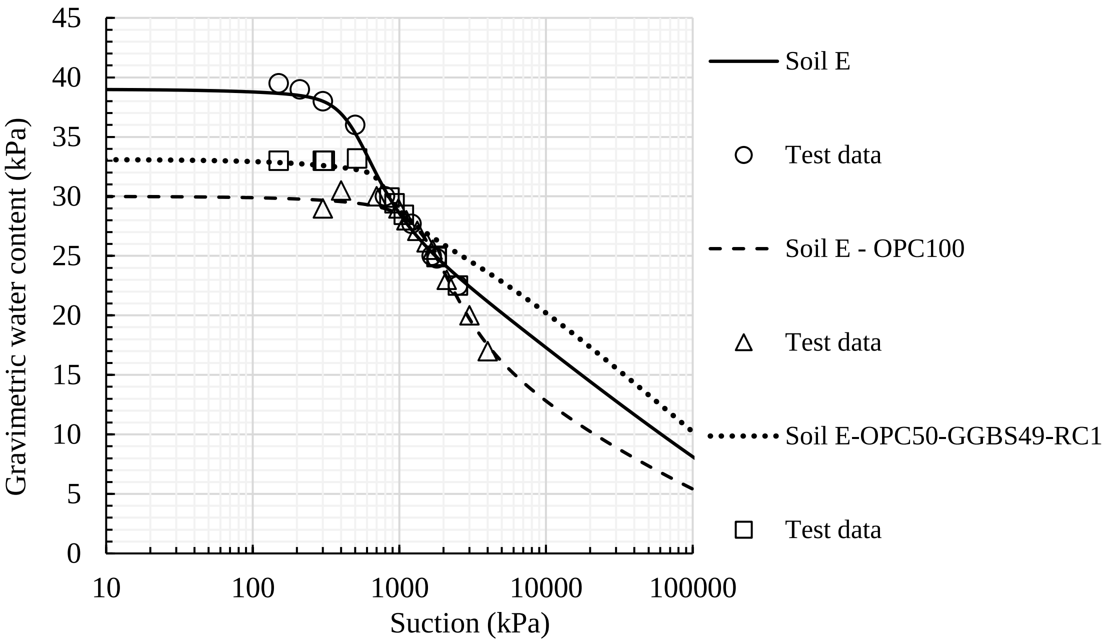


Figure 6.25 SWCC of stabilised soil

6.6.4.1 Effect of RC on SWRC

An investigation of the influence of the addition of RC to **Soil E** is shown plotted in Figure 6.26 by a comparison of the stabilised soil with and without RC. It could be observed that the SWRCs of the stabilised soil samples with RC become relatively 'flatter' (demonstrated by the higher '*n'* values of Table 6.3) which clearly demonstrates the effect of the RC in retaining more moisture. Initially though, the water-holding capacity of the stabilised soils having the proportion of RC is higher but tend to reduce as the saturation level increases with reduced suction. Hence, further hydration may have possibly occurred with more saturation leading to the formation of a water-proof structure with reduced porosity at reduced suction. The greater moisture retention property is promising for contaminant encapsulation during dredging activities as suggested by Zhang et al. (2018) while the relatively reduced porosity (compared to the combination without RC) at low suctions is desirable for swell reduction in the subgrade of pavement structures. But it should be recalled that at reduced suction levels, the rapid hardening OPC used solely to stabilise the clays do possess slightly more reduced porosity and higher AEV as compared to the stabilised clays with the RC included.

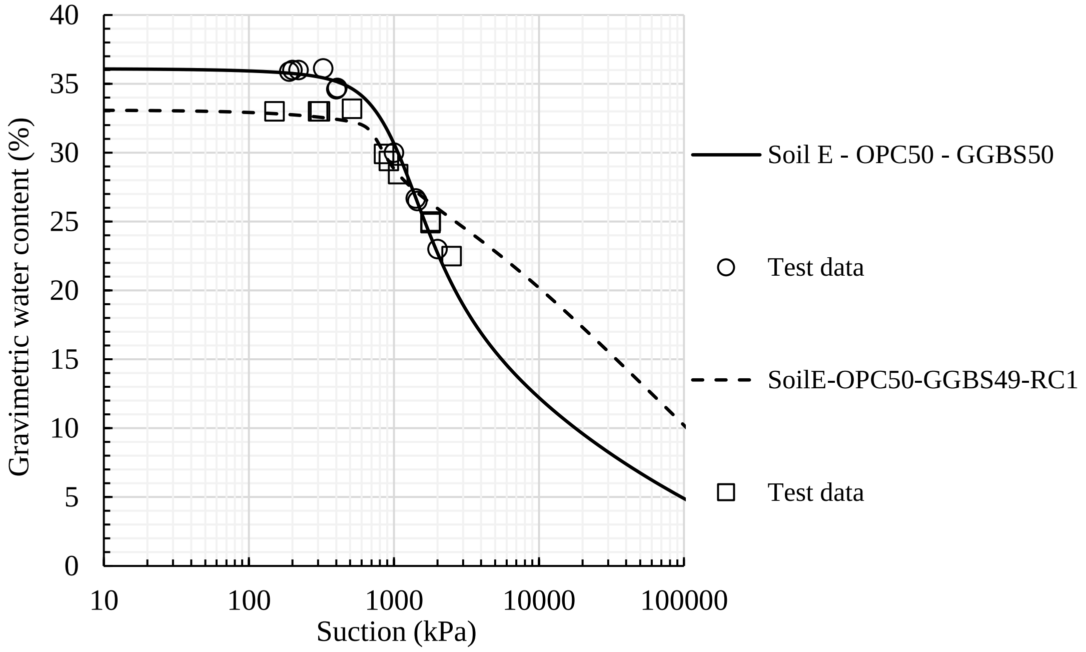


Figure 6.26 SWCC of stabilised soil showing effect of RC

Table 6.3 FX fitting model parameters of stabilised Soil E

Samples	FX parameters		
	a	*n*	*m*
	(kPa)	-	-
Soil E	1114	4.81	0.10
Soil E – OPC100	1529	4.17	0.30
Soil E – OPC50 – GGBS50	963	3.19	0.41
Soil E – OPC50 – GGBS49 – RC1	706	12.31	0.06

6.7 RELATIONSHIP BETWEEN SWRC AND STABILISED CLAY PROPERTIES

Some of the fitting parameters proposed by FX model have been known to bear important relationships with properties such as strength and swell of natural clays at least empirically (Thakur and Singh, 2005; Thakur et al., 2005; Rao et al., 2011). However, with the clay stabilised by binders, the mechanism of hydration and production of pozzolanic products (CASH or CSH) does intrinsically alter the behaviour, not least the pore size structure and distribution (Puppala et al., 2006; Lin and Cerato, 2012; Zhang et al., 2018). The FX model parameter n is one of the shaping functions of the SWRC that depends on the rate of extraction (for desorption curve) or imbibition (for adsorption curve) of water from or into the soil particles. It determines the slope portion of the SWRC, the portion of the curve that also invariably influences the nature of the void structure of the soil. A semi-empirical relationship between the FX model parameter n and the stabilised engineering properties is shown in Fig. 6.27. The best correlation occurs with the swelling potential indicating the dependence of this property on the pore morphology of the stabilised clays. An increase in the parameter n which may be invariable suggests a better retention property of the stabilised soils and eventual reduction in swelling as the suction reduces to zero is clearly depicted in Fig. 6.27. On the other hand, the parabolic fitting line seems to give the best fit even though this is still a rather unsatisfactory relationship between the parameter n and the UCS as seen in the reduced coefficient of determination (R^2). No clear description of this poor trend can be given except that unlike swelling, the stress path followed for determination of the UCS is due to external compressive loading.

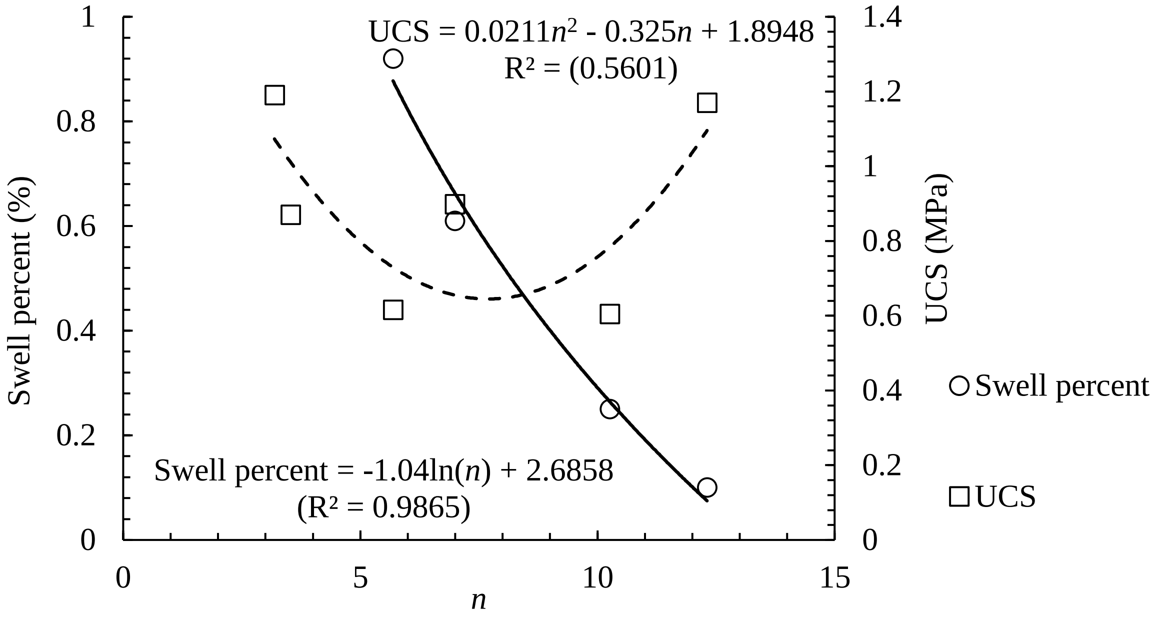
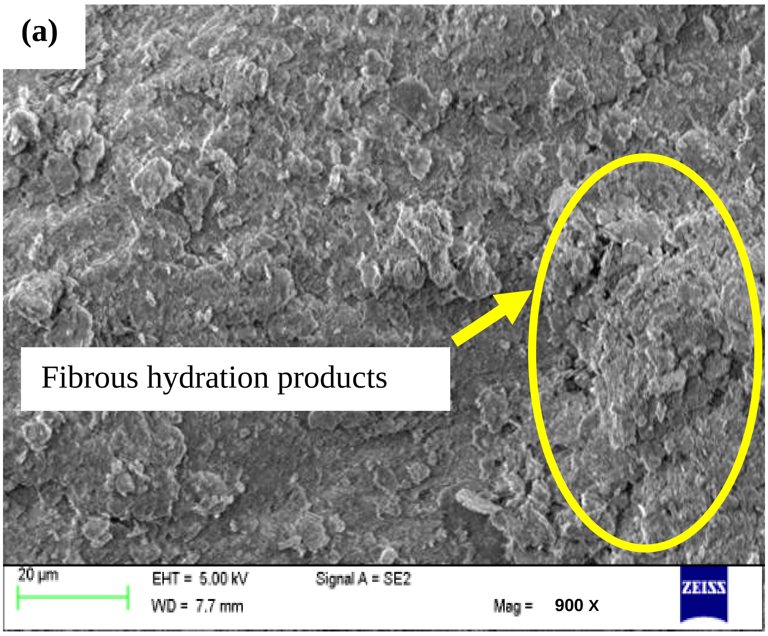


Figure 6.27 Relationship between FX parameter and the studied stabilised clay properties (UCS and swell percent)

6.8 STABILISED SOIL MECHANISM

6.8.1 General reaction mechanism of OPC-GGBS-stabilised soil

Discussions on the mechanism describing the stabilised soils' engineering behaviour using cement alone were extensively given in the previous chapter. In this section, the mechanism of stabilisation with the OPC replaced partly by GGBS and their various combinations resulting in some of the improved engineering properties discussed above are presented. As mentioned earlier in chapter 2 (see section 2.10.2), the stages of OPC-soil interactions reactions are very complex in the presence of moisture. The process of hydration is even much more complicated when GGBS is added to the OPC-stabilised soil-water system (Wild 1996; Ouf 2001). The mechanism of reactions that ensues with the addition of GGBS to an OPC-soil system has two fundamental phases namely, hydration of GGBS by hydrated lime from the OPC and soil-hydrated lime reactions. Firstly, the hydration of GGBS proceeds with the consumption of very little amounts of lime and commences soon after water is introduced and used to mix the soil-binder materials. This reaction tends to lead to the production of the calcium aluminosilicate hydrates (CASH) having low calcium to silicon ion ratio, aluminium to silicon ion ratio and calcium to aluminosilica ions ratio. The second phase involves the soil-OPC (or hydrated lime from the OPC) reaction and leads to the production of colloidal CASH again with values of the calcium to silicon ion ratio, aluminium to silicon ion ratio and calcium to aluminosilica ions ratio. Replacement of OPC by a higher percentage of GGBS whereby only a small amount of OPC is available to activate the hydration of GGBS may prevent the second phase of the soil-OPC reaction to start. However, with the OPC/GGBS ratio increased, the availability of OPC ensures the progress of the soil-OPC phase and the production of more calcium alumino hydrates (CAH) and calcium aluminosilicate hydrates (CASH). This resulting crystalline products of hydration proceeds much slower than OPC hydration and thus possesses some 'pore-blocking' effects (Figure 6.28) leading further to the increase in long-term hardening of the OPC paste and by extension an enhancement of the stabilised soil's engineering properties such as strength, swell and consolidation.



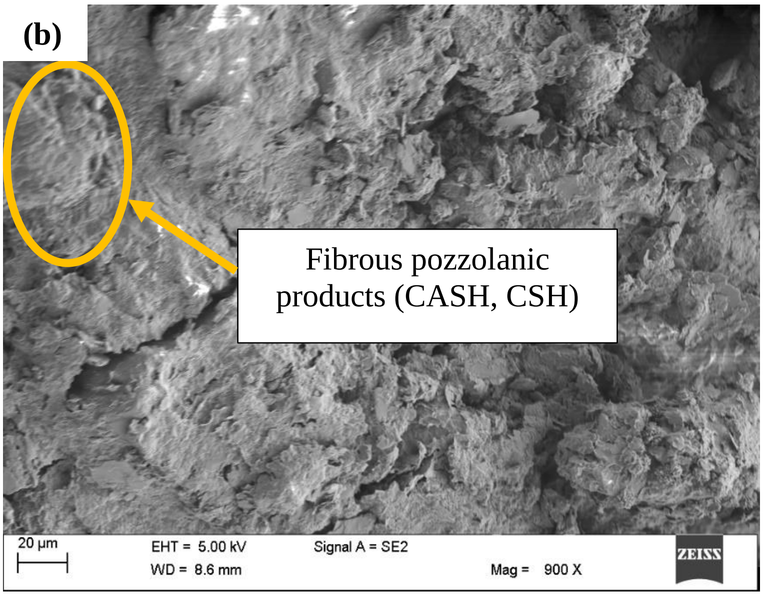


Figure 6.28 SEM of stabilised soils (a) Soil A stabilised by OPC50-GGBS50 binder combinations (b) Soil E stabilised by OPC50-GGBS50 binder combinations

6.8.2 Mechanism of soil stabilisation with the incorporated RC additive

As explained previously, soil stabilisation using cementitious materials such as OPC and GGBS or both, enables a modification of the created electrical double-diffused layer by causing a reduction of its thickness through the production of the CAH or CASH gels. It is believed that the complex hydrates form a complete spherical barrier (Figure 6.29) that prevents further reaction of the binder materials (Rahimi-Aghdam et al. 2017) as time progresses.

Moreover, depending on the amount and type of OPC or GBBS used, the decrease in swelling or increased in the strength properties could be attributed to the reduced affinity of the soil to the adsorb water within a shorter period of curing of say 7 days due to the process of continuous agglomeration and flocculation (Figure 6.30a).

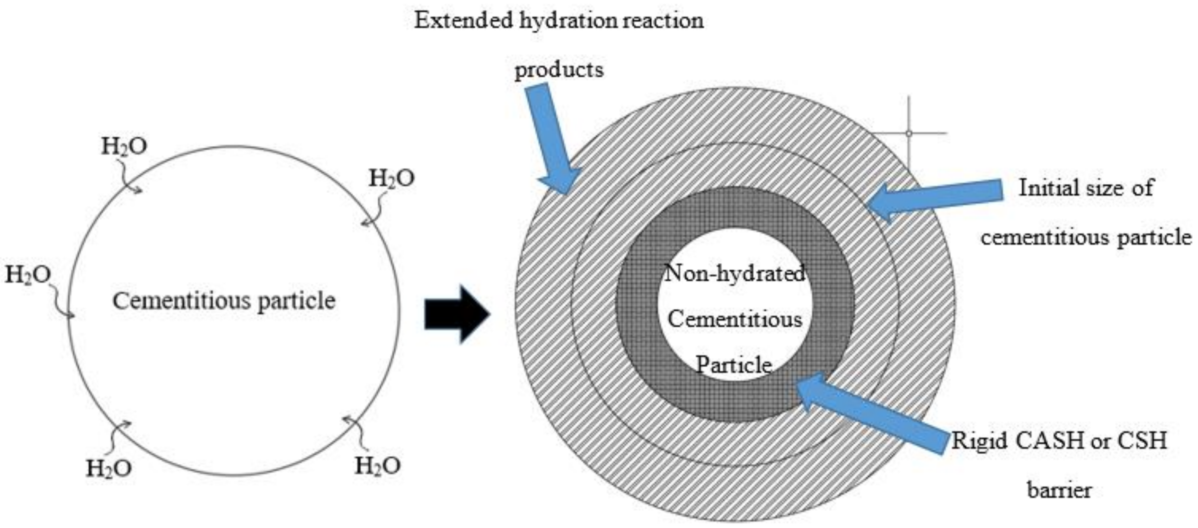
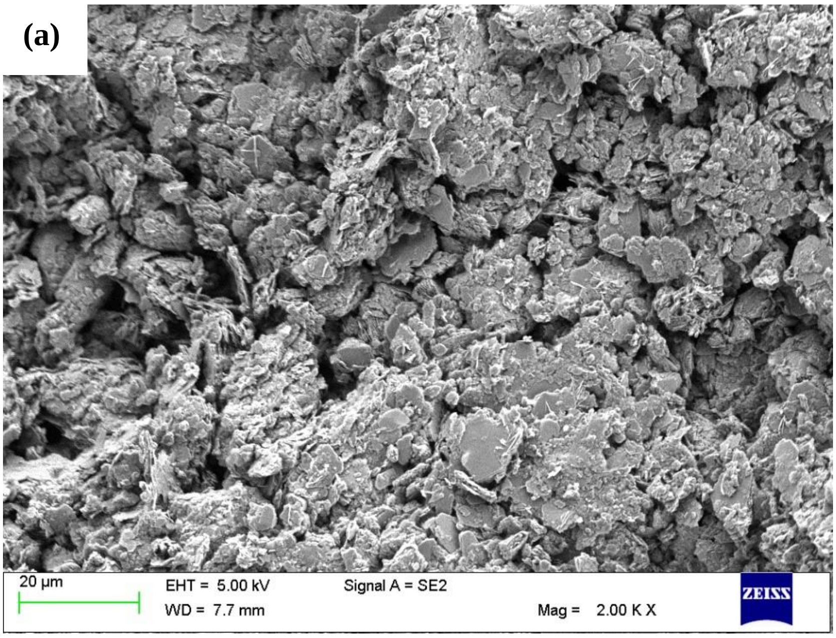


Figure 6.29 Hydration mechanism of soil stabilised by calcium-based binders such as OPC and GGBS



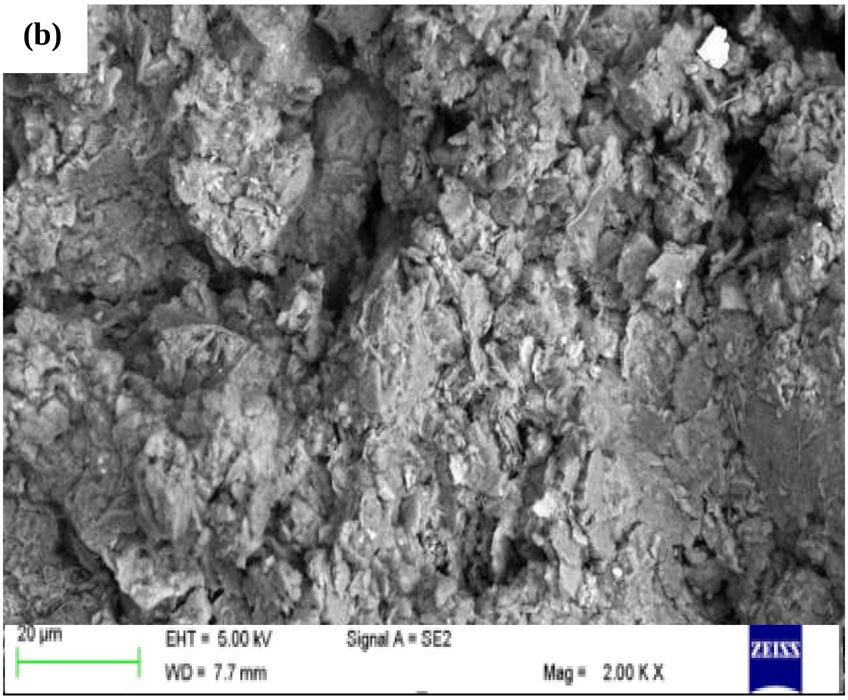


Figure 6.30 SEM of stabilised soil (a) Soil A stabilised by OPC100 (b) Soil E stabilised by OPC100

The addition of RC to the cementitious binders enables further and deep penetration of it and the water of hydration by ensuring a breakage of the formed CSH or CASH barrier and causing most of the cementitious materials to react with increased pH (Figure 6.31). A larger proportion of the water of hydration is then converted to crystalline water with more crystals growing into the spaces left in the hydration process. The extended crystallisation process coupled with a drastic decrease in the evolution of heat of hydration influences the soil-stabiliser binding mechanism which at this time would change from just “glue” to “wrapping” (matrix of interlocking filaments) – a phenomenon which is only made possible by the presence of the RC additive as an agent in the stabilisation process. The ‘wrapping’ effect and encapsulation (Figure 6.32) that are associated with the formation of the crystalline reaction product in the hydration process are also responsible for the modified cementitious product to bind very heavy clays together, a result which is nearly impossible with using cementitious binders alone as observed in the case of swell reduction when RC-modified soil is compared to the stabilised soil without RC. A decrease in the porosity during the initial hydration process and an increase in the structural crystalline matrices does lead to an increase in the compressive strength, reduction in the swelling properties and reduced settlement of the mixed product. It is also believed that the composition of the RC (mainly alkali and zeolites) also enables other processes to occur simultaneously in the clays and probably other similar materials through ionic exchanges, modifications, charge neutralization and replacements (Marjanovic et al. 2009).

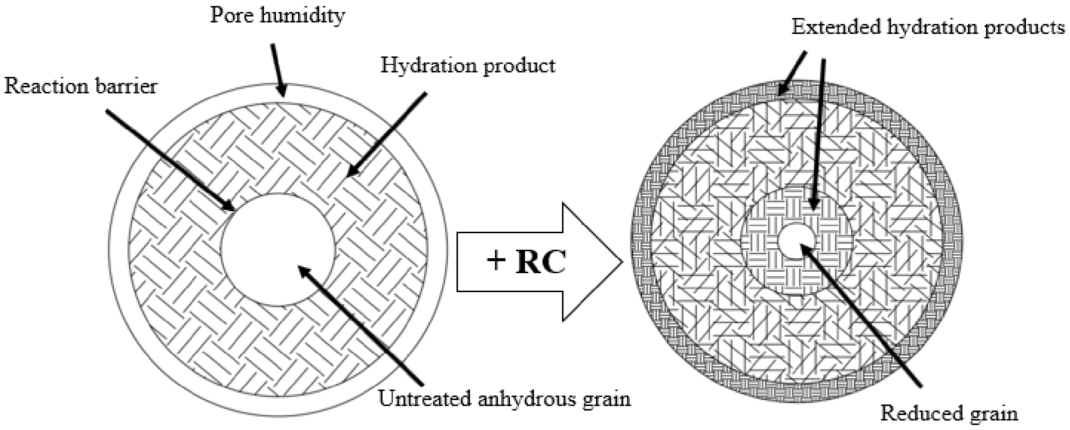
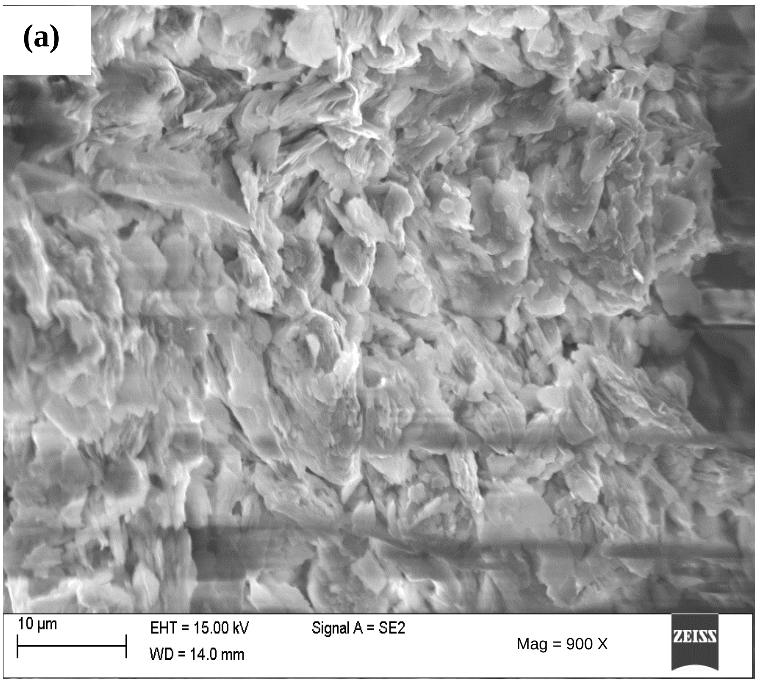


Figure 6.31 Hydration mechanism of soil stabilised by calcium-based binders and inclusion of RC



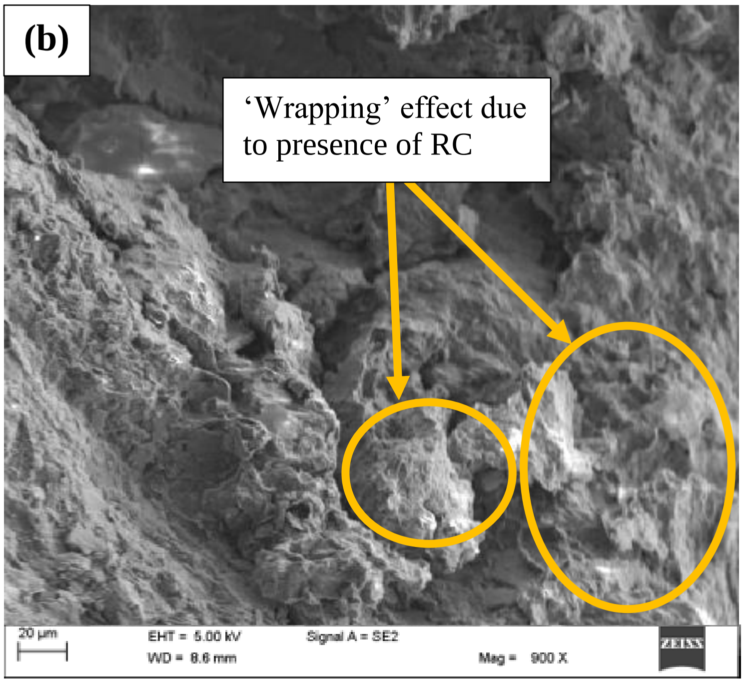


Figure 6.32 SEM of RC-modified soil (a) **Soil A** stabilised by OPC-GGBS-RC combinations (b) **Soil E** stabilised by OPC-GGBS-RC combinations

6.9 SUMMARY

Analyses were performed to assess the engineering performance of the clays by incorporation of RC in soil stabilisation in this chapter. The main objective was to evaluate the engineering (swell, consolidation & strength) and hydromechanical behaviour of stabilised **Soil A and Soil E** by a replacement of up to 50% of the OPC by the inclusion of RC. The outcome of this study are as follows:

Stabilised Soil A

i. Swelling reduced significantly with curing time for **Soil A** stabilised by inclusion of RC to the cementitious mixture. Generally, addition of RC reduced the swelling more than mixture without the RC used for both the 7 and 28 days investigated periods of curing. Moreover, the use of RC in the soil-binder mix reduced the expansion rate to almost zero as compared to the mixture without the RC as the curing was extended to 28 days.

ii. Regarding soil consolidation, from the point of view of the degree of settlement using the compression index Cc, values revealed that the mixture having the RC consolidated much less than that without the RC at 7- and 28-days curing with the differences being approximately 30% and 50% respectively.

iii. The UCS of all the OPC-by-product combinations in the stabilised soil increases generally more than the OPC-only stabilised soil at 7 days of curing. The strength of the stabilised soil with the OPC replaced by 50 and 60% of the by-products are both higher than the that of the soil stabilised using only OPC. But, with the curing extended to 28 days, the mixture without the RC tended to increase more than that having the RC. This interesting occurrence is related to the e-log p curve where the mixture without the RC had a slightly less initial void ratio at the start of consolidation as compared to the mixture that contains the RC.

iv. **Soil A** stabilised by OPC used alone seem to produce the largest AEV as compared to the natural soils and those stabilised by a combination of OPC and by-products. This indicates that greater suction (capillary behaviour) tends to occur in the soil-OPC samples (as compared to the samples having the by-products) due to a preponderance of smaller pore spaces as the wetting progresses. However, a comparison to depict the effect of the addition of RC to the stabilised mixes showed flatter SWRCs of the stabilised soil samples with RC thus demonstrating the effect of the RC in retaining more moisture as earlier claimed. Even though the water holding capacity of the stabilised soils having the proportion of RC were higher, this tended to reduce as the saturation level increases with decreasing suction.

Stabilised Soil E

i. A significant reduction in linear expansion as the curing period increases for the soil stabilised by OPC and all the binder combinations was observed. Addition of RC reduced the swelling only slightly more than mixture without the RC used for both the 7 and 28 days investigated periods of curing.

ii. From the perspective of the degree of settlement, the stabilised soil mix possessing RC consolidated more as noticed from higher values of the Cc obtained from the e log p curves for both 7- and 28-days curing duration.

iii. The stabilised soil by the OPC-by-product binder mixture with 50% of the OPC replaced are almost equal to that in which only the OPC is used over 7 days period of curing. With the period of curing increased to 28 days, the strength gained in the soil-binder mixtures containing OPC alone rose more remarkably higher than the stabilised soils with the by-products added. Generally, there tended to be somewhat of a slow gain in strength of the soil stabilised by the OPC-by-product combinations as the curing period increases.

iv. The soil stabilised by OPC used alone produced the largest AEV as compared to the natural soils and those stabilised by a combination of OPC and by-products. This phenomenon is also as a result of the presence of the very fine clay particles in the soil. However, the investigated SWRCs of the stabilised soil samples with RC becomes relatively ‘flatter’ which clearly demonstrates the effect of the RC in retaining more moisture. Initially though, the water-holding capacity of the stabilised soils having the proportion of RC is higher but tend to reduce as the saturation level increases with reduced suction. Hence, further hydration may have possibly occurred with more saturation leading to the formation of a water-proof structure with reduced porosity at reduced suction.

CHAPTER 7

INCORPORATION OF RC IN THE CEMENTITIOUS BINDERS FOR THE STABILISATION OF SULPHATE-BEARING SOIL

7.1 GENERAL

This chapter is dedicated to further investigations and discussions to authenticate and validate the use of RC in the treatment of soils exposed to harsh chemical conditions. By so doing, an application of the most effective binder mixes is used for the stabilisation of a soil that is prone to attacks by sulphates. Selection and sampling of sulphate-bearing soil are first performed followed by the treatment of the soil with the binders but with the soil stabilised by OPC alone used as a reference mix. Results of swelling of both the natural and stabilised soils under total inundation with water are then presented and analysis carried out to determine the benefits of using RC in the reduction of ettringite-induced swelling. An evaluation of the level of influence of sulphates on the stabilised soil without inundation with water is also carried out by an assessment of the degree of strength development in the stabilised soil samples. Finally, microstructural features of the stabilised soils are incorporated in order to further support and enhance the description of their engineering properties.

7.2 MATERIAL SAMPLING AND SELECTION

Given that the natural soils used in this research are commercially sourced, all the possible sulphates are known to have been skimmed out during the industrial processes (see Table 3.1, chapter 3 for chemical compositions). Hence, in order to simulate the presence of sulphate, the natural soil samples were doped with varying predetermined proportions of hydrated calcium sulphate or gypsum ($CaSO_4 \cdot 2H_2O$). The gypsum was sourced commercially and processed with 99.0% purity. Only **Soil A**, being the parent clay in this research and one with the lowest potential to swell was used as the natural soil. This was to ensure that most of the expansion property accorded to the mixture was attributed to the presence of the gypsum. Moreover, kaolinite is chemically very rich in alumina as compared to montmorillonite hence, kaolinite can give up more alumina in high pH-controlled environmental conditions to participate in the formation of the expansive ettringite mineral and are thus more susceptible to attacks by sulphates (Mitchell and Dimitris 1992; Rollings et al. 1999; Knopp and Moormann 2016).

7.3 SOIL-BINDER MIX DESIGN

Given that gypsum is a relatively less soluble source of sulphate as compared to sodium and magnesium sulphates (Wang 2002; BRE 2005; Nair and Little 2011), high predetermined quantities (4%, 8% and 12% by dry mass of soil) of the gypsum were added to **Soil A**. Expressed in ppm (parts per million), the mass concentrations of gypsum used therefore, translates to approximately 40,000, 80,000 and 120,000 ppm. The sulphate-bearing soil was stabilised by the binder mixture with 50% of OPC substituted by the by-products as discussed in the previous chapter. The choice of this amount and type of binder is based on the premise that it has the highest OPC threshold and does present the worst-case scenario for expansion even though it provided the best improvements in the engineering properties investigated in the previous chapter. In order to evaluate the performance effectiveness of RC, a comparison shall be made between the stabilised sulphate-bearing soil with the inclusion of RC and that without the RC added. The binder combinations without the RC added are those stabilised by OPC used alone and that stabilised by OPC-GGBS (i.e. in a 50:50 combination). Table 7.1 is an illustration of the binders used and the mix design adopted while Figure 7.1 is a detailed flow chart of the soil-binder design proportions including their nomenclature.

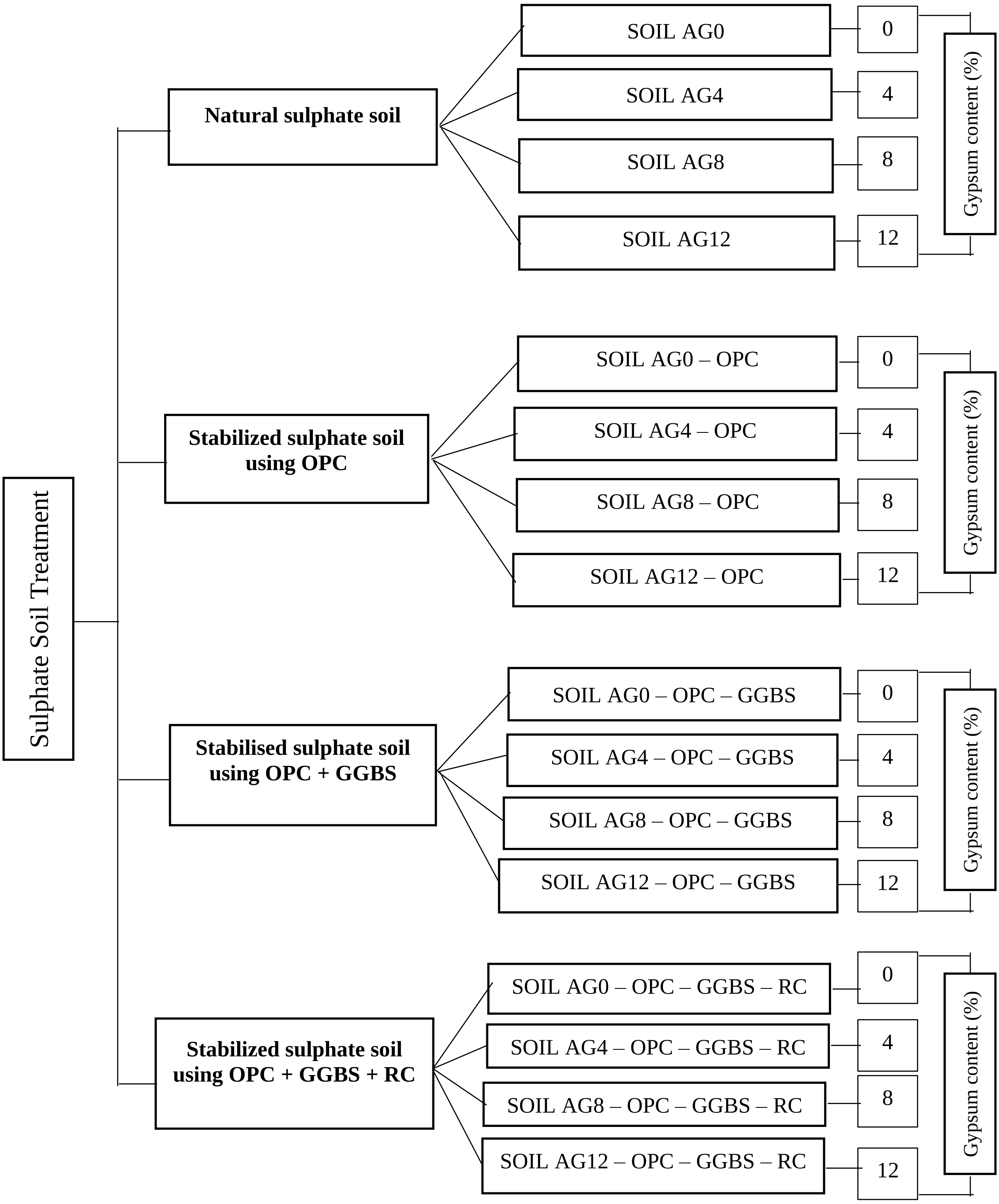


Figure 7.1 Flow chart of the novel binder mixture for sulphate soil stabilisation

Table 7.1 Cement replacement mix proportion

Binder	Designation	Replacement ratio (%)	Total binder % by dry wt. of soil
Cement	OPC	100:0	8
Cement: GGBS	OPC-GGBS	50:50	8
Cement: GGBS: RoadCEM	OPC-GGBS-RC	50:49:1	8

7.4 METHOD STATEMENT FOR MATERIAL PREPARATION AND TESTING

7.4.1 Soil-binder Preparation

Mixing of the soil materials and binders were thoroughly carried out manually using hand-mixing to ensure a homogenous composition. The amount of water added to the soil-gypsum-binder system was determined based on the optimum moisture content of the natural soil derived from the standard proctor compaction test (ASTM D 1557) but with an additional 2% of water to accommodate for the binders used based on research and experience (Wild et al. 1998). Further mixing of the materials with water was intimately performed for about 10 minutes in order to prevent the development of ‘hot spots’ and non-uniformly migrated ions as well as improving the chances of uniform ettringite nucleation site distribution (Little et al. 2010). Extended mellowing was not carried out because the amount of sulphate used is presumed to be sufficient for the development of ettringite nucleation sites (Little et al. 2010).

7.4.2 Solubility of Gypsum

It has been established by research that the soluble sulphate threshold needed for sufficient ettringite formation and which may create a substantial risk of sulphate-induced damage is about 0.3% and above (Mitchell and Dimitris 1992; National Lime Association 2000; The Highways Agency 2000; Britpave 2005). Gypsum has been known to have a solubility of 2.58 g per litre of water (Burke et al. 1999; Little and Syam 2009). However, due to its relatively low solubility property, the amount of water that is required to dissolve gypsum becomes very crucial. One mole of gypsum is known to have a mass of about 172g while containing approximately 96g of SO_4. By using this analogy, if 100g of dry soil contains about 0.3% (0.3g) of gypsum or 0.167% sulphate and 22g of water (which is the optimum moisture utilised for the mixtures in this research to accommodate the binders) then the amount of the water-soluble sulphates that can be solubilized is given as:

$$22g(H_2O) \times \frac{2.58g\ (gypsum)}{1000g\ (H2O)} \times \frac{96g\ (SO_4)}{1\ mole\ (gypsum)} \times \frac{1\ mole\ (gypsum)}{172g\ (gypsum)} =$$

$$0.0317g\ (SO_4) = 0.0317\%\ SO_4 \qquad (7.1)$$

It could then be seen that the optimum moisture used as derived from the proctor test only permits about 0.0317% of soluble sulphate which is well below the threshold of 0.3% for sulphate-induced disruption as suggested in literature (Mitchell and Dimitris 1992; National Lime Association 2000; The Highways Agency 2000; Britpave 2005).

Hence, in order to allow for an adequate sulphate dissolution, a total soaking with water of the compacted samples will have to be ensured and this was carried out as in section 7.4.3.

7.4.3 Sample preservation and curing

After extrusion of the samples from the compaction moulds using the extractor, the samples were preserved and cured as prescribed in section 3.3.1.6 of chapter 3 for 7 and 28 days prior to subsequent testing.

7.4.4 Oedometer Swell Test

The free swell oedometer testing procedure (ASTM D4546-14e1, see section 3.3.1.7) which ensures tested samples are completely soaked in water was carried out on the cured samples. This method simulates field conditions were water replenishment from groundwater flow and precipitation can result in the constant dissolution of gypsum thus, providing a suitable environment for the formation and growth of ettringite (Britpave 2005; Little and Syam 2009; Knopp and Moormann 2016). Moreover, molar volume calculations have shown that water migration from an external source into a system of stabilised soil after thorough mixing and compaction, can be responsible for huge amounts of volumetric expansion in the layer of stabilised sulphate bearing-soils (Little et al. 2010).

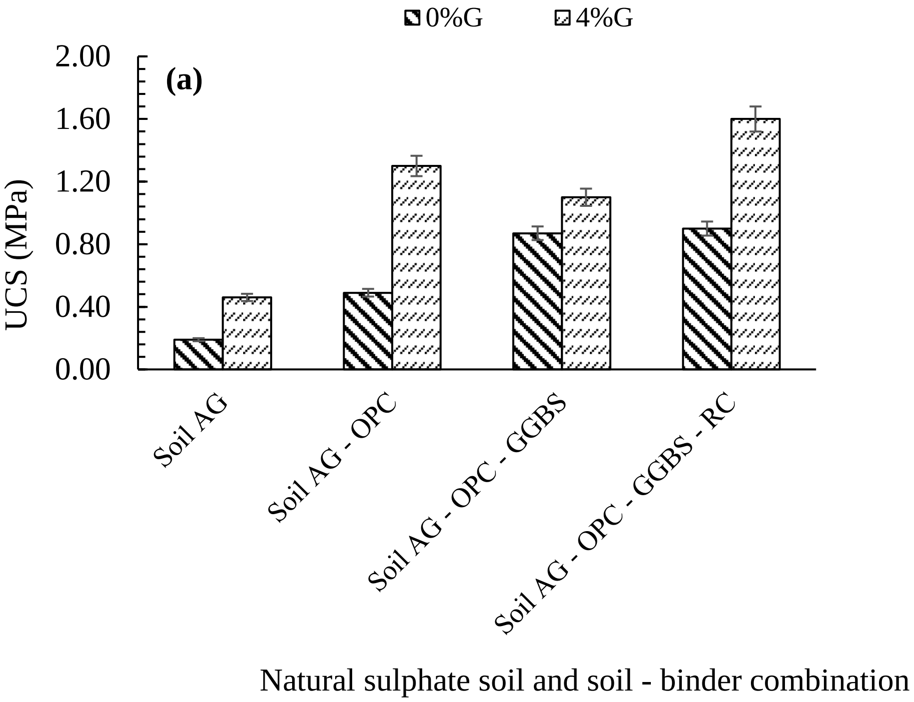
7.4.5 Unconfined Compression test

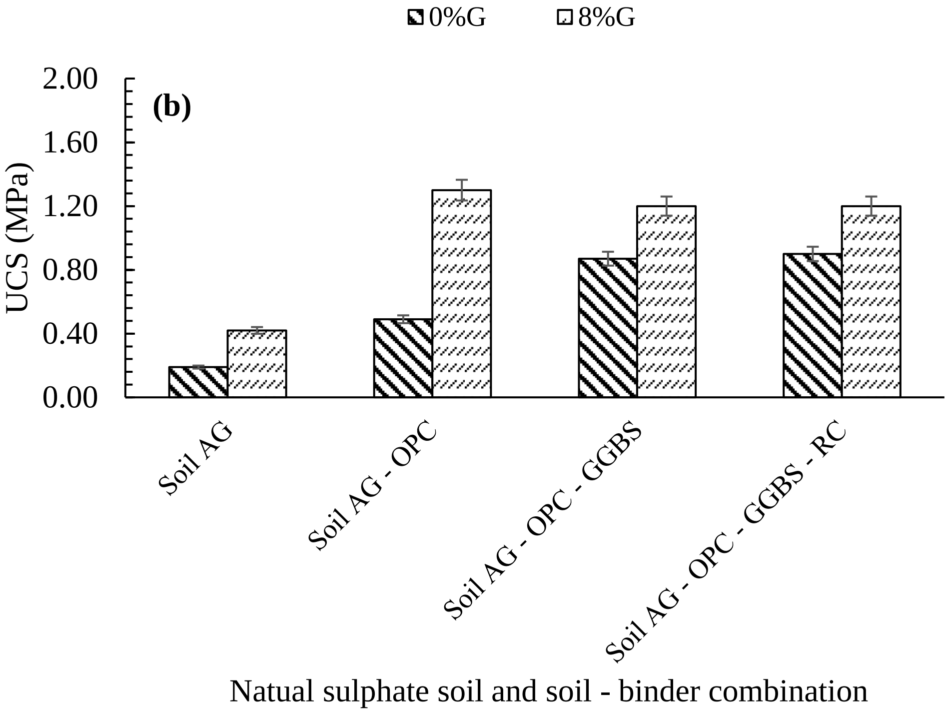
This test was performed in accordance with ASTM D2166-00 (see section 3.3.1.9) on the compacted but unsoaked samples. Since there was nowhere to account for any sort of swell (which may have either occurred or not) on the compacted samples treated with sulphates (without soaking) given that only the optimum moisture was used, the standard UCS test was conducted after 7 days of curing to ascertain the effect of sulphate on strength.

7.5 ANALYSIS OF TEST RESULTS

7.5.1 Strength

A comparison between the stabilised soil with and without gypsum added are shown plotted in Figure 7.2. For the stabilised soil without the gypsum added, it could be seen that there is a progressive rise in the UCS but with the mix having the RC being the highest. Moreover, for the mixes with the natural soil enriched with 4% gypsum, there is an even greater percentage increase in the strength (approx. 140% for Soil AG4, 165% for Soil AG4 -OPC, 26% for Soil AG4 – OPC - GGBS and 80% for Soil AG4 – OPC – GGBS – RC) as compared to the mixes without the gypsum. Again, Figure 7.2a indicates that the highest strength is obtained when the RoadCem additive is added to the mix. The same trend of the rise in the UCS with the sulphate-enriched soil is observed in Figure 7.2b and Figure 7.2c with 8% and 12% of the gypsum in the soil respectively. It is however evident that an increase in the amount of sulphate in the soil does not correspond to sufficient strength increase. Although, the strength increase seems to be only slightly apparent in the sulphate-enriched system with the OPC binder added as the gypsum levels increase to 8 and 12%. In general, it appears the presence of sulphate in the compacted (unsoaked) soil and the soil-binder system does aid the enhancement of strength. Further elucidation of this phenomenon is given in subsequent discussion in section 7.6.


▧ 0%G
▨ 4%G
(a)
UCS (MPa)
2.00
1.60
1.20
0.80
0.40
0.00
Soil AG
Soil AG - OPC
Soil AG - OPC - GGBS
Soil AG - OPC - GGBS - RC
Natural sulphate soil and soil - binder combination


▧ 0%G
▨ 8%G
(b)
UCS (MPa)
2.00
1.60
1.20
0.80
0.40
0.00
Soil AG
Soil AG - OPC
Soil AG - OPC - GGBS
Soil AG - OPC - GGBS - RC
Natual sulphate soil and soil - binder combination

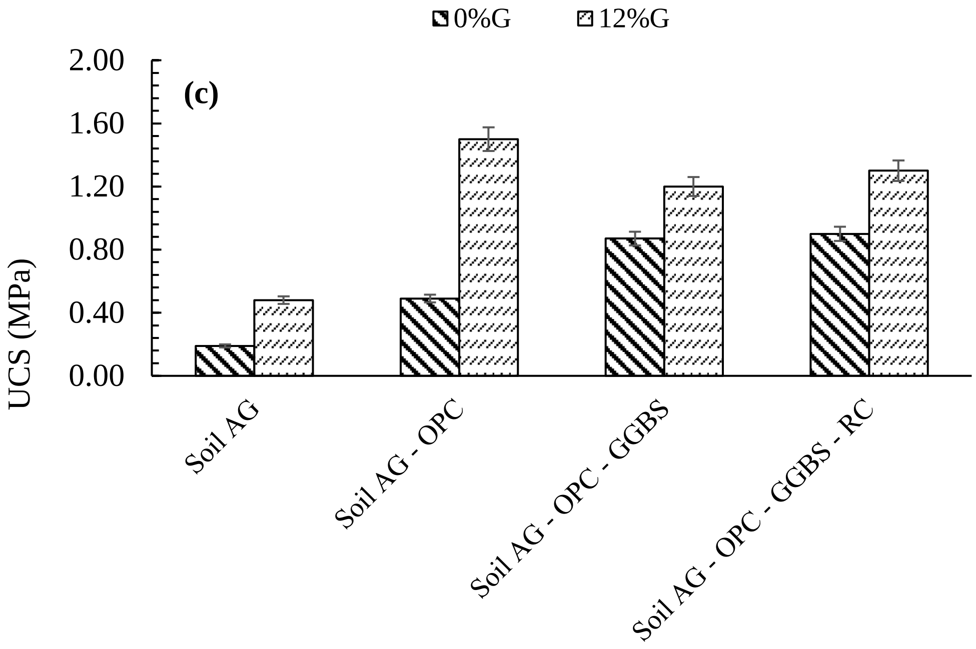


Figure 7.2 Strength of stabilised soil (a) with 4% gypsum and without gypsum (b) with 8% gypsum and without gypsum (c) with 12% gypsum and without gypsum

7.5.2 Swell

Stabilised sulphate-bearing soil

The natural sulphate-rich clays having different proportions of the gypsum cured for 7 and 28 days are shown plotted in Figure 7.3. The percentage of swelling of the soils enriched by the sulphates seem to be more than the pure natural soil for the 7 days of curing duration. However, as the curing period increased to 28 days, the swelling did appear to reduce as compared to the pure natural soil. Generally, just as in the case of the studied soil strength, the variation in swelling seems to be marginal and does not correspond to the proportion of gypsum added.

Figure 7.4 shows the sulphate soils stabilised by the various binder combinations. Generally, the stabilised sulphate-rich soils all seem to swell higher than their stabilised counterparts without the gypsum added. It could also be noticed that the increase in the amount of swelling does not appear to correspond with the increase in the quantity of gypsum added for 7 days curing duration. In fact, the reverse seems to be the case as the 7-day cured samples containing the lowest quantity of gypsum (4%) tend to swell more than the samples having supposedly higher quantities of the sulphates. However, for the 28-day cured samples (with the exception of natural samples, the rise in the level of sulphate is consistent with the amount of swell in the samples stabilised with the binders and their combinations.

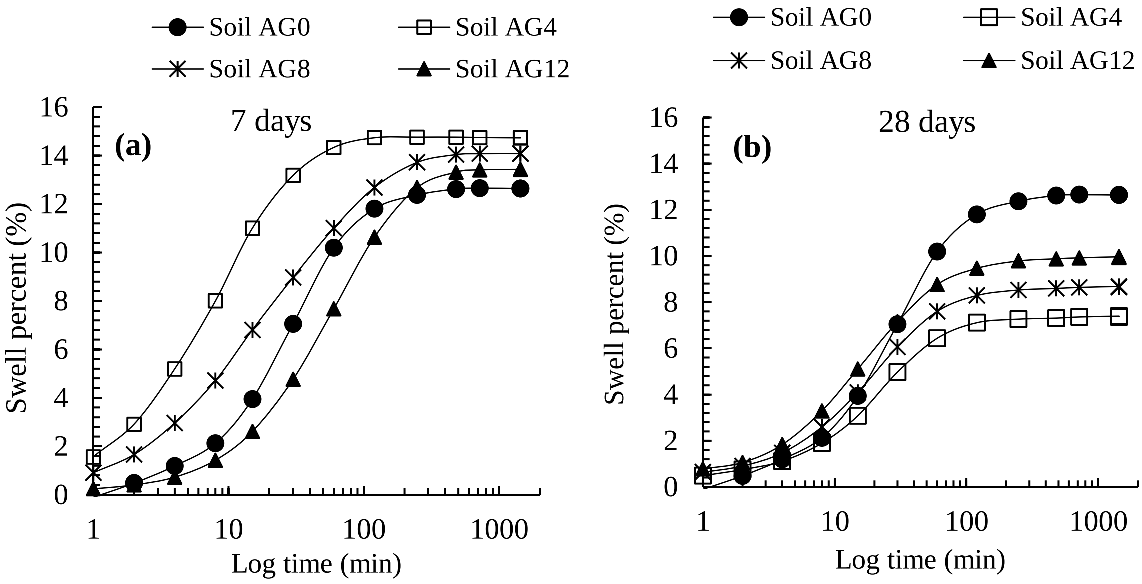
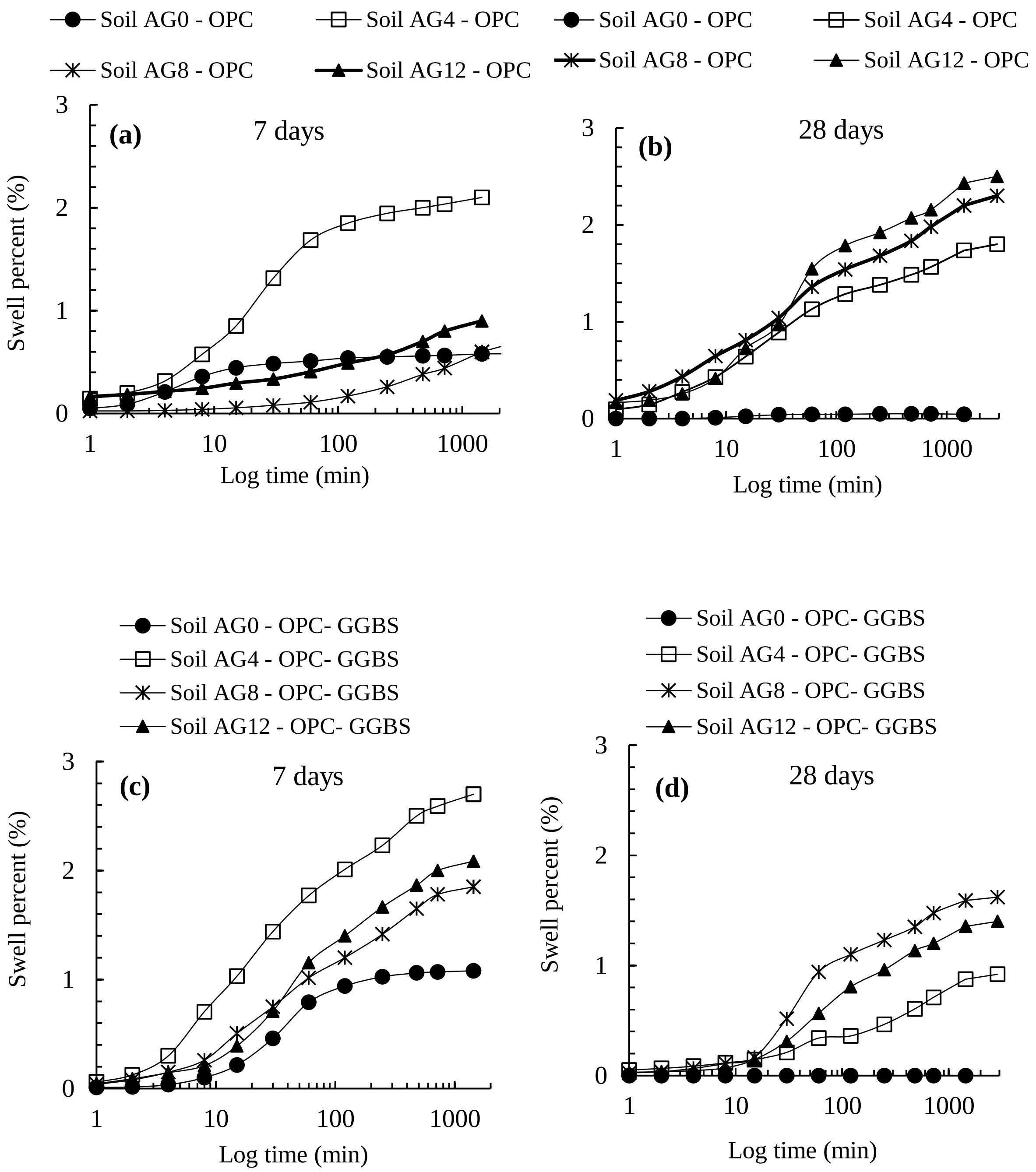


Figure 7.3 Swell percent of natural sulphate soil with various gypsum content (a) 7 days curing (b) 28 days curing


Soil AG0 - OPC
Soil AG4 - OPC
Soil AG8 - OPC
Soil AG12 - OPC
(a)
7 days
Swell percent (%)
0
1
2
3
1
10
100
1000
Log time (min)
Soil AG0 - OPC
Soil AG4 - OPC
Soil AG8 - OPC
Soil AG12 - OPC
(b)
28 days
0
1
2
3
1
10
100
1000
Log time (min)
Soil AG0 - OPC- GGBS
Soil AG4 - OPC- GGBS
Soil AG8 - OPC- GGBS
Soil AG12 - OPC- GGBS
(c)
7 days
Swell percent (%)
0
1
2
3
1
10
100
1000
Log time (min)
Soil AG0 - OPC- GGBS
Soil AG4 - OPC- GGBS
Soil AG8 - OPC- GGBS
Soil AG12 - OPC- GGBS
(d)
28 days
Swell percent (%)
0
1
2
3
1
10
100
1000
Log time (min)

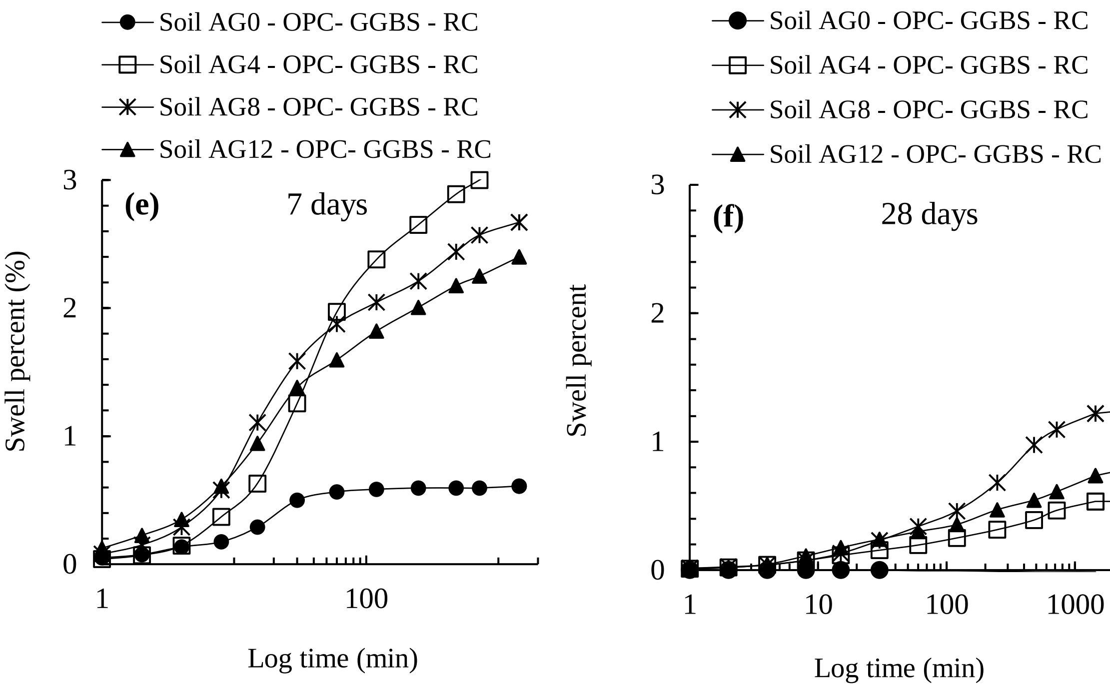
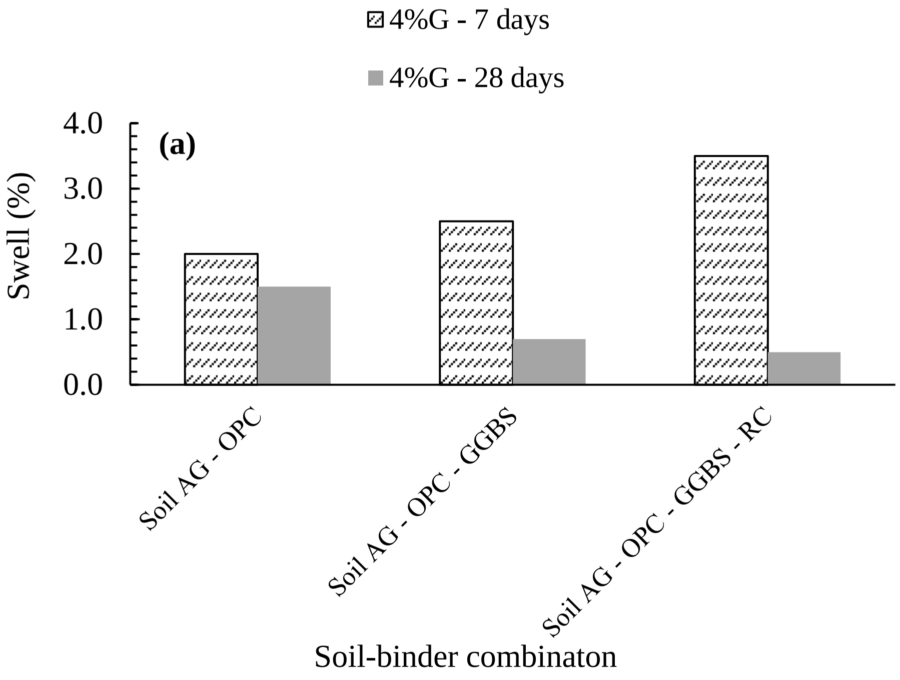


Figure 7.4 Swell percent of stabilised sulphate soil with various gypsum content (a) OPC-stabilised soil at 7 days curing (b) OPC-stabilised soil at 28 days curing (c) OPC-GGBS-stabilised soil at 7 days curing (d) OPC-GGBS-stabilised soil at 28 days curing (e) OPC-GGBS-stabilised soil at 7 days curing (f) OPC-GGBS-stabilised soil at 28 days curing

Investigation of the effect of the type of binders used and the curing duration is further carried out based on Figure 7.5 which indicates the maximum swell per cent values at equilibrium. Stabilisation of the sulphate-enriched soils generally acted to reduce the percentage of swelling with the passage of time (due to pozzolanic reactions) as compared to the pure natural soil. As also observed by Abdi et al. (2019) the extended curing of a treated or stabilised sulphate-bearing soil even before inundation can reduce the swelling as compared to the natural non-sulphate bearing soil.

However, the effect of sulphate heaving can be appreciated by an examination of the adopted different duration of curing (7 & 28 days) of the stabilised soaked sulphated-bearing samples. Firstly, the heaving of the unstabilised sulphate-rich soils is noticed to be reducing with the increase in curing from 7 to 28 days for all proportions (4, 8 & 12%) of the gypsum added as stated previously. Figure 7.5a shows that for the stabilised containing 4% of the gypsum, the percentage of swelling is observed to be decreasing with curing duration. At 7 days, the amount of swelling seems to be slightly lower for the OPC stabilised sulphate-bearing soil mix as compared to the those stabilised with 50% of OPC replaced by either GGBS or GGBS and RC. But at 28 days curing, the effect of GGBS and RC tend to act to reduce the swelling further. On the other hand, with an increase in the quantity of gypsum to 8 and 12%, the heaving of OPC stabilised sulphate-rich soil does seem to increase at 28 days of curing (Figure 7.5b and Figure 7.5c). Nevertheless, at 28 days of curing of the stabilised soaked sulphate-rich samples', heaving is seen to reduce with 50% of the OPC replaced by the by-product's additives but with the RC having the most influence as the stabilised soil containing it produces the lowest percentage of swelling.



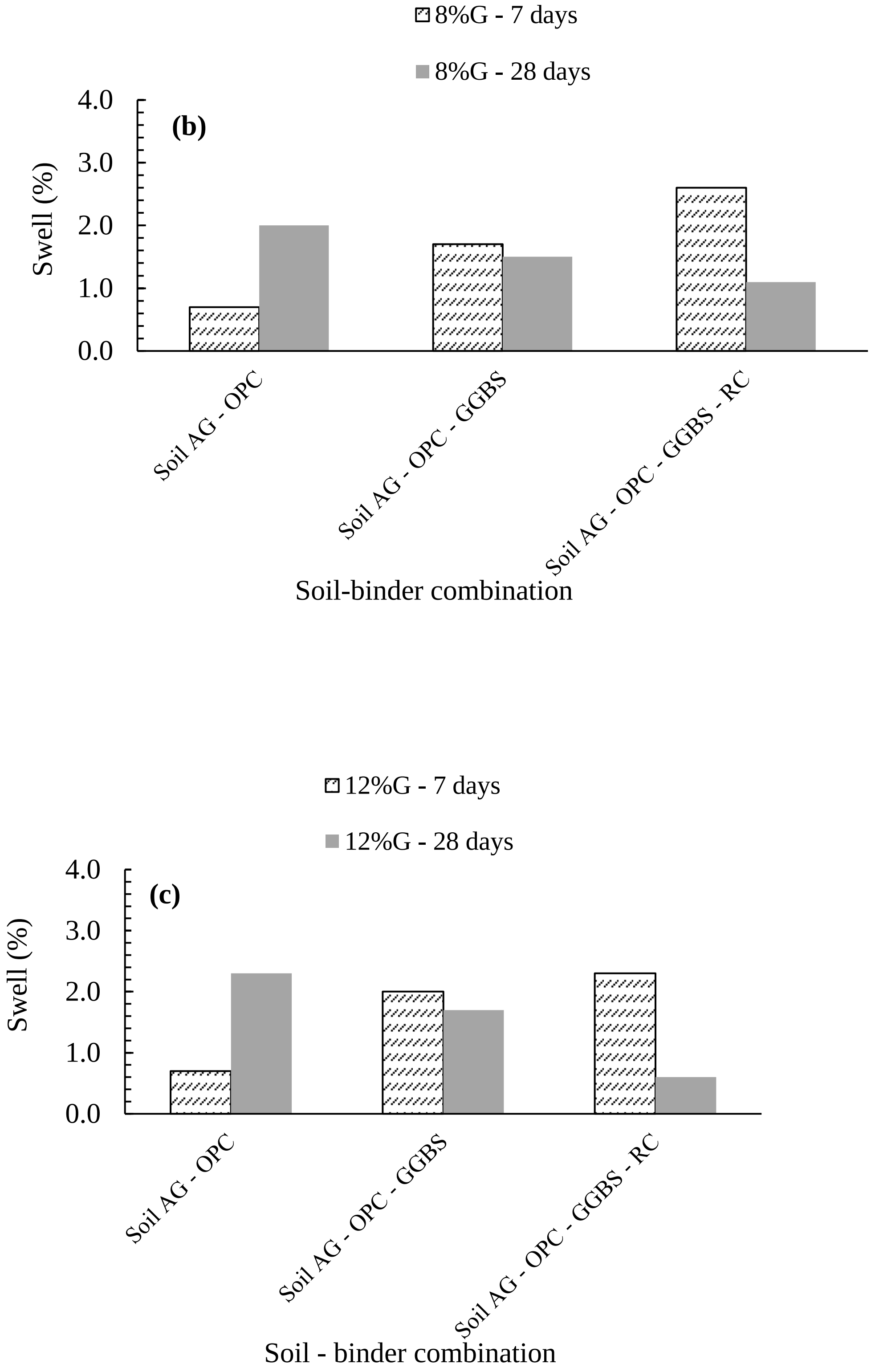


Figure 7.5 Maximum swell percent of stabilised sulphate soil with various gypsum content (a) 4% gypsum (b) 8% gypsum (c) 12% gypsum

7.6 DISCUSSION

In the discussion of the mechanism of changes occurring in both the compacted natural sulphate and stabilised soil, SEM images are provided to support an explanation of the observed hydration phases. Moreover, given that the different proportions of gypsum used did not produce sufficient variation in the properties of the natural and stabilised soils, only the micrographs showing 8% of the gypsum added are provided.

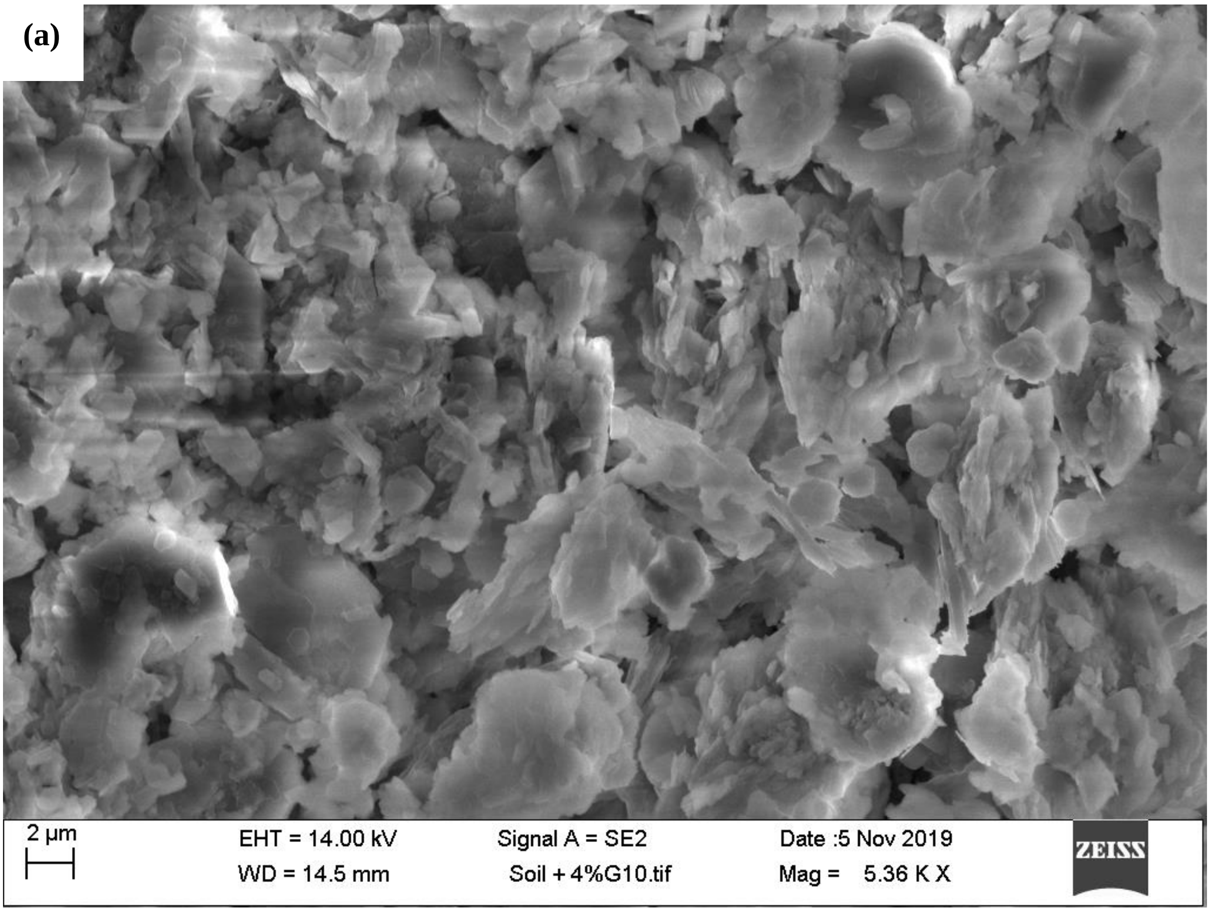
7.6.1 Strength

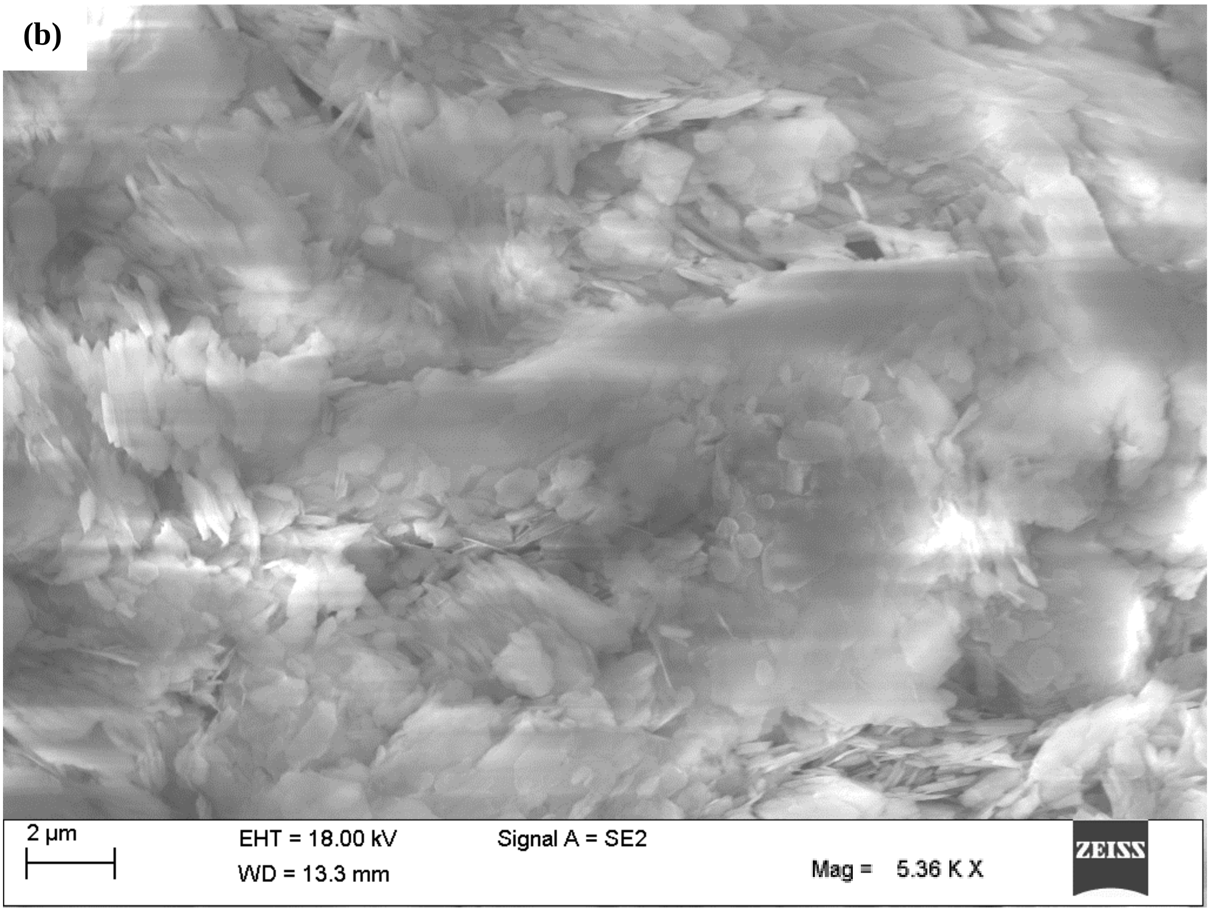
The presence of sulphates seemed to have caused an increase in UCS for all the percentages of gypsum added to the natural soil (Figure 7.2). A similar trend was observed by Yilmaz and Civelekoglu (2009); Jha and Sivapullaiah (2015b). Given that gypsum contains much of the mineral calcium hydroxide, the process of ionic exchanges between the gypsum (Ca^{2+}) and the soil ions (aluminium and silicon) and the formation of hydration products is presumed to result to a rise in strength. Yilmaz and Civelekoglu (2009) indicated that increasing the amount of gypsum in the natural soil beyond 5% may not result in significant increase or changes in the UCS. Hence, the increase in the quantity of gypsum from 4% to 12% as is in the case of this research did not result in much strength gain (Figure 7.2). On the other hand, a greater rise in the UCS of the sulphate-enriched soils was noticed when stabilised with the binders. It should be noted here that the compacted samples used for the UCS test were not soaked in water prior to the test. Except for the soil enriched by 4% of the gypsum, the stabilised sulphate soil consisting of OPC used alone seems to have the highest strength. The additional supply of calcium ions to OPC can result in strength increase for a short curing duration (Jha and Sivapullaiah 2015b). In this case, gypsum seems to act as a catalyst that accelerates the initial hydration rates of the stabilised soil leading to the formation of a sufficient amount of the compounds of cementation (CSH and CASH). Moreover, the sulphate ion that is contained in the gypsum reacts with the alumina and calcium ions present in the soil and cement respectively to form the complex calcium silicate aluminate hydrate compound or mineral called “ettringite” including the cementitious compounds of hydration (CSH and CASH) (Figure 7.6b) (Abdi and Wild 1993; Wild 1996; Wild et al. 1998). The formation of ettringite aids the reinforcement and interlocking of the particles in the soil while the produced cementitious compounds lead to the formation of a well-bound cemented and compacted matrix that all result in the increase in strength (Wild et al. 1998; Puppala et al. 2005; Jha and Sivapullaiah 2015b).

Figure 7.2 also demonstrates the increase in strength of the stabilised sulphate soil composed of OPC and GGBS combinations as compared to the stabilised soil without the gypsum added. It has also been proven that gypsum can serve as a catalyst to enhance the acceleration of the mechanism of hydration for soils stabilised by a calcium-based binder such as OPC in the presence of GGBS (Gollop and Taylor 1996; Sivapullaiah and Jha 2014). In the first instance, without the gypsum, the main determinant in the formation of the cementitious products in the stabilised Soil – OPC – GGBS system is the GGBS’s hydration properties (Wild et al. 1998). Activation of the GGBS by OPC and some of the soil’s constituents does enhance the hydration of GGBS and the formation of the cementitious compounds. But, in the presence of the sulphate (gypsum), the sulphate tends to promote further, the activation of the GGBS through OPC-gypsum activation. This process then tends to enhance and accelerate the hydration of the GGBS so that strength is ultimately and significantly increased in a short period (Figure 7.6c).

Even though the increase in strength in the soil-gypsum-OPC-GGBS system as is the case in this study was made possible by the 50-50% OPC-GGBS combination however, Wild (1996); Wild et al. (1998) stated that the amount of strength would depend on the ratio of the GGBS to lime or cement ratio and vice versa. According to Wild (1996); Wild et al. (1998), gypsum addition could improve strength (due to ettringite formation) at low GGBS to lime or cement ratios hence, indicating that gypsum has a higher accelerating effect on the lime or cement-soil reaction than it will do on the reaction involving GGBS hydration.

In the case of the stabilised soil-gypsum-binder consisting of the OPC partially replaced by both the GGBS and the RC, a combination of the multiple but sometimes competing mechanisms of hydration resulting from the soil-gypsum-OPC-GGBS system and RC inclusion as mentioned previously, would lead to the production of more crystalline compounds of hydration through further and deeper penetration of the RC and water molecules of hydration to increase strength (Figure 7.6d).

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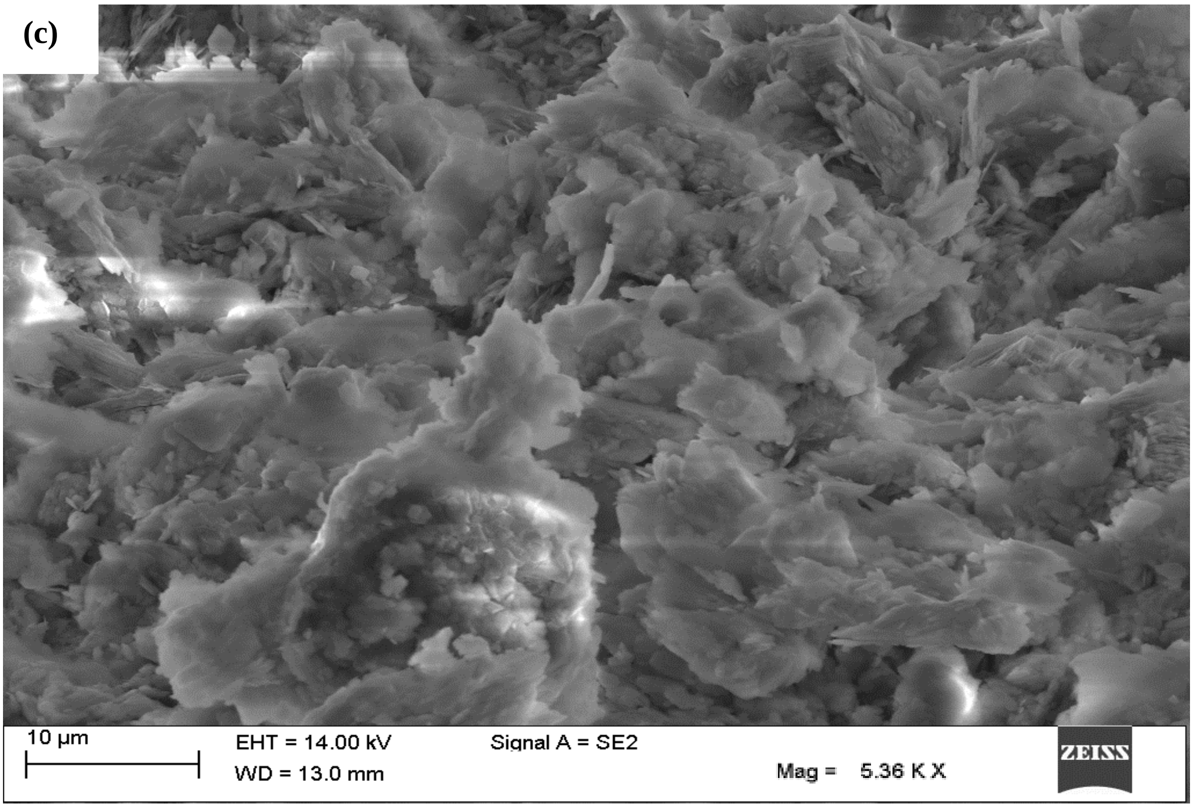
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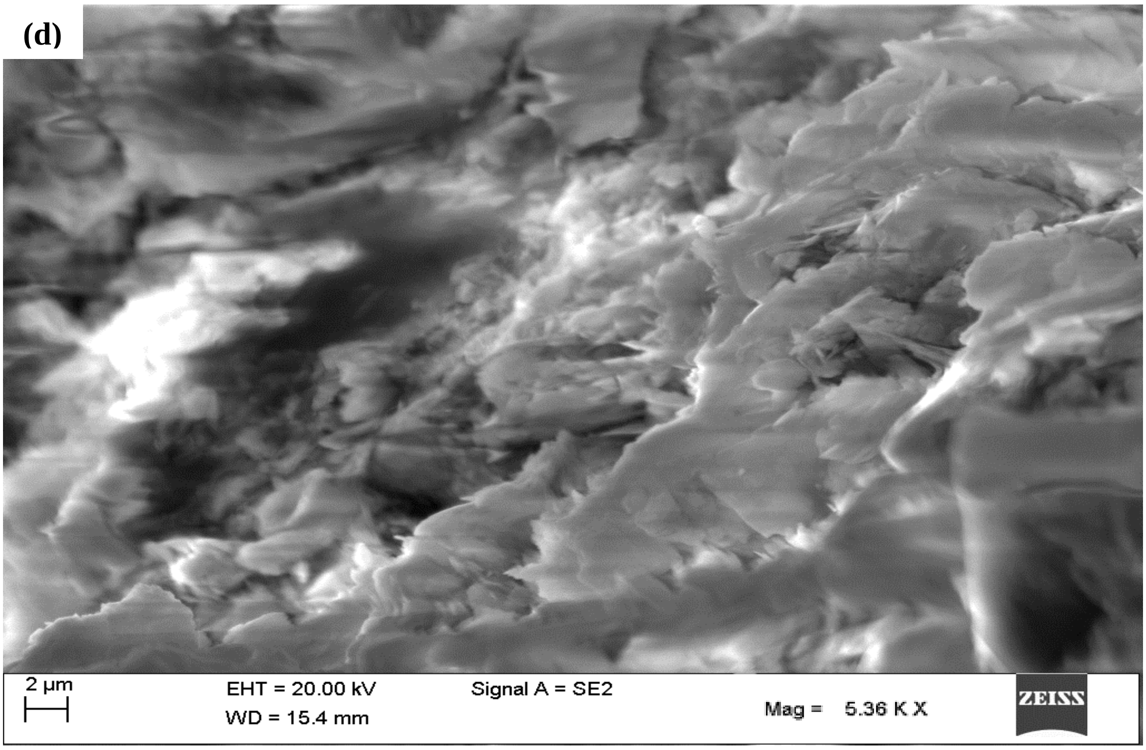


Figure 7.6 SEM of natural soil and soil stabilised by binder combinations (a) natural sulphate soil (b) sulphate soil stabilised by OPC (c) sulphate soil stabilised by OPC-GGBS combination (d) sulphate soil stabilised by OPC-GGBS-RC binder combinations

7.6.2 Swell

7.6.2.1 Natural sulphate soil

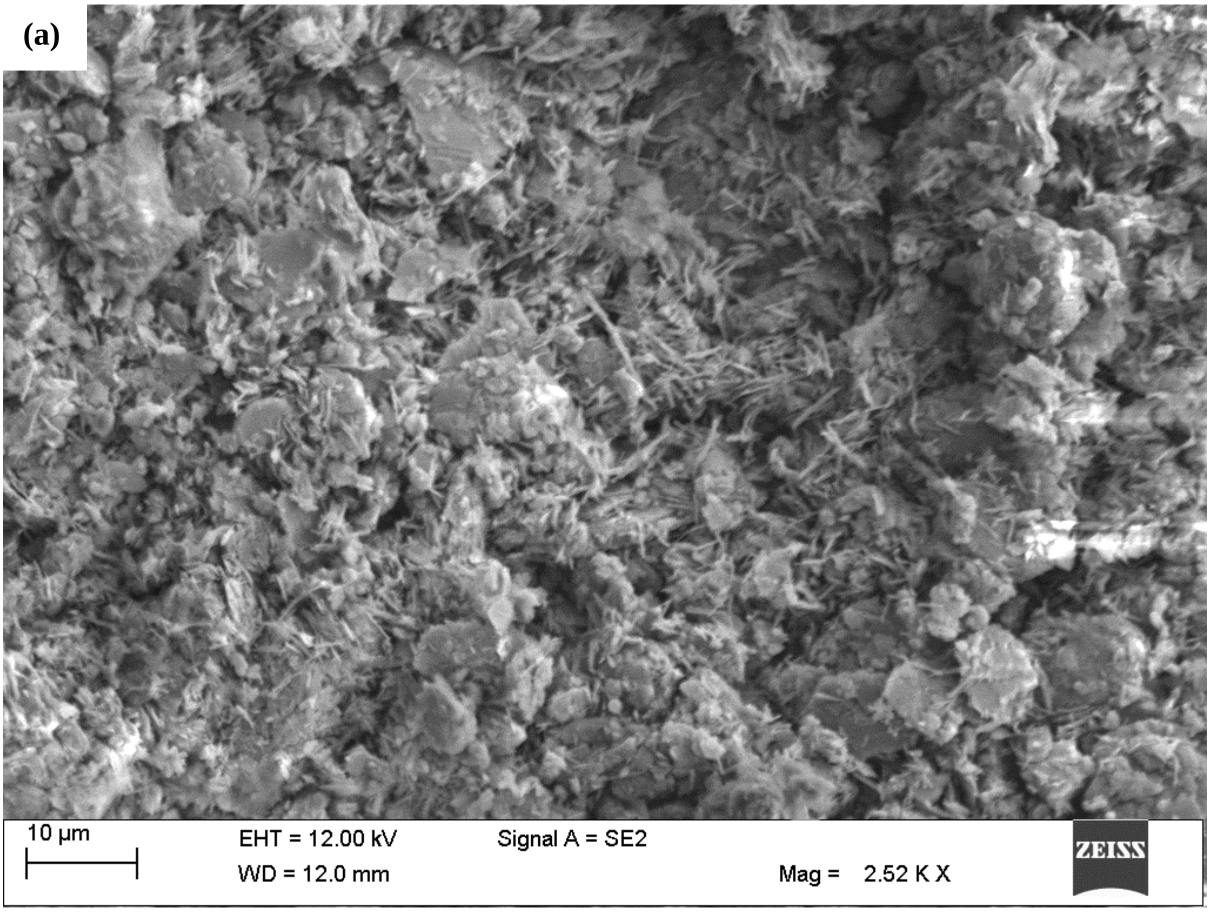
Upon soaking of the compacted samples in water immediately after 7 days curing, Figure 7.3a indicates only a slight increase in the swell of the natural sulphate-rich soil for all the amount of gypsum considered as compared to the pure natural soil. Even though the relatively stronger bonds formed due to the exchange occurring in the hydration process between the calcium ions (from the gypsum) and the weaker cations (from the soil), led to strength increase (of the compacted samples without inundation) of the sulphate rich soil (Figure 7.2), however, the same forces could not withstand the constant penetration of external water upon soaking within the 7 days curing period. A similar expansion effect of the presence of sulphate in natural kaolinite was observed by Abdi et al. (2019) who rather attributed the outcome to a slight decrease in the pH level (rise in acidity) of the soil-gypsum pore solution which engenders more adsorption of the water hence, resulting in increased swell potential.

Nevertheless, as the curing duration increased to 28 days, the swelling of the sulphate-rich soil did appear to reduce as compared to the pure natural soil (Figure 7.3b) indicating that an extended period of time was indeed required for more hydration and formation of stronger bonds which acted to reduce the swelling.

7.6.2.2 Stabilised-sulphate soil swelling

The sulphate-rich soils stabilised by the binders and their various combinations all seemed to swell higher than the stabilised soils in the absence of sulphates at both 7 and 28 days of curing (Figure 7.4). Prior to soaking, the sulphate ions and alumina existing only in limited amounts in the stabilisation reactions would bring about the precipitation of ettringites which within this time only serve as nucleation sites for the potential future growth of the crystals when both the sulphates and alumina become readily available (Little et al. 2010). Upon soaking, the movement of water into the stabilised sulphate soil will tend to solubilise the unreacted cementitious binders as well as acting as an avenue for more ionic migration hence, becoming a continuous source of supply of reagents at the nucleation sites (Hunter 1988; Burke et al. 1999). At 7 days, the amount of swelling seems to be the lowest for the OPC stabilised samples (Figure 7.5). Within this relatively short period, the initial hydration resulting in cationic exchanges and flocculation/agglomeration reactions may have been experienced more by the OPC-stabilised sulphate soils as compared to the sulphate soil stabilised by the by-products regardless of the amount of gypsum used. However, with the curing extended to 28 days, the effect of the by-product additives (GGBS and RoadCEM) tended to act to reduce the swelling more than the OPC used alone. Figure 7.7a shows the formed ettringites as a result of stabilisation of the sulphate soils by OPC at 28 days curing upon soaking with water. As stated previously, just as in the case of strength development, it is known that for the Sulphate soil-

OPC-GGBS system, sets of reactions occurring either simultaneously or in lieu of the other will ensue (Wild 1996). The hydration of GGBS which is activated by OPC in the presence of the sulphate resulting in the C-S-H and ettringite products, soil-OPC reaction in the presence of sulphates resulting in C-S-A-S-H and ettringite products as well as development of competing nucleation sites on the soil surfaces and the GGBS surfaces. For the partial replacement of OPC by the GGBS alone without the RC added, it has been suggested that the GGBS tends to contribute more in the production of the cementitious gels which are needed in hydration reaction mechanism to further reduce or some instances stop the growth of ettringites as seen in Figure 7.7b over the 28 days curing upon water soaking. Moreover, the protective influence against the attack of sulphates can become very much intense when higher amounts (up to 80%) of the GGBS are used to replace the OPC (Tasong et al. 1999; Wild et al. 1999). However, as seen previously in Figure 7.5 only 1% of the RC used in the stabilisation process would serve to reduce the swell further than the GGBS at 50% of the OPC replaced. It is believed that RC addition acts as a catalyst to promote further, the above-mentioned hydration reactions involving OPC and GGBS which does ultimately result in the prevention of swelling. The presence of more external water to the sample enables the formation of more of the crystalline pozzolanic products with the RC included thus eliminating any possibility of ettringite formation as observed in Figure 7.7c for the stabilised soil having 8% of the gypsum at 28 days curing upon soaking in water.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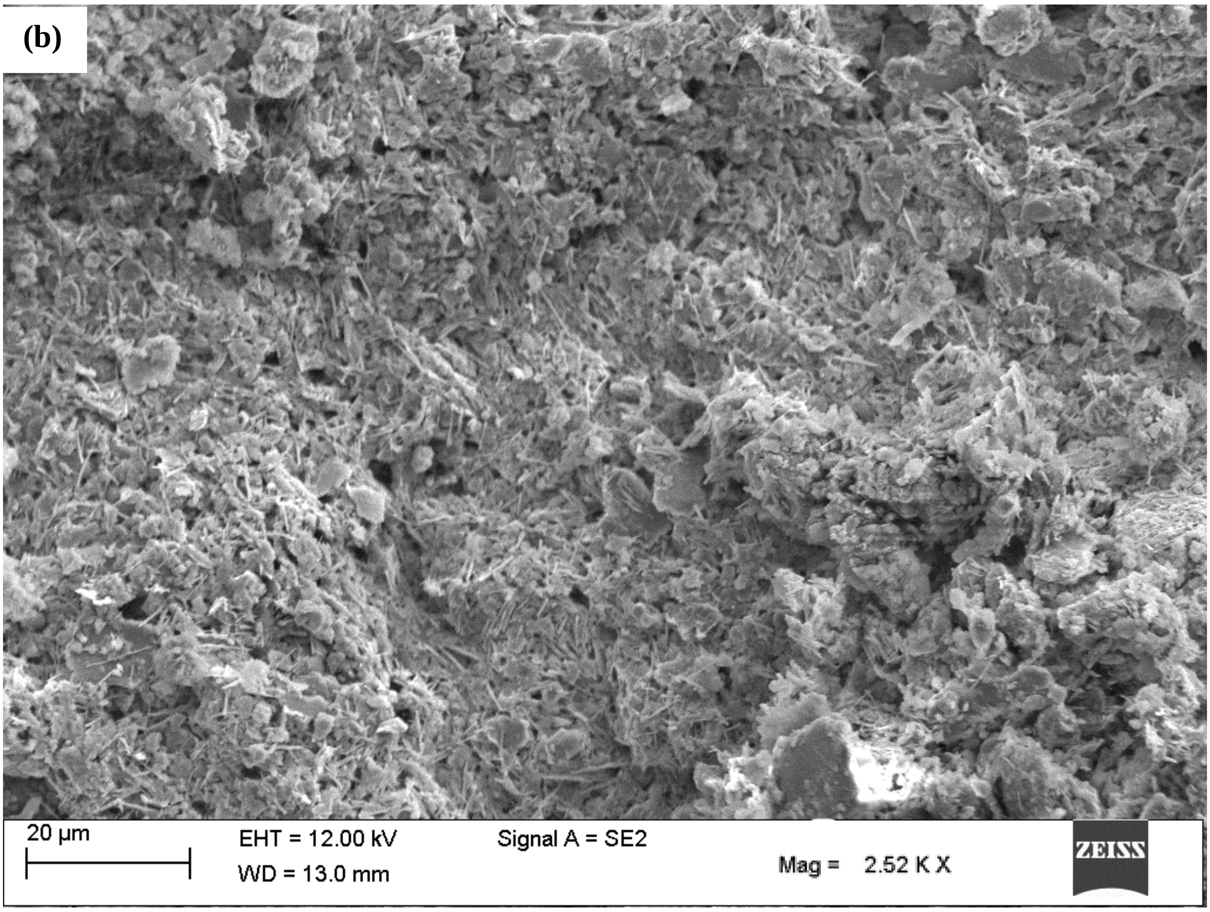
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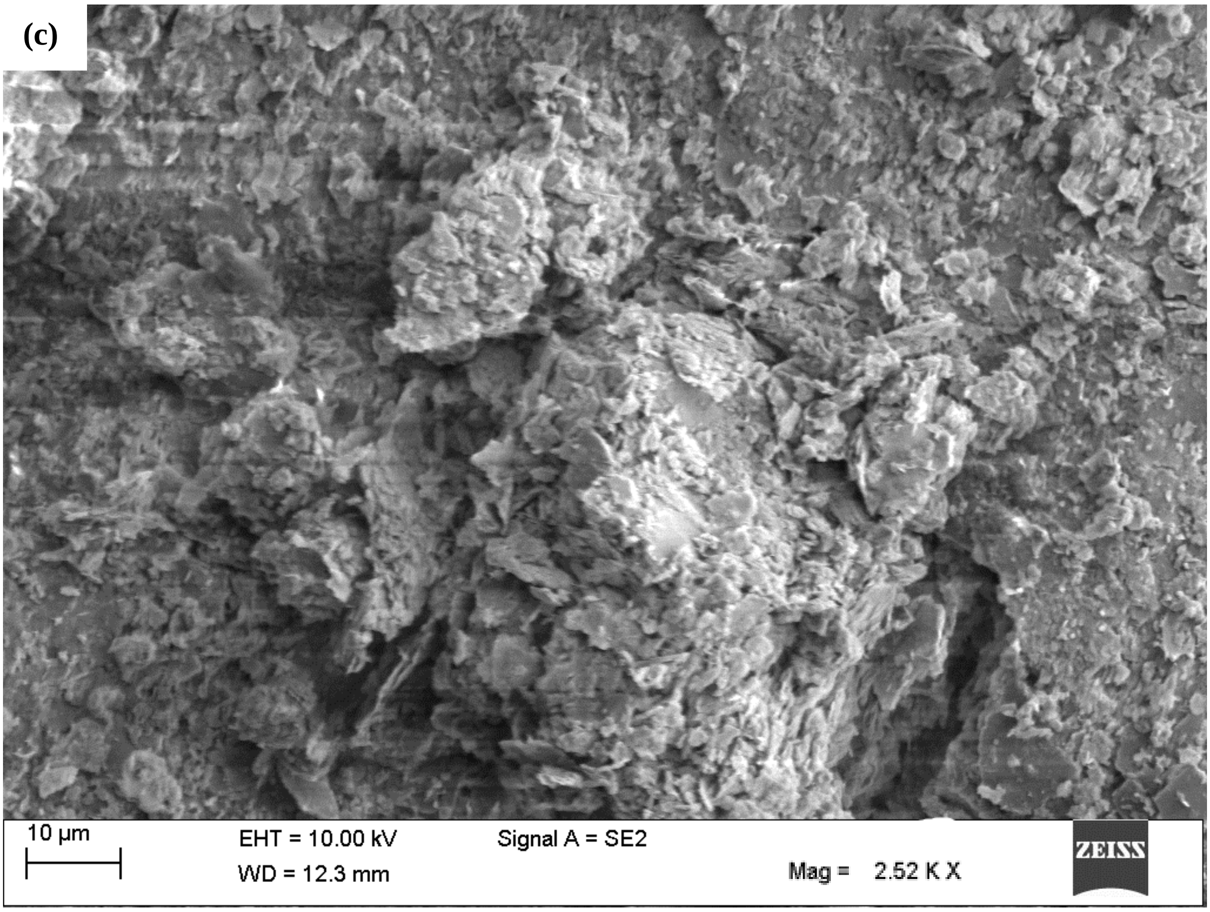

Figure 7.7 Stabilised sulphate soil with the binder combinations (a) OPC-stabilised sulphate soil (b) OPC-GGBS-stabilised sulphate soil (c) OPC-GGBS-RC - stabilised sulphate soil

7.6.3 Effect of gypsum content

The increase in the amount of swelling does not appear to correspond to the increase in the quantity of gypsum added (Figure 7.4). There could have been some limiting factors that prevent both the 8 and 12 % of gypsum added swelling linearly more than the 4%. As stated above, the mechanisms of ettringite formation (precipitated nucleation sites and subsequent crystal growth) tend to proceed very quickly the extent of which depends on the available limiting reagent present in solution (Little et al. 2010). Given that chemical reactions could only be judged as being as fast as their rate-limiting steps (or their slowest steps), this limiting factor does manifests either at the interface of the solid-solution through a topochemical mechanism or at the through-solution (or hydration) mechanism depending on ettringites' formation mode (Ivan and Michael 1988; Min and Mingshu 1994). It could have been that the saturation level of the gypsum had reached its limit at 4% (or slightly above that) or that the curing duration was relatively short, to allow for the competition of the process that would bring about an increase in the of quantities of soluble sulphates. James (2013) did suggest that the nature of the dissolved alumina and hydroxide ions may have also been the cause. With respect to the curing duration, it has been suggested that the process of ettringite formation that begins with the development of nucleation sites and subsequent growth of the crystal is time-dependent (Little and Syam 2009; Nair and Little 2009). As was noticed in Figure 7.4 that at 28 days, the rise in the level of gypsum (or sulphate) added seems to be slightly consistent with the amount of swell expected in the samples stabilised with the binders and their combinations. The higher gypsum content in the soil in this case provides a continuous source of the sulphates for the formation of a greater amount of ettringite. But again, this could be also dependent on other factors such as the soil structure, as well as those stated above regarding the formation mechanisms.

7.7 SUMMARY

An investigation of the effect of RC on soil exposed to sulphate attack was carried out in this chapter. This was done to further validate the authenticity of RC in soil stabilisation. In order to simulate the presence of sulphates, predetermined quantities (4%, 8% and 12% by dry mass of soil) of gypsum were added to **Soil A**. The sulphate-bearing **Soil A** was then stabilised by the binder mixture with 50% of OPC substituted by the with the RC included in various combinations. A brief summary of the improvement of swelling (upon soaking in water) which is considered one of the most useful engineering properties when dealing with sulphate soils, are presented as follows:

i. The percentage of swelling of the non-stabilised sulphate soil was more than that of the pure natural soil (without the gypsum added) for over 7 days of curing and testing under inundation with water. However, as the curing period increased to 28 days, the swelling did appear to reduce as compared to the pure natural soil. Generally, the variation in swelling seems to be marginal and does not correspond to the proportion of gypsum added.

ii. Generally, the stabilised sulphate-rich soils all expanded more than their stabilised counterparts without the gypsum added. In this instance, the rise in the level of sulphate was seen to be consistent with the amount of swelling in the samples stabilised with the binders and their combinations.

iii. A reduction of the swelling was observed when GGBS was partly replaced by the OPC. The GGBS contributed more in the production of the cementitious gels needed in the hydration reaction mechanism to further reduce ettringite formation as compared to OPC used alone in the soil stabilisation.

iv. An inclusion of RC to the cementitious binders (including OPC and GGBS) in the stabilised soil served to the swelling even further than the GGBS at 50% of the OPC replaced. The presence of RC enabled the formation of more crystalline pozzolanic products thus eliminating any possibility of ettringite as observed from the SEM measurements.

v. Gypsum also resulted in an increase in the strength of the natural soil and stabilised without soaking in water.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.1 SUMMARY

Using cement as a traditional binding agent for soil stabilisation can pose serious environmental and technical challenges in civil engineering construction. Current reported figures of carbon released as greenhouse gases due to cement production and usage are certainly very ominous and threatening to the environment. Moreover, sulphate-rich expansive clays treated by OPC could result in unwanted heaving instead of reducing it. Consequently, significant developments in knowledge have prompted a consideration of a non-traditional industrially-developed additive called “RoadCEM” (RC) as an alternative in soil stabilisation and construction that can mitigate both disadvantages.

This research aimed to investigate the geotechnical properties of expansive clays stabilised by substitution of most of the cement used in soil stabilisation by incorporation of RoadCEM.

Current mix specification guidelines recommend the usage of marginal quantities (1-2% by weight of OPC) of RoadCEM to improve cement hydration in soil stabilisation. However, in order to further reduce the percentage of cement that can be potentially used, this research utilised GGBS as a complementary cementitious and environmentally-friendly material while maintaining the industry recommendation of 1% of RoadCEM in the mixture. Hence, an investigation into the firmly established sustainability credentials of RC vis-à-vis its potential impact on mostly the volume change and hydromechanical behaviour of stabilised soils was the major thrust of this study.

8.2 CONCLUSIONS

Following preliminary studies on five expansive soils (having differing Na-montmorillonite contents), two model soils of extreme plastic properties were subsequently utilised for the application of the stabilising agents with the incorporated RoadCEM. Cement proportion in the binder mixtures was reduced by up to 50% and evaluation of the effect of RoadCEM on the volume changes and mechanical properties of the stabilised soil carried out along with the soil stabilised by cement used alone as a reference. The realisation of the main goals and objectives involved laboratory exercises such as index property tests, oedometer swell and consolidation tests, unconfined compression tests and suction tests on natural and stabilised clay samples. Microscopic examination and analyses using the SEM, EDS and XRD were also embarked upon to explain some of the mechanisms of change in the samples as well as to support the results and outcome of investigations on the tested samples. The following are the key findings from this study:

i. A unique application of the particle size analyser (Mastersizer) and the grain size statistics programme (GRADISTAT) enabled an identification and evaluation of the five natural expansive soils' behaviour by confirming the presence of clay-sized samples as the major cause of expansive soil swelling within a 95% confidence interval.

ii. The variability in the engineering properties of the natural and cement-stabilised clays is minimal and does not correspond to the percentage proportion of the clay-sized Na-montmorillonite particles present in the clays. Moreover, as the compacted clays become more plastic due to an increased montmorillonite content, the improvement in their mechanical properties may not only be attributed to cement used alone as a binding agent but also on the phenomenon of thixotropy.

iii. Addition of RC as an additive to the stabilised soil mixture reduced the swelling significantly more than the stabilised soil mixture without the RC used for all the investigated varied cement proportions. Moreover, the use of RC in the stabilised soil with the cement reduced by 50% led ultimately to zero expansion.

iv. From the point of view of the degree of settlement using the compression index Cc, values to compare **Soil A** stabilised with 50% of OPC replaced with the by-product possessing RC and without RC reveals that the mixture having the RC consolidates much less than that without the RC. 50% OPC substitution by the by-products containing the RC additive caused a reduction in Cc of approximately 80% in 7 days but with an even greater reduction of about 90% occurring at 28 days of curing.

v. The moisture-retention capacity of the stabilised soils having the proportion of RC are higher than those without the RC used. This phenomenon demonstrates a promising contaminant encapsulation feature for soils stabilised by inclusion of RC especially in dredging activities. Also, at ultimate saturation (zero suction), the moisture withholding capacity of soil stabilised by the addition of RC to cementitious products reduced, confirming its much-reduced swelling capacity.

vi. Soil stabilisation by partial replacement of cement with RC increased the unconfined strength of the stabilised soils within a short-investigated period of 7 days. Although the peak strength of the OPC replaced by 50% of the by-product containing RC was more than the OPC-only stabilised soil, the greatest strength development was experienced by stabilised soil with 40% of the cement reduced in the soil-binder mixture.

vii. The evolution of extended crystallisation in the hydration mechanism of the stabilised products as revealed by SEM indicated a “wrapping” (matrix of interlocking filaments) – a phenomenon which is only feasible with the presence of the RC additive as an agent in the stabilisation process. The ‘wrapping’ effect is responsible for the improved mechanical properties due to the inclusion of RC.

viii. As a means of validation of the use of a nanotechnology product as well as the optimum proportion of the binder mixture adopted, RoadCEM was incorporated in the stabilisation of a sulphate-bearing soil that is prone to sulphate heaving. Results indicated the efficacy of RC in reversing the heaving trend on a stabilised sulphate soil as compared to the stabilised mix that contains only cementitious products (cement and GGBS).

ix. SEM revealed a reduction in the ettringites formed in the stabilised soils resulting from the addition of RC as compared to the soil mixture in which the RC was not used.

8.3 STUDY LIMITATIONS

It is very pertinent to reiterate that this research relied mostly on standard laboratory-controlled methods and practices to achieve its aims and objectives. Hence, although this study recognizes soil stabilisation operations such as: mixing equipment capability, process of mixing, skills used by contractors and accurate geometric layout used in sound in – situ soil stabilisation schemes, it does not however provide some of the detailed specifications for field logistics and implementation. Nevertheless, the factors that make up the limitations of this study are as follows:

i. Some of the above-stated conclusions may only be restricted to the types of soils chosen for stabilisation by the binders. The soil types adopted for this study were industrially processed and of known composition. This was done intentionally to ensure the non-interference of some intrinsic chemicals, organics, etc especially when stabilised using the binders in this study. Hence, it might be possible that the use of naturally occurring soil samples could produce slightly different results of the geotechnical properties studied in this research.

ii. The results obtained from the consistency limit tests (liquid and plastic limits) of the natural and cement-stabilised soils can also be regarded as a constraint in this research even though the relevant standard laboratory practice was duly followed. This is because the actual performance of these tests relies solely on the skills and expertise of the operator.

8.4 RECOMMENDATIONS FOR FUTURE RESEARCH

i. In the development of a rational and comprehensive means of identifying and predicting natural expansive soil swelling, this study has depended on one of the most precise techniques in the analysis of particle sizes by using laser diffractometry. However, it is recommended that more validation studies using other methods of pore analysis such as mercury intrusion porosimetry (MIP) be explored and carried out.

ii. For the validation of some of the claims made herein regarding the improvement in the engineering properties of using RoadCEM, it is suggested that a broader range of undisturbed site soils be used in the stabilisation.

iii. Given that not all possible geotechnical characteristics of the stabilised soil were covered in this study, it might also be necessary to investigate the potential effect of RoadCEM on other properties of the stabilised soils such as, the California bearing ratio (CBR), resilient modulus of elasticity and permeability. Additionally, the period of curing of the stabilised soil samples can also be extended beyond the 28 days limit adopted in this study.

iv. This study made use of GGBS as a complementary material to cement, however, it is recommended that other environmentally friendly by-products, wastes, polymers and organics be employed, and their geotechnical performance compared to those in this study with the RoadCEM incorporated.

v. Fixed quantities of the OPC and RC proportions were utilised for this study based on frequent field practice, recommendations and experience for the types of soils studied in this research. It is suggested that as more additives or binders are adopted as stated in (iv) above, a more rigorous data training, regression and optimisation be performed in order to arrive at mixed proportions that are suitable for the stabilisation of the chosen soils based on predetermined engineering properties.

vi. Several other external and soil intrinsic factors that are not investigated in this study could bear possible influence on the soils stabilised by inclusion of RoadCEM. Hence, it is proposed that variables such as cycles of freezing & thawing, wetting & drying cycles, acidic conditions and other contaminants be considered in future research.

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APPENDICES

Appendix A: Soil water retention curves of natural and stabilised soils

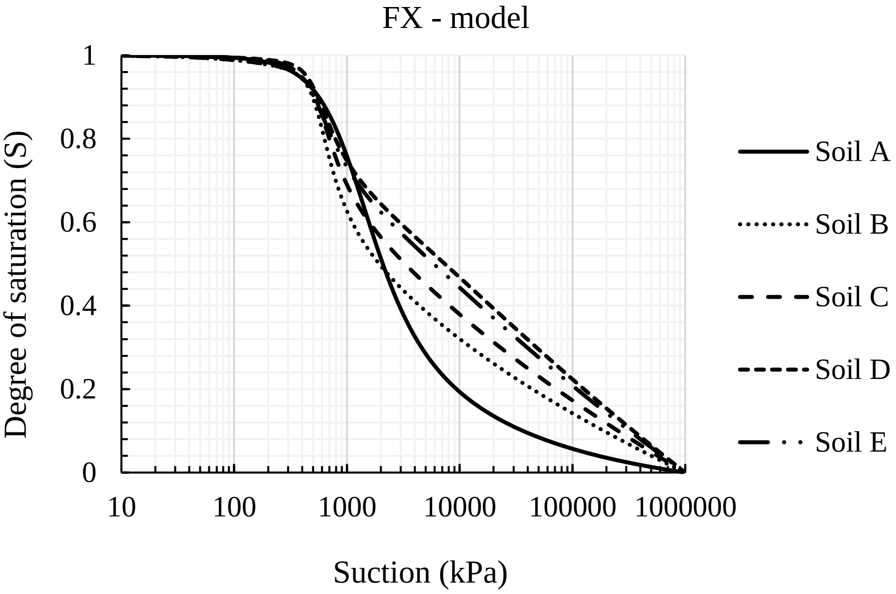
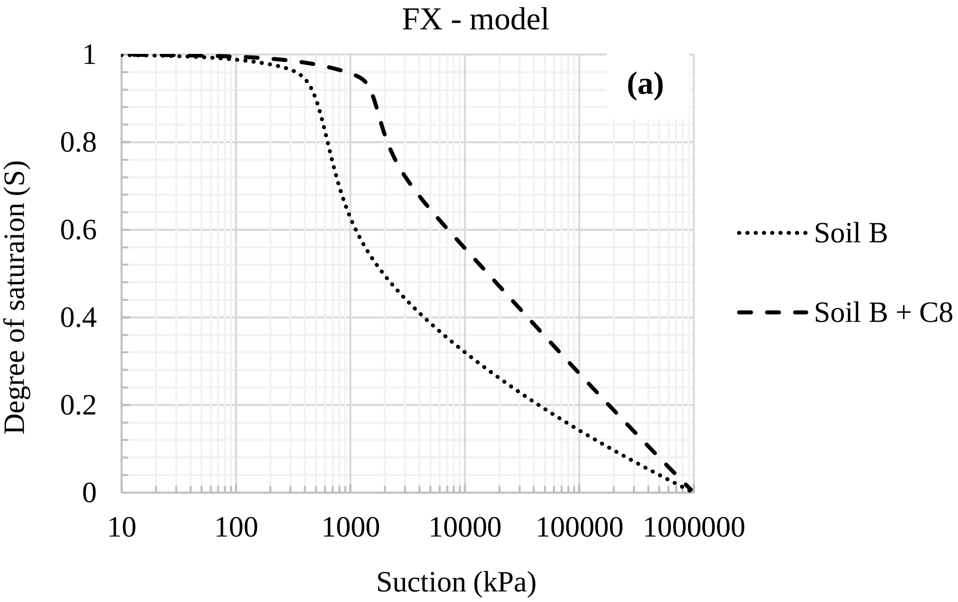


Figure A-1 SWRC model for natural clays.



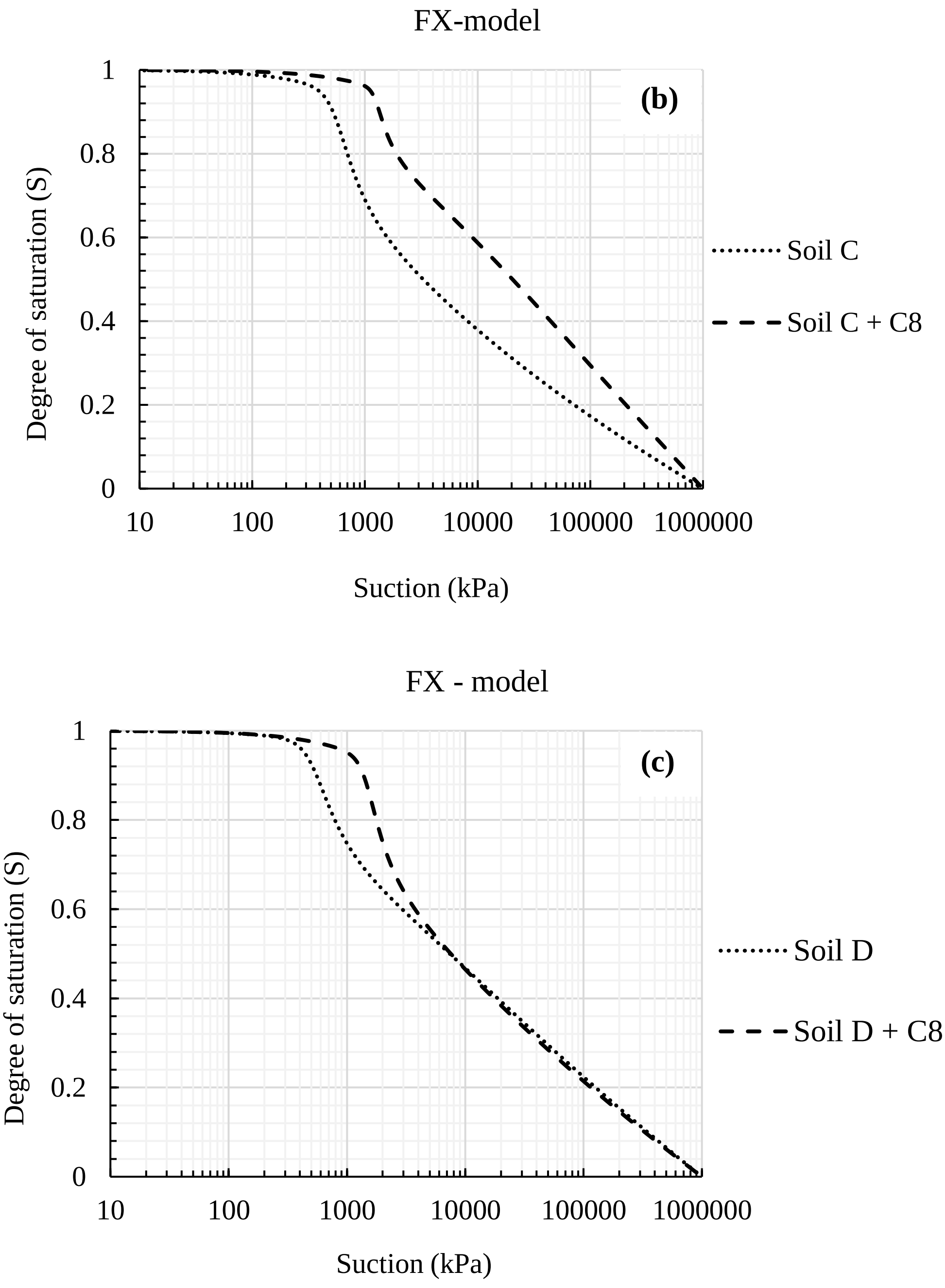


Figure A-2 SWRC model for natural and OPC-stabilised clays (a) natural and stabilised Soil B (b) natural and stabilised Soil C
(c) natural and stabilised Soil

Appendix B: Awards and Conference Attendance Certificates

Best Presentation – EEC Faculty Research Symposium, Coventry University

EEC Faculty Research Symposium Attendance Certificate

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DFI-EFFC Conference Participation Certificate

Content removed on data protection grounds.