LABORATORY INVESTIGATION ON STRESS-STRAIN BEHAVIOUR OF CEMENT-TREATED EXPANSIVE SOILS

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การทดสอบพฤติกรรมความเค้นและความเครียดของดินบวมตัวที่ผสมด้วยปูนซิเมนต์ใน
ห้องปฏิบัติการ

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วิทยานิพนธ์นี้เป็นส่วนหนึ่งของการศึกษาตามหลักสูตรปริญญาวิศวกรรมศาสตรดุษฎีบัณฑิต
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ซอเพียพ พอร์ : การทดสอบพฤติกรรมความเค้นและความเครียดของดินบวมตัวที่ผสมด้วยปูนซิเมนต์ในห้องปฏิบัติการ (LABORATORY INVESTIGATION ON STRESS-STRAIN BEHAVIOUR OF CEMENT-TREATED EXPANSIVE SOILS) อ.ที่ปรึกษาวิทยานิพนธ์หลัก: สุเชษฐ์ ลิขิตเลอสรวง, 182 หน้า.

วิทยานิพนธ์ฉบับนี้ดำเนินการศึกษาอิทธิพลของสัดส่วนผสมของดินเหนียวไม่บวมตัวและซีเมนต์ต่อพฤติกรรมของดินเหนียวบวมตัวด้วยวิธีการทดสอบในห้องปฏิบัติการเป็นหลัก โดยใช้ดินเหนียวกรุงเทพและเบโทไนต์เป็นตัวอย่างของดินเหนียวไม่บวมตัวและดินเหนียวบวมตัวในการศึกษาตามลำดับ มีการปรับเปลี่ยนส่วนผสมระหว่างดินเหนียวบวมตัวกับดินเหนียวไม่บวมตัวตามสัดส่วนต่างค่ากัน โดยเลือกสัดส่วนดินเหนียวบวมตัวต่อดินเหนียวไม่บวมตัวที่ 40:60 และ 20:80 มาปรับปรุงด้วยการผสมซีเมนต์ที่ 5 และ 10% ผลการศึกษาพบว่า พฤติกรรมของดินบวมตัวผสมจะขึ้นกับสัดส่วนของดินไม่บวมตัวเป็นหลักโดยเฉพาะในช่วงการผสมที่ 20-40% โดยการเพิ่มสัดส่วนของดินไม่บวมตัวจะส่งผลให้กำลัง ความแข็ง และค่าซีบีอาร์ เพิ่มขึ้น และสมบัติการบวมตัวและหดตัวลดลง ผลการศึกษาดินที่ไม่ปรับปรุงด้วยซีเมนต์ยังสอดคล้องกับงานวิจัยในอดีต ส่วนพฤติกรรมของดินบวมตัวที่ปรับปรุงด้วยซีเมนต์จะทำให้ค่ากำลังและความแข็งเพิ่มขึ้นอยากมาก และยังไปลดสมบัติการบวมตัวและหดตัวอย่างมีนัยสำคัญ การศึกษานี้ยังมีการพัฒนาเครื่องมือการอัดตัวหนึ่งมิติให้สามารถวัดค่าแรงดันแนวดิ่งและแนวราบได้ ตลอดจนสามารถนิยามค่าสัมประสิทธิ์แรงดันด้านข้าง จากผลการปรับปรุงดินบวมตัวด้วยซีเมนต์พบว่าสัมประสิทธิ์แรงดันด้านข้างของดินเพิ่มขึ้น เนื่องจากผลของซีเมนต์จะไปลดการบวมตัวในแนวดิ่งได้ดีกว่าแนวราบ ดังนั้นในการออกแบบโครงสร้างป้องกันดินวิศวกรควรตระหนักถึงค่าสัมประสิทธิ์แรงดันดินด้านของของดินบวมตัว และการลดลงของแรงดันด้านข้างด้วยการพิจารณาผลของการลดลงของหน่วยแรงในแนวดิ่งประกอบด้วย เนื่องจากผลของการปรับปรุงดินบวมดัวด้วยซีเมนต์

ภาควิชา วิศวกรรมโยธา ลายมือชื่อนิสิต ....

สาขาวิชา วิศวกรรมโยธา ลายมือชื่อ อ.ที่ปรึกษาหลัก ....

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KEYWORDS: BANGKOK CLAY / BENTONITE / SHRINKAGE & SWELLING / CEMENT-TREATED SOILS / LATERAL COEFICIENT OF SWELLING PRESSURE

SOPHEAP POR: LABORATORY INVESTIGATION ON STRESS-STRAIN BEHAVIOUR OF CEMENT-TREATED EXPANSIVE SOILS. ADVISOR: PROF. SUCHED LIKITLERSUANG, 182 pp.

This dissertation investigates an influence of non-swelling clay and cement contents on geotechnical properties of swelling soils. The Bangkok clay and bentonite were used as the non-swelling and swelling clays in this study. The swelling and non-swelling clays were mixed at different proportions. The 40:60 and 20:80 mixture clays were selected to be treated with 5% and 10% cement. It was found that engineering properties of expansive soils undergo favourable changes when mixed with a modest dose (20-40%) of non-swelling clays. However, a relatively large increase of non-swelling clay content is necessary to obtain markedly larger strength, stiffness, and CBR value, as well as reduce the shrink-swell properties. Correlations established among the untreated soil properties were tested against data from some previous study and found to be applicable satisfactory in most cases. On the other hand, while cement added to the mixture clays, the strength and stiffness of soils markedly increase along with a positively reduce of shrink-swell properties. In this study, a modified oedometer apparatus was developed to measure the vertical and lateral pressures, and then the lateral coefficient of swelling pressure can be defined. By adding cement to the swelling soils, the coefficient was markedly increased. It seems that the cement will affect to reduce swelling potential more in vertical direction. Hence, in the design of retaining structure on expansive soils, the engineer should aware the lateral coefficient of swelling pressure and the reduction of the lateral swelling pressure by considering the very rapid decrease of the vertical stress of cement-treated soils while compare that to untreated soils.

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Field of Study:	Civil Engineering	Advisor's Signature	....
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CHAPTER 1: INTRODUCTION

1.1 Background

Expansive soils is a problem that exists in many countries in the world, being particularly noticeable recently in some Asian countries such as India (Katti & Katti, 1994; Shelke & Murty, 2010), China (Ramaswamy & Anirudhan, 2009), Banglasdesh (Siddique & Hossain, 2013), Indonesia (Java, Sumatra, and Sulawesi) (Bukit, Frida, & Harahap, 2013), and Thailand (Sawangsuriya, Jotisankasa, Vadhanabhuti, & Lousuphap, 2012). Expansive soil is often formed via the in-situ alteration of volcanic ashes through hydrothermal processes. Expansive soils' features, which are significant volumetric expansion upon wetting and shrinkage upon drying, are a very common cause of problems that can cause failure of structures constructed above them (Bahia & Ramdane, 2012; Lim, Gomes, Kadir, & Abidin, 2013; Puppala, Manosuthikij, & Chittoori, 2013). The damages occur mostly in building foundations (Dasgupta, 2013), slopes (Zhan, Ng, & Fredlund, 2006), roadways, subgrades (Muntohar, 2006), highways, airports, seaports, retaining walls, and other residential buildings (Ramadas, Kumar, & Yesuratnam, 2012). The damage due to shrinkage and swelling of expansive soils that cost billions of dollars are estimated by various part of the world (Amer A Al-Rawas, 1999; Miller, 1997; Nuhfer, 1994). For example, in the U.S., the cost estimate associated with damage due to shrinkage and swelling of expansive soil was between \$6 billion and \$11 billion in damage annually to building, road, airport, pipelines, and other facilities (Nuhfer, 1994). In Asia, the costs of the damage have not yet been well-estimated.

Due to the fluctuation of underground water between dry season and rainy season, there is substantial variation of moisture content or suction in the ground, causing expansive soils to change in volume. Such volume changes and resulting shrink-swell movements or deformation often distresses infrastructure that is not designed to resist those swelling pressures or forced movements (Sudjianto, Suryolelono, & Mochtar, 2011). The potential of soil swelling deformation depends mainly on the nature of the soil, i.e. clay fraction, mineralogy, dry density, water content, etc. and the amount of water intake (Ferber, Auriol, Cui, & Magnan, 2009; Katti & Katti, 1994). Generally, the predominant mineral content in expansive soils includes montmorillonite or smectite (Radhakrishnan, Kumar, & Raju, 2014). Recently, the shrinkage characteristic of expansive soils, among other aspects, on natural and stabilised expansive clayey soils was investigated using two different measurement methods, manual and digital imaging, which focused on linear and three-dimensional volumetric shrinkages expansive soils (Puppala, Pokala, Intharasombat, & Williammee, 2007). The shrinkage and swelling movement or swelling pressure of expansive soil does not take place in only one direction (L.-J. Wang, Liu, & Zhou, 2015). Expansive soils swell laterally as well as vertically, and in fact, the lateral swelling pressure can be 2-10 times larger than the vertical swelling pressure (Fourie, 1989; Mohamed, Taha, & El-Aziz, 2014). If these pressures are greater than the foundation pressure, then uplift, often uneven and differential, will occur. This might cause cracks or damage to structures, especially retaining structures, which are normally not designed to endure the enormous swelling pressure. The swelling pressures depend on the loading and wetting conditions as a consequence of the different microstructure changes that occur under different conditions (Q. Wang, Tang, Cui, Delage, & Gatmiri, 2012).

Wetting of expansive soils causes such a great uplift force that suppressing their actions on structures purely by mechanical means (with piles and rigid linings, for example) is not a realistic option. Recently, measuring and assessing the swelling pressure has been given particular importance in designing foundations on expansive soils. In evaluating the swelling pressure, some researchers have suggested a technique to allow soil to heave until attaining equilibrium and then applying pressure to bring it to the original volume (Katti & Katti, 1994), while other researchers have proposed a zero swelling test (constant volume) and one-dimensional consolidation test for predicting heave in swelling soils (Fredlund, 1969; Q. Wang et al., 2012). However, these methods generally concern only the vertical swelling pressure measurement. There is still no reliable method available that allows the designer to predict the lateral swelling pressures on geotechnical structures due to swelling soils (Mohamed et al., 2014). The lateral coefficients of earth pressure of different soils and conditions have been investigated by some researchers, including studies on the lateral earth pressure of expansive black cotton soil (Katti & Katti, 1994), and on the effect of lateral swelling pressure of expansive soil on retaining structure (Mohamed et al., 2014). However, there is still limited research on the lateral coefficient of earth pressure of swelling soil. On the other hand, for non-swelling soils, the established theory of earth pressure based on force equilibrium and perfect plasticity may be adopted without a need to give further consideration to the additional pressure due to the mineral swelling. Therefore, many geotechnical engineers opt for excavating and replacing the entire soil, or to improve the existing ground. The ultimate countermeasure would be the total replacement of problematic expansive soils with non-expansive ones. However, it will be more practical and cost-effective to alleviate the expansive characteristics by mixing the problematic soils with less active, non-expansive soils at an appropriate ratio, thus limiting the amounts of both newly purchased soils and surplus soils that would be generated upon full replacement. If added soils have a cementing effect, it is considered to be advantageous in restricting the swelling and shrinking of expansive soils.

1.2 Statement of the problems

Civil engineering work cannot be separated from the most important aspects of the soil. A number of problems of civil engineering structures that are often found at construction sites are the result of the properties' low quality soil, which can be characterised by high moisture content, high compressibility, low bearing capacity, high swelling pressure during wetting and high shrinkage while the moisture content is squeezed out. The degree of swelling-shrinkage potential, as well as reversibility, is not well-known, and there may be differences in the effects or distribution. One of the types of soil that has disadvantageous characteristics is soil that is prone to high swelling-shrinkage. Some types of soils that have high swelling-shrinkage potential can undergo significant volume changes due to changes in water content. Such soil types include clay that contains minerals (smectite or montmorillonite) that have high swelling and shrinkage potential. The type of soil with this condition is often referred to as expansive soil, and has a large number of structures, especially lightweight structures (Figure 1-1) which are associated with serviceability performance, mainly in the form of cracks or permanent deformation.

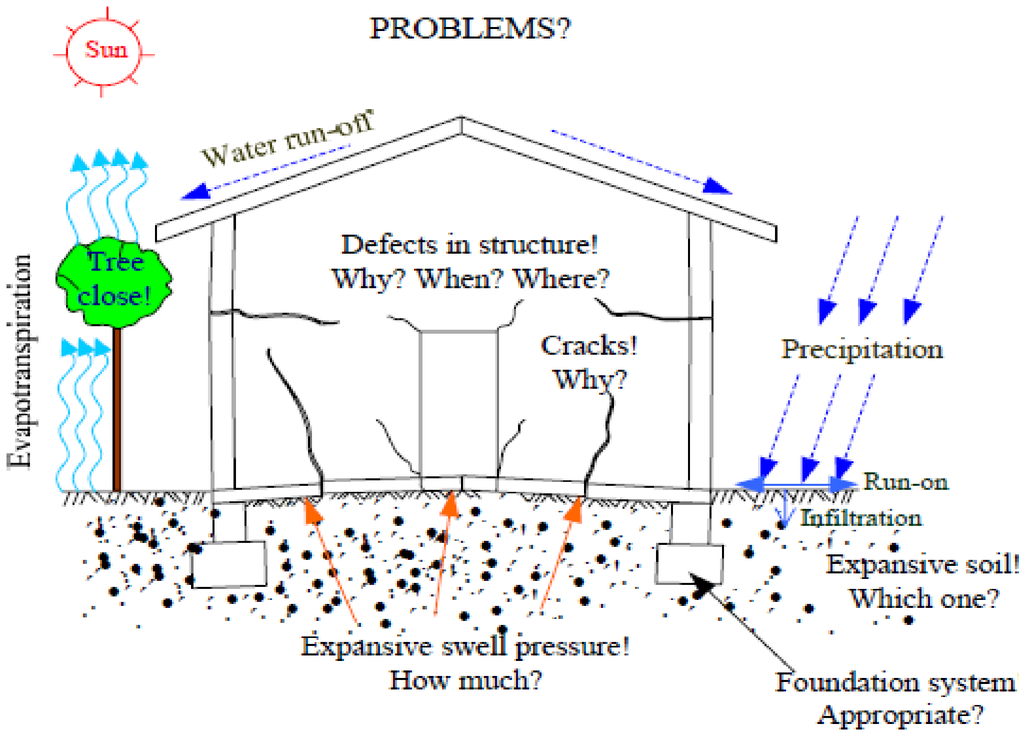


Figure 1-1: Idealized building with problems (Lucian, 2008).

1.3 Research objectives

The main objectives of this research are to show and characterise the influence of non-expansive soil and cement contents on physical, engineering, and mechanical properties of expansive soil by paying particular attention to its effect on swelling and shrinkage properties. The detail of the works carried out here are concentrated on the following subjects:

1. To investigate the influence of non-expansive clay on soil consistency (liquid limit, plastic limit, shrinkage limit), compaction and CBR characteristics; one-dimensional compression and swelling behaviour, strength, stress-strain and stiffness characteristics, and shrinkage-swelling strain behaviour of the expansive soils.

2. To investigate the influence of cement on strength, stress-strain and stiffness behaviours and shrinkage-swelling strain characteristics of the expansive soil using unconfined compression, areal shrinkage strain, and vertical free swelling strain tests, as well as the index properties tests. Recent research on the compaction of cement-treated and untreated expansive soils has also concentrated on the measurement/control of vertical stress and/or the control of horizontal stress during wetting under a confined condition using confined swelling pressure equipment. The effects of non-expansive soil and cement content on the lateral coefficient of earth pressure (K_s) and stress path, *q-p'* plan, were also investigated in this study.

1.4 Scope of work and limitation

To achieve the above objectives, the scope of work was restricted to research materials of artificial expansive soil, composed of non-swelling Bangkok clay, mixed with various percentages of expansive Na-montmorillonite bentonite clay to simulate an expansive soil at different levels of plasticity and swelling phenomena. A comprehension of the clay mineral type and influencing factors was reviewed, as well as the complexity of the shrinkage and swelling, mechanisms of soil-water interaction, and swelling pressure measurement methods. The Bangkok clay was firstly air dried, powdered and sieved though a No. 40 sieve (425 μm). The soil specimens were prepared either by reconstitution or compaction at its optimum moisture content and maximum dry density. A laboratory soil testing program was prepared, and numbers of experiments were conducted, including index properties, compaction and CBR, unconfined compression, consolidation, areal shrinkage strain, free swell and swelling under constant pressure (surcharge), and confined swelling pressure tests, including X-Ray diffraction and scanning electron microscope analysis. The analysis focused mainly on the stress-strain and shrink-swell characteristic of expansive soils as engineering materials at different soil plasticities, which result from mixing non-swelling Bangkok clay and bentonite. The study was also focused on the analysis of the effects of added cement as a stabiliser agent for engineering properties, stress-strain and shrink-swell characteristics, including the coefficient of lateral swelling pressure and stress path on *q-p’* plan of the expansive soil. Correlation between the index properties and mechanical properties were also made. Last is the comprehensive research report summarising the present research findings.

1.5 Research methodology

This research was conducted by means of literature review of expansive soils. The literature review was undertaken in order to provide a basic framework of understanding and available information regarding expansive soil and their damage to civil structure. Sources included hard copy books, proceeding conference’s paper, journals, dissertations, and other available online materials. A case study on the swelling mechanism of compacted bentonite was carefully selected to provide rich understanding on expansive soil behaviour and solutions to geotechnical problems related to expansive soils. To obtain the smooth operation of the research, the methodology was divided into four main steps as follows (Figure 1- 2):

1. Desk study: In the first step, after the statement of problems and the research objectives was set up, a literature review and data collection needed to be prepared in advance to develop more understanding and to find solutions that respond to those problems.

2. Materials selection: As the objectives of this research focus on expansive soil and the effect of non-swelling soil and cement on their engineering properties, the expansive soil used in this study should be a highly swelling soil. Hence, in this research work, the available non-swelling Bangkok clay and commercial Na-montmorillonite bentonite representing highly expansive soil, both locally produced in Thailand, were selected as the study materials.
3. Laboratory soil testing: In this step, artificial expansive soils made by mixing various proportions of bentonite and Bangkok clay were tested with and without cement treatment. The details of laboratory investigations on the shrinkage and swelling of expansive soil was performed to determine the engineering, physical, and

mechanical properties of the expansive soils and the effects of non-swelling Bangkok clay and the cement.

4. Analysis and Discussion: The analysis focused mainly on the stress-strain and shrink-swell characteristics of expansive soil and the influence of non-swelling Bangkok clay and cement added to expansive soils. Swelling pressure characteristics, both lateral and vertical, were also discussed, including the lateral earth pressure (Ks) and stress path in the *q-p'* plan. Some correlations between the index properties, engineering properties, and mechanical properties were made.

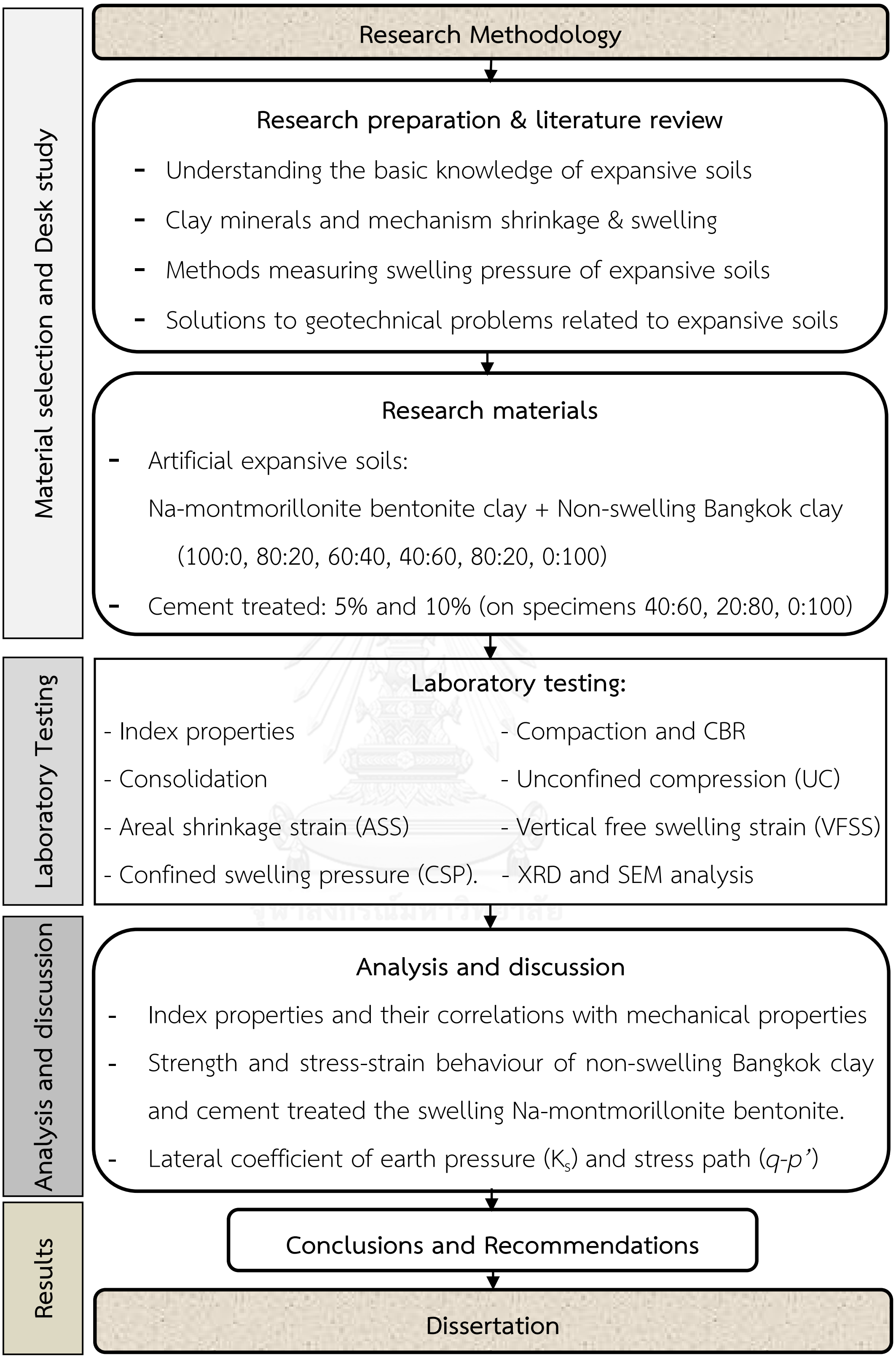


Figure 1- 2: Flow chart of the research work.

1.6 Structure of the thesis

This dissertation consists of six chapters.

- Chapter 1: Provides background on the research related to the distribution of expansive soils around the world, cost of damage annually due to swelling and shrinkage of expansive soils including a quick review of some previous researcher's methods to reduce shrinkage and swelling of expansive soils, definition of the problems, set up of the main objectives of the research, research methodology, and finally the scope and structure of this dissertation.
- Chapter 2: A comprehensive literature review was carried out and is summarised in the chapter 2. The objective of the literature review was to obtain a state of the art understanding of clay mineral types and mechanism of swelling, the factors influencing the shrinkage and swelling of expansive clays, clay-water interaction and water adsorption mechanisms. Existing methods investigating and interpreting the shrinkage and swelling mechanism of expansive soils during wetting and drying were also quickly reviewed. These were concerned with the methods measuring the shrinkage strain of expansive soil, swelling strain and swelling pressure both vertical and lateral direction, including the existing studies of lateral coefficient of earth pressure. The solutions to geotechnical problems related to shrinking and swelling soils were also presented, which provides ideas to help design the experimental portion of the research.
- Chapter 3: In chapter 3, a description of works related to material selection and material description is presented. The basic properties of the materials, such as grain size distribution and the index properties including mineralogical properties and scanning electron microscope (SEM) photography, are also described. A testing program of the untreated and cement-treated expansive soils was also prepared in advance. Reconstituted sample preparation and testing methods were performed. The detailed description of the methods and testing procedure are presented in this chapter. The methods consisted of compaction and CBR, unconfined compression, consolidation,

areal shrinkage strain, free swell and swelling under a constant pressure (surcharge), and confined swelling pressure, including the index properties tests.

- Chapter 4: All the testing results are presented in chapter 4. The results are separated in to two main parts. The first part focuses on the description of the effects of non-swelling Bangkok clay on the index properties of swelling Na-montmorillonite bentonite, compaction characteristics and CBR (soaked and unsoaked), stress-strain and strength characteristics, compression and swelling behaviour, areal and volumetric shrinkage strains characteristics, vertical free swelling strain and volumetric swelling characteristics in one-dimensional direction, including some correlations between the index properties; engineering properties, and mechanical properties. The second part focuses mainly on the effect of cement on the index properties of expansive soil (bentonite and Bangkok clay mixtures), stress-strain and strength characteristics of compacted untreated and cement-treated expansive soil. The effect of the cement on the shrinkage strain (areal and volumetric shrinkage strains) and swelling pressure characteristic, both lateral and vertical, were presented, including the lateral earth pressure (Ks) and stress path in the *q-p'* plan.
- Chapter 5: Discussion of the findings of this investigation is presented in chapter 5. The shrinkage and swelling properties of soil at different liquid limits, artificially prepared from a mix of bentonite and Bangkok clay, are discussed along with the effect of 5% and 10% cement-treated soils. The discussion also focuses on how the soil index properties correlate with other properties, such as swelling and shrinkage, as well as engineering and mechanical properties. This research also tries to show how the correlations obtained from the artificially expansive soil, bentonite and Bangkok clay, apply to the naturally occurring expansive soil by comparing the results of this study with some other previous studied.
- Chapter 6: Summarizes and concludes the findings of this research, and points to additional paths for further research.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

The variety of geomaterials encountered in geotechnical engineering problems is almost limitless (Mitchell & Soga, 2005). Mineralogy is the primary factor controlling the size, shape, and physical and chemical properties of soil particles. The range of solid particle size in soil can be great, from gravel and cobble to very small colloidal particles, which generally known as clay mineral. The term clay is somewhat ambiguous, in that it is used to refer both to a size and to a mineral type. A size term, it refers to all constituents of a soil smaller than a particular size, usually 0.002 mm (2 μm) for engineering applications. As a mineral term, it refers to specific clay minerals, which are distinguished by (1) small particle size, (2) a net negative electrical charge, (3) plasticity when mixed with water, and (4) high weathering resistance. Not all clay particles are smaller than 2 μm, and not all non-clay particles are coarser than 2 μm; however, the amount of clay mineral in a soil is often closely approximated by the amount of materials finer than 2 μm (Mitchell & Soga, 2005). A further important different between clay and non-clay minerals is that the non-clays are composed primarily of bulky shaped particles; whereas, the particles of most of the clay mineral species are platy, and in a few cases they are needle shaped or tubular. Clays can be divided into three main groups on the basis of the crystalline arrangement namely, kaolin, illite and montmorillonite groups, as summarised in Table 2-1.

A better understanding of the relationships between the microstructure and bulk physical properties of illite and kaolinnitic clays, especially in respect to the importance of physical state of the porewater has been studied by many investigators (Mitchell & Soga, 2005). Much less was known about montmorillonite clays to a very large extent because of the limited use of the material. Montmorillonite clay known as one type of expansive clays can both swelling and shrinkage or collapse on wetting and drying, respectively, depending upon the applied stress level and stress history, and therefore foundations or other lightweight structure built on these soils undergoes movement with swelling and shrinkage of the soil. As the result, there is considerable cracking and other forms of distress in the buildings, road and other structure. In these soils important physic-chemical interactions occur in the vicinity of the active clay mineral. Proper understanding of these interactions can explain some of the observed feature of soil behaviour. This help the constructing a conceptual image of the swelling and shrinkage processes. The aims of this chapter, therefore, some fundamental aspects of expansive clay behaviour at a micro-level (such as salient features of unit layers, cation exchange, and clay expansion theories) including their mechanisms are first briefly reviewed.

Table 2-1: Clay mineral groups (Murty, 2007)

No	Name of mineral	Structure formula
I	Kaolinite group 1. Kaolinite 2. Halloysite	 $Al_4Si_4O_{10}(OH)_8$ $Al_4Si_4O_6(OH)_{16}$
II	Illite group Illite	$K_y(Al_4Fe_2.Mg_4.Mg_6)Si_{8-y}$ $Al_y(OH)_4O_{20}$
III	Montmorillonite group Montmorillonite	 $Al_4Si_8O_{20}(OH)_4nH_2O$

2.2 Clay mineral and mechanisms of swelling

The clay minerals are formed through a complicated process from as assortment of parent materials. The parent materials include feldspars, mica, and limestone. The alteration process that take place on ground is referred to as weathering and that on the sea floor or lake bottom as halmyrolysis (Chen, 1975). The presence of various clay minerals such as montmorillonite in the expansive soil is determined by the use of x-ray diffraction (XRD), cation exchange capacity (CEC), thermal gravimetric analysis (TGA), infrared spectroscopy (IR), specific surface area, etc. The basal spacing values (in Angstroms, Å) determined by XRD, specific surface area (SSA) and cation exchange capacity (CEC) for different clay mineral groups are given in Table 2-2.

Table 2-2: Some of clay minerals characteristics (Mitchell & Soga, 2005)

Mineral	Interlayer bond	Basal spacing	Specific surface area (m^2/gm)	Cation exchange capacity (meq/100g)
Kaolinite	Hydrogen; Strong	7.2 Å	10-20	3-15
Montmorillonite	Oxygen-oxygen; Very weak	9.6 Å	700-840	80-150
Illite	K ions; Strong	10 Å	65-100	10-40

A surface area per cubic centimeter gram of a clay mineral is usually expressed in terms of specific surface area. A given volume, or weight of 2 μm clay has 50 times more surface area than the same quantity of very fine sand and 10 times more surface area than the same weight of silt. Colloidal clay (< 0.001 mm) exhibits 20 times the surface area than the same weight of silt and 1000 times the surface of fine sand. Non-expanding clay minerals commonly have a size smaller than 2 μm. The most common non-expanding clay minerals are illite, chlorite and kaolinite (Pusch & Yong, 2006). The SSA and CEC of illite and kaolinite is much smaller than that of montmorillonite (Table 2-2), which mean that it is significant compared to non-clay minerals, and hence important to the water uptake and retention. The clay with lower SSA and CEC has lower swelling potential during water intake. Clay minerals are complex aluminum silicates composed of two basic units, silica tetrahedron and alumina octahedron. Figure 2-1(a) illustrated a single unit of silica tetrahedron which consists of a silicon atom surrounding by four oxygen atoms.

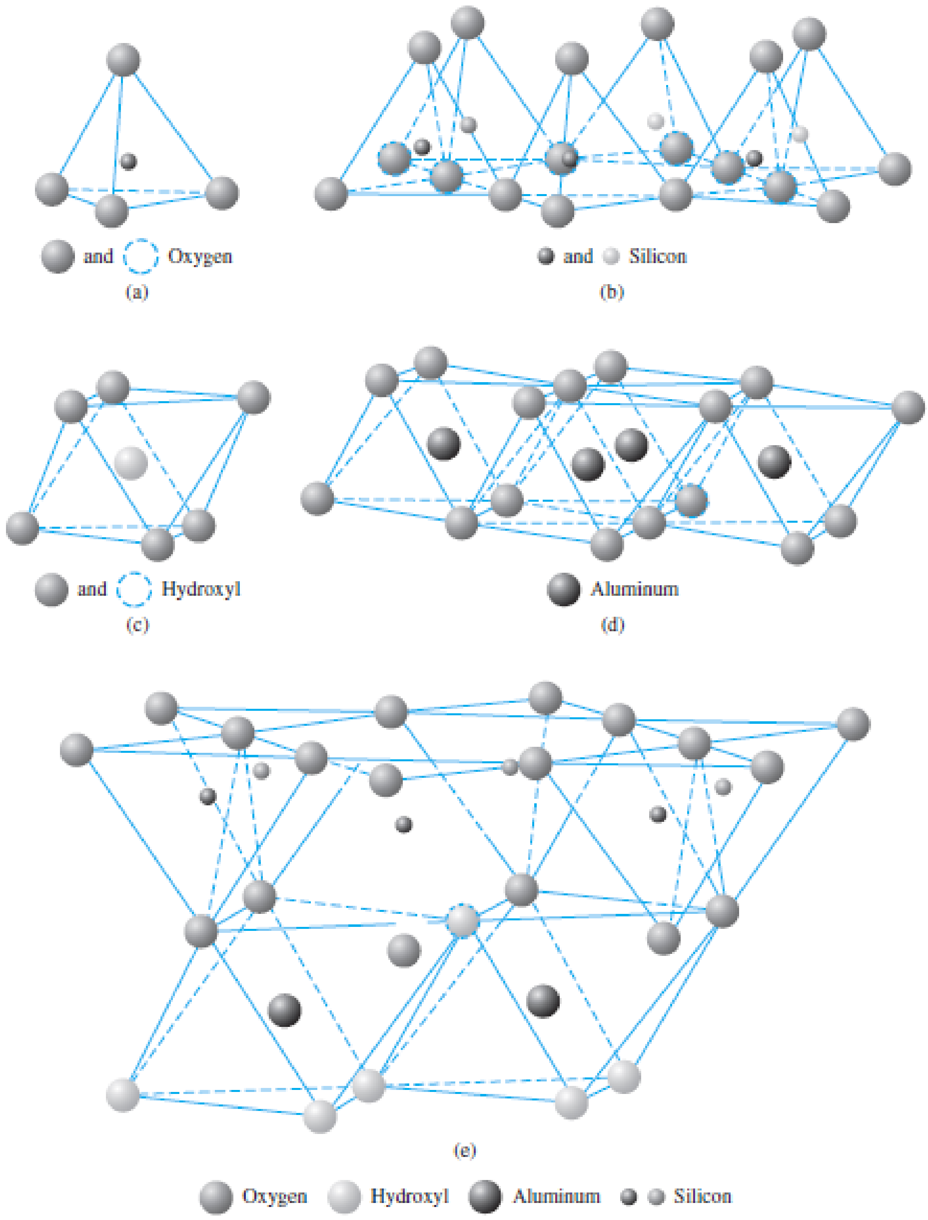


Figure 2-1: (a) Silica tetrahedron; (b) Silica sheet; (c) Alumina octahedron; (d) Octahedral (Gibbsite) sheet; (e) Elemental silica-gibbsite sheet (Das & Sobhan, 2013).

A silica sheet is the combination of tetrahedral silica units as illustrated in Figure 2-1(b). The octahedral units consist of six hydroxyls surrounding an aluminum atom (Figure 2-1c), and the combination of the octahedral aluminum hydroxyl units gives octahedral sheet or gibbsite sheet (Figure 2-1d) while Figure 2-1(e) show a staking between silica sheet and Gibbsite sheet (octahedral sheet) to form an elemental Silica-Gibbsite sheet. These element can be easily drawn as illustrated in Figure 2-2.

2.2.1 Kaolin group

The kaolin groups of minerals are the most stable of the groups of minerals. This group includes the dioctahedral minerals kaolinite, dickite, nacrite, and halloysite, and the trioctahedral minerals antigorite, chamosite, chrysotile, and cronstedite. All members of the kaolinite group form primarily during hydrothermal alteration or weathering of feldspars under acid conditions, but kaolinite and halloysite are most likely the only members formed in soils (Poppe, Paskevich, Hathaway, & Blackwood, 2001). The kaolin mineral consists basically repeating layers of elemental one tetrahedral (silica) sheet and one octahedral (alumina or gibbsite) sheet (silica-gibbsite sheets). Because of the stacking of one layer to each of the two basic sheets, kaolinite is called a 1:1 clay mineral (Figure 2-2a). The two sheets are held together in such a way that the tips of the silica sheet and one of the layers of the octahedral sheet form a single layer. This layer is about 7.2 Å thick and extends indefinitely in the other two directions. A kaolinite crystal, then, consists of stack of several layers of the basic 7.2 Å layer. Successive layers of the basic layer are held together by hydrogen bonds between the hydroxyls of the octahedral sheet and the oxygen of the tetrahedral sheet. Since the hydrogen bonds are comparatively strong, it prevents hydration and allows the layers to stack up to make a rather large crystal that is difficult to dislodge. The mineral is, therefore, stable and water cannot enter between the sheet to expand the unit cells.

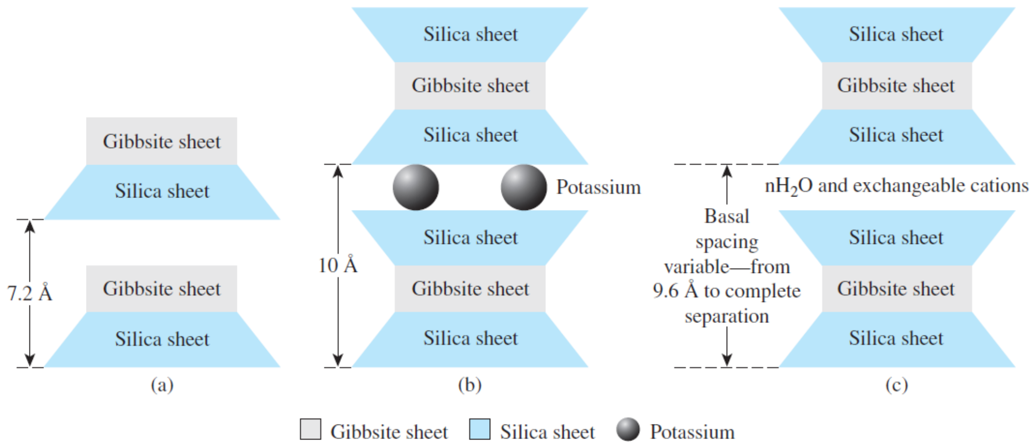


Figure 2-2: Diagram of the structure of (a) Kaolinite; (b) Illite; (c) Montmorillonite (Das & Sobhan, 2013).

2.2.2 Illite group

Illite is essentially a group name for non-expanding, clay-sized, dioctahedral, micaceous minerals. The structural arrangement of the illite mineral is composed of two silica tetrahedral sheets and one alumina (gibbsite) sheet it has a 2:1 structure and at the interlayers are bonded together with a potassium atom (Figure 2-2b). Illites, which are the dominant clay minerals in argillaceous rocks, form by the weathering of silicates (primarily feldspar), through the alteration of other clay minerals, and during the degradation of muscovite (Poppe et al., 2001). Illite exhibits more plasticity than kaolinite, and has little tendency to change volume when exposed to a change in moisture content unless there is a deficiency in potassium, in which case the illite particle will exhibit an increased tendency for volume change. The presence of potassium as the bonding materials between units makes the illite mineral swell less.

2.2.3 Montmorillonite group

Members of the montmorillonite (smectite) group include the dioctahedral minerals montmorillonite, beidellite, nontronite, trioctahedral minerals hectorite (Li-rich), saponite (Mg-rich), and sauconite (Zn-rich) (Poppe et al., 2001). The structural arrangement of the montmorillonite mineral is composed of two silica tetrahedral sheets and one alumina (gibbsite) sheet (Figure 2-2c). Thus montmorillonite is also called 2:1 mineral structure similar to illite. The silica and gibbsite sheet are combined in such a way that the tips of the tetrahedrons of each silica sheet and one of the hydroxyl layer of the octahedral sheet from a common layer. The atoms common to both the silica and gibbsite layers are oxygen instead of hydroxyls. The thickness of the silica-gibbsite-silica unit (2:1 layer) is about 9.6 Å thick structure units (Figure 2-2c), and like kaolinite the layers extend indefinitely in the other two directions. Because of the bonding by van der Walls' force between the tops of the silica sheets is weak and is a net negative charge deficiency in the octahedral sheet, water and exchangeable ions can enter and separate the layers (Mitchell & Soga, 2005). This mineral exhibits considerable variation in characteristics because of the interchange between elements within each sheet. This mineral exhibits the highly undesirable characteristic of undergoing considerable change in volume when moisture is added to or lost from the soil mass. This characteristic can lead to very serious problems of heaving or of settlement. Soils containing montmorillonite are very susceptible to swelling as they change (increase) water content (Miller, 1997; Mitchell & Soga, 2005), and the swelling pressure developed can easily damage light-weight structure and highway pavement.

2.3 Factors influence shrink-swell potential of expansive clays

Expansive clays derive their high degree of shrinkage and swelling potential mainly depend upon the presence of active clay minerals which known as montmorillonite. The other important factors controlling the shrinkage and swelling potential are the initial dry density (void ratio) and water content of the soil specimen, the nature of pore fluid, the type of exchangeable cations, the overburden pressure, and the wetting and drying effects (Chen, 1975; Miller, 1997). The parent minerals for the formation of montmorillonite are residual soils formed by the weathering of basaltic rocks under alkaline environment often consist of ferromagnesium mineral, calcic feldspars, volcanic glass, and many volcanic rocks. Due to the extended periods of dry climate cause desiccation of the soil but rain cause swelling near the ground surface. The depth of the swelling zone is not much, being less than 5m in most case. However, the active zone over which there are seasonal changes in moisture content varies from 1.5 m to 4 m (O'Neill & Poormoayed, 1980). The moisture content variations with depth below the ground surface are shown in Figure 2-3.

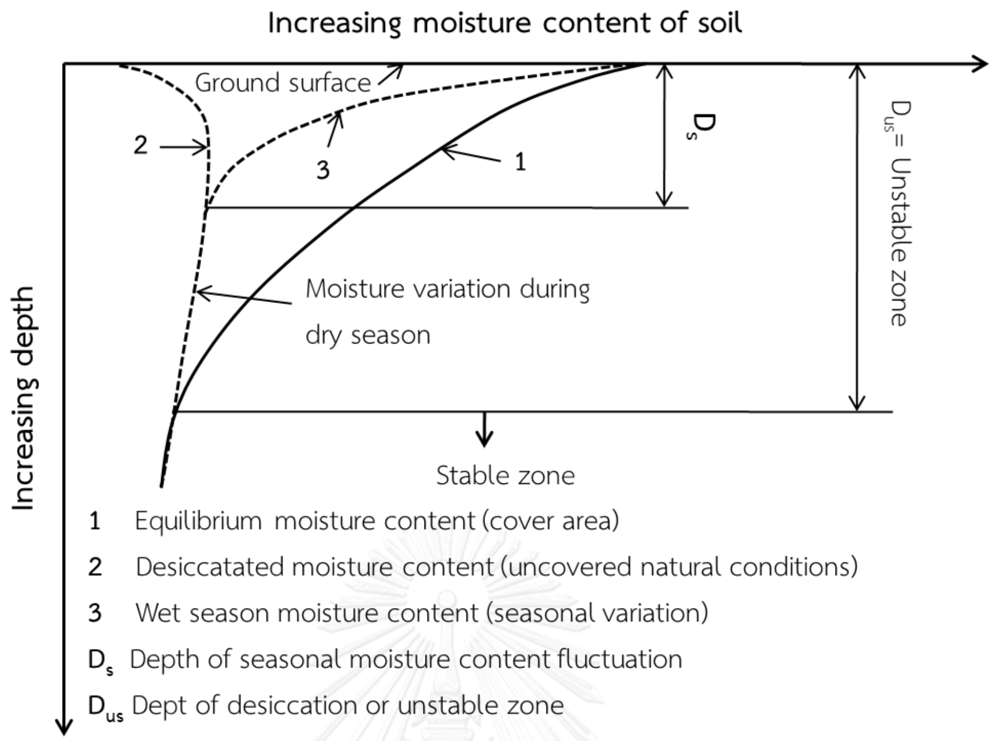
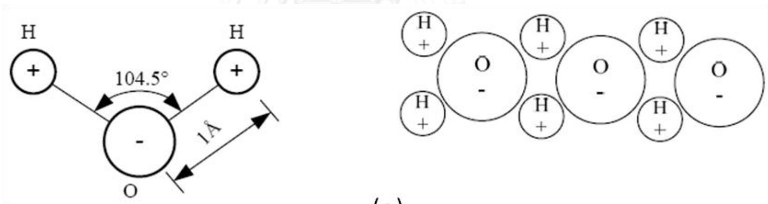


Figure 2-3: Moisture content variation with depth below ground surface (Murty, 2007).

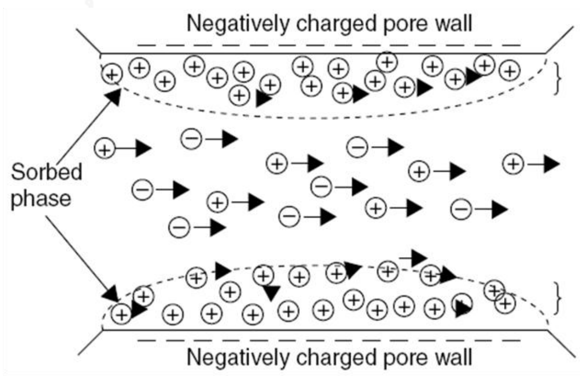
The swelling potential and swelling pressure are known to increase with increase in expansive clay content and dry density and decrease with increase in initial water content, overburden pressure, pore salt concentration, and exchangeable cation valence (Chen, 1975). It has hence been suggested that clay soils could be compacted at water contents in excess of the optimum moisture content (OMC) values to control their swelling potentials (Gromko, 1974).

2.4 Mechanisms of soil-water interaction

Interaction between clay particles, dissolved ions, and water are caused by unbalanced force fields at the interfaces between clay and water. The properties of water are known as clear, colorless, odourless, tasteless liquid that has a density very nearly equal to unity, freezes at 0 ^{o}C, boils at 100 ^{o}C, and has defined viscosity and thermal properties. In reality, the situation is different because neither water, nor soil surfaces are inert chemically and they interact with each other influencing the soil behaviour (Mitchell & Soga, 2005). The water (H_2O) is the polar molecule composed of a V-shaped arrangement of dipoles (two hydrogen nuclear poles of positive charge) attracted to one negatively charged oxygen atom resulting into ion hydration, with an average H-O-H angle of 104.5° (Figure 2-4a). Since opposite charges attract, the positive molecules attract the negative molecules forming a chain of water molecules. Clay particles, because of their very small size and platy shapes, have large surface areas, especially montmorillonite clay (Table 2-2), when external surfaces of each unit layer (basal spacing) of clay mineral come close to each other, interacting diffuse electrical double-layers are formed with a relatively high concentration of cations near the surface. As a result of negative charges in the clay surfaces, the cations present in the solution (water) are attached to the immediate vicinity of the clay particles. This affects the organisation and physical state of the hydrates (Figure 2-4b). Dried clay swells when give access to water, and temperatures above 100 °C are need to remove all the water from clay. Possible mechanisms for clay-water interaction include the following and show graphically in Figure 2-5.



(a)



(b)

Figure 2-4: (a) Water molecule; (b) Interacting electrical double layer (Pusch & Yong, 2006).

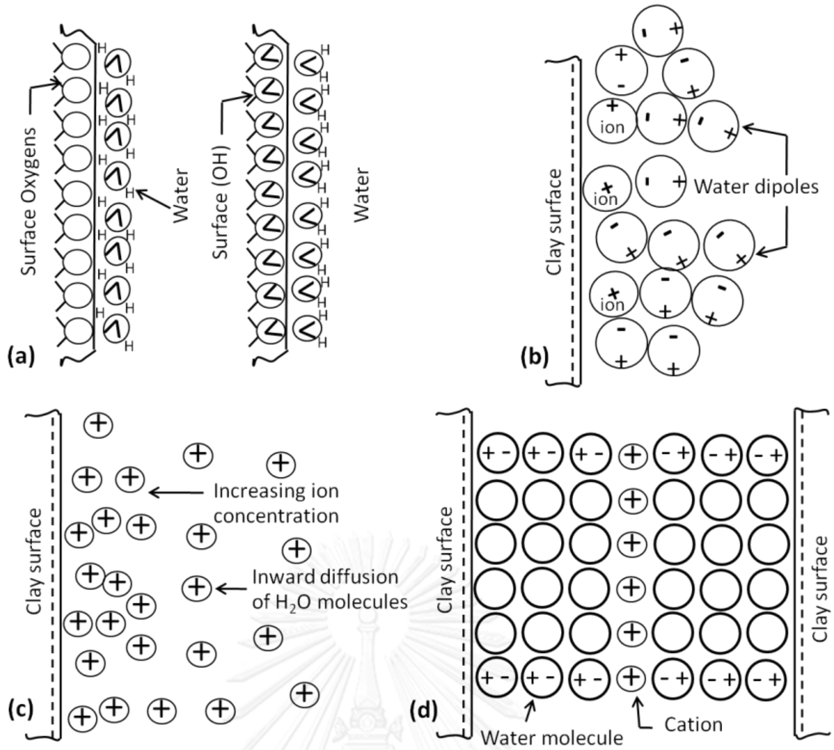


Figure 2-5: Possible mechanisms of water adsorption by clay surface: (a) hydrogen bonding, (b) ion hydration, (c) attraction by osmosis, and (d) dipole attraction (Mitchell & Soga, 2005).

a). Hydrogen bonding

The term hydrogen bond is the attractive force between the hydrogen attached to an electronegative atom of one molecule and an electronegative atom of a different molecule. Usually the electronegative atom is oxygen, nitrogen, or fluorine, which has a partial negative charge. The hydrogen then has the partial positive charge. Because the surface of clay mineral is a layer of either oxygens or hydroxyls, hydrogen bonding can easily develop. Since a hydrogen ion form the positive end of a dipole, its attraction to the negative end of as adjacent molecule is termed a hydrogen bond (Figure 2-5a).

b). Hydration of exchangeable cations

Hydration of exchangeable cations is the process in which the cations are attached to negatively charged clay surfaces as shows in Figure 2-5(b). This mechanism is significantly occurred at a lower moisture contents.

c). Attraction by osmosis

The concentration of cations increases as negatively charged clay surfaces are approached as shows in Figure 2-5(c). Because of this increased concentration and the restriction of diffusion of ions from the vicinity of the surface, as a result of electrostatic attraction, water molecule tends to diffuse toward the surface in an attempt to equalise concentrations.

d). Change surface-dipole attraction

Because of clay particle representing as a negative condenser plate. Water dipoles would then orient with their positive poles directed toward the negative surfaces and with the degree of orientation decreasing with increase distance from the surface. However, at the midplane between parallel plates there would be structural disorder because like poles would be adjacent to each other (Figure 2-5d). This is because of the high hydration number and energy of aluminum in the clay structure, thus water is strongly attracted to the surfaces that it interposes itself between the surfaces and the counter ions, with the counter ions removed as far as possible from the surface, that is, to the midplane between opposing parallel sheets. With this model the structure shown in Figure 2-5(d) can be conceived. The same type of arrangement could result simply from ion hydration. In dry clay, adsorbed cations occupy positions in holes on the move to the central region between clay layers.

2.5 Methods measure and identify potential shrink-swell of expansive soils

The different clay minerals groups exhibit a wide range of engineering properties. Within any one group, the range may also be great and is a function of such factors as particle size, degree of crystallinity, type of adsorbed cations, pH, the presence of organic matter, and the type and amount of free electrolyte in the pore water (Mitchell & Soga, 2005). Generally, the importance of these factors increase in the order kaolin < illite < montmorillonite. In order to select an appropriate design and methods of construction, identification of potential shrinkage and swelling of subsoil as their foundation is an important step. There are several geological and geotechnical methods to identify the shrinkage and swelling potential of expansive soils. Those are visual identification, geological and geomorphological identification, and identification by grain size analysis, Atterberg limits, and some other direct and indirect methods. The direct methods such as free swell test, swell in the oedometer, cation exchange capacity (CEC), and X-ray diffraction (XRD) microscopy. The indirect methods include Casagrande's plasticity chart, plasticity table, colloid content (clay contents), soil activity, plasticity index, etc.

2.5.1 Visual identification

The shrink-swell potential of expansive soil can be estimated by observing the extent of desiccation cracks (Figure 2-6). Clayey soils rich in montmorillonite develop of desiccation cracks greater than the soil with none or less montmorillonite minerals, i.e. halloysite, kaolinite and illite (Miller, 1997; Mitchell & Soga, 2005). Generally, high desiccation cracks develop in the sun-parched ground surface during dry season (Day, 1999). The large size of the cracks and more frequent polygon arrangement of cracks, the higher degree of swelling potential are while that the small and thin crack indicates low shrink-swell (Mitchell & Soga, 2005). Other visual identification can be easily observed by shoes or tires of vehicles, the high potential of shrink-swell soil when wet become very sticky and plastic than that the lower and attached to the shoes or tire of the vehicles.

Figure 2-6: Deformation and cracking of road structure during dry season, Ngawi, Indonesia (Por & Karnawati, 2012).

2.5.2 Geological identification

Expansive soil are widely distribute over the world and generally losses of thousands of millions of dollars per year due to shrinkage and swelling of soil foundation, result crack or permanence deformation to civil structure (Katti & Katti, 1994; Miller, 1997; Nuhfer, 1994). The distribution of expansive soils are closely related to regional geological background, climate, hydrology and geomorphology (Habib, 2013). Generally, the parents' rocks of expansive soils are ultrabasic igneous rocks, metamorphic rocks, limestone, sandstone and mudshale which those type of rock contain plenty of silicate minerals and easily convert into clay minerals such as montmorillonite, illite and kaolinite during weathering. A good geological mapping could provide rich information related to method of forming a mass into size, shape and their behaviour. The site history could well understanding which provide valuable idea of the soil composition as the preliminary information for further investigations.

2.5.3 Identification based on soil consistency

The different clay minerals presence in different size range because mineralogical composition is a major factor in determining particle size. The shape of the most common clay mineral are platy, except for halloysite which occur as tubes (Figure 2-7a). Particles of kaolinite are relatively large, thick, and stiff (Figure 2-7b). The montmorillonite (smectite) are composed of small, very thin, and filmy particles (Figure 2-7c). Illite are the intermediate between kaolinite and montmorillonite (Figure 2-7d).

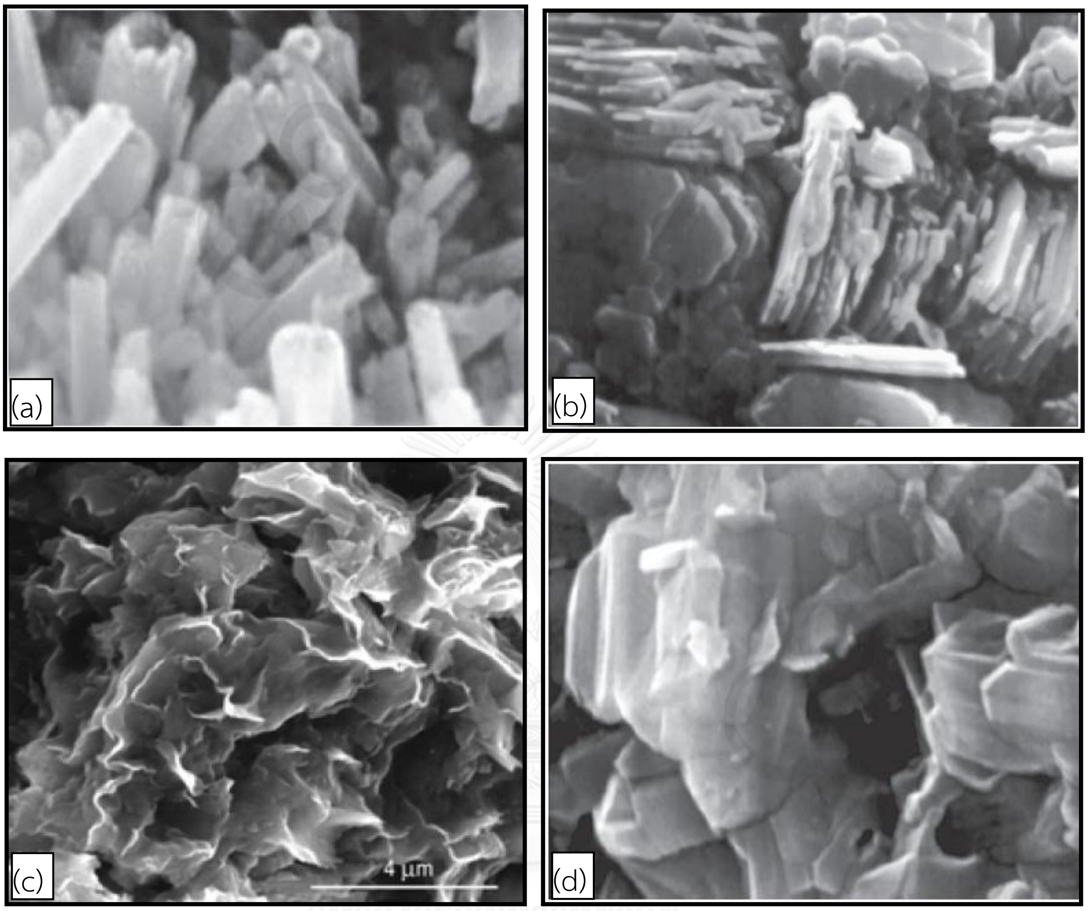


Figure 2-7: Electron photomicrograph of (a) alloysite (picture width is 2 μm), (b) kaolin (picture width is 17 μm), (c) montmorillonite (picture width is 10 μm), and (d) illite (picture width is 7.5 μm) (Mitchell & Soga, 2005).

There is some concentration of different clay minerals' particle sizes within certain ranges smaller than 2 μm, as summarised in Table 2-3.

Table 2-3: Mineral composition of different particle size ranges in soils (Mitchell & Soga, 2005)

Particle size (μm)	Predominating Constituents	Common Constituents	Rare Constituents
0.1	Montmorillonite Beidellite	Mica intermediates	Illite(traces)
0.1-0.2	Mica inter mediates	Kaolinite Montmorillonite	Illite Quartz (traces)
0.2-2.0	Kaolinite	Illite Mica intermediates Micas Halloysite	Quartz Montmorillonite Feldspar
2.0-11.0	Micas Illites Feldspars	Quartz Kaolinite	Halloysite (traces) Montmorillonite (traces)

The type and amount of clay influence a soil's properties, and the Atterberg limits reflect both of these factors. To separate them, the ratio of the plasticity index to the clay size fraction (percentage by weight of particles finer than 2 μm), termed the activity could provide a classification of clay mineral as presented in Table 2-4. Another method to identify expansive soil could be referred to the Atterberg limit. The Atterberg limits are a basic measure of the critical water contents of a fine-grained soil, such as its liquid limit (LL), plastic limit (PL), and shrinkage limit (SL). Among them, the LL and plasticity index (PI= LL-PL) are the most effective parameter for classifying the potential of shrink-swell clay. For example, the montmorillonite clay is known as a highly shrink-swell clay mineral has a very high LL (LL= 100% to 900%) while that the kaolinite mineral is known as a non or low shrink-swell clay mineral has a lower LL (LL= 30% to 110%) as presented in Table 2-4. The LL and PL values of any one clay mineral may vary over a wide range. At different clay minerals group present different plasticity values as listed in Table 2-4. Clays rich in montmorillonite minerals tend to absorb more water and thus exhibit greater swelling than non-expansive clays such as chlorite, illite, and kaolinite. In general, finer soils have a greater capacity to hold

moisture due to their greater particle surface area (Miller, 1997; Mitchell & Soga, 2005). On the other hand, clayey soils rich in montmorillonite retain plasticity at lower moisture contents than non-expansive clays such as chlorite, illite, and kaolinite.
Reported by Miller (Miller, 1997), Skempton suggested three groups of clays according to their activity value as inactive (non to low swelling potential), for activity values less than 0.75; normal (medium to high swelling potential), for activity's value range from 0.75 and 1.25; and active (high to very high swelling potential), for activity's value greater than 1.25. Active clay provides the most potential for expansion. A typical value of the activity's value of various clay minerals is presented in Table 2-4.

Table 2-4: Activity of various clay minerals (Miller, 1997; Mitchell & Soga, 2005)

Mineral	Activity, A (-)	Liquid limit, LL (%)	Plastic limit, PL (%)	Shrinkage limit, SL (%)
Montmorillonite	1.5-7.2	100-900	50-100	8.5-15
Illite	0.5-1	60-120	35-60	15-17
Kaolinite	0.5	30-110	25-40	25-29
Halloysite	0.1-0.5	50-70	47-60	-
Attapulgite	0.5-1.2	160-230	100-120	-
Allophane	0.5-1.2	200-250	130-140	-

A wide variety of soil engineering properties have been correlated to the LL and PL, and these properties are also used to classify a fine-grained soil. The study of soil composition and its relationship to soil properties are approached in two ways. In the first, natural soils are used, the composition and engineering properties are determined, and correlations are made. This method has the advantages that measured properties are those of the soil in nature. Disadvantages, however, are that compositional analyses are difficult and time consuming, and that in soils containing several minerals or other constituents such as organic matter, silica, alumina, and iron oxide the influence of any one constituent may be difficult to isolate. In the second approach, the engineering properties of artificial soils are determined. Soils of known composition are prepared by blending different commercially available clay minerals of relatively high purity with each other and with silts and sands. Although this approach is much easier, it has the disadvantages that the properties of the pure minerals may not be the same as those of the minerals in the natural soil, and important interactions among constituents may be missed. Whether the influences of constituents such as organic matter, oxides and cementation, and other chemical effects can be studied successfully using this approach has not yet been thoroughly investigated (Mitchell & Soga, 2005). Based on the moisture content stage range on the Atterberg limit chart from solid state up to the liquid state, stress-stain behaviour could be quickly evaluated as illustrated in Figure 2-8.

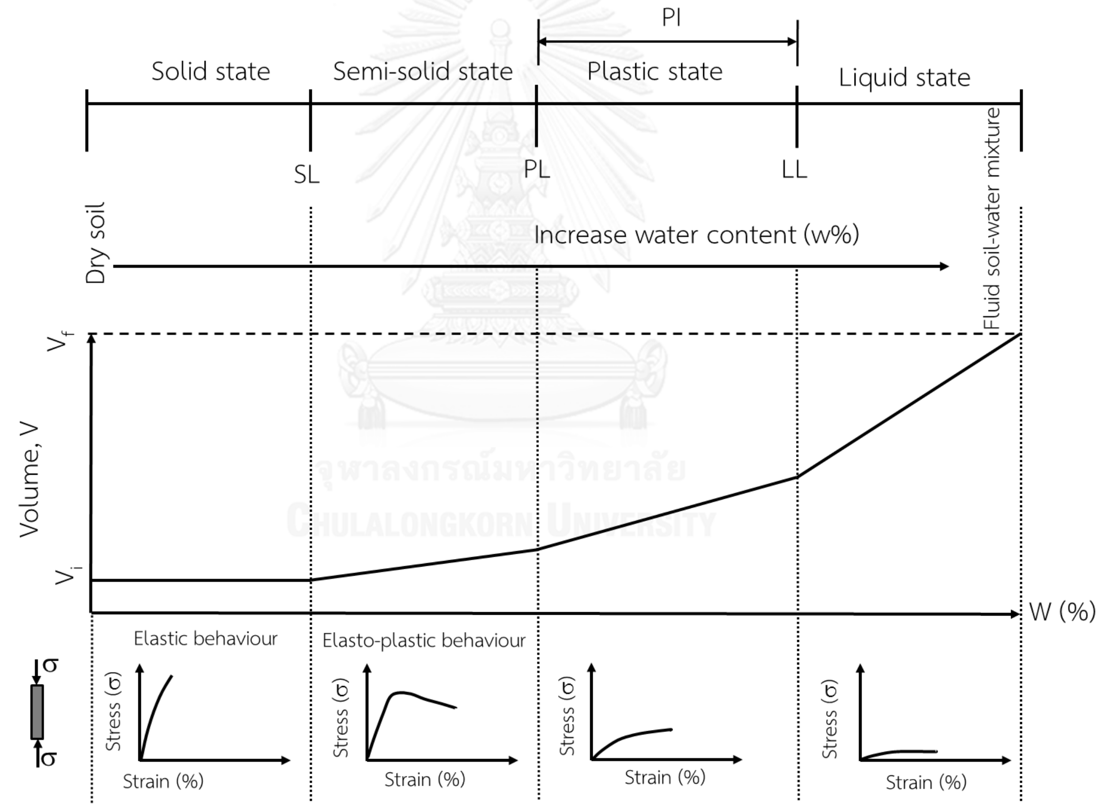


Figure 2-8: Atterberg limit description, volume change and generalised stress-strain response of expansive soils (Das & Sobhan, 2013; Lucian, 2008).

2.5.4 Direct measurement shrink-swell of expansive soils

The direct methods comprise of mineralogical identification of the soil or quantitative evaluation of the shrinkage and swelling characteristics of an undisturbed sample in the laboratory. The methods commonly need a special equipment and are considered not suitable for quick identification the potential of shrinkage and swelling of expansive soils. There are several standard laboratory methods have been developed to directly measure and determine the shrinkage and swelling properties of expansive soils. Those include one-dimensional expansion and shrinkage test (ASTM-Standard-D3877, 1996), expansion index (Nuhfer, 1994), California Bearing Ratio, CBR, (ASTM-Standard-D1883, 1999), potential volume change (PVC), coefficient of linear extensibility (COLE), free swell test (Amer A Al-Rawas, 1999). Recently, a linear shrinkage bar test was proposed by some researchers (Puppala et al., 2013; Puppala et al., 2007) in cooperating with image processing technique suggested that the measurement could provide more accurate results than that from the manual measurement i.e. ASTM and BS methods (ASTM-Standard-D3877, 1996; BS-1377, 1990).

2.5.5 Indirect measurement shrink-swell of expansive soils

Indirect measurement of potential swell of expansive soil can be done by using correlation between soil properties such as index properties, engineering properties and mechanical properties. The potential shrink-swell of expansive soil can be detected by plot of plastic limit again liquid limit according to Casagrande’s plasticity chart (Figure 2-9). For example, a soil sample with liquid limit (LL) 40% and plasticity index (PI) 25% plots in the zone typical for smectites (montmorillonite) implying that it has a high potential for swelling. Soils that plot above the A-line are fat or plastic clays and those, which plot below it, are organic soils, silts and clayey soils. The U-line is approximately indicates the upper bound of the relationship of the plasticity index to the liquid limit for natural soils, thus no soil should plot above U-line. The equations A-line and U-line are presented in Figure 2-9. There is another use for the A-line and U-line. Casagrande suggested that, the shrinkage limit of a soil can be approximately determined if its plasticity index and liquid limit are known (see Holtz and Kovacs, 1981).

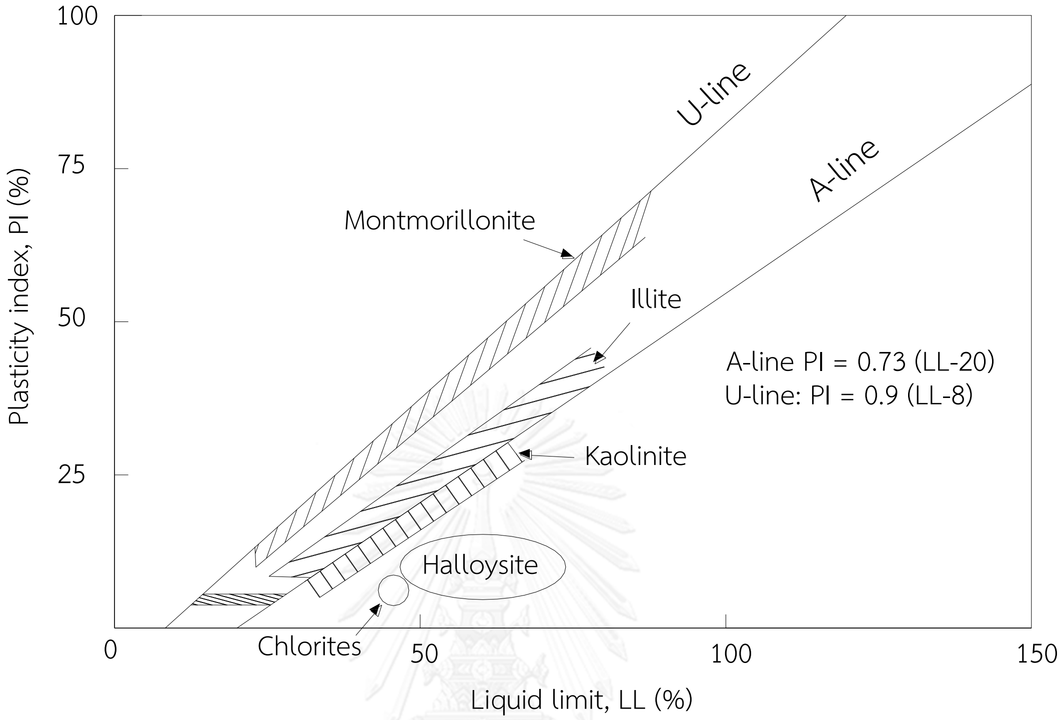


Figure 2-9: Plot of clay minerals on Casagrande's chart (Jaleel, 2011).

The actual amount of volume change of a clay in response to a change in stress depends on the environmental factors and on the cation type and electrolyte type and amount. The compositional factors included type of minerals, amount of each mineral, type of absorbed cation, shape and size distribution of clay particles, and pore water composition. The environmental factors included moisture content, density, confining pressure, temperature, fabric, and availability of water. In general, the shrinkage and swelling properties of clay minerals follow the same pattern as their plasticity properties; i.e., the more plasticity mineral, the more potential swell and shrinkage. The actual amount of swell or shrinkage as a result of wetting or drying depends on factors in additional to mineralogy, such as particle arrangement, initial moisture content, and confining pressure. The potential expansions of the different clay minerals follow closely their relative plasticity indices; i.e., the higher the plasticity index the greater the swelling potential (Mitchell & Soga, 2005).

Because of the problems encountered in the performance of structures founded on high shrink-swell soils, various attempts have been made to develop reliable methods for their identification. The most successful of these are based on the determination of some factors that is related directly to the clay mineral composition, such as shrinkage limit, plasticity index, activity, and percentage finer than 2 μm. The relationship between swell or swell pressure and these parameters that reflect only the type and amount of clay are not possible because of the dependence of the behaviour on initial state (moisture content, density, and structure) and the other environmental factors. An example illustrated by Figure 2-10, which shows four different correlations between swelling potential and plasticity index (Chen, 1975). The two curves showing the Chen correlations were obtained for different natural soils compacted to dry densities between 100 pcf and 110 pcf (15.7 and 17.3 kN/m^3) at moisture content between 15 and 20 percent. The curve showing the Seed correlation was obtained for artificial mixture of sand and clay minerals compacted at optimum moisture content using Standard AASHTO compactive effort allowed to swell under a surcharge of 1 psi (7 kPa). For the curve showing the Holtz & Gibbs correlation was obtain using data from both undisturbed and remolded samples which allowed to swell from an air-dry state to saturation under a surcharge of 1 psi (7 kPa). A correlation between compositional factors that reflect both the type and amount of clay, that is the activity A, defined as ΔPI/ΔC, and the percent clay size C (%< 2 μm), tested on artificial sand-clay mineral mixtures is suggested as the following equation (Seed, Woodward Jr, & Lundgren, 1962).

$$S = 3.6 \times 10^{-5} A^{2.44} C^{3.44} \tag{2-1}$$

Where: S is the percent swell for samples compacted and tested as indicated earlier. A diagram based on this relationship is shown in Figure 2-10. In the case of compacted natural soils the swelling potential might be related to the plasticity index with an accuracy of ±35% according to the following equation

$$S = 2.16 \times 10^{-3} (PI)^{2.44} \tag{2-2}$$

Somewhat different relationships have been found to better classify the swell potential of some soils, and no single relationship is suitable for all conditions. Thus, while the above relationships and plots such as Figure 2-10 illustrate the influences of compositional factors and provide preliminary guidance about the potential magnitude of swelling, reliable quantification of swell and swell pressure in any case should be based on the results of tests on representative undisturbed samples tested under appropriate conditions.

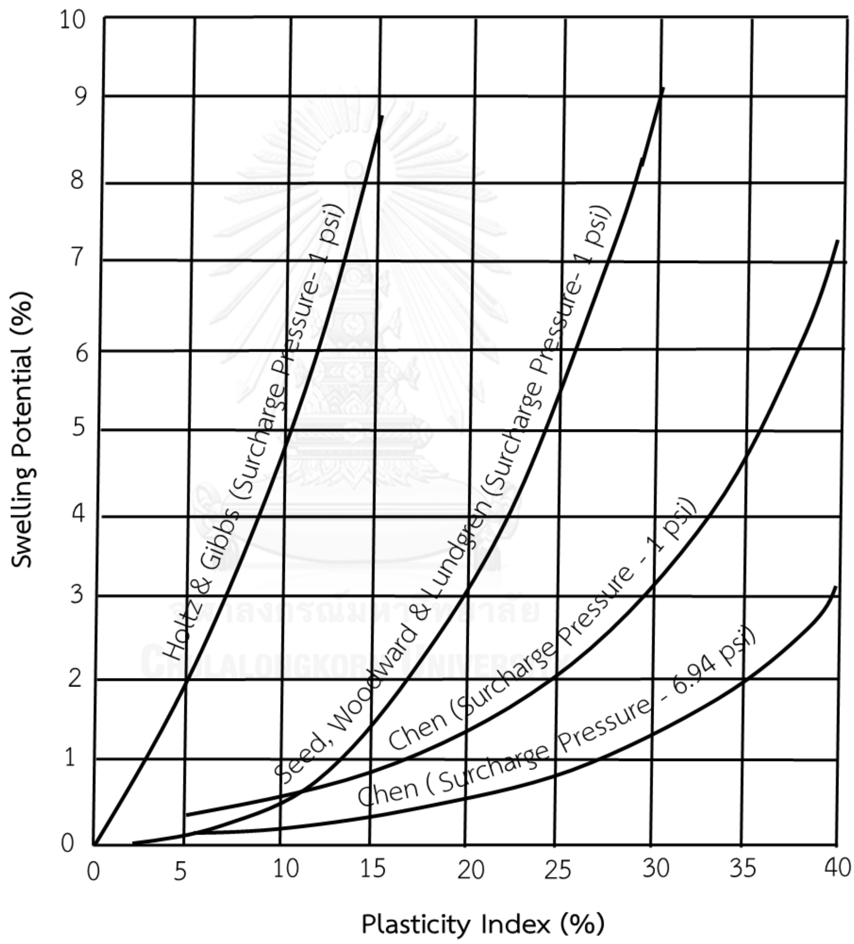


Figure 2-10: Four correlations between swelling potential and plasticity index (Chen, 1975).

2.6 Shrinkage strain of expansive soils

The shrinkage is relatively related to moisture loos from the expansive soil mass, it is influenced by both external and internal factors. The external factors affecting of moisture loss from the expansive soil mass including air temperature, relative humidity and wind velocity, which affect the loss of moisture from the soil surface. Different environmental condition results in different drying out, hence different shrinkage size and shape are occurring. Higher shrinkage is to be expected with rise in temperature, with a decrease in relative humidity, with the increase in air velocity around the soil mass, and with increase in the length of time for which the expansive soil subjected to drying conditions. The size and thickness of the expansive soil mass is affected to the rate of moisture content loos, a larger and thicker soil more slowly dry out than that the small and thin ones. The internal factors affecting the shrinkage of expansive soil mass are those related to its constituents: type and amount of minerals content, clay fraction, moisture content, etc.

A research study undertaken by (Puppala et al., 2007) using image technology to investigate shrinkage characteristics of various compost materials treated expansive clay including municipal solid was (a mixture of wastes include paper, glass, wood, plastics, reusable goods, soils food waste, plant debris, metal, textiles and rock, with the organic material), biosolids compost, and dairy manure compost. The effect of initial moisture content, set at optimum moisture content, wet optimum moisture content and liquid limit, on the shrinkage characteristic of the different mixture materials explain earlier were investigated. The result from the investigation observed that the present of organic matter enhanced shrinkage resistance and shear strength at low compost proportion between 20%-30%, while a higher proportion beyond 30%, the shear strength reached plateau condition. The linear shrinkage strain potential of the expansive soils indicated larger shrinkage strain at the initial moisture content set at their liquid limit while that the optimum moisture content is the lowest.

The image processing technique were used along with the manual measurement method to evaluate the shrinkage characteristic of natural and stabilised expansive clayey soils, focusing on the linear and three-dimensional volumetric shrinkage strain at various initial moisture content (Puppala, Pathivada, Bhadriraju, & Hoyos, 2001). The result showed that measured by digital imaging were provided volumetric shrinkage strains higher that those measured by manual methods. The author suggested that the digital measurement method were provided reliable and consistent results than those from the manual.

2.7 Methods measuring swelling pressure of expansive soils

The volume change in soils are important because of their relation to settlements due to compression, heave due to expansion, and their contributed to deformations. Four different methods namely constant-volume method, pre-swell method, zero-swell method and swell-consolidation method were performed to measuring the swelling pressure upon wetting of bentonite/clay-stone mixture (Q. Wang et al., 2012). The effects of the water chemistry, the hydration procedure, the pre-existing technical voids and the experimental methods on the swelling pressure were investigated. The stress paths of these methods are presented in Figure 2-11. For the “constant-volume” method, stress path OA, the sample was restricted by a cell and allowed to swell in a confining volume during wetting. The top part of the sample was incorporated a total pressure sensor to monitor the vertical swelling pressure.

For the “pre-swell” method is presented by stress path “OBB” in the Figure 2-11, the specimen was first allowed to swell freely in the axial direction to a certain value (Point B, noted as “pre-swell”), then piston was fixed permitting the generation of swelling pressure that was monitored by the load transducer. This method was used to investigate the effect of pre-existing technical voids (simulated by the pre-swell allowed in the tests) on the swelling pressure. For the “zero-swell” and “swell-consolidation” methods, the equipment employed was a conventional oedometer. Firstly, a low initial load of 0.1 MPa was exerted on the specimen prior to water flooding. As soon as the specimen wetted up it attempted to swell. When the swell exceeded 10 μm (equivalent to 0.1%), extra pressure was added in small increment to bring the volume of soil specimen back to its initial value. This procedure was repeated until the specimen ceased to swell (OCC′ in Figure 2-11). The maximum swelling pressure was defined as the stress under which no more swelling strain was observed. The "swell-consolidation" method consists of re-saturating the soil under a low vertical pressure of 0.1 MPa until full swell was achieved. After swell completion, conventional oedometer test was conducted. The pressure required to compress the specimen back to its original void ratio is defined as the swelling pressure. The corresponding stress path ODD′ is shown in Figure 2-11. In this path, point D also represents the maximum swelling strain of the specimen.

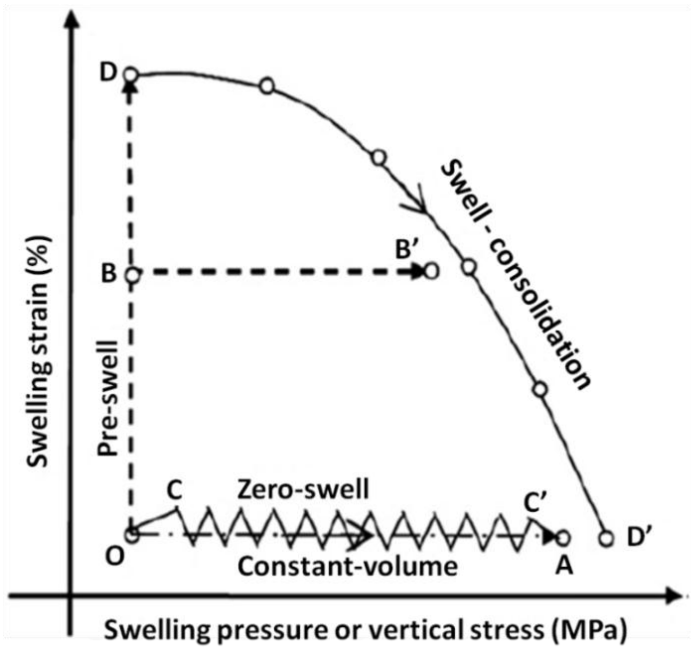


Figure 2-11: Stress path of various experimental methods used to determine the swelling pressure (Q. Wang et al., 2012).

A study by Azam (Azam, 2006) using a constant volume method to investigate the swelling and consolidation behaviour of an undisturbed field sample of AI-Qatif clay. A large-scale circular and square sample was tested in a in a highly instrumented large-scale odometer. The result was compared to the conventional circular sample and observed that the vertical swelling pressure were reduced by 35% and 60% for the large-scale circular and square sample, respectively. The study suggested that, the variation vertical swelling pressure is mainly due to the presence of soil discontinuities affecting by large cross-sectional areas. The decreasing of the observed value by 35%

and 60% for the two large-scale samples due to the fissure, joints, and cracks in the clay which consumed a significant portion of the movement during swelling thereby decreases the observed value. According to Azam (Azam, 2006) the constant volume test give an accurate estimate of vertical swelling pressure and best matched actual field measurements due to the complete confinement of the soil specimen related to the mold rigidity. A swelling pressure of the undisturbed and remolded sample was investigated by Attom (Attom, Abu-Zreig, & Obaidat, 2006) following the zero swell test. The result found that the swelling pressure of the undisturbed specimen were about two times higher than that the remolded sample.

The swelling parameters, percent of swell and swelling pressure, were investigated on expansive soil using oedometer and triaxial loading apparatus (Al-Shamrani & Dhowian, 2003). The data obtained from these two methods also examined using the results from experiments in the field using pressure, suction, and moisture prediction methods. The data obtained from the oedometer is markedly overestimate compared to the three field measurement methods mention earlier, while that using triaxial swell parameters gave a satisfactory agreement between observed and predicted heave values. A reasonable agreement was achieved between the field measurement and oedometer data by applying a correction factor to the heave estimated based on oedometer data.

There are several methods have been presented for the measurement of swelling pressure of expansive soils. The zero swell test and swell-consolidation test are commonly used, however, due to the development technique of measuring the swelling pressure of expansive soil, two relatively new techniques, namely “restrained swell test” and “double oedometer swell test” are introduced (Basma, Al-Homoud, & Husein, 1995). The restrained swell test consist of successively increasing load on the soil specimen allowing it to reach an equilibrium swelling strain at each applying stress level. During applying stress level, the specimen was submerged in water and allowed to fully swell. The process is repeated with various applying stress on the submerged soil specimen. The swelling stress at the equilibrium swelling (no expansion) is defined as the maximum swelling pressure. For the double oedometer test involved two identical specimens. For the first specimen is loaded as a compacted state by incremental vertical stress with equilibrium deformation recorded at each stress level while the second specimen is submerged without applied load and the maximum swell is recorded. The swelling potential defined by the term different between the percent changes in specimen height at each stress level. The measure swelling pressure is the stress at which the percent settlement of the first specimen equals the percent swell of second specimen. Comparing the above two testing methods (restrained swell and double oedometer tests), the restrained swell test was suggested more reasonable results for swelling pressure measurement, on the other hand, is considered closely to the field conditions (Basma et al., 1995).

2.8 Swelling mechanism of compacted expansive soils

The swelling deformation and swelling pressure of compacted expansive soils during wetting were studied by many researchers (Komine & Ogata, 1994; Mohamed et al., 2014; Muntohar, 2006). Figure 2-12(a) show the swelling deformation of compacted bentonite clay under a constant vertical pressure (surcharge) while Figure 2-12(b) show the swelling pressure with swelling deformation restricted of the compacted bentonite clay (Komine & Ogata, 1996). The compacted bentonite clay contain the swelling montmorillonite, non- swelling minerals (e.g. quartz, calcite, etc), and voids (Komine & Ogata, 1996). The volume of void before water intake, the voids is mainly engaged by air and free water. During water intake, the volume of montmorillonite minerals increases. Water molecules are adsorbed at active site on the layers of the crystal lattice (Na-montmorillonite), and the voids in the compacted bentonite clay are filled by this volume increase of swollen montmorillonite. Hydration of the exchangeable cation is largely responsible for this adsorption and for the resulting increase of the interlayers spacing. The phenomena provided the compacted bentonite clay able to swell at a constant vertical pressure, the volume of compacted bentonite clay increases as the volume of montmorillonite minerals increases until the swelling pressure of the montmorillonite minerals equals the vertical pressure (Figure 2-12a). In the case of the total volume of compacted bentonite clay is restricted, the montmorillonite minerals swell and filled the voids in the compacted bentonite clay without increasing the overall volume of the compacted bentonite clay. Whenever the void are filled, the volume of montmorillonite minerals can be measured as swelling pressure of the compacted bentonite clay (Figure 2-12b).

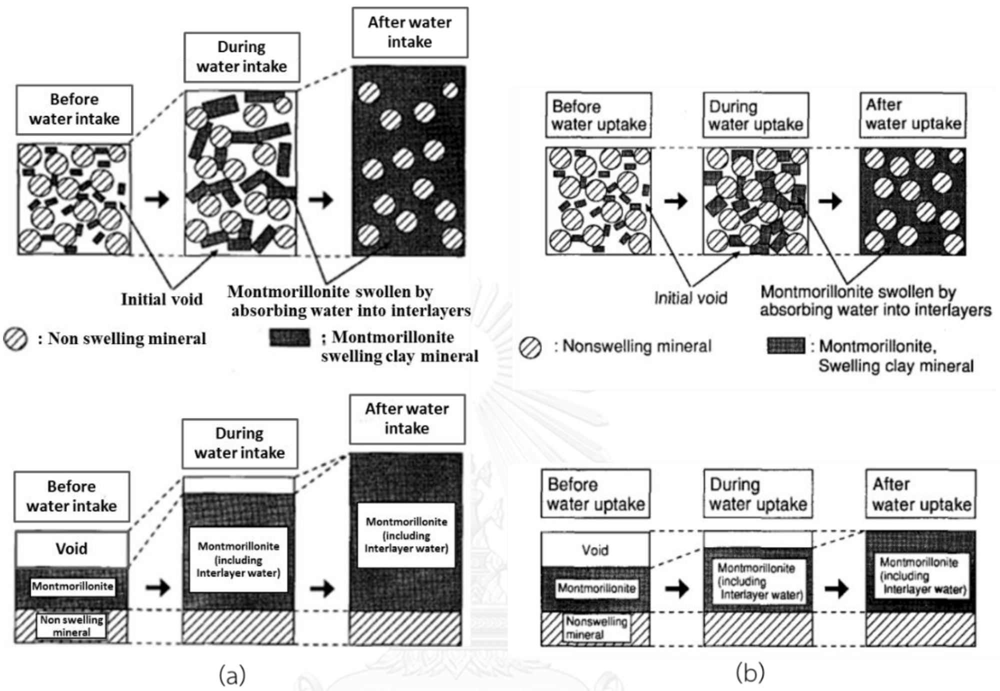


Figure 2-12: Mechanism on the swelling deformation and swelling pressure of compacted bentonite clay; (a) Swelling deformation at a constant vertical pressure; (b) Swelling pressure with swelling deformation restricted (Komine & Ogata, 1996).

Swell and swelling pressure, rate of swell and water absorption of compacted Kaolinite-bentonite mix and san-bentonite mix specimens were investigated by (Hashim & Muntohar, 2006). A conventional oedometer was employed for the swelling deformation. The study observed that the time required to reach a stable stage value varies depending on the percentage of bentonite and the type and amount of non-swelling fraction. A smaller bentonite content, the rate of swelling was slower with sand, but the rate was increased gradually with a decrease in the particle size of non-swelling fraction. Suggested by Dakshanamurthy (Dakshanamurthy, 1978), two stages of swelling are noted. The first stage is the hydration of clay particles, a successive monolayers on the surface of swelling clay which refer to interlayer or intercrystalline swelling. The second stage of swelling is due to double-layer repulsion, a large volume change was occurred in this stage.

A saturated expansive clayey soil exhibits differ properties from those of saturated non-expansive clayey soil (Katti & Katti, 1994). During saturation from dry to saturated condition, under a normal load, non-expansive clays soil settles while that the expansive soil heave upwards. As the load increase, more settles are obtained for the non-expansive soil, while the expansive soil heaves go on reducing and reaches zero at some load and then start settling. In natural condition, at a saturated condition the moisture content of an expansive soil is about their liquid limit and goes on decrease rapidly up to around 1 to 1.5 m depth and then remains constant.

2.9 Methods measuring of lateral swelling pressure

Expansive soil swell not only in the vertical direction, but also laterally (Mohamed et al., 2014). Any pavement structure or foundation especially lightweight structure, located or buried in expansive soil, may be subjected to large magnitudes of lateral swelling pressure as a result of increase in moisture content. Number of experimental testing methods are proposed and available to measuring the swelling pressure of expansive soils during wetting such as constant vertical pressure, swell consolidation, zero swell, constant volume (Attom et al., 2006; Azam, 2006; Q. Wang et al., 2012; Wayne & David, 2004), restrained swell and double oedometer swell tests (Basma et al., 1995) however, those are concern on the vertical swelling pressure measurement while there is less methods are available for the estimation or measuring the lateral swelling pressure of expansive soils. Black cotton soil, known as a highly expansive soil, which located in India has been investigated using a large scale test (Katti & Katti, 1994) to measure the lateral swelling pressure during wetting. Figure 2-13 presented the large scale laboratory technique to measure the lateral swelling pressure the expansive soil.

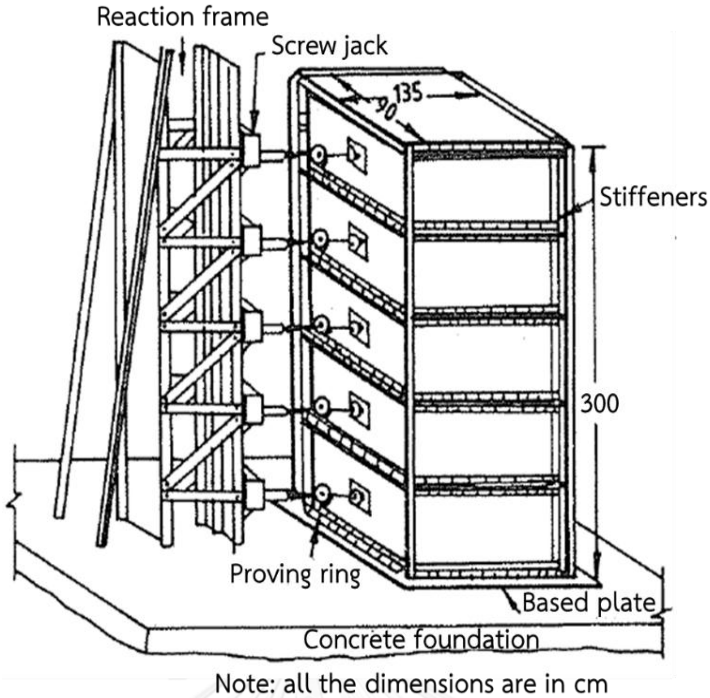


Figure 2-13: Large scale apparatus measuring lateral swelling pressure of expansive soil (Katti & Katti, 1994).

The lateral-pressure devised consist of reaction jack, proving ring, and measuring piston were placed on the wall of the apparatus with 60 cm interval as illustrated in Figure 2-13. The swelling pressure was recorded during saturation of the soil specimen which took about 40 to 60 days to complete 100% saturation and reach a stable condition. The swelling pressure at the end of saturation was recorded as the maximum swelling pressure of the soil specimen. Seed et al. (1960), Komornik and Zeitlin (1965), Bansilal and Palit (1969), and Katti (1978) have developed methods to measure swelling pressure. Table 2-5 presents a brief description of the experimental work conducted on measurement of lateral and vertical swelling pressure by the various researchers.

Table 2-5: Swelling pressure measurement (Katti & Katti, 1994)

Author	Principle	Method
Komornik and Zeitlin, 1965	Measurement of lateral pressure in consolidation ring	Strain gauge fixed on lower portion of consolidation ring, turned down to 0.3 mm thickness. Swelling pressure measured after saturation of sample.
Seed, Mitchel and Chan, 1960	Measurement of swelling pressure in a mould of 10 cm diameter through a proving bar	Pressure transmitted to proving bar recorded though a dial gauge as deflection of proving bar
Bansilal and Palit, 1969	Measurement of lateral swelling pressure and effect of lateral confining pressure on vertical swelling pressure.	Water filled in hollow cylindrical soil specimen generates swelling pressure all along height of specimen measured by proving ring in triaxial set-up incorporates measurement of vertical swelling pressure under different confining pressure applied to water in hollow portion.
Katti, 1978	Triaxial system for lateral pressure, vertical swelling pressure measurement.	Special attachments make it possible to restrict the volume change in lateral direction as well as in vertical direction by externally applied pressure.

Recently, a new technique measuring the lateral swelling pressure of expansive soil has been developed, namely "thin-wall oedometer" (Abbas, Elkady, & Al-Shamrani, 2015). A laboratory testing was conducted on compacted highly expansive soil using a thin wall oedometer measuring vertical and lateral swelling pressure during wetting. The main components of this device are presented in Figure 2-14. Firstly, the thin-wall oedometer was calibration, correction, and verification using strain gauges. The thin-wall ring in Figure 2-14 was instrumented with 4 electrical strain gauges attached on the outer surface of the thin-wall ring. Though applying an appropriate calibration factor, the thin-wall ring provided an indirect method for the measurement of lateral pressure which, together the vertical pressure applied.

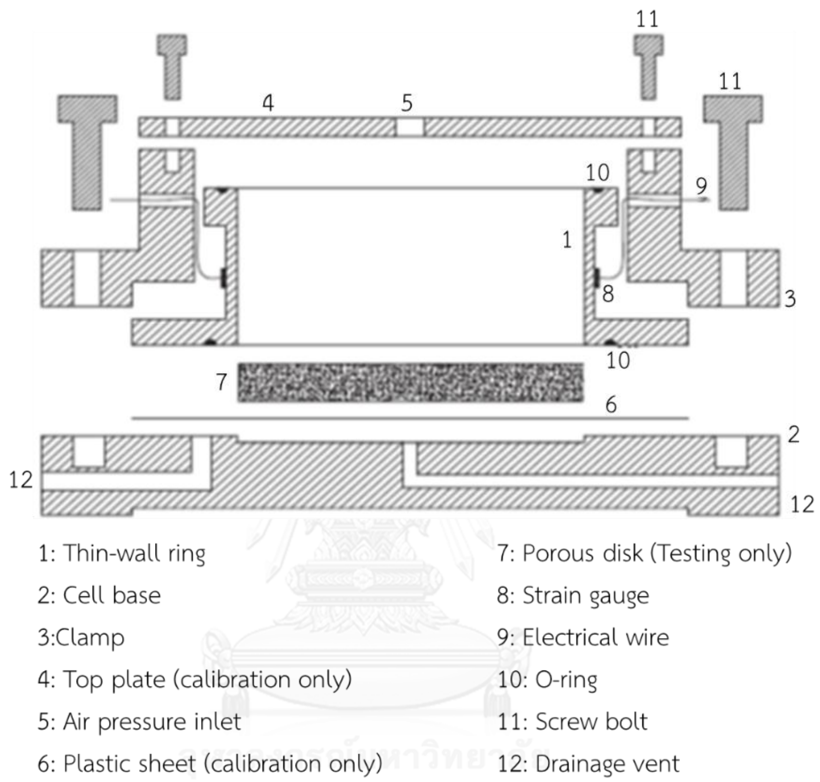


Figure 2-14: Schematic diagram showing the components of the thin-walled oedometer cell (Abbas et al., 2015).

2.10 Lateral coefficient of earth pressure

Measurement or prediction of lateral earth pressure in expansive soil has been a problem to geotechnical engineering for long time ago. Recently, several researchers investigated the lateral coefficient of earth pressure at different soil and condition. The lateral coefficients of earth pressure of different soils under various conditions have been investigated by some researchers; i.e. Kati (Katti & Katti, 1994) on the lateral earth pressure (active, passive and at rest) of the expansive black cotton soil, Tian (Tian, Xu, Zhou, Zhao, & Kun, 2009) on the coefficients of earth pressure at rest in thick and deep swelling clay, Hayashi (Hayashi, Yamazoe, Mitachi, Tanaka, & Nishimoto, 2012) on coefficient of earth pressure at rest (K_0) for normal and overconsolidated peat ground in Hokkaido area, Mohamed (Mohamed et al., 2014) on the effect of lateral swelling pressure of expansive soil on retaining wall. The total lateral pressure, acting to a retaining wall which is constructed within expansive soil, consist of lateral earth pressure, surcharge pressure and lateral swelling pressure as illustrated in Figure 2-15.

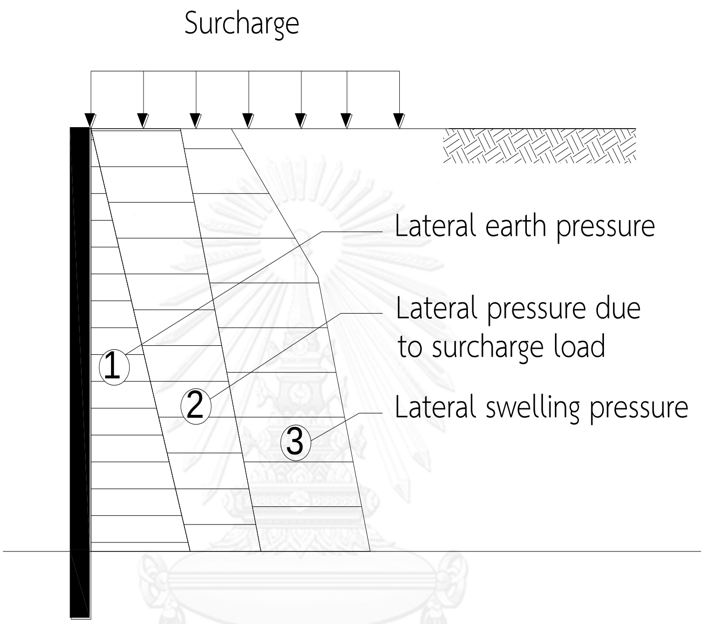


Figure 2-15: Schematic of diagram pressure acting on sheet pile, after Sapaz (Sapaz, 2004).

The magnitude of the lateral swelling pressure is very high might be 2-10 time that of the swelling vertical (Fourie, 1989). The coefficient of lateral earth pressure at rest (K_0) for a saturated soil is less than 1 for non-expansive soil and varies from 15 to 20 for expansive soil in the depth between 1 to 1.5 m and increase in depth the K_0 also greater that 1.2 to 3 (Katti & Katti, 1994). A study on K_0 consolidation properties has been investigated by number of researchers (Tian et al., 2009). The author name and a brief description of the tests are summarised in Table 2-6.

Table 2-6: Investigation on K_0 consolidation properties (Tian et al., 2009)

Author	Apparatus and test description
Cheng, 1999	A rigid pressure chamber with a flexible lateral confining pressure medium. The experiment conducted using incompressible degassed water or oil as the confining pressure medium.
Cheng, 2000	A rigid pressure chamber with a servo-control system and a radial deformation sensor that controls the sample with no lateral deformation.
Cheng, 2001	A rigid pressure chamber with rigid lateral confinement.
He et al., 2003	Adjusting the confining pressure according to sample axial and volumetric deformation. The criterion for the sample is no lateral deformation so that the displacement equals the product of the initial sample area times the axial deformation.
Wang et al., 2003	Double-cylindrical pressure chamber. The deformation of the specimen is equivalent to a water-level fluctuation of the inner core. Based on the variation in the water level, the soil sample is prevented from deforming laterally by controlling the chamber and axial pressure.

According to Tian (Tian et al., 2009), thick and deep soils, exist in an initial, dense high-pressure state with low moisture content close to the plastic limit. A study the coefficient of earth pressure at rest in thick and deep swelling clay by Tian (Tian et al., 2009) presented results that, during loading process, K_0 increase and becoming greater at high pressure applied. However, in the unloading process, K_0 increased only at the initial stage and then decrease under unloading. Moreover, the incremental magnitude defined by $K_0= d\sigma_3/d\sigma_1$ gives greater values than the total magnitude, $K_0= \sigma_3/\sigma_1$. The above investigators also concluded that the earth pressure at rest in deep and thick soils could be estimated by a power function of axial confining stress.

2.11 Solution to geotechnical problems related to expansive soils

There are several methods can be used to minimise shrink-swell and settlement of an expansive soil so that difficulties of construction could be overcome. Those are compaction, pre-wetting, constant moisture content, and chemical stabilisation, etc. The application of these methods will keep intact over a long period of time the engineering structures such as buildings, highways, pavement, dam embankments, and shallow foundations. The descriptions of these methods are presented in the following section.

2.11.1 Compaction

The best solution (but uneconomical due to high cost) is to excavate and remove the expansive soil before starting the construction and replace it with compacted, non-expansive material. In general, a compacted clay will expand less under a given load than the same clay in an undisturbed state. For example, a sample of dried clay from central Oklahoma which in its natural state expanded under load up to 468 kPa required only about 104 kPa to prevent expansion when compacted on the dry side with a standard hammer (Stavridakis, 2006). The initial void ratio of expansive soils has profound influence on corresponding swelling pressure and volume change associated with the adsorption and desorption of water. Previous works have documented that cylindrical specimens compacted at a lower density undergo less total axial strain when following a wetting path. In some cases where it has been positively established that the expansive layer is only of limited thickness, only the upper strata of the expansive soil should be removed and replaced by inert material compacted in 23 cm layers. Excavation and removal deeper than 1.5 m, however, have proved uneconomical.

2.11.2 Pre-wetting

The main purpose of pre-wetting is to allow dried foundation soils to swell prior to placement of a structure. There are several wetting methods. One of the most common wetting methods is ponding or submerging of an area in water. However, wetting the foundation subsoil by ponding may require many months or even years to increase the moisture content to the required depths unless the clay contains a fissure system to pass water percolation through the soil. Furthermore, the completed structure may continue to rise for some time after pre-wetting. An easier way to pre-wetting the soil foundation can be performed by installing of a grid of vertical wells (drainage system) prior to flooding. This can decrease the time necessary to adjust the soil moisture content up to the point where maximum heave will occur within a few months of flooding.

2.11.3 Constant moisture content and conceptual of drainage system

Water is the main contributor to the wear and damage of roads, slabs or any other pavement structure. The water can be in the form of ground water, surface water (streams and rivers) or rain (Figure 2-16) and it can damage the road in several ways by washing away the soil (erosion and scouring), making the road body less resistant to traffic (i.e. weakening the load bearing capacity), depositing soils (silting) which may obstruct the passage of water, or washing away entire sections of the road or its structures, and swelling or shrinkage soil foundation when it get wet or dry condition that cause of cracking or permanence deformation the pavement structure built on expansive soil. Damage and wear to the road can be reduced if the flow of water is controlled. In the case of soil foundation is expansive soil type, most moisture control methods are applied around the perimeter of structures in an attempt to minimize edge wetting or drying of soil foundations. One of the more common methods of maintaining constant moisture is through the installation of impermeable layers or barriers, sufficient drainage systems, and control of vegetation coverage. Various drainage measures are applied to effectively deal with the water arriving at the road. Surface water arrives directly on the road as rain, as runoff from the surrounding areas, or in streams and rivers. In flat terrain, the entire area around the road may be inundated with water during the rainy season. In addition, water also travels underground, which can have an impact on the quality of the road due to the groundwater table fluctuation. If the soil foundation is expansive soil type, due to the groundwater fluctuation, the soil will swell or shrinkage as the moisture content increase or decrease, respectively. An efficient drainage system is therefore essential to allow water to flow off and away from the road as quickly as possible (Orr, 2003). This is achieved by a system consisting of the following components:

- Road surface drainage which enables the water to flow off the road surface,
- Side drains and mitre drains which collect and lead the water away from the road,
- Road embankments in flood prone terrain, lifting the road surface well above the highest flood levels,
- Catch-water drains which catch surface water before it reaches the road,
- Scour checks, preventing erosion in the ditches by slowing down the flow of the water,
- Culverts which lead the water from the side drains under the road to the other (lower) side,
- Bridges and drifts which allows the road to cross rivers and streams in a controlled manner throughout the seasons.

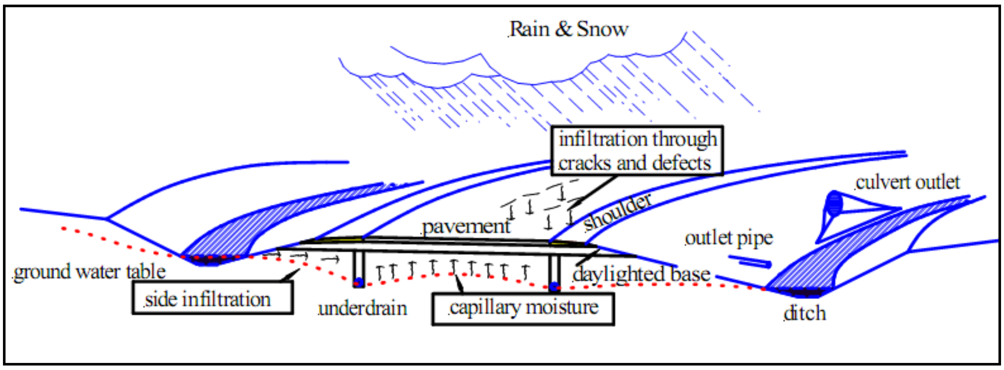


Figure 2-16: Conceptual road drainage system (Orr, 2003).

2.11.4 Chemical stabilisation

Extensive studies have been carried out on the stabilisation of expansive soils using various chemical additives such as lime, cement and fly ash (Radhakrishnan et al., 2014) including organic compound and compost materials (Puppala et al., 2007), electrosmosis, and electro-kinetic (Thuy, Putra, Budianta, & Hazarika, 2013). Lime stabilisation develops because of base exchange and cementation processes between clay particle and lime. The primary effect of small lime additions, 2% to 8%, is to decrease significantly the liquid limit, plasticity index, maximum dry density, and swelling pressure, and to increase the optimum moisture content, strength, and durability of expansive type clays (Stavridakis, 2006).

Recently, the modification of soil to improve their geotechnical properties is well reorganised. Among various stabilising agents the most prominent is Portland cement. Cement is one of the most effective materials in ground stabilisation. Cement stabilisation develops from the cementitious bonds between the calcium silicate (three-calcium silicate) and aluminate hydration products of cement and the soil particles. The action of cement reduces both the swelling types: either the interlayer or the intermicellar swelling (Stavridakis, 2006). The basic strategy of cement stabilisation is to reduce the liquid limit, plasticity index, permeability, deformation, and potential volume change and to increase the shrinkage limit, strength, and durability. Organic compounds (chemical substances) stabilise expansive soils by water proofing, by retarding water adsorption, or by hardening (increase of strength and durability) the soil with resins. Particularly, the acrylic resin prevents, by making the clay-cement system more impermeable, from the adsorption of deleterious substances such as sulphates or probably organic compounds from ground, with small chains. On the other hand, the estimated costs of this method can be as much as twice that of cement or lime in similar applications' problems. Recent years, new methods using cement stabilization has been developed in order to optimize and effective use of recycled geomaterials with appropriate considerations of environmental impacts (Kasama, Zen, & Iwataki, 2006). Those methods including grudged soils mixed with cement, sandy soil mixed with cement by pre-mixing method in order to mitigate against liquefaction in Japan.

CHAPTER 3: MATERIALS AND METHODS

3.1 Introduction

This chapter starts with a description of works related to material selection and characterisation, followed by the description of materials and testing methods used. The first consideration in the material selection was that the soil specimen should be strongly expansive. Another important consideration was that the test duration should fall within the time limits of the research period. Not only is the behaviour of expansive soils more complex than that of non-expansive soils, but the testing of the expansive soils in the laboratory is a very slow process. This is mainly because of the fact that the expansive soil has lower permeability than soils with less or non-expansive soil (i.e., sand, silt, or non-expansive clay). Hence, it took a very long time to reach a stable and saturated condition (S_r= 100%). The two basic materials selected in this study were a Na-montmorillonite bentonite, used as a highly expansive soil, and a natural Bangkok clay as a representative of non-expansive soil. The bentonite and Bangkok clay were mixed at various levels to simulate low, medium to high, and very highly expansive soils. In the soil mixtures, the Bangkok clay content (BKK) was varied at 0, 20, 40, 60, 80, and 100% by dried weight. And then, two mixtures (60 and 80% BKK), representing expansive soils at different liquidities, were selected to be treated by the addition of 5% and 10% Portland cement by dried weight. The pure Bangkok clay was also investigated for the purposes of comparison. The material properties and testing methods are described in the following sections.

3.2 Materials

3.2.1 Bentonite

Bentonite is a clay mineral whose major mineralogical component is formed by the montmorillonite (smectite) group (Pusch & Yong, 2006). As a result, bentonite is a clay with highly expansive characteristics. Bentonite is used in various engineering applications, such as a barrier layer for dam construction, soil contamination, mine waste, disposal of radioactive wastes (Komine, 2010), nuclear waste disposal (Komine, 1999), buffer/backfill material (Cui, Zhang, & Zhang, 2012; Ito, 2006). It is also commonly used as a material in conventional water-based drilling fluids (Abbasi, Javadi, & Bahramloo, 2012) due to its high plasticity and low permeability. Sodium montmorillonite bentonite (Na-montmorillonite bentonite) has a higher capacity for swelling than calcium montmorillonite bentonite (Ca-montmorillonite bentonite) (Pusch & Yong, 2006). Hence, in this research, a commercial Na-montmorillonite bentonite, which is locally produced in Thailand, was selected as a highly expansive material. According to Mitchell & Soga, Na-montmorillonite bentonite is a high swelling clay composed mainly of Na-montmorillonite, which is the result of chemical alteration of igneous materials, usually tuff or volcanic ash (Mitchell & Soga, 2005; Poulose, Ajitha, & Sheela Evangeline, 2013; Přikryl, 2004). Montmorillonite has low permeability and marked expansibility due to stacked lamellae, each of which consists of two sheets of SiO_4 tetrahedrons sandwiching an octahedral layer (Figure 2-2) of hydroxyls and Fe, Mg or Li ions (Pusch & Yong, 2006). Observed by X-ray diffraction (XRD) analysis, the chemical compounds and chemical elements of Na-montmorillonite bentonite used in this study were identified as summarised in Table 3-1 and 3-2, respectively.

Table 3-1: Chemical compounds of Na-montmorillonite bentonite and Bangkok clay

Chemical compounds	Bentonite (%)	Bangkok Clay (%)	Portland Cement (%), (Horpibulsuk et al., 2011)
Quartz (SiO_2)	50	63	21
Aluminum oxide (Al_2O_3)	19	21	5
Iron oxide (FeO)	15	8	4
Calcium oxide (CaO)	5	-	65
Magnesium oxide (MgO)	3	2	1
Potassium oxide (K_2O)	-	3	1
Sodium oxide (Na_2O)	3	1	-
Other minerals	5	3	4

3.2.2 Bangkok Clay

Bangkok clay is a soft marine silty clay lying beneath the low flat plains of the central area of Thailand (Abuel-Naga, Bergado, & Rujivipat, 2005; Horpibulsuk, Yangsukkaseam, Chinkulkijniwat, & Du, 2011; Seah & Lai, 2003; Surarak et al., 2012), characterized by a high water content close to its liquid limit, low shear strength, and large potential for settlement (Sunitsakul, Sawatparnich, & Apimeteetamrong, 2010; Surarak et al., 2012). Due to its low swelling characteristics (Consoli, Prietto, da Silva Lopes, & Winter, 2014), Bangkok clay was used to represent non-swelling clay in this study. The chemical compounds and chemical elements of Bangkok clay are presented in Table 3-1 and 3-2, respectively.

3.2.3 Portland Cement

Expansive soils exhibit high plasticity, high compressibility, and high shrinkage and swelling potential due to certain minerals and environmental conditions, which results in reduced strength, low coefficient of internal friction, and high swelling pressure during wetting. Consequently, they are low quality materials for construction and present difficulties. Recently, due to the increasing population and the necessity of larger areas for construction, modifying ground at any particular site which presents expansive soils to improve their geotechnical properties is very important before any construction can begin. Among various stabilising agents, the most prominent is Portland cement (Consoli et al., 2014; Khemissa & Mahamedi, 2014). The cement is an agent which helps to increase shear strength, frictional angle, and reduce the shrinkage and swelling potential of expansive soils. In this study, a commercial Portland cement (PC) was selected as the stabilizing agent of expansive soils (Bentonite-Bangkok clay mixtures), as was described earlier. Table 3-1 and 3-2 lists the chemical compounds and chemical elements of the cement obtained from X-ray diffraction analysis (XRD), respectively.

Table 3-2: Chemical element of bentonite, Bangkok clay, and cement

Chemical element	Bentonite (%)	Bangkok Clay (%)	Cement (%)
O	35.9	40.0	36.3
C	31.5	29.3	20.4
Na	2.6	-	3.0
Mg	1.2	-	2.6
Al	5.3	6.4	2.9
Si	11.3	16.7	4.4
Cl	0.3	-	0.8
K	-	0.3	0.7
Ca	0.7	0.2	22.4
Ba	1.4	-	1.9
Fe	6.3	3.2	1.7
Cu	3.6	3.9	3.2

3.2.4 Scanning electron microscope of the materials study

Figure 3-1 shows the scanning electron microscope (SEM) photographs of the bentonite, Bangkok clay and cement with 10 μm and 1 μm scale bars. Based on these images, clear shapes and particle arrangements of the bentonite, Bangkok clay and cement can be identified. It was observed that the particles of bentonite are stacked together to form a thicker flake structure, as shown in Figure 3-1(a) while those of the Bangkok clay are separated from each other, as can be seen in Figure 3-1(b). Figure 3-1(c) shows flocculated particles, with big floccules of the cement particle. SEM photos of the bentonite, Bangkok clay and Portland cement also show that the particles of Bangkok clay are much smaller than those of bentonite and Portland cement, with the Portland cement particles being the biggest. This may be due to the predominance of calcium oxide (CaO) and quartz (SiO_2) in the cement (Table 3-1). The bentonite and Bangkok clay particles are flat, whereas the Portland cement particles are irregular in shape.

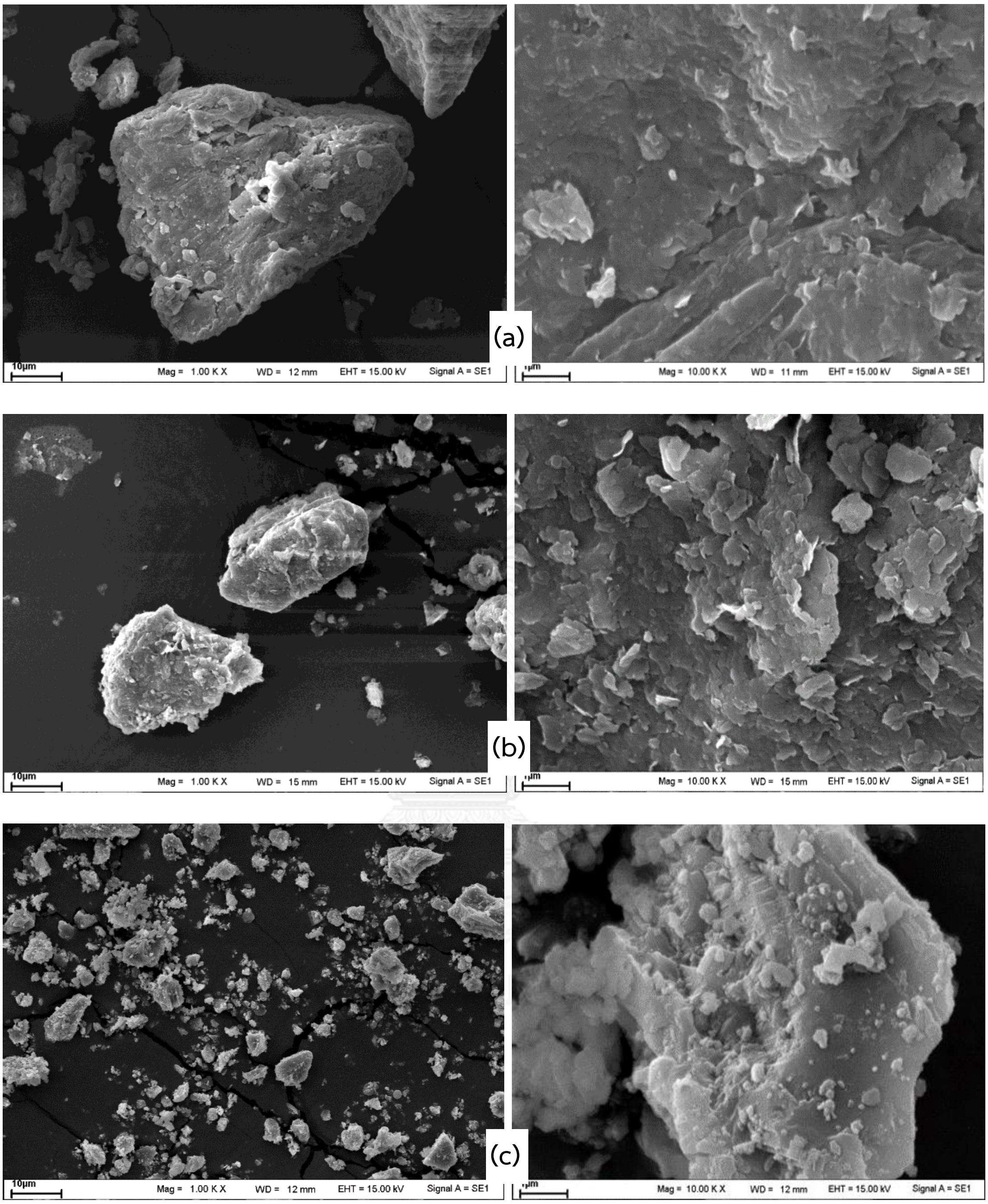


Figure 3-1: SEM photographs (a) Na-montmorillonite Bentonite; (b) Bangkok clay; (c) Portland cement, with 10 μm and 1 μm scale bars.

3.2.5 Basic geotechnical properties of the materials study

3.2.5.1 Grain size distribution

Grain size distribution curves of the bentonite, Bangkok clay, and cement are shown in Figure 3-2. These curves were obtained from hydrometer analysis following the ASTM (ASTM-Standard-D422, 2002). The bentonite had 57% clay fraction (i.e. smaller than 2 μm) while the Bangkok clay's was 76%, and Portland cement's was 12%, respectively. The highest percentage of clay particles smaller than 0.001mm or 1 μm, colloidal size (Das & Sobhan, 2013), was found in the Bangkok clay (68%) followed by bentonite (54%), with the lowest proportion being in Portland cement (5%). The uniformity coefficient of the Portland cement is C_u = 4.7 while the coefficient of gradation is C_c = 0.94, where $C_u = \frac{D_{60}}{D_{10}}$ and $C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}}$, respectively. The aggregation of the cement particles observed in Figure 3-1(c) explains why the predominately Illitic Bangkok clay and montmorillonite bentonite has apparently finer particle distribution than the Portland cement.

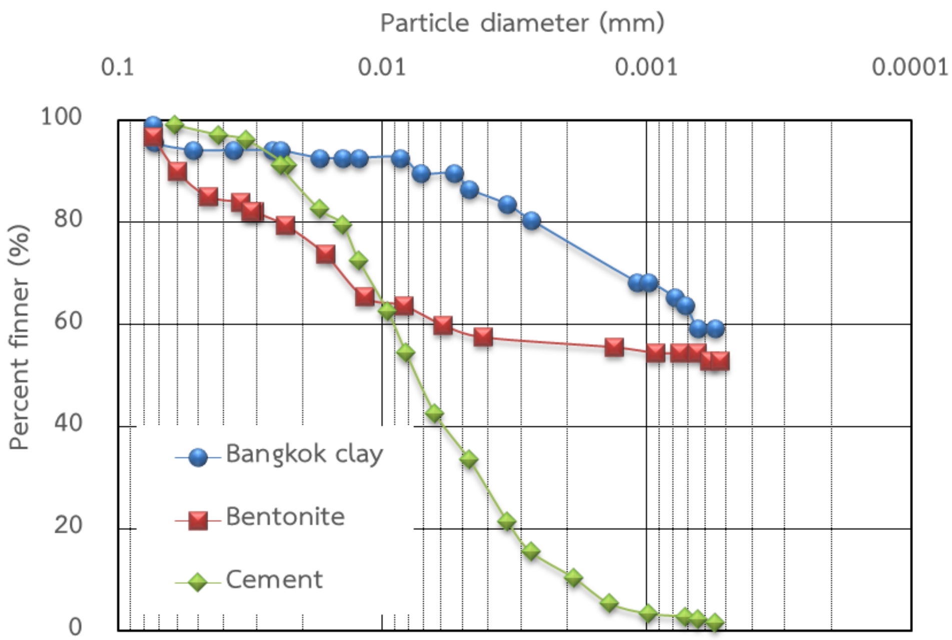


Figure 3-2: Grain size distribution curves of bentonite, Bangkok clay, and cement.

3.2.5.2 Index properties of the materials study

The index properties of the Na-montmorillonite bentonite/Bangkok clay mixtures and the cement are summarised in Table 3-3, following the ASTM standard.

Table 3-3: Index properties of bentonite/Bangkok clay mixtures and Portland cement

Index Properties	Bangkok clay content, BKK (%) (Bentonite and Bangkok clay mixtures)						Portland cement
	0	20	40	60	80	100	
Liquid limit, LL, (%)	509	368	281	191	102	72	-
Plastic limit, PL, (%)	54	50	42	39	37	31	-
Plasticity index, PI	455	318	239	152	65	41	-
Shrinkage limit, SL (%)	23	22	12	12	16	14	-
Specific gravity, Gs	2.87	2.82	2.78	2.76	2.69	2.64	3.15
Activity, (A)	8.00	5.07	3.70	2.23	0.92	0.55	-
Optimum moisture content, OMC (%)	15.5	16.6	20.5	23	26	27	-
Maximum dry density, $\rho_{d,\max}$ (t/m^3)	17.0	16.7	15.9	15.2	14.4	14.2	-

3.3 Reconstituted sample

Frequently the researchers are faced with sample variability and soil disturbance in the natural soil obtained from the field. To achieve better control in sampling variability, laboratories frequently employ the reconstitution of soil samples (Burton, Pineda, Sheng, & Airey, 2015; Tanaka & Kamei, 2011; Ukritchon, Seah, Budsayaplakorn, & Lukkunaprasit, 2006). In this study, the Na-montmorillonite bentonite-Bangkok clay mixtures were prepared either by reconstitution or compacted at its optimum moisture content and maximum dry density for various tests as summarised in Table 3-4 and 3-5. Firstly, the Bangkok clay was air-dried under the sun for about one week, and the crushed, powdered and sieved though a No. 40 sieve (425 μm) similar to the study conducted by Tanaka & Kamei, who researched reconstituted Ariake clay (Tanaka &

Kamei, 2011). The Na-montmorillonite bentonite and Bangkok clay were mixed in proportions of 100:0, 80:20, 60:40, 40:60, 20:80, 0:100 by dried weight, respectively. The testing methods involved in the case of cement-untreated soils were Atterberg limit, unconfined compression, consolidation, compaction, California Bearing Ratio (CBR), areal shrinkage strain (ASS), swelling under a constant pressure, and vertical free swelling strain with the parameters measured as summarised in Table 3-4.

A large consolidometer (Figure 3-3) was used to prepare reconstituted samples with dimensions of 160mm in diameter and 300mm in height. The main components of the apparatus consist of a base plate, chamber, top plate, top cap and loading piston. The stainless steel top plate, the base plate and the bronze chamber are held together by four tie rods. There are two porous stones attached with filter papers placed at the top and bottom of the chamber to create double drainage condition during loading. The filter papers are used in the purpose of preventing the clogging of porous stones by the clay slurry. The load (surcharge) placed on the loading piston was transferred to the clay slurry thought the top cap. An o-ring was also placed on the outer rim of the top cap for preventing flow of the slurry between the gaps.

Bangkok clay was air-dried, powdered, and then sieved through a No. 40 sieve (425 μm) similar to the study conducted by Tanaka & Kamei, who researched reconstituted Ariake clay (Tanaka & Kamei, 2011). The Bangkok clay powder was mixed with bentonite powder by a mixer, adding an appropriate amount of water (approximately to reach the liquidity index of 1.5) to prevent segregation or bleeding. After being mixed to be completely uniform, the slurry was placed into the large consolidometer chamber and an initial vertical stress of 5 kPa was applied. The stress was increased to a maximum vertical stress of 30 kPa in increments of 5 kPa.

For each load increment, the load was maintained until the full dissipation of pore water pressure within the sample had been achieved. The water ponding in the cell chamber (on the top cap) was drained out from the chamber after each step of loading was achieved. About four weeks were required to complete all the loading steps. After completing the loading steps, the load and the piston are removed, and the clay cake was extruded from the chamber with a hydraulic jack. The clay cake was then trimmed into smaller pieces, wrapped with plastic, and sealed with paraffin. The specimens were stored in a humid environment before subsequent tests were performed.

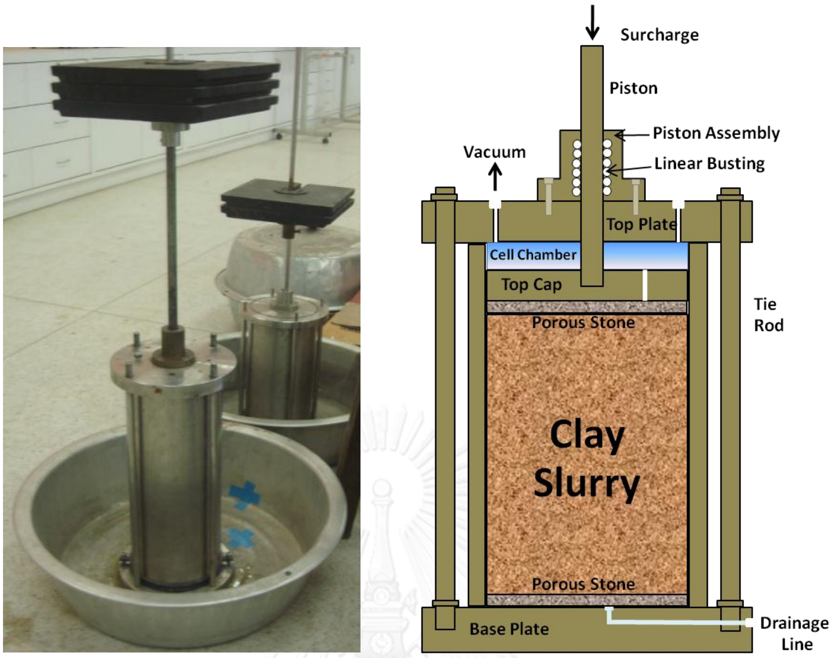


Figure 3-3: Large consolidometer for reconstitute soils preparation.

3.4 Testing programs

As the aim of this research works is to show and characterise the influence of non-expansive soil and cement contents on physical, mechanical and engineering properties of expansive soil, the experimental portion of the work involved the measurement of either reconstituted or compacted expansive clay volume change, strength, stress-strain and stiffness, swelling and shrinkage strain, swelling pressure (both vertical and lateral directions) under a confining boundary condition, including the index properties of the expansive soils, both with and without cement treatment.

3.4.1 Cement untreated soils

Firstly, the Bangkok clay was air-dried under the sun for about one week, and then crushed, powdered and sieved through a No. 40 sieve (425 μm) similar to the study conducted by Tanaka & Kamei, who researched reconstituted Ariake clay (Tanaka & Kamei, 2011). The Na-montmorillonite bentonite and Bangkok clay were mixed in proportions of 100:0, 80:20, 60:40, 40:60, 20:80 and 0:100 by dried weight, respectively. The testing methods involved in the case of cement-untreated soils were Atterberg limit, unconfined compression, consolidation, compaction, California Bearing Ration (CBR), areal shrinkage strain (ASS), swelling under a constant pressure, and vertical free swelling strain with the parameters measured as summarised in Table 3-4.

Table 3-4: Testing methods for specimens of Na-montmorillonite bentonite/Bangkok clay mixtures

<table>
<tr><th rowspan="2">No.</th><th rowspan="2">Testing methods</th><th rowspan="2">Parameters investigated</th><th>Bangkok clay content (%)</th></tr>
<tr><th>0, 20, 40, 60, 80, 100</th></tr>
<tr><td>1</td><td>Atterberg limit</td><td>LL, PL, SL</td><td rowspan="9">✓</td></tr>
<tr><td>2</td><td>Unconfined compression (reconstituted sample)</td><td>q_u, E_{50}</td></tr>
<tr><td>3</td><td>Unconfined compression (compacted sample)</td><td>q_u, E_{50}</td></tr>
<tr><td>4</td><td>Consolidation</td><td>c_c, c_s, c_v</td></tr>
<tr><td>5</td><td>Compaction</td><td>OMC, MDD</td></tr>
<tr><td>6</td><td>California bearing ratio, CBR</td><td>CBR</td></tr>
<tr><td>7</td><td>Areal shrinkage strain</td><td>ASS, V_{ss}</td></tr>
<tr><td>8</td><td>Swelling under a constant pressure</td><td>ε_v</td></tr>
<tr><td>9</td><td>Vertical free swelling strain</td><td>ε_{vf}</td></tr>
</table>

Note: LL= Liquid limit; PL= Plastic limit; SL= Shrinkage limit; q_u= Unconfined compressive strength; E_{50}= Young's modulus at half the failure stress; c_c= Compression index; c_s= Swelling index; c_v= Coefficient of consolidation; OMC= Optimum moisture content; MDD= Maximum dry density; CBR= California bearing ratio; ASS= Areal shrinkage strain; V_{ss}= Volumetric shrinkage strain; ε_v and ε_{vf}= Vertical strain.

3.4.2 Cement treated soils

Two proportions of bentonite and Bangkok clay mixtures, 40:60 and 20:80, were selected to represent expansive soils, as described earlier. The cement treatment was performed on the clay mixtures, including the pure Bangkok clay, by 5% and 10% additional dried weight. The pure Bangkok clay was also tested for the purposes of comparison. The testing methods, including Atterberg limit, unconfined compression, areal shrinkage strain, vertical free swelling strain, and confined swelling pressure tests with the parameters measured as summarised in Table 3-5. The details of the equipment used and laboratory testing procedures for the untreated and cement-treated soils are presented in the following section (section 3.5).

Table 3-5: Testing methods for specimens of Na-montmorillonite/Bangkok clay mixtures with an anditional 5% and 10% cement

No.	Testing methods	Parameters investigated	Bangkok clay content (%)	Cement added (%)	
-	-	-	60, 80, 100	5	10
1	Atterberg limit	LL, PL, SL		✓	✓
2	Unconfined compression	q_u, E_{50}		✓	✓
3	Areal shrinkage strain	ASS, V_{ss}	✓	✓	✓
4	Vertical free swelling strain	ε_{vf}		✓	✓
5	Confined swelling pressure	σ'_v, σ'_h, K_s, c_s, κ, K/p'		✓	-

Note: σ'_h = Horizontal effective stress; σ'_v = Vertical effective stress; κ = mean slope of e-ln p' swelling line; K= Bulk modulus; p'= mean effective stress.

3.5 Testing methods

3.5.1 Compaction and CBR tests

Compaction and California Bearing Ratio (CBR) tests were performed on the bentonite-Bangkok clay mixture as specified in Table 3-4. The purpose was to identify the compaction and CBR characteristics and their variation trend according to differences in soil composition and resulting plasticity. In this investigation the compaction tests followed the modified routine (ASTM-Standard-D1557, 2000). The CBR tests were performed on samples compacted at the optimum moisture content (OMC) and the maximum dry density (MDD), following the ASTM (ASTM-Standard-D1883, 1999). The compacted samples, still in the molds, were soaked for 65 days in a tank filled with water to simulate wetting and possible weakened conditions in the field. The CBR values were measured both for soaked and unsoaked specimens. The expansions of the samples were measured in the vertical direction during soaking by a dial gauge installed at the top of the soaking specimen to quantify the vertical swelling strain during soaking under a surcharge of 2.5 kPa. The surcharge was applied to the top surface of the specimen to represent the weight pavement above the subgrade, following the ASTM (ASTM-Standard-D1883, 1999).

3.5.2 Oedometer test

In order to evaluate the compressibility and the volume change characteristics of the expansive clays at saturated, high-water content states, one-dimensional compression and swelling was conducted on reconstituted samples of the mixture clays (described in section 3.3) in standard oedometers (ASTM-Standard-D2435, 2003). The samples, taken after pre-consolidation in the consolidometer (Figure 3-3), had various initial water contents ranging from 60% to 372%, slightly less than their liquid limits of 72% to 509%, respectively.

3.5.3 Unconfined compression test

A series of unconfined compression tests (UC) was performed either on the reconstituted or compacted samples (Table 3-4 and 3-5) by following the ASTM (ASTM-Standard-D2166, 2000). For the test on the reconstituted soils, the samples were prepared by trimming to the dimensions of 50mm in diameter and 100mm in height, while the test on the compacted soil, the specimens were prepared to 70mm diameter and 150mm height and tested at a loading rate of 0.75mm/min. The compacted soil specimens were prepared at the optimum moisture content of the cement-untreated soil using modified effort (ASTM-Standard-D1557, 2000). In the case of cement-treated soil, the UC tests were performed after 28-day curing. The unconfined compressive strength (q_u) and Young's modulus at half the failure stress (E_{50}) of the soil specimens were determined.

3.5.4 Areal and volumetric shrinkage strains tests

The shrinkage of expansive clay can be measured with a one-dimensional expansion and shrinkage test (ASTM-Standard-D3877, 1996) or by the linear shrinkage test (BS-1377, 1990). However, a linear shrinkage bar test (in this study called areal shrinkage strain test) was suggested by (Puppala, Intharasombat, & Qasim, 2004; Puppala, Pathivada, Bhadriraju, & Hoyos, 2006; Puppala et al., 2007) in conjunction with the image processing technique. These authors recommended that using the image processing technique could provide more accurate measuring results than that from manual measurements. Therefore, the areal shrinkage strain test with the image processing technique was selected in this study. The mixtures of bentonite and Bangkok clay, included in the case of treated cement, were uniformly mixed with water at their liquid limits (LL of the cement untreated soils) and placed in a 120mm x 20mm x 5mm mold, as shown in Figure 3-4. The untreated soil specimen was allowed to hydrate by keeping the moisture content constant for about 48 hours to allow the soil particles to swell while that in the case of cement-treated, the soil specimen was cured for a period of 28 days before placing into the air drying at a controlled temperature room of 30 ± 1°C and a relative humidity of 50 ± 2 %. The air drying of the soil specimens were taken for a period of 7-10 days, depending on the soil specimen which could reach its stable state. The soil specimens without hydration were also tested for the purpose of comparison.

The moisture lost during desiccation of the soil specimens in the mold were recorded every 24h by using a balance with accuracy of 0.01g, until it reached a stable moisture content via room air drying. After drying, cracks formed in the soil specimens in irregular patterns, as shown in Figure 3-4. Photographs of the whole sample were taken with a digital camera, then analyzed with image processing to determine the areal shrinkage strain of the soil specimens. The image analysis provided both cracked and un-cracked areas as reflected by black and white pixels relatively, after setting binarisation with an appropriately selected threshold. By taking the ratio of cracked areas (area of void) after air-drying (A_c) to original area (area of sample before being cracked) of the sample (A_t), areal shrinkage strains were determined(Por, Likitlersuang, & Nishimura, 2015; Puppala et al., 2004).

$$Areal\ shrinkage\ strain,\ ASS = \frac{A_c}{A_t} \qquad (3\text{-}1)$$

Figure 3-4 (A) and 3-4(B) show the Bangkok clay and the bentonite before and after seven days of air-drying, while Figure 3-4(c) shows the image thresholding after air-drying.

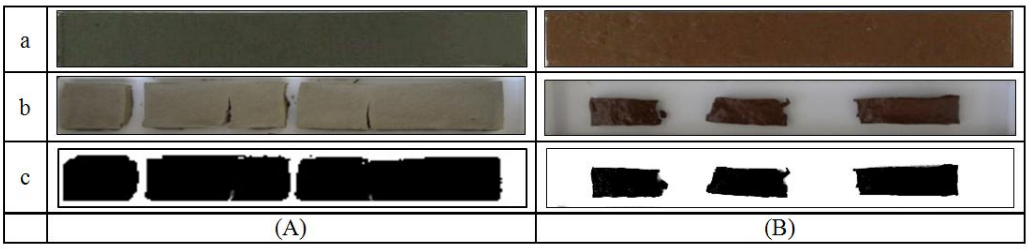


Figure 3-4: Typical photograph of soil specimen surface area seven days before and after air dry (a = before air-dried, b = after air-dried, c = binarised to black and white after image thresholding on air-dried specimens).

The soil specimens after being air-dried in a controlled-temperature room of 30 ± 1°C and a relative humidity of 50 ± 2 % were then used to determine the volumetric shrinkage strain. The volumetric shrink of the dried soil pat were determined by removing the pat from the shrinkage mold and immersing it in a glass cup full of mercury (Figure 3-5). The volume of displaced mercury was then recorded as the volume of the dried soil pat (V_v). The volumetric shrinkage strain (V_{ss}) was determined by dividing the volume of shrinkage between the air-dried soil to that of the pre-dried (wet soil) as follows:

$$Volumetric\ shrinkage strain, V_{ss} = \frac{V_v}{V} \quad (3\text{-}2)$$

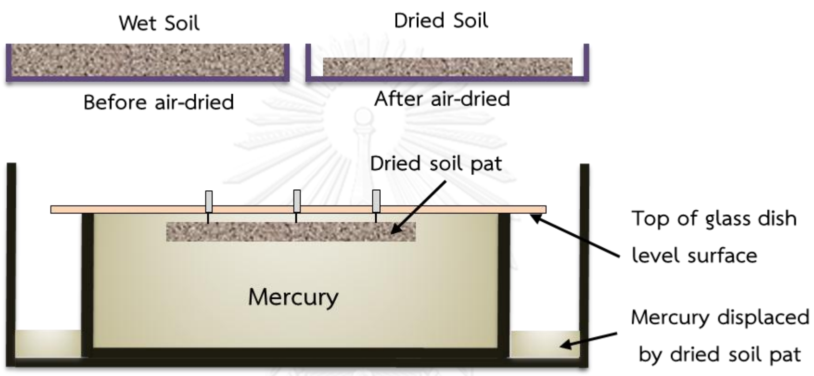


Figure 3-5: Apparatus for measuring volumetric shrinkage strain.

3.5.5 Vertical free swelling strain test

This test was performed on all the compacted specimens at the same conditions that were used in the UC tests. The specimen was trimmed to 60 mm diameter and 24 mm height, and then placed into a rigid mold with an open face at the top. The specimen in the mold was then submerged in water. The specimen was in direct contact with water at the top. The specimen bottom also had free access to water through a sheet of filter paper laid between it and the percolated mold base (Figure 3-6). The vertical free swell of the specimen was recorded by using an electronic digital calliper until the deformation reached a stable state. The vertical free swelling strain (ε_{vf}) was defined as the ratio between the increases in height (Δh) to the original height (h_0) of the laterally confined specimen (ε_h= 0%), where

$$\varepsilon_{vf} = \frac{\Delta h}{h_0} \tag{3-3}$$

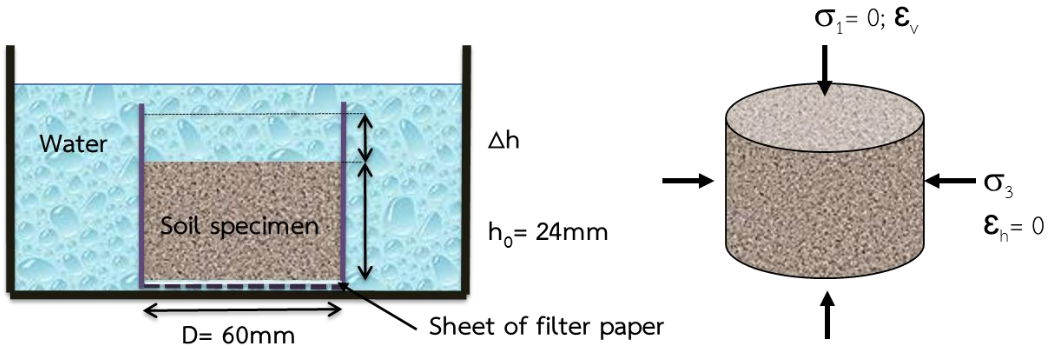


Figure 3-6: Apparatus for measuring vertical free swelling strain.

3.5.6 Confined swelling pressure test

The vertical and lateral swelling pressures and the vertical swelling strain of the cement-treated and untreated soils were measured in a medium-scale oedometer equipped with a gauged confining ring to record the horizontal reaction pressure. The specimen size was 75 mm in diameter and 25 mm in height. The apparatus is illustrated in Figure 3-7. The vertical swelling force of the soil specimen was recorded by using a low-compliance load cell with a large capacity (20 kN). The gauged confining ring, manufactured of stainless steel with a thickness of about 2.5 mm, was calibrated by applying air pressure when the chamber was completely sealed. The structure and configuration of the confining ring made it susceptible to offset shifts when the bolts were tightened to assemble the whole apparatus. Through a number of calibration trials, however, it was confirmed that the linearity of the response and the calibration factor were unchanged. The offset were therefore readily corrected simply by noting the shifts. The lateral swelling strain was thus restrained (no lateral swelling strain) while the vertical strain was allowed to develop in steps.

Initially, the specimen was installed in the apparatus and the swelling strains were kept to zero in both lateral and vertical directions (ε_h= 0%, ε_v= 0%) by fixing the vertical ram. Then the specimen in the apparatus was allowed to freely suck the water from a burette connected to the bottom of the specimen through a porous stone, as shown in Figure 3-7. This process guaranteed that the matric suction at the end of each equilibration stage was always zero. The air pressure in the chamber was always equal to the atmospheric pressure in a controlled temperature room of 25±1°C. The pressure vent in the chamber was loosely plugged to prevent a humidity loss. The testing procedure was divided into 5 steps, in which the lateral swelling strain was always restrained to be zero (ε_h= 0%) and the vertical swelling strain (ε_v) was allowed to develop to 2%, 4%, 6%, and 8% by raising the ram. In the first step, the specimen in the apparatus was not allowed to swell (no volume change) until the specimen stopped sucking the water from the burette and reached a stable condition in both vertical and lateral swelling pressure. The process in each step typically took 7 to 14 days. The swelling pressure of the specimen in the apparatus in the first step was the maximum in both vertical and horizontal directions. For step 2 to step 5 (ε_v= 2, 4, 6 and 8%, respectively), the procedures were similar.

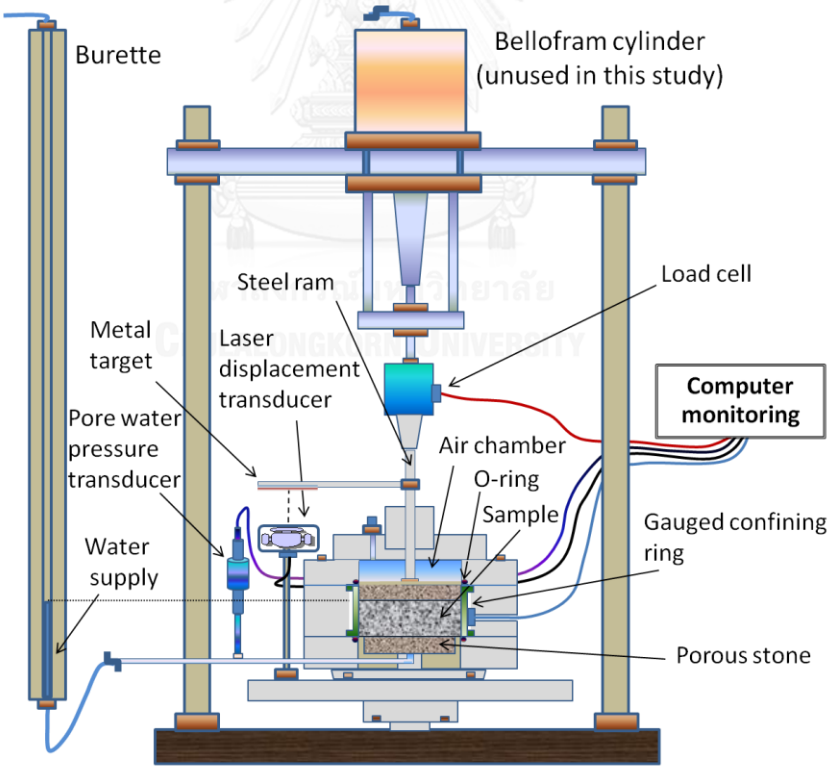


Figure 3-7: Confined swelling pressure (CSP) oedometer measuring lateral and vertical swelling pressures.

CHAPTER 4: TESTING RESULTS

4.1 Bentonite and Bangkok clay mixtures

4.1.1 Index properties

American standard testing methods (AUSTROADS, 2000) were followed to determine the liquid limit (LL), plastic limit (PL) and shrinkage limit (SL). The tests were performed on all proportions of Bangkok clay and bentonite, and the LL, PL and PI were determined as summarised in Table 3-3. The results obtained from the Atterberg tests indicate that adding a proportion of Bangkok clay to bentonite has log-proportional effects on the LL and PL, as shown in Figure 4-1. The effect was also similarly found on the optimum moisture content (OMC) obtained from a modified compacted test (ASTM-Standard-D1557, 2000). As the Bangkok clay content (BKK) increased, the LL of the mixture clays decreased over a wide range, leading also to the decrease of the PL, PI, and OMC values. A study on bentonite/sand mixtures (PCA, 1956) concluded that the reductions of the above properties are related to the presence of finer particles that contribute to the plasticity. However, although the Bangkok clay has smaller particles smaller than that of the bentonite (Figure 3-2), it does not contribute to plasticity as much as bentonite does. On the other hand, a study on lime-stabilised expansive soil (ASTM-Standard-D2478, 2010) found that a decrease in the LL and PI, as well as an increase in the PL and OMC, are obtained while increasing lime content. Compared to such behaviour, adding non-swelling clays seems to have the simpler effect of bringing the plasticity log-linearly towards that of the added clay upon 100% replacement.

The influence of the Bangkok clay content is more significant on the LL and PI than on the PL and OMC. As shown in Figure 4-1, the LL and PI decreased significantly while the PL and OMC only slightly decreased with the increase of BKK. This might reflect the sensitivity of swelling clay content to its plasticity. Similarly, Mitchell & Soga reported that the variation of LL for different clay mineral groups is greater than the variation of the PL (Mitchell & Soga, 2005). Figure 4-1 also illustrates that the OMC-line lies below the PL-line, and they show almost parallel lines between each other. The log-linearity observed in these relationships means that the greatest change of the LL value is observed when a first 20% dose of Bangkok clay is introduced to the pure bentonite. The decreases of the LL, PI and OMC became gradually modest as the proportion of BKK grew. The implication of these reductions in the plasticity properties of a swelling clay added with a non-swelling clay is that the swelling clay becomes relatively friable and workable with a modest replacement.

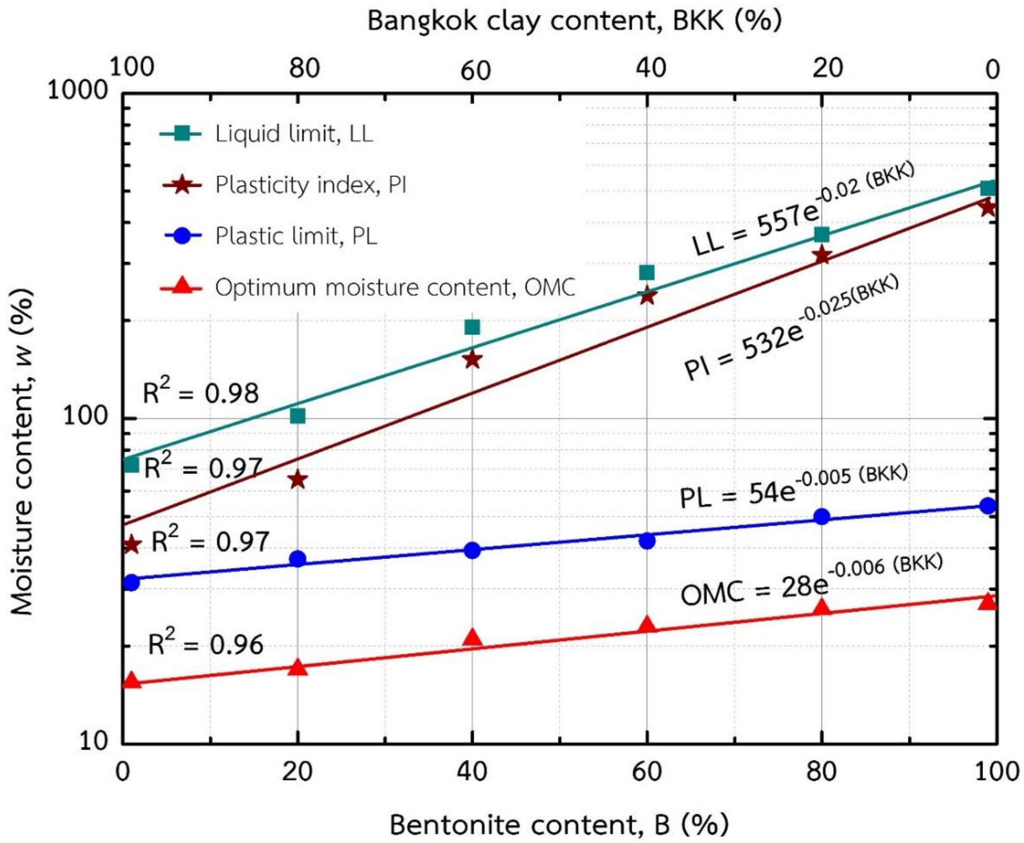


Figure 4-1: Effect of Bangkok clay content (BKK) on LL, PI, PL, and OMC of the bentonite-Bangkok clay mixtures.

All the Atterberg limit's results (LL and PI) are plotted in the Casagrande's chart (Figure 4-2). It can be seen that all the data points of the matching between LL and PI lie above the A-line and are slightly lower than the U-line while increasing the bentonite content in the soil mixtures up to pure bentonite. This might be due to a very high montmorillonite mineral content in the bentonite, which could lead the data points to being placed almost on the U-line when the bentonite content of soil mixtures is above 40% soil mixtures (Figure 4-2). The U-line is the upper bound of the relationship between the plasticity index and liquid limit for any soils.

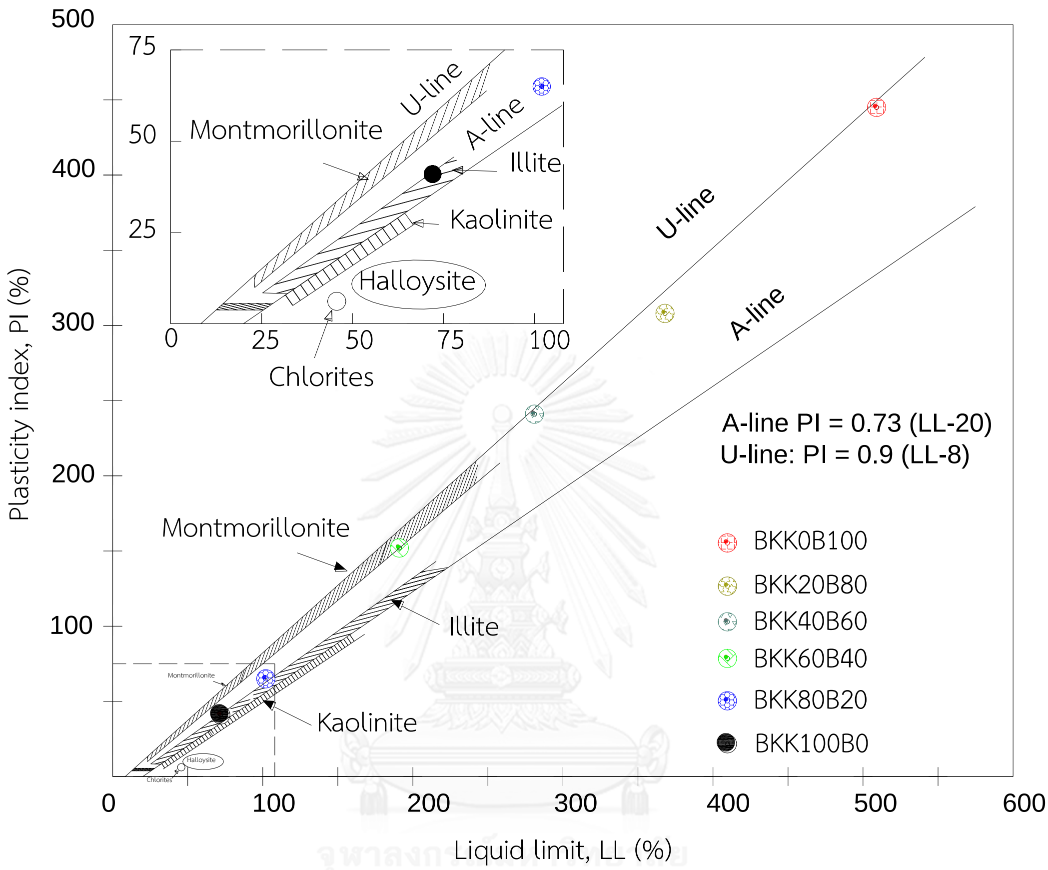


Figure 4-2: Plot of clay minerals (kaolinite, illite and Bangkok clay) on Casagrande's chart after (Mitchell & Soga, 2005).

Figure 4-3 shows the interaction obtained from each proportion of Na-montmorillonite bentonite and clay mineral (kaolinite, Illite and Bangkok clay) mixtures versus Na-montmorillonite bentonite content present by liquid limit. The figure is modified after (Mitchell & Soga, 2005). The dashed line represents the LL values to be expected if each mineral contributed in proportion to the amount present while the data points and solid lines show the actual values measured in the laboratory. The bentonite-kaolinite and bentonite-Bangkok clay mixtures gave values close to theoretical while those of the bentonite-illite mixtures were much lower than predicted. Compared to the theoretical values, although the bentonite-Bangkok clay mixtures gave LL values slightly less than the bentonite-kaolinite mixture, the pure Bangkok clay presents slightly higher LL values than the kaolinite, illite and the theoretical values (Figure 4-3).

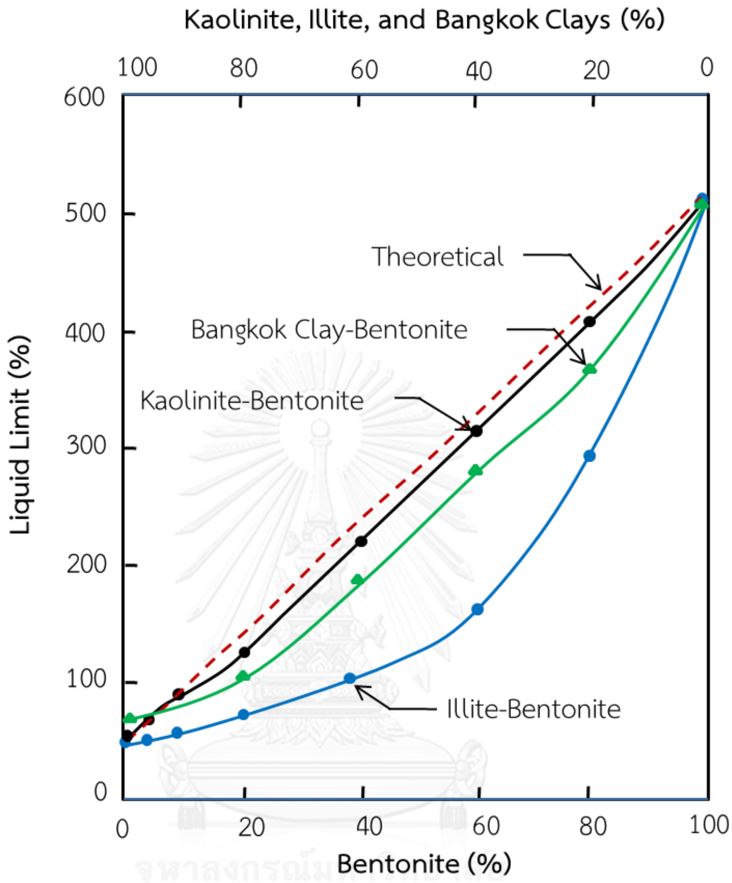


Figure 4-3: Interaction between clay minerals (Kaolinite, Illite and Bangkok clays) versus bentonite clay as indicated by liquid limit, data modified after (Mitchell & Soga, 2005).

4.1.2 Compaction characteristic

Compaction is one of the methods used in soil improvement. Due to compacted energy, the soil particles are artificially arranged and packed together in a denser state, leading to a decrease in the void ratio of the soil and thus increasing its dry density. In this research, a modified compaction energy (ASTM-Standard-D1557, 2000) was applied on 6 different soil properties, bentonite-Bangkok clay mixtures, (Table 3-4) and the optimum moisture content and maximum dry density were determined. The soil mixtures had liquid limit varying from 72% to 509% depending on the amount of bentonite content. The dry density and moisture content at different levels of Bangkok clay content are plotted graphically in Figure 4-4 in order to find the optimum moisture content and the maximum dry density of the soil mixtures. The figure revealed that the maximum dry density (MDD) increases and the optimum moisture content (OMC) decreases monotonically as BKK increases. The decrease of OMC is related to the reduction of LL, as illustrated in Figure 4-1.

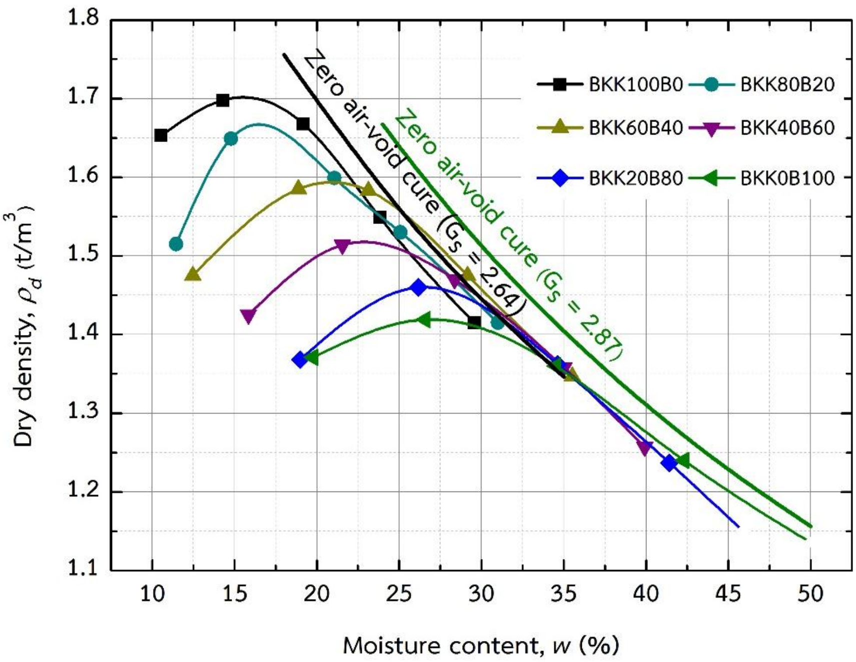


Figure 4-4: Effect of Bangkok clay content on compaction behaviour of the bentonite-Bangkok clay mixtures.

This is possibly due to the fact that non-swelling Bangkok clay requires less water for inter-particle lubrication than the swelling clay (bentonite). The reduction in the OMC when increasing the proportion of BKK can be explained as follows: the cation exchange between non-swelling Bangkok clay and Na-montmorillonite decreases the thickness of electric double layers and promotes flocculation. The flocculation of the clay particles implies that the water-additive-soil mixtures can be compacted with lower water content, and the OMC will be reduced. As explained in the literature review, the montmorillonite clay particles, due to their very small size and platy shapes, have large surface areas Table 2-2, when external surfaces of each unit layer (basal spacing) of the clay mineral come close to each other, interacting diffuse electrical double-layers are formed with a relatively high concentration of cations near the surface. As a result of negative charges in the clay surfaces, the cations present in the solution (water) are attached to the immediate vicinity of the clay particles. Consequently, higher levels of bentonite content (higher montmorillonite) require higher levels moisture content (OMC) to obtain maximum dry density.

The zero air-void curves illustrated in Figure 4-4 represent the line envelopes for the zero air-void cures of all the mixture between bentonite and Bangkok clay, while the zero air-void cures of the bentonite lie above the Bangkok clay. As shown in Figure 4-4 and the Table 3-3 found that for compaction curve of higher PI soil, flatter curve are obtained. For example, the compaction curve of the pure bentonite (PI= 455%) is the flattest curve compared to all the other soil mixtures. Hence, in the compaction process the bentonite needs higher levels of water content between each compaction step than that of lower PI soil (i.e. Bangkok clay) in order to attain a significant change of state. The relationship between OMC and MDD obtained from Figure 4-4 is presented as a linearly in a relatively wide range as shown in Figure 4-5. The relationship can be simplified by the following equation:

$$MDD = -0.025(OMC) + 2.08 \tag{4-1}$$

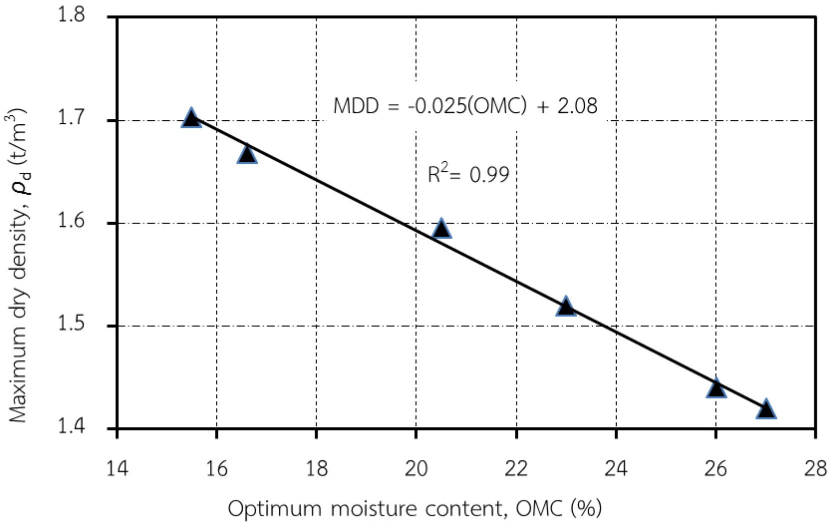


Figure 4-5: Relationship between OMC and MDD of bentonite and Bangkok clay mixtures.

Figure 4-6(a), (b), and (c) plotted between MDD, OMC, and CBR versus PI, respectively. These relationships can be simplified by the following equations:

$$MDD = -0.0007PI + 1.7 \qquad [t/m^3] \qquad (4\text{-}2)$$

$$OMC = -6x10^{-5}(PI)^2 + 0.05PI + 13.2 \qquad [\%] \qquad (4\text{-}3)$$

$$CBR = 128.4(PI)^{-0.43} \qquad [\%] \qquad (4\text{-}4)$$

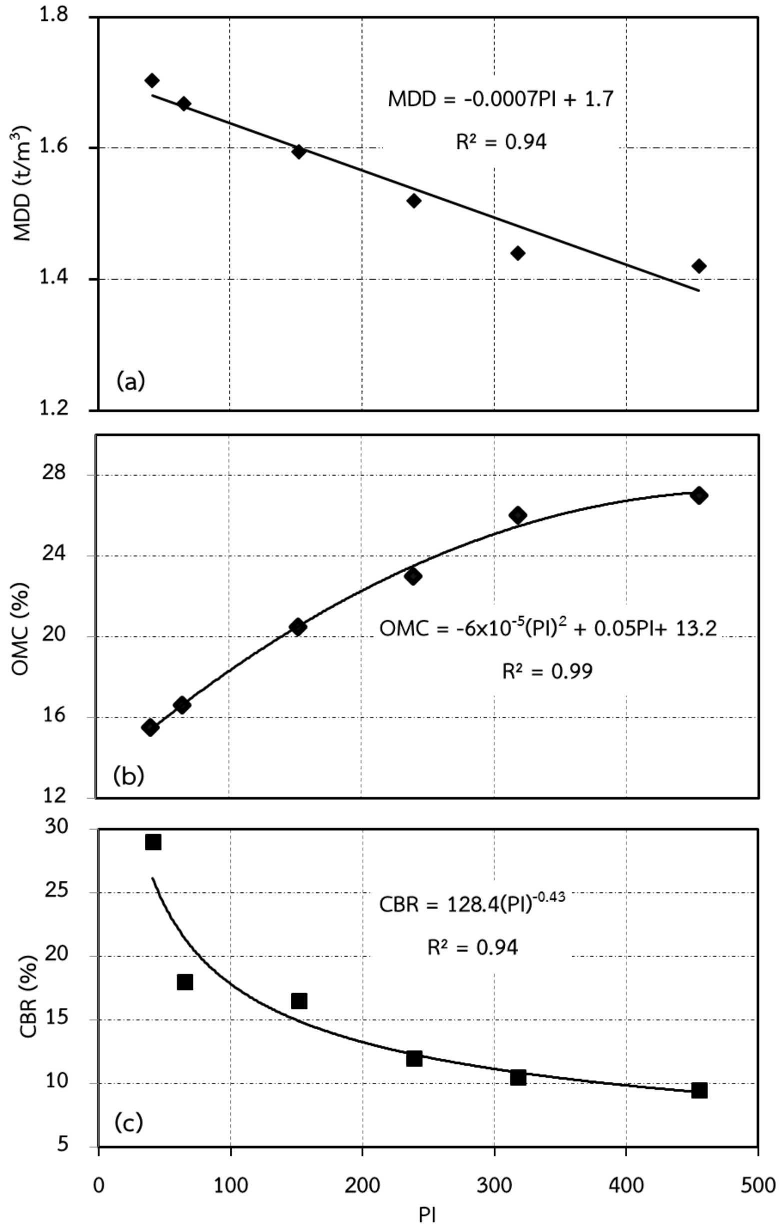


Figure 4-6: Correlation between MDD, OMC and CBR versus PI of bentonite and Bangkok clay mixtures.

These relationships show that MDD and CBR increase in relation with the decrease of OMC while the PI decreases. In all the clay mixtures, the highest MDD and the lowest OMC values were obtained at the lowest PI. This means that the highest MDD and the lowest OMC were obtained when the swelling clay (bentonite) is fully replaced by the non-swelling clay (Bangkok clay).

4.1.3 California bearing ratio (CBR)

The results obtained from CBR tests for un-soaked and soaked specimens using the modified compaction test at various percentages of bentonite and Bangkok clay mixtures are summarised in Table 4-1. As the percentage of BKK increases, CBR values increased under both un-soaked and soaked conditions. In the un-soaked condition, the result shows a higher CBR value rating, between 9.5% and 27.0%. On the other hand, when the same mixed proportion with the same compacted condition specimen were soaked for 65 days to simulate a possible weakness condition, they exhibited very low values of CBR, between 0.5% and 2.1% (Table 4-1). The Bangkok clay doesn't have much of an effect on the CBR value of the swelling soil (bentonite) in the case of soaking. The CBR reduction ratio due to soaking, R, could be qualified as the following:

$$R(\%) = \frac{CBR_{un} - CBR_{s}}{CBR_{un}} \times 100 \qquad (4\text{-}5)$$

Where CBR_{un} = CBR value under the unsoaked condition and CBR_s = CBR value under the soaked condition at 65 days.

The CBR_{un}, CBR_s, and R values are summarised in Table 4-1. Observed that the CBR reduction ratio of all the mixture soils are almost the same, varying from 93% to 95%.

Table 4-1: Unsoaked and soaked CBR values of bentonite/Bangkok clay mixtures

Bangkok clay content (%)	0	20	40	60	80	100
CBR_{un} (%)	9.5	10.5	12.0	16.5	18.0	29.0
CBR_s (%)	0.50	0.65	0.75	0.9	1.1	2.1
R (%)	95	94	94	95	94	93

The study on gypsiferous subgrade soil (Razouki, 2003) and on soft fine-grained red soils stabilised by using fly ash (Sarkar, Islam, Alamgir, & Rokonuzzaman, 2012) have confirmed that the soaking period has a large effect on reductions of CBR values, in which the CBR value decreased with an increase in soaking period. Figure 4-7 presents the variations of the vertical swelling strains versus time of the compacted soils in the CBR mold under 2.5 kPa during soaking. During soaking, the bonding force between clay particles becomes weaker due to the interaction between basal space (Figure 2-2) and water molecules leading the clay particle swell, especially montmorillonite mineral, which is contained in the soil specimen. Hence, the structure of the clay sample becomes loosened as evidenced by observing from swelling test results of each sample, such as the increased vertical swelling strain due to increases in soaking period (Figure 4-7). The void ratio of the soil specimen increases, leading the specimen to become soft, and, as a result, the penetration stress becomes smaller, with a decreased CBR value.

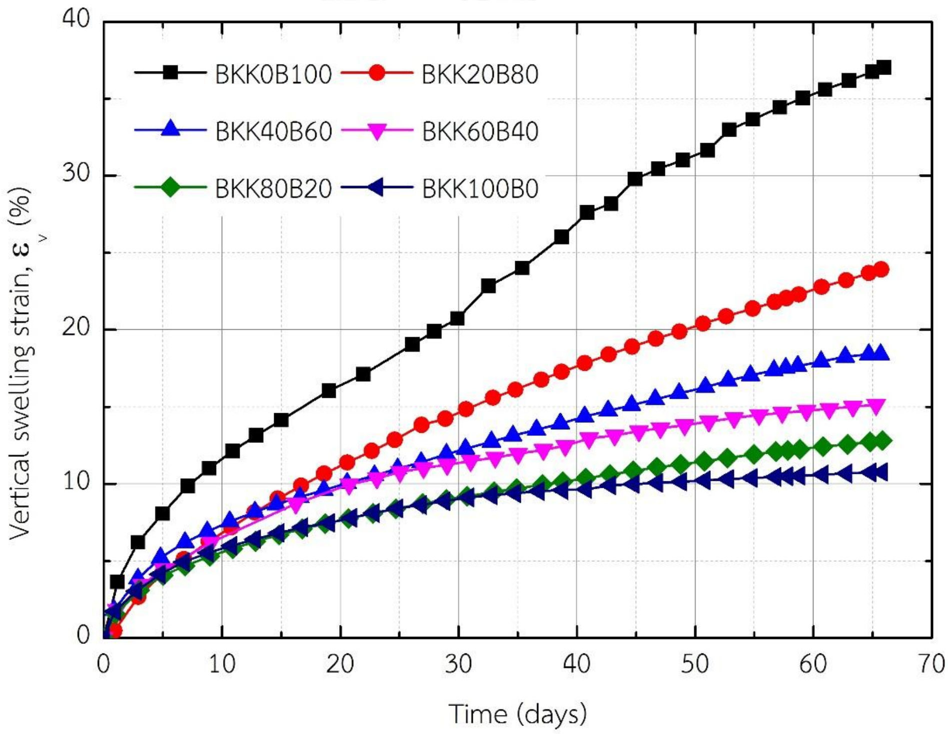


Figure 4-7: Vertical swelling of bentonite-Bangkok clay mixtures under a surcharge of 2.5 kPa during 65 days of soaking.

Figure 4-7 observed that the swelling strain decreased significantly with increases in the BKK. For each 20% increment of BKK from 0% to 100%, the effect on the vertical swelling diminished gradually. The rate of vertical swelling became smaller, and almost reached a stable stage within 65 days when the BKK was more than 60%. Samples with less than 40% BKK continued to swell after 65 days. This may be due to the high swelling potential of bentonite coupled with a smaller degree of permeability, which resulted in slower water infiltration into the sample, requiring more time to reach a stable stage. As with plasticity, a modest addition of non-swelling clay has a significant effect on reducing the swelling potential, as well as the time needed to reach a stable state.

4.1.4 Compressibility behaviour of the reconstituted soils

In order to qualify the compression and swelling characteristics of the expansive soils at high levels of moisture content, an oedometer apparatus was adopted, using a test procedure that follows the ASTM (ASTM-Standard-D2435, 2003). The test was performed on the reconstituted samples explained in the previous section (section 3.3). The moisture content and void ratio obtained at the end of the reconstituted sample process under a vertical effective stress of 30 kPa is represented as the initial moisture content and an initial void ratio at the beginning of the consolidation process. The moisture content was slightly lower than its liquid limit, as summarised in Table 3-3 and Table 4-2.

Table 4-2: Initial moisture content and the compressibility of the bentonite/Bangkok clay mixtures

Bangkok clay content, BKK (%)	0	20	40	60	80	100
Initial moisture content, w_i (%)	372	267	246	189	88	60
Initial void ratio (e_i)	11.65	9.74	6.87	4.31	3.80	1.55
Compression index (c_c)	6.2	5.0	3.4	2.1	1.7	0.6
Swelling index (c_s)	1.66	0.99	0.77	0.33	0.21	0.12

4.1.4.1 Compression behaviour

The void ratio-vertical effective stress $(e-\log\sigma_v^{'})$ curves for each soil specimen with different LL in one-dimensional compression were obtained as shown in Figure 4-8. The specimens were moulded with high moisture content, which resulted in a relatively large initial void ratio at seating load. With the specimen with higher bentonite content, the void space was higher and filled up by air or/and water. At this moisture content and void ratio condition the resistance of soil specimens to the applied vertical stress is very low, presenting very high compressibility, as shown in Figure 4-8. Due to the fact that bentonite has very high LL compared to the Bangkok clay, the water retention capacity of Bangkok clay is much less than that of the bentonite. As presented in the Table 4-2 the moisture content of the higher LL soil (i.e., bentonite) is much higher than the lower LL soil (i.e., Bangkok clay) for the specimen has the same experience under a vertical effective stress set at 30 kPa. The initial void ratios of mixtures of bentonite and Bangkok clay decrease with increase in Bangkok clay content Table 4-2. The total compressibility was found relatively in the order of the LL of the soil mixtures. The lower LL soil presented a smaller compression index (c_c), as well as a smaller swelling index (c_s) relatively lower swelling mineral content (i.e., montmorillonite) in the soil mixtures. The smaller in c_c and c_s values indicate less potential of settlement with increase non-swelling soil content (i.e., Bangkok clay).

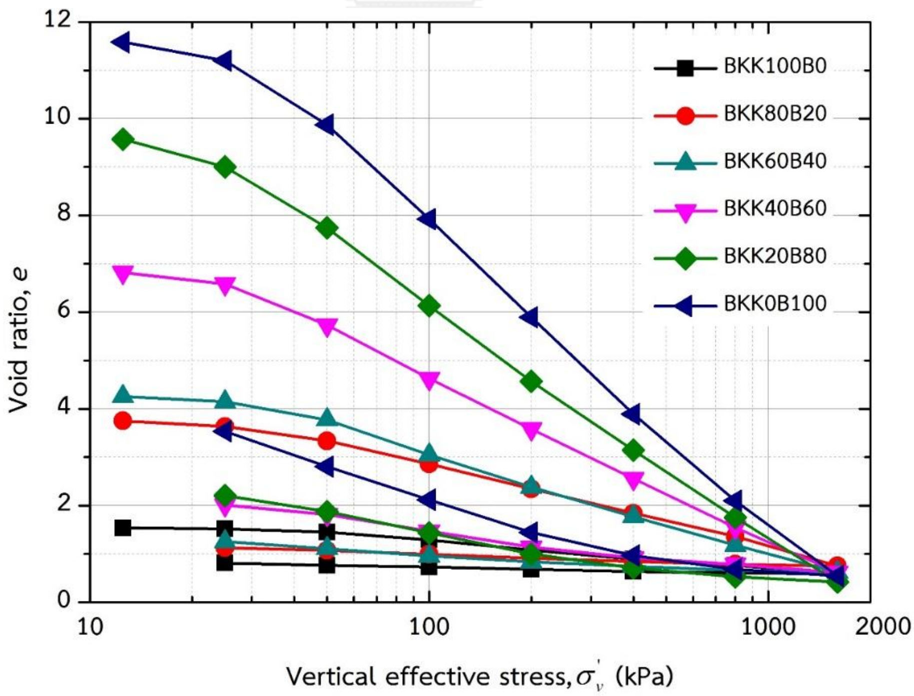


Figure 4-8: Compression behaviour of bentonite-Bangkok clay mixtures.

The compressibility of a clay soil with given pore fluid is generally controlled by the net repulsive pressure related to the development of a diffuse double layer around the clay particles (Sreedharan & Puvvadi, 2013). With a decrease in the bentonite content of the soil mixtures, the repulsive pressure decreases and the specimen reaches equilibrium with the applied effective pressure at lower void ratios.

4.1.4.2 Coefficient of consolidation

Following the square root time method, the vertical coefficients of consolidation (c_v) for all the soil specimens were determined for each loading and unloading step. These were plotted between c_v versus the log of the vertical effective stress $(c_v - \log \sigma_v^{'})$ of the soil specimen with different BKK, as presented in Figure 4-9.

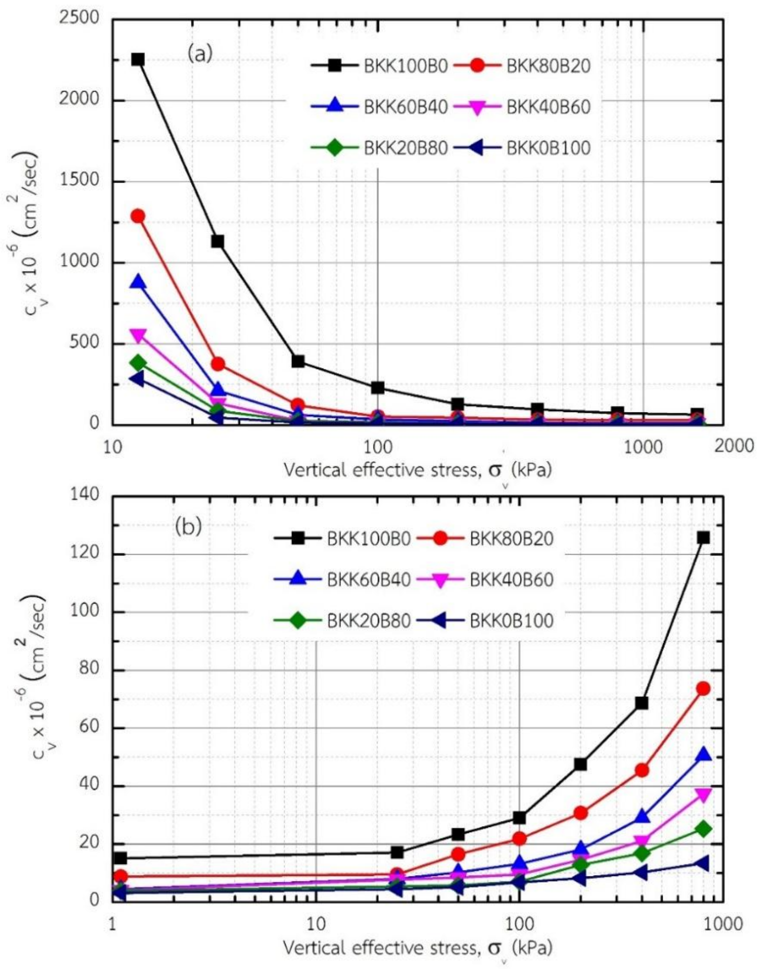


Figure 4-9: Vertical coefficient of consolidation versus vertical effective stress of the bentonite-Bangkok clay mixtures at (a) loading and (b) unloading stages.

The results presented in Figure 4-9(a) indicated that the c_v values decreases significantly with an increase in $\sigma_v^{'}$ and decrease in BKK. In contrast to these results, the study on reconstituted organic soil (Rabbee & Rafizul, 2012) found that the c_v value increased with an increase in both the organic content and the effective stress $(\sigma_v^{'})$. However, at higher effective stress levels, a similar behavior to this study was observed with c_v converging to a constant value (Figure 4-9a). Unlike the loading stage, the c_v in the unloading stage (Figure 4-9b) seemed to decrease significantly with decreases in both the BKK and vertical effective stress. The relationship between c_v with varied plasticity index (PI) for each incremental loading and unloading stages are presented in Figure 4-10 and 4-11, respectively. For all the applied vertical effective stress, the decreasing trends of c_v were similar in terms of shape. From the figures observed, c_v markedly increased with decreases in PI. Thus, the order of increase in c_v is practically related to the order of decrease in the PI in both cases, loading and unloading stages.

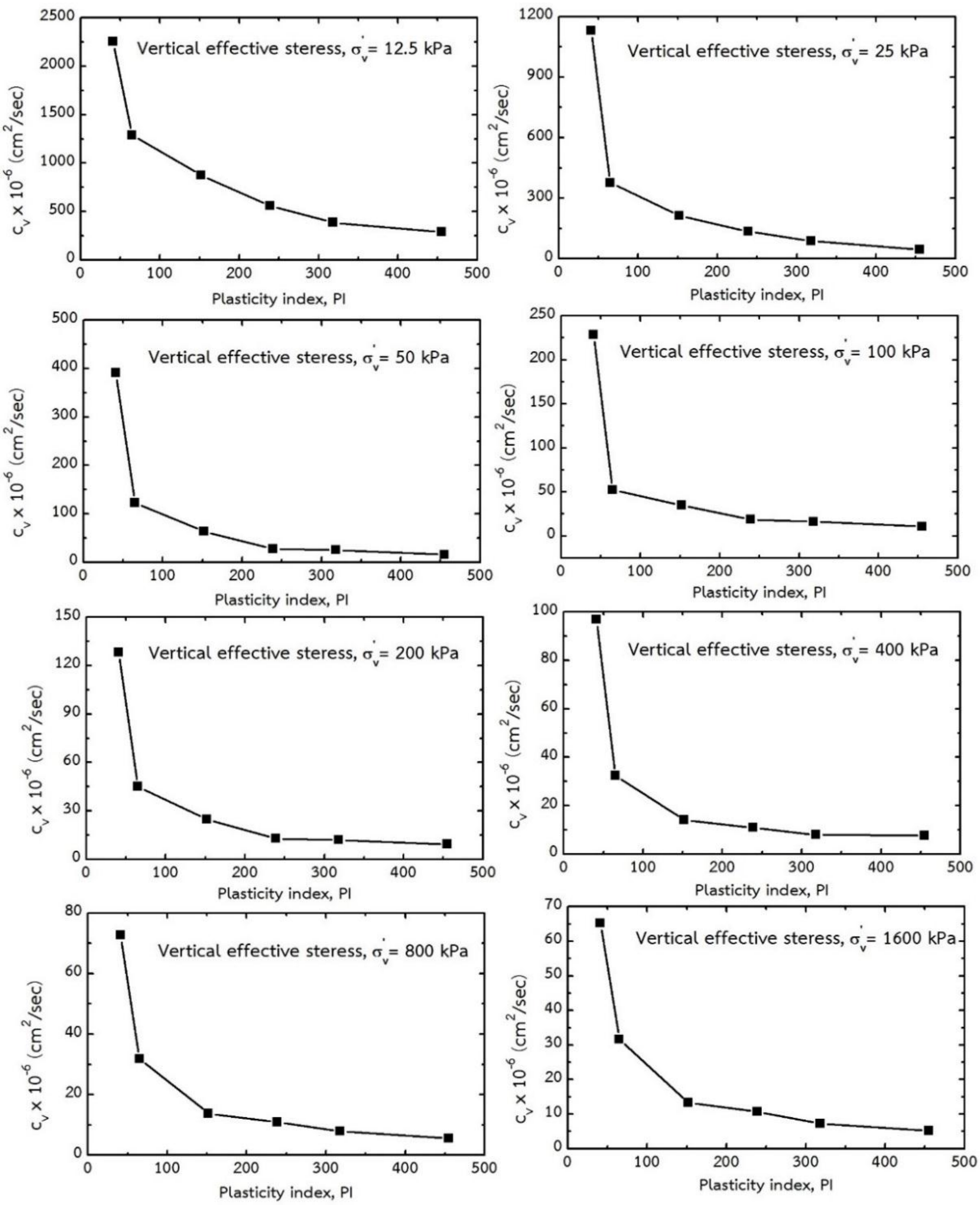


Figure 4-10: The relationship between c_v and PI at loading stage of the soils with different PI obtained from bentonite and Bangkok clay mixtures.

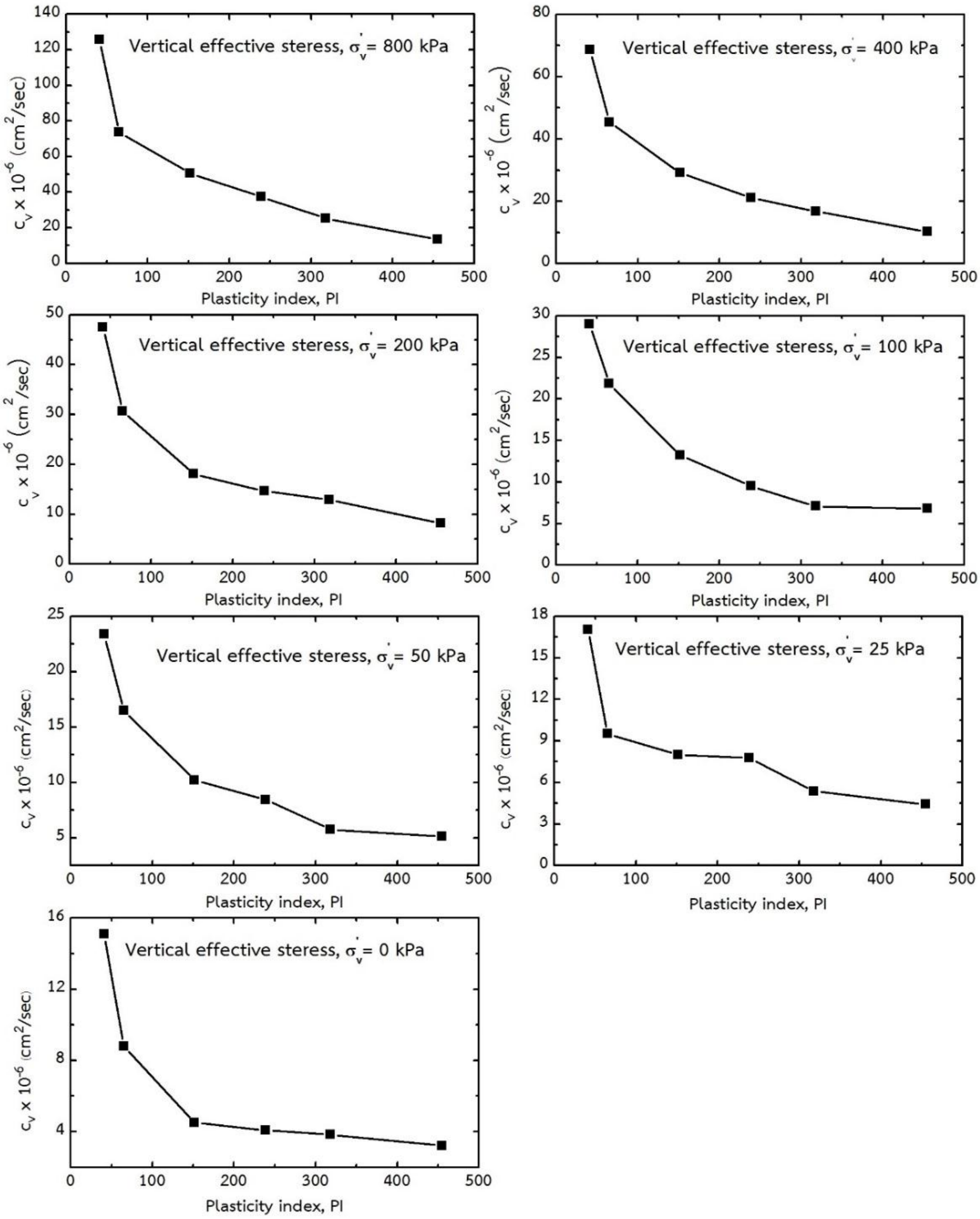


Figure 4-11: The relationship between c_v and PI at unloading stage of the soils with different PI obtained from bentonite and Bangkok clay mixtures.

4.1.4.3 Correlation between c_c versus c_s and LL

Correlation and empirical relationships are used widely in geotechnical engineering because it provides a fast, cost effective means of predicting the value of one parameter based on the values of some other, possibly more easily performed test and/or determinative method. Existing correlations established from extensively sourced data may not be applicable to local conditions, and most are available only a non/low expansive soil, while results for a medium to very high expansive soil may or may not be applicable or available. In this investigation, the expansive soils used were varied in LL from low to very high (LL= 75% to 509%), and the empirical relationship between the compression index, c_c, and swelling index, c_s, and that between c_c and LL obtained from all the mixture conditions are employed as presented in Figure 4-12(a) and (b), respectively.

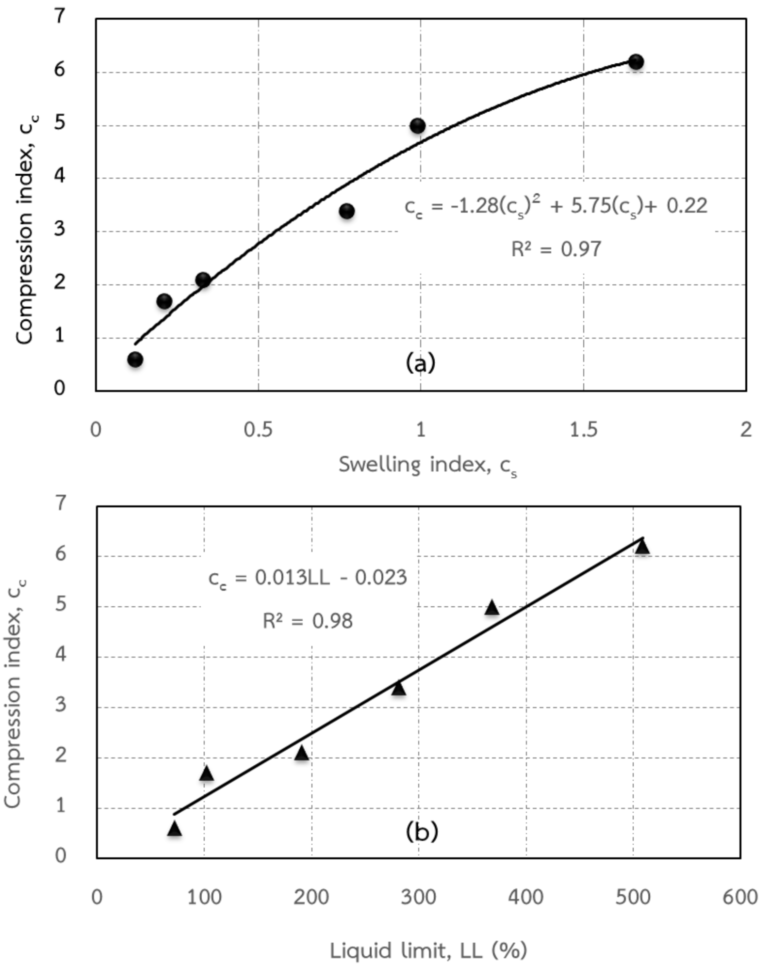


Figure 4-12: Correlations between c_c, c_s and LL of bentonite-Bangkok clay mixtures.

It can be perceived from the figure that when c_s and LL increase, c_c also increases. Generally, the swelling index is appreciably smaller in magnitude than the compression index, c_s/c_c = 0.1-0.2 (Das & Sobhan, 2013). The compressibility properties, expressed by c_c and c_s, were expressed as the empirical equations of (4-6) and (4-7), respectively.

$$c_c = -1.28(c_s)^2 + 5.75(c_s) + 0.22 \tag{4-6}$$

$$c_c = 0.013LL - 0.023 \tag{4-7}$$

4.1.5 Stress-strain behaviour of reconstituted soil

The strength and stress-strain behavior at varying levels of BKK with high initial moisture content slightly lower than its LL (Table 4-2) has been investigated by conducting an unconfined compression test (ASTM-Standard-D2166, 2000). The stress-strain results at varying levels of BKK are presented in Figure 4-13. Figure 4-13a reveals that stress increased in relation to increases of axial strain, and showed similar behaviour for all soil mixtures. The figure also depicts that strength increases with increases in the level of BKK (decrease LL). Clear failure peaks strength after which strain-softening occurred were observed in specimens with 80% and 100% BKK, with the q_u of 12.8kPa and 17.6 kPa at a lower axial stain of 5.4% and 4.8%, respectively. There was no clear peak strength observed, and behaviour was ductile, when the level of BKK in the clay mixtures was lower than 80%.

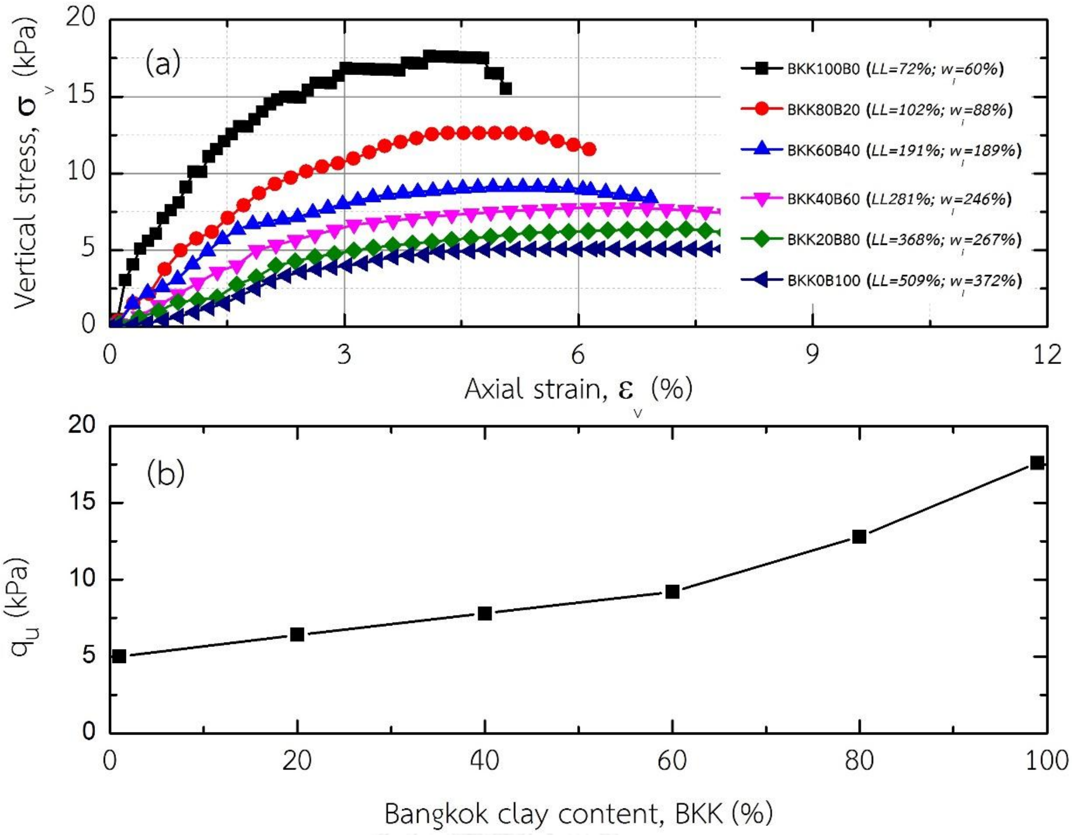


Figure 4-13: Stress-strain behaviour of reconstituted expansive soil.

4.1.6 Swelling strains characteristic

The swelling strains in this research work focused on the vertical swelling strain of a confining lateral strain with free swell in the vertical direction ($\sigma_v' = 0$). The main intent of the testing program was to better understand the effectiveness of various available empirical tests to provide realistic predictions of vertical heave of expansive soils when subjected to seasonal change. The important soil properties in relation to the swelling expansive soil during soaking are the initial water content and dry density as well as the LL of the soil. In this investigation, the initial condition of the soil specimens was set at their optimum moisture content (OMC) and maximum dry density (MDD). The OMC and MDD of the compacted soil mixtures depended on the amount of BKK. The higher the level of BKK in soil mixtures, the lower the levels of OMC and higher MDD obtained, i.e., bentonite has OMC = 27% and MDD = 14.2 t/m^3, while the Bangkok clay has OMC = 15.5% and MDD = 17.0 t/m^3 (Table 3-3). Swelling tests in a lateral confining strain (ε_h= 0%) with a free vertical swelling strain mold of the compacted bentonite and Bangkok clay mixtures were conducted (Table 3-3). The vertical free swelling strain (ε_{vf}) of the soil specimens during soaking were plotted versus time (days) as presented in Figure 4-14(a), while Figure 4-14(b) presented the maximum vertical free swelling strain versus LL varied from 72% to 509% obtained from the soil mixtures. A study on compacted bentonite (Wayne & David, 2004) showed that the maximum swelling strains of the compacted soil increase while the initial moisture content of the soil decreased with increases of dry density. The OMC set as the initial moisture content of the Bangkok clay is much lower than that of the bentonite, but the swelling strain of the Bangkok clay during soaking is much lower, because the Bangkok clay has very low LL compared to the bentonite and hence the ε_{vf} decreases significantly with increases of BKK in the soil mixtures (Figure 4-14a). The relationship between ε_{vf} and time (day) has shown that the greater the time is, the greater ε_{vf} expansive soil will occur, as shown in Figure 4-14(a). The swelling time to reach its stable stage decreases for the specimen with lower OMC and LL (i.e., higher BKK in the soil mixtures). This characteristic is possibly caused by the decreasing of montmorillonite mineral content in the soil mixtures replaced by illitic or kaolinite, which are predominant in Bangkok clay, leading to lower swelling strain. The montmorillonite mineral has a very weak bonding force between the unit layers (basal space, Figure 2-2c). Water and exchangeable ions can enter and separate the layers easily, hence the result of high swelling properties.

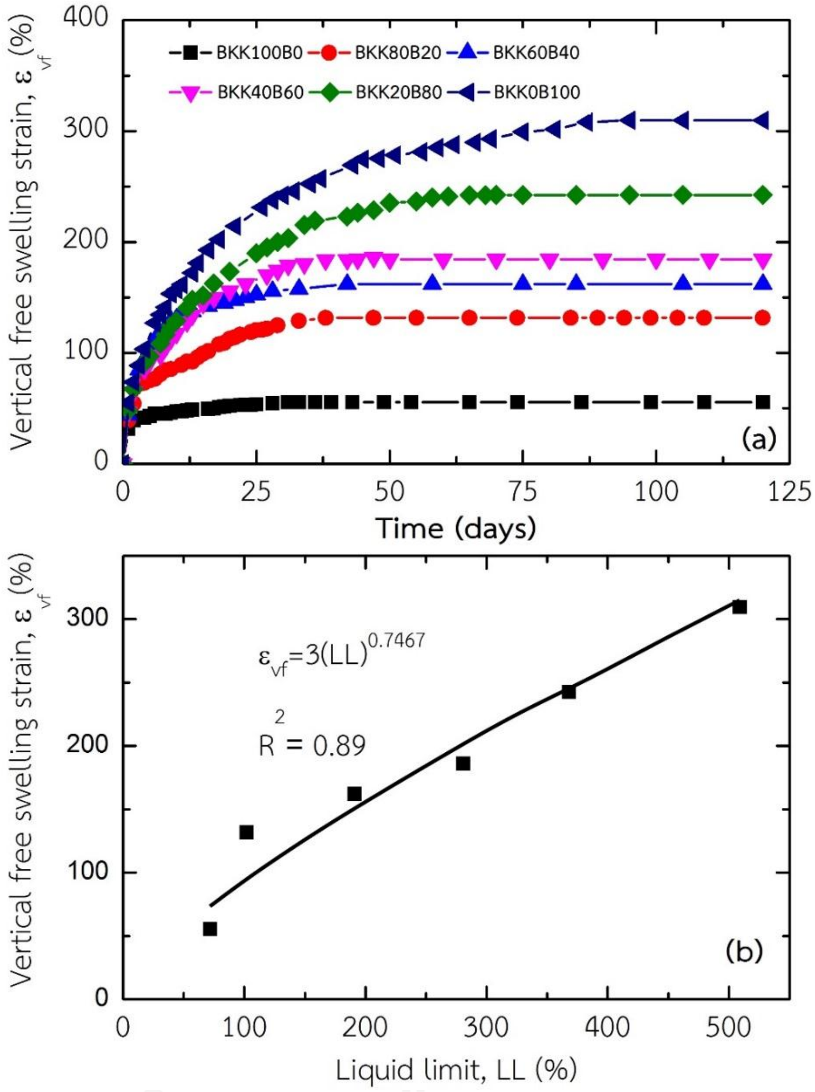


Figure 4-14: Vertical free swelling strain (ε_{vf}) of expansive soil.

Figure 4-14(b) reveal that the ε_{vf} increase significantly with the increase of LL of the soil. Many soil properties and engineering behaviours have been correlated with the liquid limit, including swell-shrinkage properties (Nayak & Christensen, 1971; Yilmaz, 2006). An empirical relationship has been found between ε_{vf} and LL of the compacted bentonite and Bangkok clay mixtures, with LL varying from 72% to 509% and expressed as in the following equation:

$$\varepsilon_{vf}\,(\%) = 3(LL)^{0.7467} \quad (4\text{-}8)$$

The correlation (Eq.4-8) should be reasonably reliable for a medium, high to a very high swelling soil.

4.1.7 Shrinkage strains characteristic

4.1.7.1 Areal shrinkage strain

The areal shrinkage strains of the expansive soils with 48 hour-curing and without curing were investigated. The purpose of curing is to allow the soil particles to become completely swollen after being mixed with water before being air-dried, and a comparison with the specimens without curing are made. After air-drying specimens from the initial moisture contents set at their LLs until reaching stable states, the maximum areal shrinkage strains (ASS) were observed by using an image processing technique as explained earlier (section 3.5.4). The measured ASSs of both specimens with and without curing are shown in Figure 4-15. For both testing conditions, the ASS of bentonite-Bangkok clay mixture decreased with an increase in the amount of BKK. This is because the moisture content at the LL of the clay mixtures, set as the specimen's initial condition, decreased with increasing amounts of BKK. Similarly, as mentioned by some researchers (Mitchell & Soga, 2005; Puppala et al., 2013), the greater the plasticity is, the greater the shrinkage that can be expected. The study of expansive clay mixed with biosolids compost, BS, and dairy manure composts, DMC, (Puppala et al., 2007) showed that the linear shrinkage strain, which in this study is called areal shrinkage strain (ASS), decreased with an increasing amount of BS and DMC. These authors observed that, with higher moisture contents at initial stages, larger shrinkage strains upon drying were obtained. The decrease of ASS for each 20% increment of BKK was larger for greater amounts of BKK (Figure 4-15), indicating that an initial modest dose of a non-expansive clay mixed into an expansive clay has disproportionally large effects in reducing the shrinkage potential.

In both cases, with and without curing, it was observed that at the levels of BKK larger than 60% (LL less than 191%) the ASS markedly decreased. The ASS of the specimen with curing is smaller than that without curing. In the same liquid limit soil, the different in ASS between the specimens with and without curing was smaller for lower LL soil, i.e., for the specimen with LL = 509% (pure bentonite) the different ASS between cured and non-cured mixtures was 11%, while that of the soil with LL= 72%

(Bangkok clay) the difference in ASS was only 2% (Figure 4-15). It is obvious that the ASS tends to be a unique value for the lower LL soils. Hence, curing has a greater effect on ASS for higher LL soils.

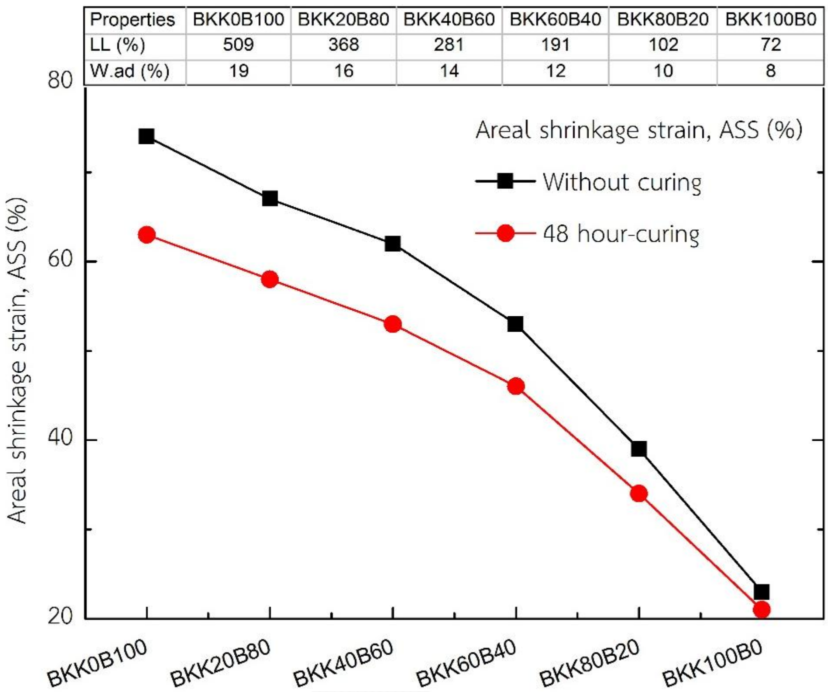


Figure 4-15: Areal shrinkage strain at air-dried in a control temperature room 30±1 ^{o}C and humidity 50 ±2% for 7 days with 48 hour-curing and without curing of bentonite and Bangkok clay mixtures.

The study measured the residual moisture content of clay mixture with different clay constituent proportions during room-air drying. The residual moisture content was recorded daily until reaching a stable moisture content in the room-air drying (Figure 4-16a). Desiccation took about 7 days to allow all the soil specimens to reach the stable moisture content at a controlled temperature room of 30 ± 1^{o}C and a relative humidity of 50 ± 2 %. The residual moisture content ($w_{a.d}$) after 7 days of room-air drying was also measured by oven drying and the results are shown in Figure 4-15. The residual moisture content after room-air drying was greater for the soil with higher LL as well as greater ASS. Figure 4-16(a) shows a relationship between the variation moisture content and time during desiccation. All the specimens were greatly decreased in moisture content after the first 3 days, and then slightly decreased before reaching a stable moisture content at about 7 days. Soil specimens with lower LL could reach a stable moisture content at the room-air drying earlier than soils with higher LL, i.e., the Bangkok clay has LL= 72%, and reaches a stable moisture content at about 5 days at the control room-air drying while that the bentonite (LL= 509%) could reach at about 7 days.

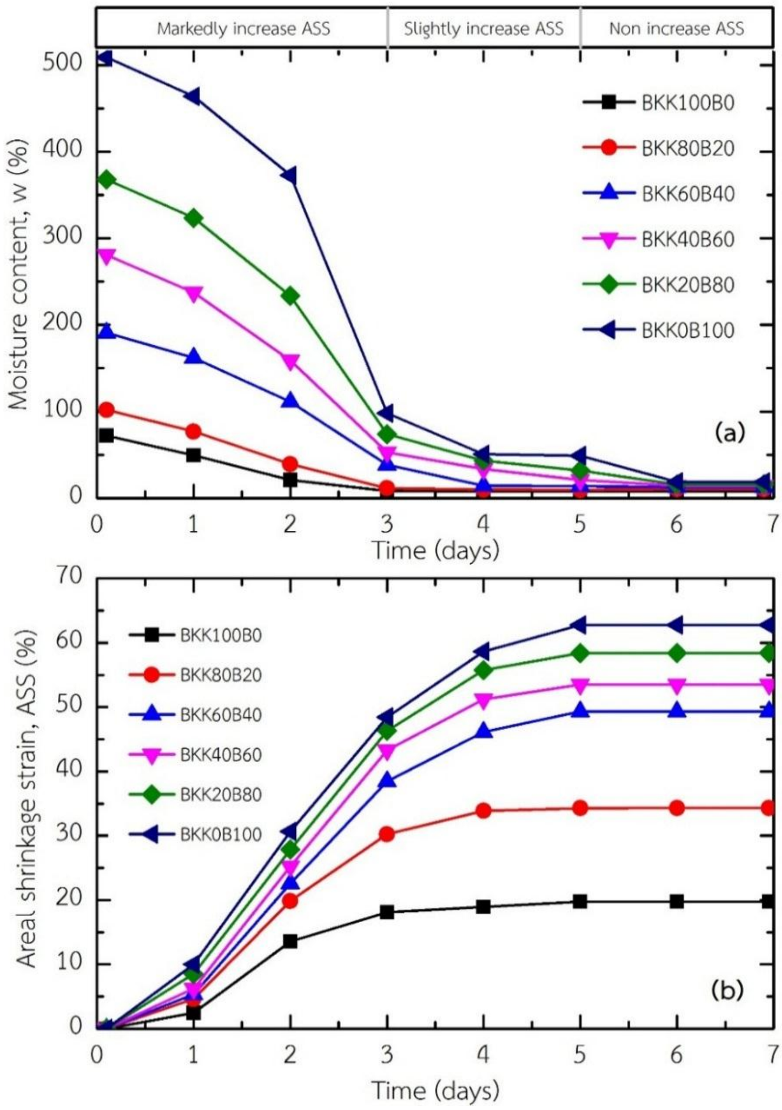


Figure 4-16: Variation of moisture content and areal shrinkage strain versus time during desiccation at a controlled temperature room of 30 ± 1^{o}C and a relative humidity of 50 ± 2 %.

The ASSs for all the soil specimens were greatly increased in the first 3 days in the room-air drying, which occurred in relation to the greatly decreasing moisture content

(Figure 4-16). The ASS continued slightly increase until reaching a constant ASS, considered as no further volume change at this stage, even when the moisture content of the higher LL soil (i.e., bentonite) still slightly decreases to reach a stable moisture content at day 7. Figure 4-17 shows the relationship between areal shrinkage strains (ASS) versus moisture content (*w*) during desiccation of different LL soils set as its initial condition during room-air drying. From the figure, the ASS increases with the decrease of moisture content. At the same air drying condition, all the soil specimens set at their LLs as the initial moisture content presented greater ASS for the higher LL soil. Moreover, the incremental ASS of the soil specimen during air dry was greater for the specimen with higher bentonite content (higher LL). From the figure, it is important to note that a greater change in ASS was observed as moisture content varies between their LL and PL. At the moisture contents less than its PL, the ASS was slightly increased and tended to be a constant value at level of moisture content slightly higher than its stable moisture content (ASS constant at moisture content: PL> $w \geq w_{a.d}$).

Properties	BKK0B100	BKK20B80	BKK40B60	BKK60B40	BKK80B20	BKK100B0
LL (%)	509	368	281	191	102	72
Wa.d (%)	19	16	14	12	10	8

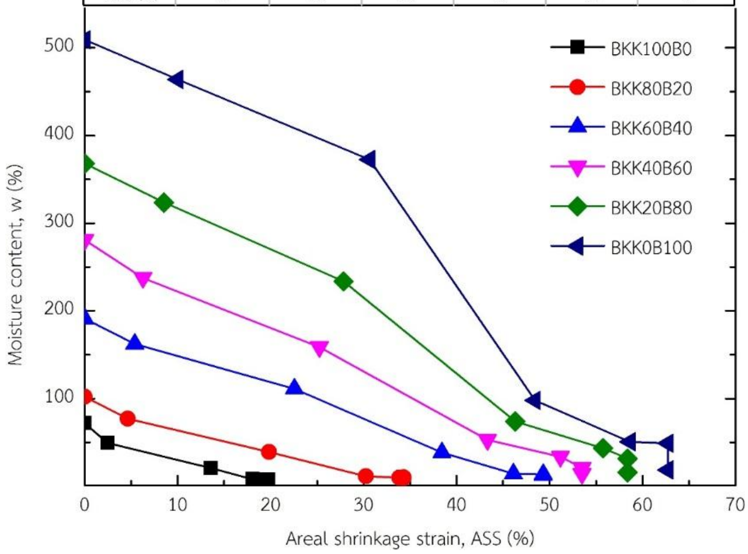


Figure 4-17: Areal shrinkage strain (ASS) versus moisture content (*w*) of bentonite and Bangkok clay mixtures.

Lateral movement of an expansive soil not only swell, but also shrinkage. Currently, there is no models are available in the literature to predict areal shrinkage strain and the residual moisture content of expansive soils. Hence, an attempt are made to develop correlations between areal shrinkage strain (ASS) and residual moisture content after air drying ($w_{a.d}$) versus LL properties of expansive soils. The correlations between LL and the ASS and that between $w_{a.d}$ and LL of the expansive soils are made and presented in Figure 4-18(a) and (b), respectively. The relationships are expressed by the following equations:

$$LL = 24.71\, e^{0.046(ASS)} \tag{4-9}$$

$$LL = 16.16 e^{0.193(w_{a.d})} \tag{4-10}$$

It is clear that with higher LL, higher $w_{a.d}$ and ASS were obtained. The ASS and $w_{a.d}$ decrease exponentially with the decrease of LL in a normal axis scale. The decrease of the above properties is directly related to the decrease in the amount of expansive clay content. The above correlations presented high coefficient of determination values, suggesting that these are good correlations. The results also indicated that both areal shrinkage strain and the residual moisture content show a strong dependency of the LL value of the expansive soils, and these trends appeared to display exponential behaviour. Overall, even the trends shows a good relationship between the ASS and $w_{a.d}$ versus LL, more soil with different LL values than those used in this research would be needed to enhance the developed areal shrinkage strain and the residual moisture content correlations. On the other hand, the test with varying air temperature and humidity should be performed. However, the trends obtained here show that these correlation can be used to estimated approximate areal shrinkage strain and residual moisture content at air drying (temperature 30 ± 1^{o}C and a relative humidity of 50 ± 2 %) of medium to highly expansive soils.

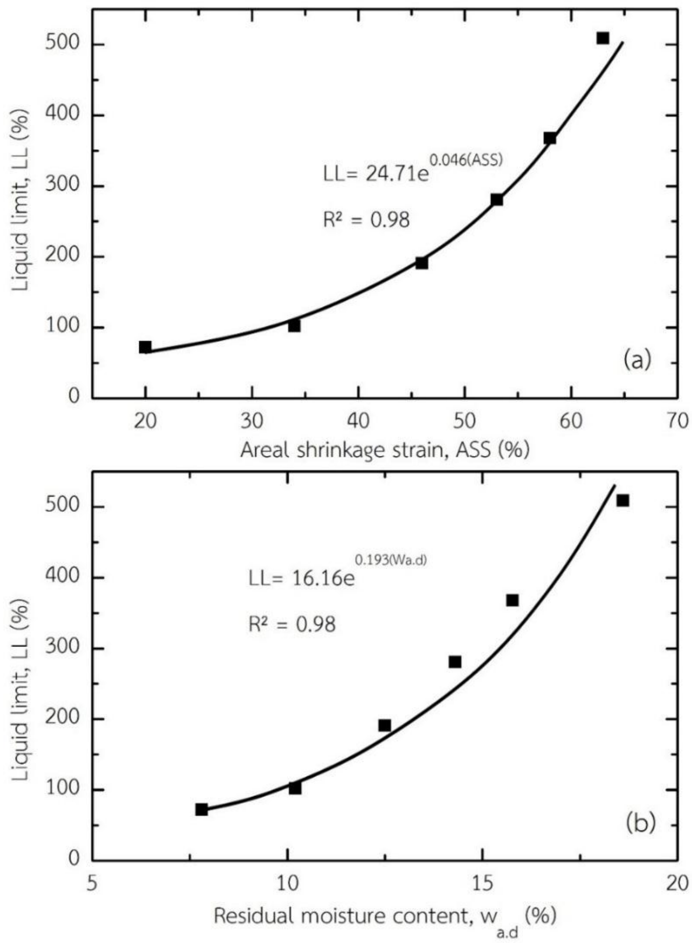


Figure 4-18: Correlation between (a) ASS versus LL and (b) $w_{a.d}$ versus LL of the expansive soils.

4.1.7.2 Volumetric shrinkage strains

A volume change characteristic of expansive soils with different LL set as their initial condition before and after air dried was also measured. The different proportions of bentonite and Bangkok clay with volumetric shrinkage strain (V_{ss}), including the results of ASS in the case of 48h specimen's curing before being placed in to air dry discussed earlier, were plotted in the same figure (Figure 4-19a). Observe that both ASS and V_{ss} decreased with the increases of the amount of BKK (decrease LL) in the soil mixtures. The ASS and V_{ss} decreased parallel to each other with the decrease of LL, as well as the residual moisture content after room-air drying ($w_{a.d}$). For the same soil specimen after room-air drying, the percent of V_{ss} was greater than that of the ASS for all the soil mixtures (Figure 4-19a). The total volume of moisture loss increased with increases in the initial moisture content. However, the final values of residual moisture content ($w_{a.d}$) after the room-air drying obtained is higher for the higher LL soil set as its initial condition. It follows that the amount of removing the moisture content for complete shrinkage will get bigger and consequently the volumetric shrinkage will increase as the initial moisture content increase. The volumetric shrinkage strain are plotted again the areal shrinkage strain as presented in Figure 4-19(b). The relationship shows linearly as represented by the following equation:

$$V_{ss} = 0.86(ASS) + 35.17 \qquad (4\text{-}11)$$

Properties	BKK0B100	BKK20B80	BKK40B60	BKK60B40	BKK80B20	BKK100B0
LL (%)	509	368	281	191	102	72
W.ad (%)	19	16	14	12	10	8

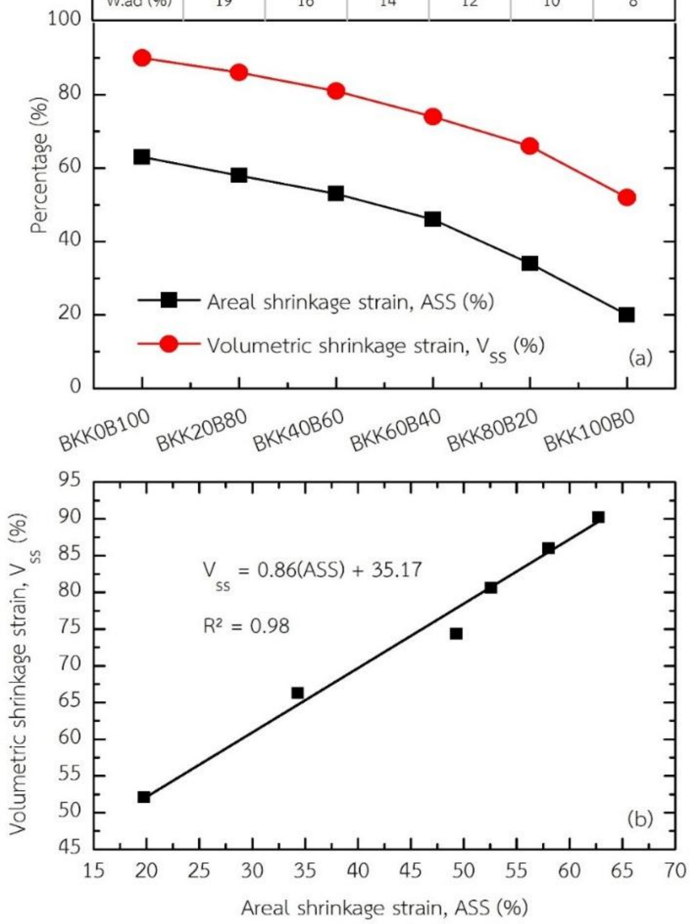


Figure 4-19: Areal and volumetric shrinkage strains at air-dried in a control temperature room 30±1 °C and humidity 50 ±2% for 7 days of bentonite and Bangkok clay mixtures.

It is postulated that V_{ss} increases with increases in initial moisture content. It is also seen that there exists critical volumetric shrinkage strains at moisture content ranges close to their shrinkage limit. The upper limit is less than their plastic limit and the lower limit is about their shrinkage limit. Any further change of moisture content does not cause further change in volume as evidenced by looking at the ASS value, the ASS became a constant value at the residual moisture content ranking between the plastic limit to the shrinkage limit of the soils (Figure 4-17 and Table 3-3). The practical significance of this behaviour is that in natural conditions, due to seasonal changes, especially in the dry season any expansive soil located in a zone beneath a pavement may subjected to changes in moisture content within the critical volumetric change while other zones may be affected by change beyond this critical range. This leads to different settlement or movement of the pavement subgrade, which in turn is the main cause of structural deformation up to the damage. Figure 4-20 show the relationship between volumetric shrinkage strains versus the liquid limit. Observe that the V_{ss} increases with exponential increases in LL of the soil. The relationship can be easily expressed as in the following equation:

$$LL = 3.95e^{0.053(V_{ss})} \tag{4-12}$$

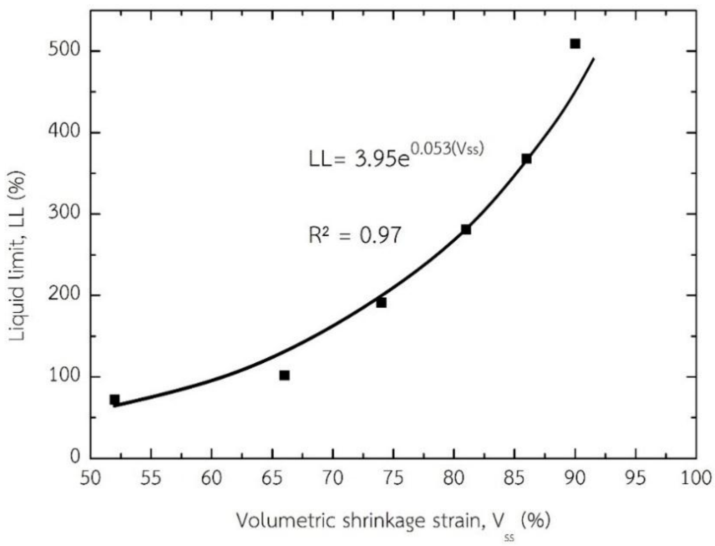


Figure 4-20: Volumetric shrinkage strain versus liquid limit at room-air drying in a control temperature room 30±1 °C and humidity 50 ±2% for 7 days of bentonite and Bangkok clay mixtures.

4.1.8 Relationship between clay Activity value versus shrinkage and swelling strains

Soil consistency limits such as liquid limit (LL), plastic limit (PL), shrinkage limit (SL) and plasticity index (PI) caused by activity value are the indicators of the mineralogical composition of fine particle in soils. Previous investigators established the consistency limits in indicating shrinkage and swelling characteristics of expansive clays, in which the high plasticity indicates the presence high activity values of clay minerals (Chen, 1975; Miller, 1997; Mitchell & Soga, 2005; Seed et al., 1962).

The clay activity value is defined according to the following ratio:

$$Activity, A = \frac{PI}{Clay\,(less than 2\mu m)\%} \qquad (4\text{-}13)$$

For the mixture between bentonite and Bangkok clay, the activity values were ranged from 0.5 to 8.0 (Table 3-3). According to Mitchell (Mitchell & Soga, 2005) the smaller activity value's the clay the lower swelling and shrinkage potential the clay's are. The kaolinite is known as a non-swelling clay which has the same activity value, A = 5.5, to the Bangkok clay as presented in Table 2-4. Hence, the Bangkok clay is also classify as a non-swelling clay (Table 2-4 and 3-3), which has similar plasticity behaviour as the kaolinite clay (Figure 4-3). Figure 4-21 illustrated the relationships between soil plasticity indexes (PI), areal shrinkage strain and vertical free swelling strain (ε_{vf}) with varying clay activity values from 0.5 to 8.0. The PI increase linearly with the increasing of clay activity value as shown in Figure 4-21a. In additional to PI parameters, activity parameters can be used to predict the areal shrinkage and vertical free swelling strains of expansive soils and the relationships can be simply by the following equations:

$$A = 0.018(PI) - 0.38 \qquad (4\text{-}14)$$

$$ASS = 15.13(A) + 33 \qquad (4\text{-}15)$$

$$\varepsilon_{vf} = 101.96(A)^{0.54} \qquad (4\text{-}16)$$

Influence the clay activity to the areal shrinkage strain (ASS) and that the vertical free swelling strain (ε_{vf}) were similar. The percentage of swelling bentonite to be a factor in increasing the value of both, the greater percentage of bentonite in the mixture soils the greater the value of activity (A) and hence the greater the values of ASS and $\boldsymbol{\varepsilon}_{vf}$.

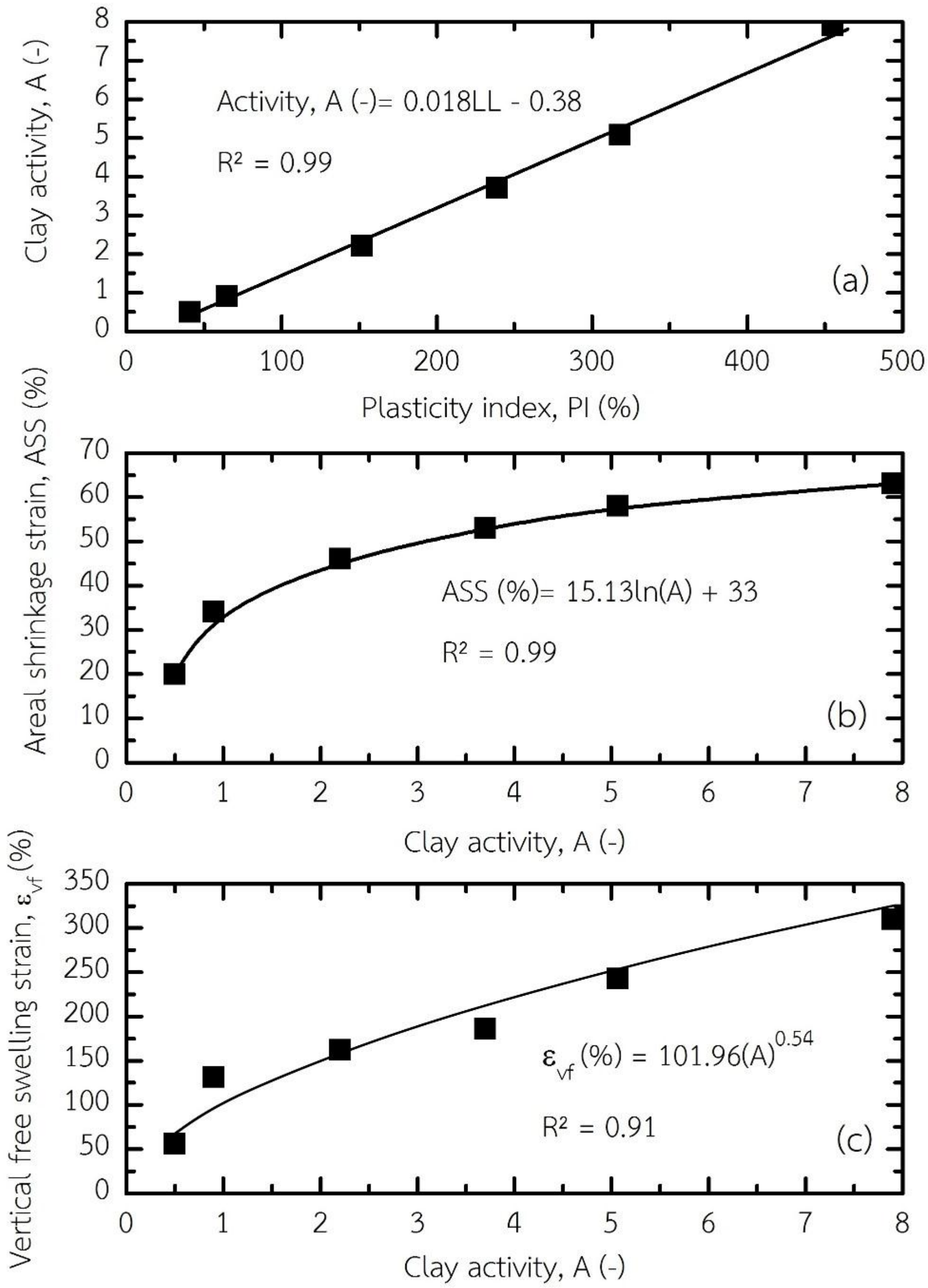


Figure 4-21: Relationship between PI, ASS, and $\boldsymbol{\varepsilon}_{vf}$ versus activity value (A).

4.2 Cement treated expansive soils

The two mixture proportions between bentonite and Bangkok clay (40:60; 20:80) disused earlier were selected to be represented as expansive soils at different liquid limits (Table 4-3), and then treated by 5% and 10% cement by dried weight. The pure Bangkok clay was also investigated for the purpose of comparison. The effect of cement on the index properties, engineering properties and shrinkage and swelling properties are presented in the following sections.

4.2.1 Index properties

Soil index properties were used extensively in various engineering purposes, especially liquid limit, plastic limit and shrinkage limit which was obtained from Atterberg limit test results, are useful properties for estimating the potential of shrinkage-swelling of expansive soil. The Atterberg limit test results of expansive soil treated with 5% and 10% cement content are summarized in Table 4-3 and graphically plotted in Figure 4-22.

Table 4-3: Atterberg limits of cement-treated expansive soils

Sample notation	Atterberg limits			
	LL (%)	PL (%)	PI	SL (%)
BKK100B0	72	31	41	14
BKK100B0C5	52	35	17	21
BKK100B0C10	48	37	11	25
BKK80B20	102	37	65	16
BKK80B20C5	65	41	24	18
BKK80B20C10	57	43	14	26
BKK60B40	191	39	152	12
BKK60B40C5	85	45	40	20
BKK60B40C10	63	48	15	23

Figure 4-22 shows that liquid limit (Figure 4-22a) decreases significantly while the plastic limit (Figure 4-22b) increases gradually with increases in cement content, thereby resulting in a decrease in the plasticity index (Figure 4-22c). A possible reason of the above results is related to the addition of cement, which aids flocculation, and aggregation of the clay particles. As shown in Figure 3-1 and 3-2, cement has a larger grain size than that of bentonite and Bangkok clay. The predominant calcium ions (Table 3-1) from the cement cause a reduction in plasticity, the clay becomes more friable and workable. The first 5% cement introduced to the soil caused the LL and PI to decrease significantly, while an additional 5% cement added to the soil seemed to have a decreased effect. The reduction factor due to the 5% cement added to the soil generally depends on the amount of swelling clay minerals (bentonite content) present in the soil.

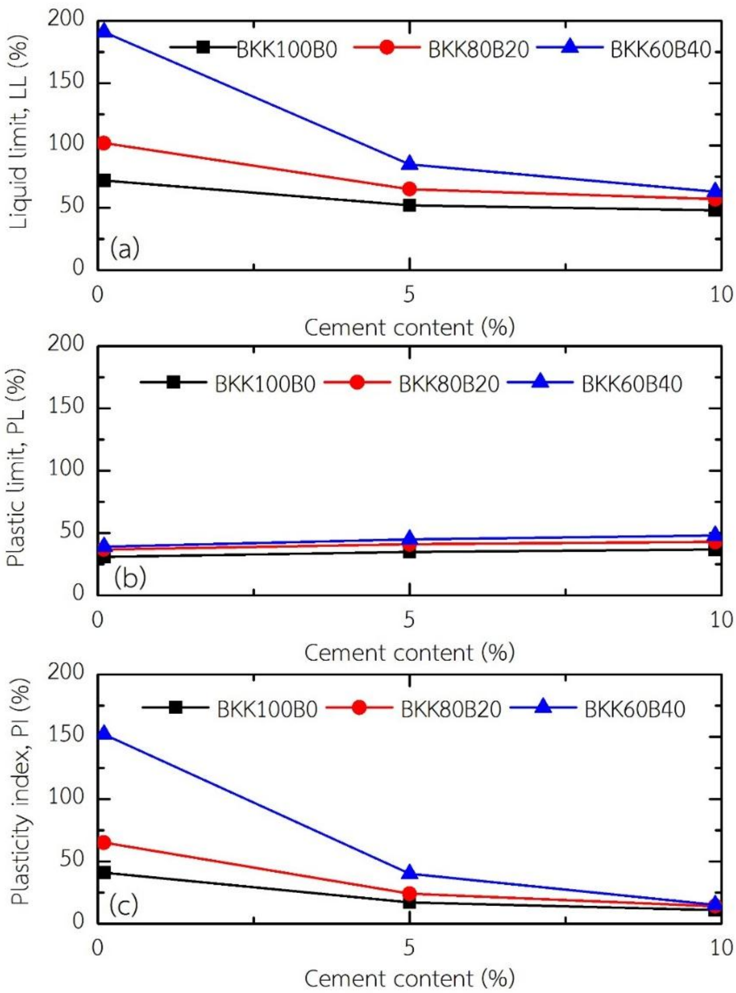


Figure 4-22: Variation of Atterberg limit with cement content.

Figure 4-22 showed that the cement seems to have a large effect on soils with higher levels of LL than that of soils with lower levels. For example, the first 5% cement added to the soil, the pure Bangkok clay (BKK100B0) reduces its liquid limit by 20%, while the higher LL soil, BKK80B20 and BKK60B40, reduced by 37% and 106%, respectively (Table 4-3 and Figure 4-22). Comparing those reductions, the cement seems to be much more effective in higher LL soil than lower. Although the absolute value of LL and PI are smaller for lower LL soil, the reduction of LL and PI due to the cement added is greater for them.

The LL and PI from the Atterberg limit test results are plotted in a plasticity chart, and the changing of the clay particles due to the effect of 5% and 10% cement is illustrated in Figure 4-23. According to ASTM (ASTM-Standard-D2478, 2010) initially all the un-treated soil specimens were clays belonging to the CH group (fat clay). The matching of LL against PI on the plasticity chart for the specimen treated fell below the A-line. For the presenting 5% and 10% cement content, the soil class shifted to elastic silt (MH). This is due to the increase in particle sizes for the agglomerations of soil particles with cement.

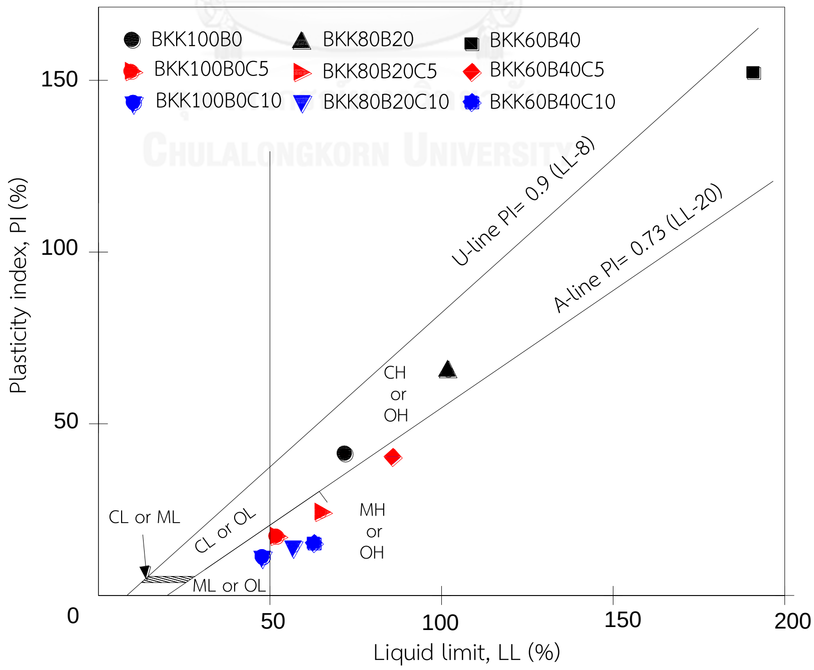


Figure 4-23: Plasticity chart showing the un-treated and cement-treated the expansive clays, bentonite/Bangkok clay mixtures, (after Das & Sobhan, 2013).

4.2.2 Strength and Stress-Strain Characteristics

The stress-strain and strength-cement relationships of the un-treated and cement-treated soils from the unconfined compression tests are plotted in Figure 4-24. From un-treated to cement-treated specimens, the failure pattern changes from ductile to brittle. The stress-strain curves obtained for different mix ratios showed a similar contrast between the un-treated and cement-treated specimens in terms of shape. All the un-treated soil specimens exhibited ductile behaviour with continuous deformation until a steady state was reached at a failure strain (ε_f) of 4.0% to 5.5%. All of the stress-strain curves of the cement-treated soils after 28-day curing had a higher peak and smaller ε_f (2.5% - 2.8%, and 2.1% - 2.5% for 5 and 10% cement adding, respectively) compared with those of the un-treated soils. The lower LL soil (i.e. Bangkok clay) presented higher unconfined compressive strength (q_u) at a lower ε_f in both cases, cement-treated and un-treated soil. The increase of q_u and decrease of ε_f correlated well with the reduction of LL and plasticity index (PI) due to the cement added to the soil (Table 4-3 and Figure 4-24). The reduction in LL and PI caused a significant increase of q_u due to the cement removal of some water that can be absorbed by clay minerals.

The q_u values of all the specimens are plotted against the percentage of cement added in Figure 4-24(b). Comparisons of all the strength improvements due to cement added to the soil with Bangkok clay ($q_u/q_{u.BKK}$) are summarised in Table 4-4 and plotted graphically in Figure 4-25(a). Observe that at the same cement amount in the soil, the strength improvement on the BKK is larger than the other soil mixtures. However, the strength improvement factor (SIF), defined by the ratio between the strength of cement-treated to that the un-treated soil (SIF= $q_u/q_{u.un}$), was greater for higher LL soil (less BKK), as presented in Table 4-4 and Figure 4-25(b). This indicates that, for the soils with higher LL, the cement had a greater effect on improving the strength and decreasing the strain at failure than for the soils with lower LL. Although the absolute value of strength is smaller for higher LL (lower BKK) specimens, the strength improvement ratio is greater for them.

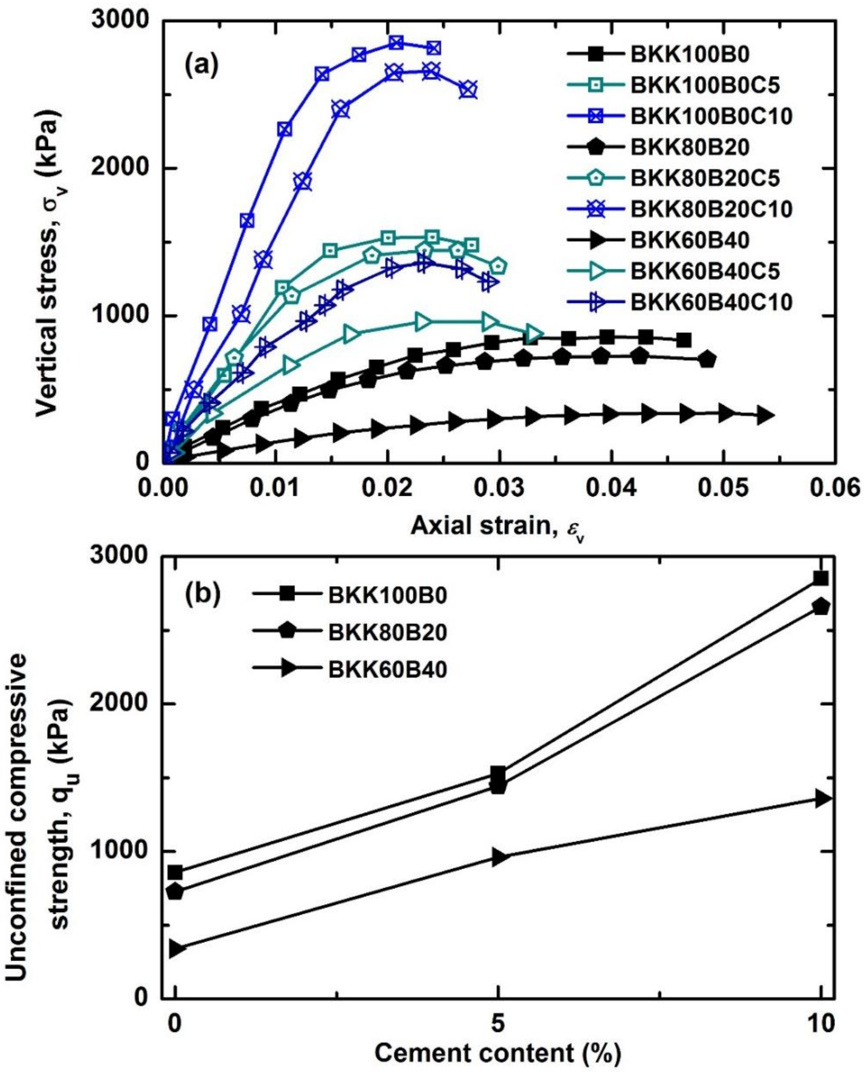


Figure 4-24: (a) Relationships between stress and strain from unconfined compression tests; (b) The unconfined compressive strength of the un-treated and cement-treated soils.

The cement addition led also to an increase of the Young's modulus (E_{50}) with the increase in cement content as summarised in Table 4-4. Studies on stabilised Swedish soils (Åhnberg, 2006) and cement-treated kaolin clays (Raftari, Rashid, Kassim, & Moayedi, 2014) similarly found that, as the strength level increased, the stiffness of the materials also increased. The E_{50} increased by 3 – 5 and 4 – 6 times, while 5 and 10% cement were added to the soils, respectively. This study implies that the addition of considerable amounts of cement to expansive soils increased stiffness, peak strength, and brittleness. The increase of q_u and E_{50} are directly related to decreases in the plasticity and optimum moisture content (OMC) of the expansive soils observed when non-swelling Bangkok clay content in the soil mixtures (Table 3-3 and 4-4) was increased. This may be due to the relative decrease of montmorillonite mineral content by increase non-swelling minerals (Kaolin or Illite, for example), which are present in the Bangkok clay. As reported in the literature (Abuel-Naga et al., 2005; Horpibulsuk et al., 2011; Surarak et al., 2012), Bangkok clay is a low-swelling clay exhibiting physical and engineering properties similar to those of kaolin, while bentonite is a high swelling potential clay, typically 60-90% consisting of montmorillonite minerals (Lee & Shackelford, 2005).

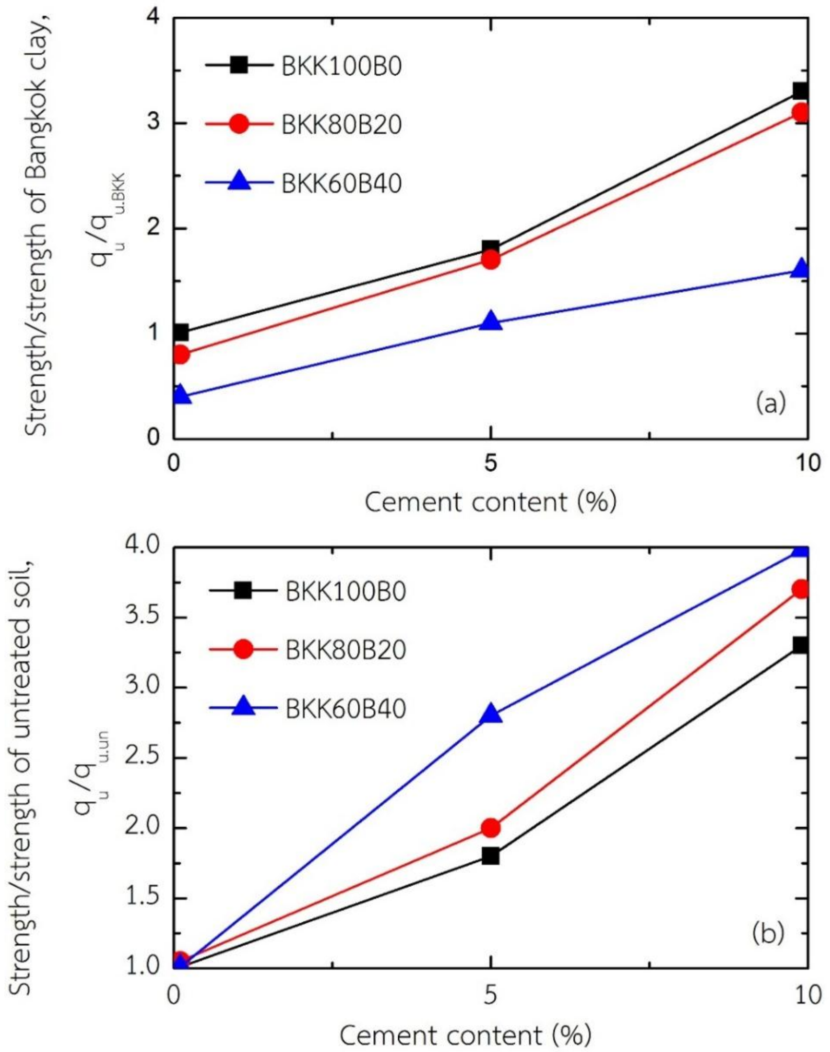


Figure 4-25: (a) Strength/strength of Bangkok clay versus cement content and (b) strength/strength of un-treated soil versus cement content.

Table 4-4: Strength (q_u) and elastic modulus (E_{50}) of cement-treated expansive soils

Sample notation	q_u (kPa)	$q_u/q_{u\text{-}BKK}$ (%)	$q_u/q_{u\text{-}un}$ (%)	E_{50} (MPa)
BKK100B0	856	100	100	40
BKK100B0C5	1,528	179	179	125
BKK100B0C10	2,850	333	333	236
BKK80B20	725	85	100	36
BKK80B20C5	1,443	167	199	119
BKK80B20C10	2,659	311	367	136
BKK60B40	339	40	100	14
BKK60B40C5	960	112	283	66
BKK60B40C10	1,359	159	401	77

4.2.3 Shrinkage Strains

4.2.3.1 Areal Shrinkage Strain

The areal shrinkage strain of cement-treated and untreated expansive soils, both with and without 28 day-curing, was investigated. After air-drying specimens from the initial moisture contents set at their LLs (LL of untreated soil) until reaching stable states, the maximum areal shrinkage strains (ASS) were determined by using an image processing technique as explained in the testing method (section 3.5.4). The areal shrinkage strain of the un-treated and cement-treated soil specimens with and without curing time are presented in Figure 4-26.

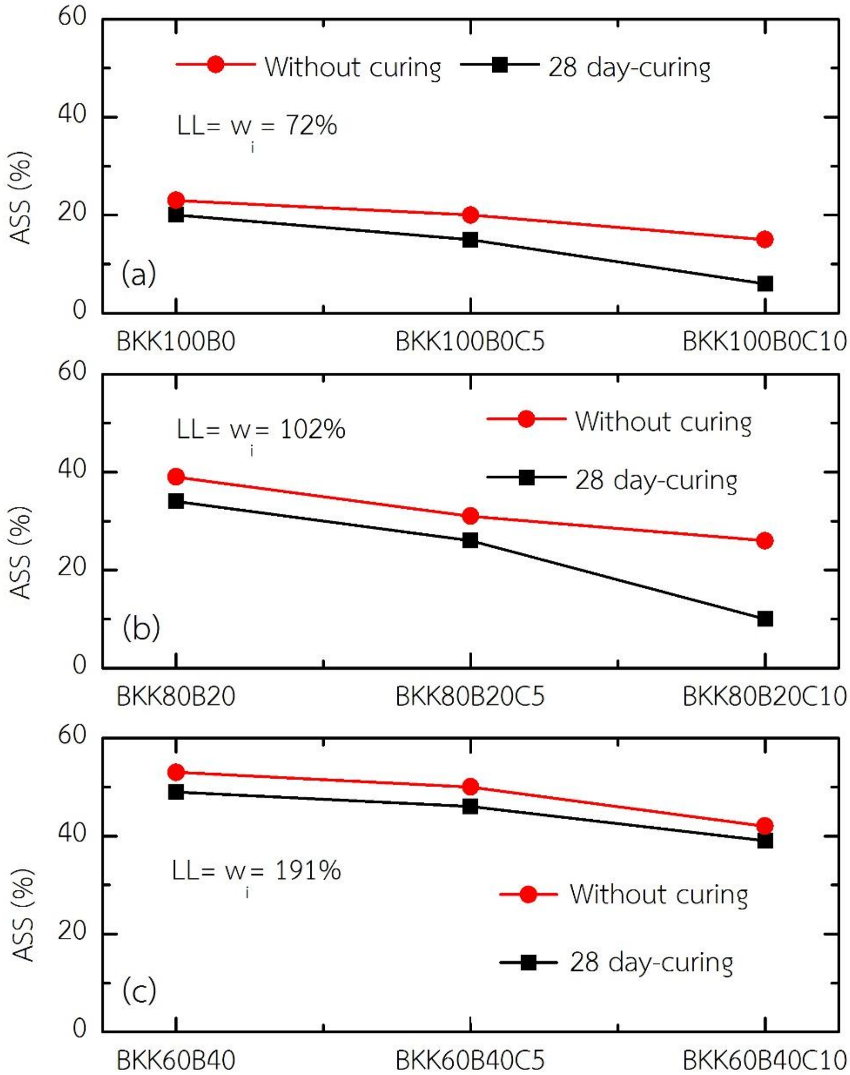


Figure 4-26: Effect of 28 day-curing on the areal shrinkage strain of the cement-treated and un-treated expansive soils.

For both testing conditions, the ASS of bentonite-Bangkok clay mixture decreased with an increase in the BKK as well as the cement content in relation with the decrease of LL. This is because the moisture content at the LL of the clay mixtures, set as the specimen's initial condition, decreased with increasing BKK. In the case of cement-treated soil, plastic limit (PL) remains almost constant, but LL decreased significantly, which mainly reduced the plasticity index (PI) of all the cement-treated soils (Table 4-3). Similar behaviours are also found in cement-treated natural expansive clay (Khemissa & Mahamedi, 2014) and on cement-treated fat and lean clays (Eskişar, Altun, & Kalıpcılar, 2015). It results in a reduction of the soil's liquid limit, i.e. reduction of its

ASS and $w_{a.d}$ as well, even though the initial moisture content of the cement-treated soil was set at the LL of the untreated soil.

All the specimens with 28 day-curing have ASS smaller than of those without curing. The effect of curing time on ASS was greater for the specimens with higher amounts of cement added. For example, the ASS decreased by 5% and 9% after 28 day-curing on the pure Bangkok clay when treated with 5% and 10% cement, respectively. Comparing the three specimens (BKK100B0, BKK80B20 and BKK60B40) when treated with 5% and 10% cement, the largest effect on ASS by curing time was observed for the specimen BKK80B20 (LL= 102%), while the curing effect on specimen BKK60B40 (LL= 191%) was lower. This is indicated in that the greatest effect by curing time of cement-treated the expansive soils on the ASS was observed for the soil with LL levels between 72% to 191% (72% < LL < 191%), while the soil with LL lower than 72% or greater than 191% the effect of curing time of the cement-treated soil on ASS seem less (Figure 4-26).

In the case of 28 day-curing of the soil specimen, the residual moisture content (*w*) during room air-drying after curing was also measured, and the ASS for each moisture state was also determined. The relationship between room air-drying time versus moisture content and ASS were plotted in Figure 4-27(a) and (b), respectively. The rapid moisture content lost was observed in the first 4 days in the air drying and then continued to slightly lose moisture until reaching a stable moisture content at day 7 for the un-treated soils and day 8 for the cement-treated soils. The rapid decrease in moisture content lead to marked increases in ASS of the soil specimens, both with and without cement-treatment. For all the soil specimens during desiccation in the room air-drying, the moisture content lost was slowly processed as well as increased gradually in ASS for the specimens with higher cement content (Figure 4-27).

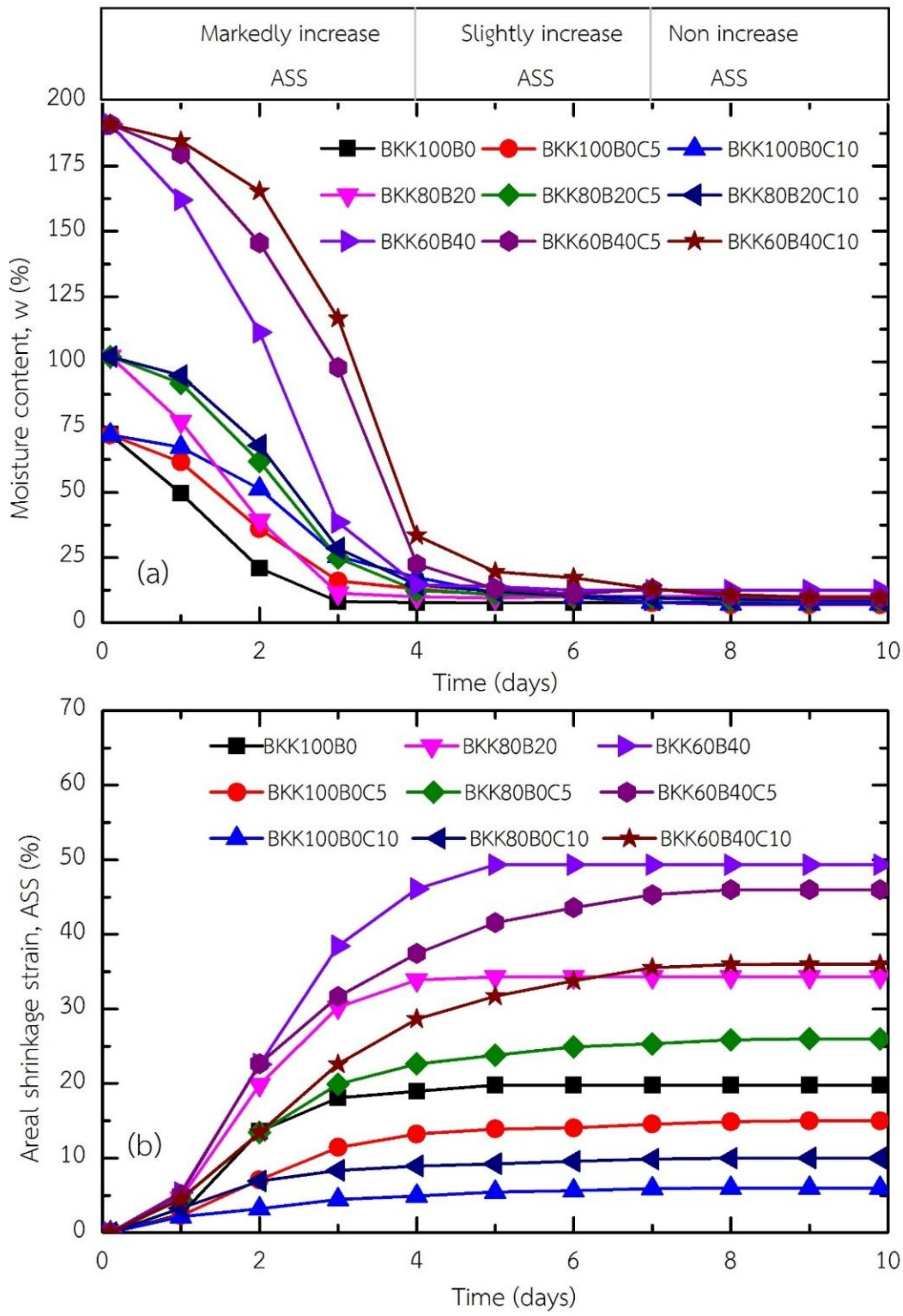


Figure 4-27: Relationship between (a) time versus moisture content, w, and (b) time versus areal shrinkage strain, ASS.

The variation of moisture content during desiccation, starting initially from their LL, of the soil specimens was observed along with the determination of ASS at each moisture content state at room air-drying. The variation of the moisture content (*w*) versus ASS is plotted in Figure 4-28. Figure 4-28(a) shows the relationship between *w* and ASS of the three un-treated soil at different levels of LL (LL= 72%, 102% and 191%) set as their initial condition obtained from the mixtures between bentonite and Bangkok clay as explained earlier. From the figures observed that the relationship shows a slightly concave shape (Figure 4-28a), while the specimens treated with 5% and 10% cement have a convex relationship (Figure 4-28b and c, respectively). The more convex shape is observed for the higher cement content as illustrated in Figure 4-28(b) and (c) for 5% and 10% cement-treated, respectively. The convex shape represents the cement preventing the shrinkage of the expansive soil during desiccation, because the moisture content lost from the soil specimen does not produce as much ASS compared to the untreated soil.

Comparing the same soil specimen between the un-treated to the 5% and 10% cement-treated soil in Figure 4-28(d), (e) and (f), it can clearly be observed that for the un-treated soil, the ASS increases rapidly in respond to the rapid loss of the moisture content during desiccation. However, when the 5% and 10% cement added in to the soil, even though the *w* rapidly decreases, the ASS doesn't increase much. The reduction is attributed to the cement content present in these soils, which enhances the tensile strengths of the treated soils and hence results in reductions of the shrinkage strains. Another reason is due to the presence of cement, LL and PI decreasing significantly (Table 4-3), leading to decreased ASS. Initially, all of the un-treated soil's behaved as fat clay (CH), according to the plasticity chart in Figure 4-23, while when treated by 5% and 10% cement, the behaviour shifted to that of elastic silt (MH), which explains why the ASS decreases with the amount of cement present in the soil.

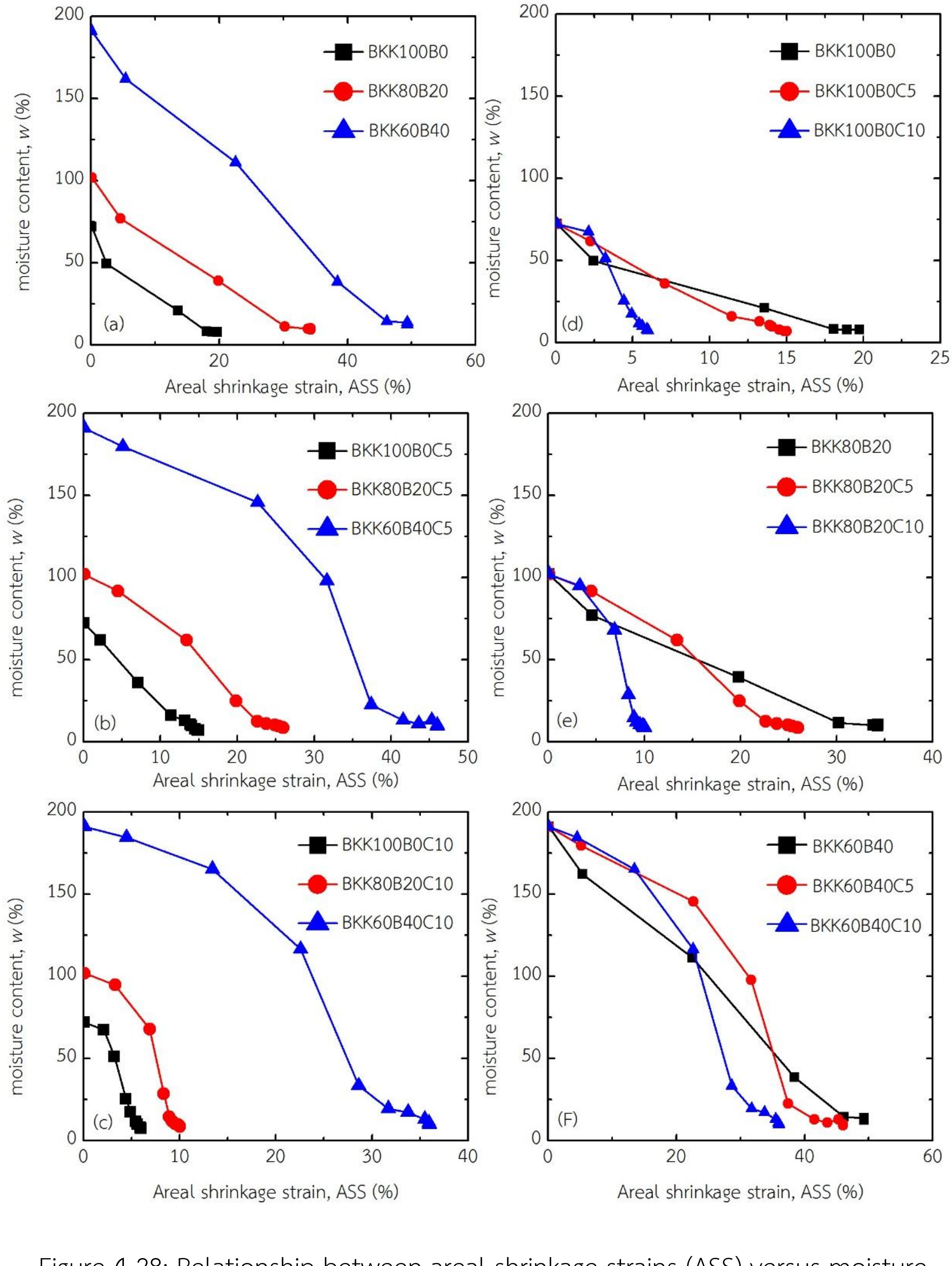


Figure 4-28: Relationship between areal shrinkage strains (ASS) versus moisture content (*w*) of cement-treated and un-treated soils.

4.2.3.2 Volumetric shrinkage strain

The volumetric shrinkage strain (V_{ss}), defined by the ratio of the volume cracked (volume of void after cracked) to the total volume of the soil before air drying (un-cracked specimen), of the untreated and cement-treated soils after the maximum shrinkage of the soil specimen in the room air-drying are plotted in Figure 4-29 along the ASS discussed earlier. From the figures observed, both ASS and V_{ss} decreased with increases of cement content. All the soil specimens of the V_{ss} are greater than that of the ASS. For the pure Bangkok clay (LL= 72%), it was observed that the value of ASS and V_{ss} decreased and tended to be close to each other when increasing the cement content, while for the higher LL soil (LL= 102% and 191% for BKK80B20 and BKK60B40, respectively), the ASS and Vss value decreased parallel to each other with increases of cement content as shown in Figure 4-29(b) and (c). Comparing the slope plotted between ASS versus V_{ss} in Figure 4-29(d) indicated that the cement has a greater effect on preventing the V_{ss} of lower LL soil than that of the higher LL.

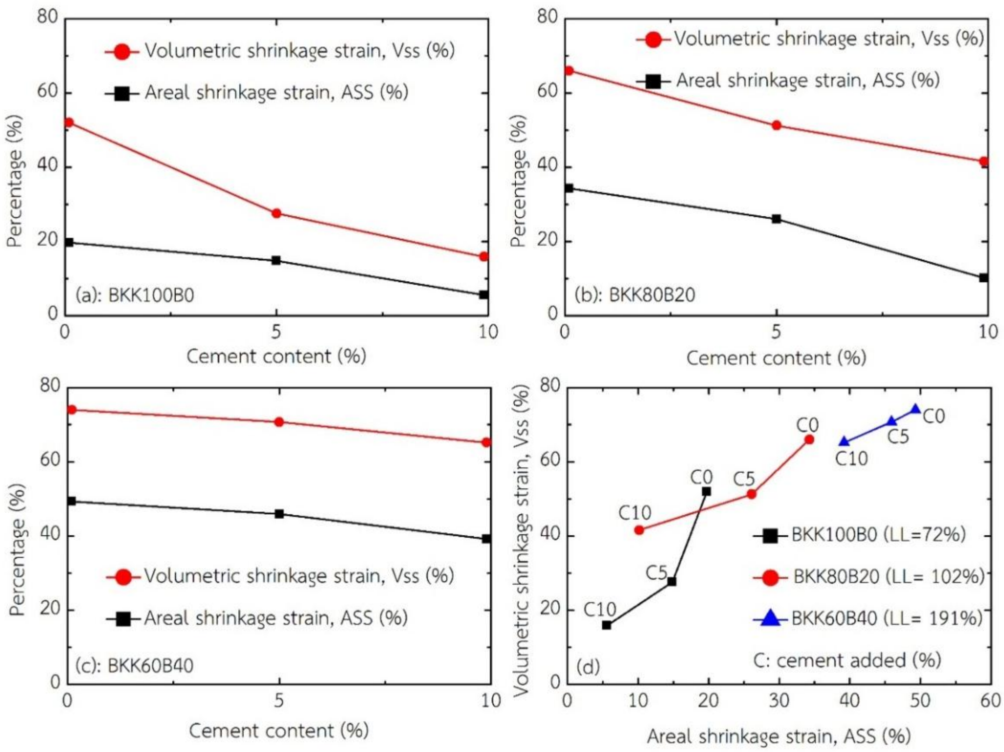


Figure 4-29: Areal and volumetric shrinkage strains of the cement-treated and untreated expansive soils.

4.2.4 Vertical Free Swelling Strain

The vertical free swelling strain (VFSS) was defined as the percentage swell of a laterally confined specimen, which has been submerged in water after being compacted to the maximum dry density and optimum moisture content by the modified compaction test (ASTM-Standard-D1557, 2000). The results of the VFSS tests of the un-treated and cement-treated soils are plotted in Figure 4-30. Figure 4-30(a) shows the variations of VFSS with time for un-treated and cement-treated soils. The early swelling strain rate of higher LL soil was greater than the lower LL soil. With all the soil specimens, the swelling strain occurred at a faster rate, and after the primary swelling was completed (i.e., at the first 5 days of swelling for the pure Bangkok clay, in Figure 4-30a), the rate of swelling continued, but at a slower pace. It was observed that the end of primary swelling for the cement un-treated soil varied between 5 to 10 days, and that markedly decreased with increases of cement content (Figure 4-30). In general, the decreased time needed for completion of the primary swelling as well as to reach the maximum swelling strain was associated with a decrease in swelling clay content (bentonite content) for both with and without cement treatment.

For the un-treated soil specimens, the VFSS decreased significantly with increases in the amount of BKK (i.e. decreased in LL). The relationship between swelling strain and the deformation rate depends significantly on initial moisture content. For the specimens with lower LL (higher BKK), the maximum VFSS was a smaller reach at a shorter period than the higher LL specimen (Figure 4-30a). The longer time required for reaching the eventual strain in lower BKK (higher bentonite content) specimens is because of the lower BKK specimen has lower c_v (Figure 4-9b), possibly due to the very low permeability and the high swelling properties of bentonite (Komine, 2004, 2010). This may be due to the decrease of the montmorillonite mineral content in the soil mixtures by the relative increase of non-swelling mineral content (Kaolin or illite, for example), which are predominant in Bangkok clay. As reported by (Lee & Shackelford, 2005; Pusch & Yong, 2006), bentonite is a clay with high swelling potential consisting of montmorillonite mineral, typically 60% to 90%.

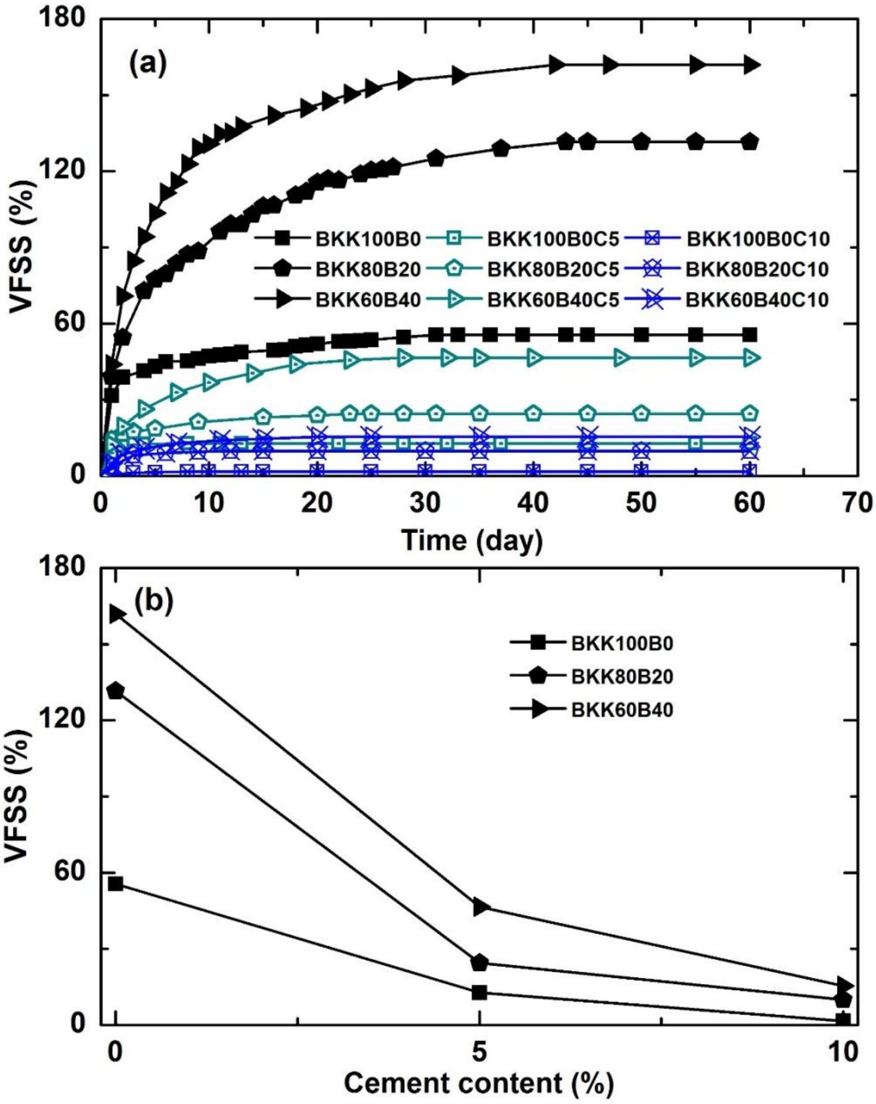


Figure 4-30: Results of vertical free swelling strain (VFSS) tests; (a) Relationships between VFSS and time; (b) Relationships between VFSS and added cement content.

Figure 4-30(b) shows the relationship between the cement content and the maximum VFSS, which demonstrates clearly that increasing the cement addition to the mixture dramatically decreases the VFSS for all the specimens. The rate of reducing the VFSS of the soil specimens was greater with the first 5% cement added, with slightly diminishing returns for additional 5% cement added to the soil. The cement lead also reduced the time for the specimen to reach a stable state, as illustrated in Figure 4-30(a). The time to reach a stable state for the cement-treated soils is governed by the coefficient of consolidation ($c_v = k / m_v \gamma_w$), where *k* is the coefficient of permeability,

m_v is the coefficient of volume compressibility, and γ_w is the unit weight of water. According to (Deng, Yue, Liu, Chen, & Zhang, 2015), cement-treating normally reduces *k*, hence, c_v must have been made greater by an even more significant m_v reduction due to cement addition. Thus, the cement addition leads to larger c_v, and therefore, the time of swelling soils to reach its stable stage decreased. The cement treatment has the positive effect of not only reducing the shrinkage strains, but also of reducing the swelling strains and swelling time to reach its stable stage for expansive soils.

4.2.5 Swelling pressure

Swelling pressure testing is performed on the compacted specimen prepared at optimum moisture content and maximum dry density of the cement un-treated soil following the modified compaction test (ASTM-Standard-D1557, 2000). The measuring of swelling pressures, both in vertical and lateral directions during wetting in a lateral confining ring of a one-dimensional confined swelling pressure apparatus were conducted at varying vertical strains from 0% to 8% with increments of 2% ($\boldsymbol{\varepsilon}_h$=0%; $\boldsymbol{\varepsilon}_v$= 0%, 2%, 4%, 6% and 8%). The variation of vertical and lateral swelling pressures with respect to the time (day) at a confining lateral strain and varying vertical strain, for 5% cement-treated and un-treated expansive soil are plotted in Figure 4-31 and Figure 4-32, respectively. It was observed that swelling pressures, both in the vertical and lateral directions, continued to occur for nearly three days for the pure Bangkok clay (BKK100B0), beyond which there was no change in swelling (Figure 4-31a and Figure 4-32a, for the vertical and lateral swelling pressures, respectively). Similar trends were obtained for tests on the higher LL soils (LL= 102% and 191% for BKK80B20 and BKK60B40, respectively), but the time to reach its stable state was increased with increases in LL of the soil. The time to reach a stable state for the soil with LL= 102% (Figure 4-31c and Figure 4-32c) and 191% (Figure 4-31e and Figure 4-32e) were 8 days and 10 days, respectively.

The rate of swelling pressure was greater for the un-treated soil, while that for the 5% cement-treated the swelling pressure occurred at a slower rate, and more quickly reached its stable state than the un-treated soil (Figure 4-31 and Figure 4-32). The maximum swelling pressure of the cement-treated soil was greatly lower than the un-treated soil. This is because LL and PI decrease significantly in cement-treated soil, (Table 4-3). Decreases of LL and PI lead to marked decreases in vertical free swelling strain (Figure 4-30) and hence, swelling pressure in a confining condition markedly decreased. As shown in the plasticity chart (Figure 4-23), the presence of 5% cement induced the soil specimen which was initially in the class of fat clay (CH) to shift towards being elastic silt (MH) at a lower LL and PI.

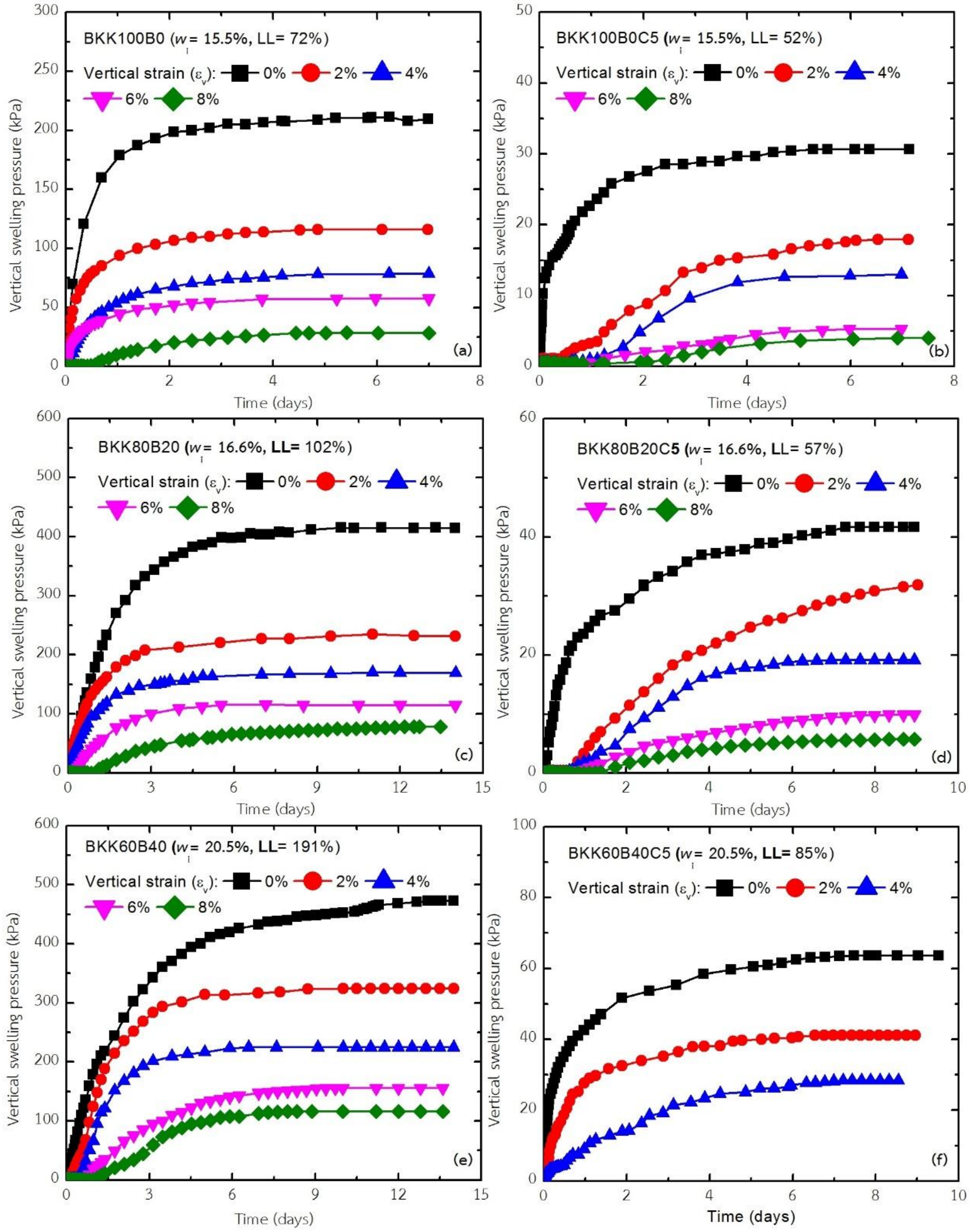


Figure 4-31: Vertical swelling pressure versus time at varying vertical strain from 0% to 8% of a confining lateral stain of 5% cement-treated and un-treated expansive soils.

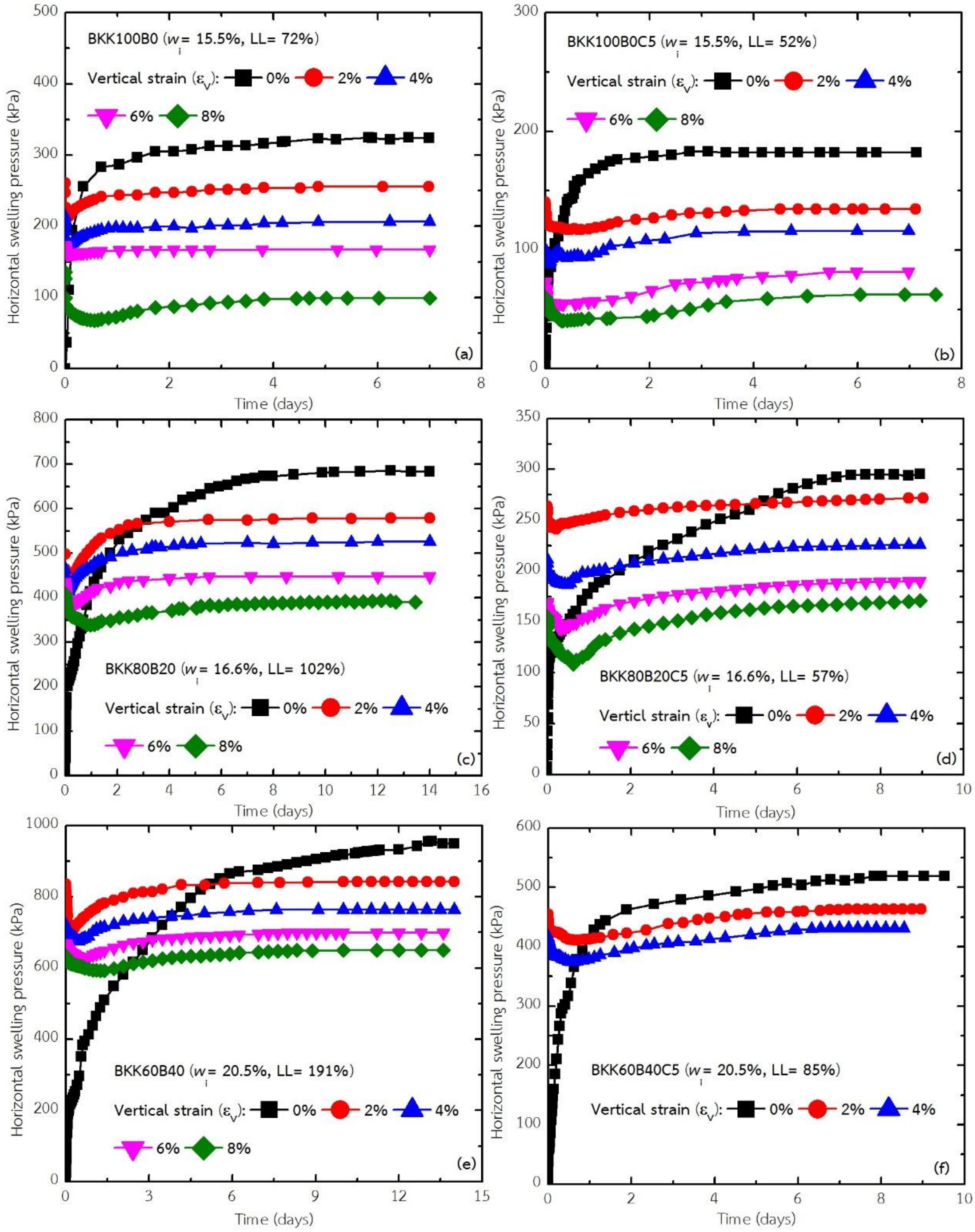


Figure 4-32: Horizontal swelling pressure versus time at varying vertical strain from 0% to 8% of a confining lateral stain of 5% cement-treated and un-treated expansive soils.

The maximum vertical and horizontal swelling pressures recorded at the end of each vertical strain are plotted in Figure 4-33. All the test results show that, for both un-treated and cement-treated specimens, the lateral swelling pressure was higher than the vertical pressure at all stages. For the un-treated specimens, the swelling pressure decreased with an increase in BKK (Figure 4-33) at all stages, possibly due to the smaller amount of swelling minerals, such as montmorillonite, contained in the specimens. Such results are also found in many materials mixed with bentonite or some other expansive soils. For example, (Komine, 1999), (Sun, Cui, & Sun, 2009), and (Cui et al., 2012), who studied bentonite-sand mixtures, reported that the maximum swelling pressure and the maximum swelling strain were lower as the sand content of the mixture became larger. Similar observations were also found in the study on lime-, cement- (Amer Ali Al-Rawas, Hago, & Al-Sarmi, 2005) and fly ash- (Nalbantoğlu, 2004) treated expansive soils. According to (Q. Wang et al., 2012), the swelling pressure of soils depends on the montmorillonite content and the type of exchangeable cations. The cation exchange capacity usually increases as the swelling clay content increases.

After being treated by 5% cement, the swelling pressure in both directions, lateral and vertical, decreased significantly. The cement binds the soil particles together and increases the stiffness of the soil (for example, see E_{50} in Table 4-4). As a result, it leads to reduced swelling under unconfined conditions and, equivalently, reduced swelling pressure during wetting under confined conditions. Figure 4-33(a) and (b) shows that the lateral swelling pressure decreased much less than the vertical swelling pressure while the vertical swelling strain, ε_v, was permitted to develop to 8% in steps. The vertical swelling pressure was reduced to very small values in the cement-treated cases during these steps Figure 4-34(a). Hence, the presence of cement in the soils led to significant increases of the lateral coefficient of earth pressure during one-dimensional swelling, K_s, where $K_s = \sigma'_h/\sigma'_v$. The K_s value of the un-treated and cement-treated soils under one-dimensional swelling ($\boldsymbol{\varepsilon}_h$= 0%; $\boldsymbol{\varepsilon}_v$= 0%, 2%, 4%, 6%, and 8%) were plotted in Figure 4-34(b).

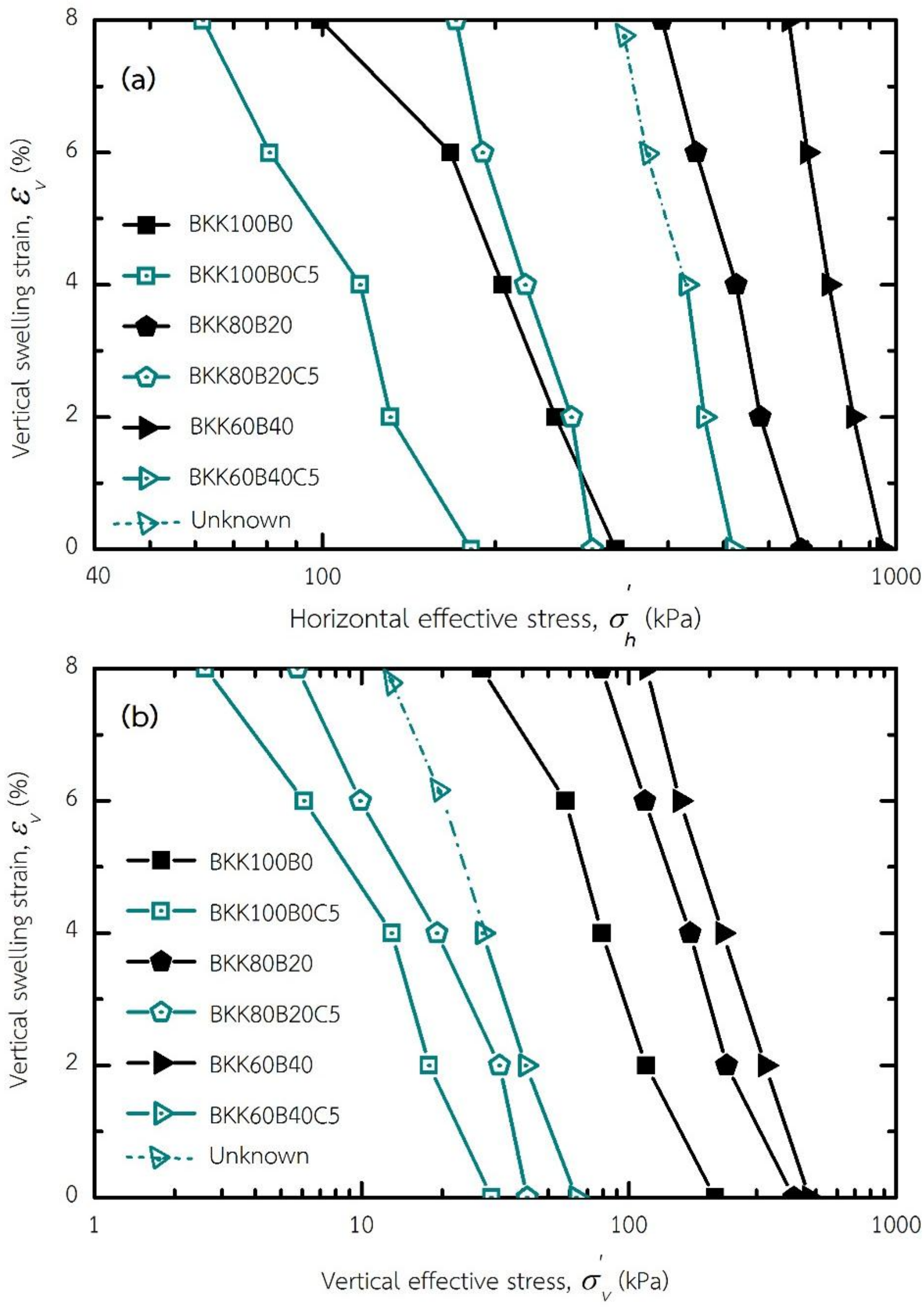


Figure 4-33: Variation of vertical swelling strain versus effective stress of un-treated and 5% cement-treated soils; (a) Vertical swelling strain versus horizontal effective stress; (b) Vertical swelling strain versus vertical effective stress.

The K_s value became larger when 5% cement was added to the soils at all stages of swelling, reaching markedly large values at eventual stages. This is because the cement-treated soils had increased shear stiffness (as represented by E_{50} in Table 4-4) and shear strength (as represented by q_u in Table 4-4). Therefore, while the vertical strain developed, the deformation to readjust the horizontal pressure was significantly subdued, preserving a large part of the initial lateral swelling pressure, which then led eventually to significantly large K_s values. Although the K_s value is higher for the cement-treated soils, the absolute values of horizontal swelling pressure are still smaller than that in the un-treated soils, as shown in Figure 4-34(a). It is hence important that, for the cement-treated expansive clays, any design of earth structure, especially retaining walls, the lateral expansive earth pressure must be assessed not only by a K_s value but by also considering the very rapid decrease of the vertical effective stress upon wetting as any confinement is released.

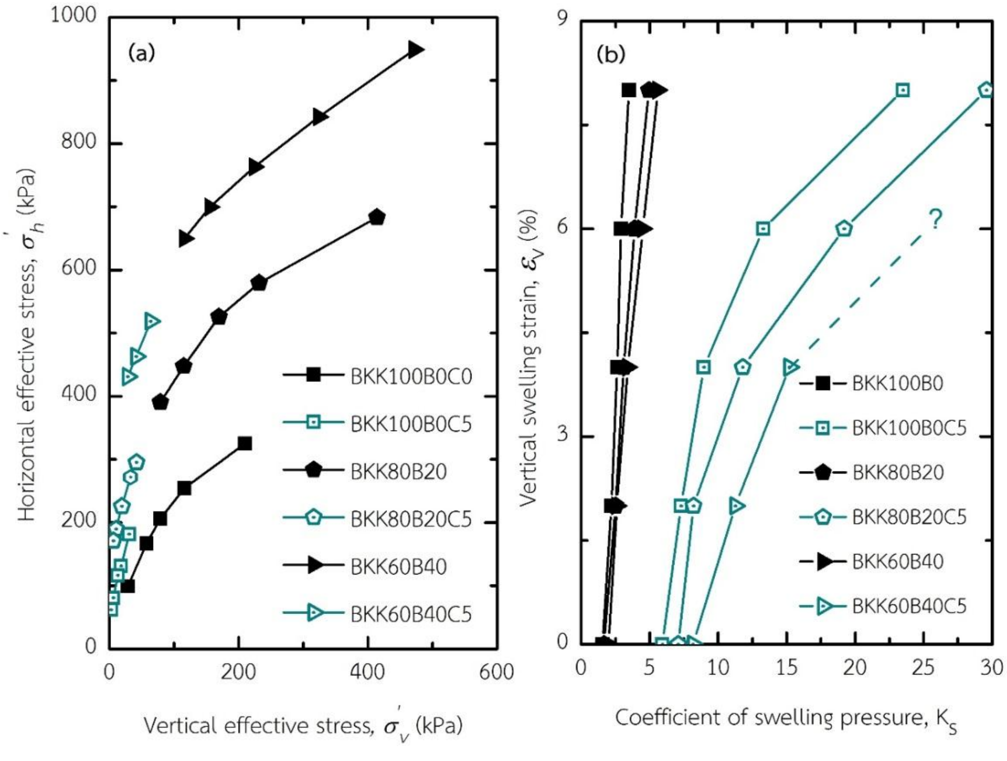


Figure 4-34: Effective stress and lateral coefficient of earth pressure (K_s) of un-treated and 5% cement-treated soils; (a) Relationship between vertical and horizontal effective stress; (b) Variation of lateral coefficient of earth pressure with vertical swelling.

4.2.6 Effective tress path

Typical *q-p'* (the deviator stress, $\sigma_v^{'} - \sigma_h^{'}$, versus the mean effective stress, $1/3(2\sigma_h^{'} + \sigma_v^{'})$ and (1+*e*)-*p'* (the specific volume versus the mean effective stress) diagrams for the un-treated and cement-treated specimens are shown in Figure 4-35(a) and (b), respectively. By permitting the vertical swelling strain to develop from 0% to 8% with the incremental steps of 2%, the un-treated soils' effective stress paths seemed to approach a stress ratio, $\eta = q/p' = 1.0$, while the 5% cement-treated specimens' paths approached the $\eta = 1.4$ line in the *q-p'* plane, as illustrated in Figure 4-35(a). For an axi-symmetric extension, the angle of shearing resistance ($\varphi^{'}$) of the un-treated and cement-treated soils can be simply determined from a relationship between $\varphi^{'}$ and η, $Sin\varphi^{'} = 3\eta/(6-\eta)$, assuming zero effective cohesion. By adding 5% cement to the soils, a significant increase in $\varphi^{'}$, from 41° at the un-treated states to 52° at the treated states, was observed. This occurred due to the produced cementitious materials, which not only binds the clay particles, contributing to the overall increase of shear stiffness, but may also fill the pore space between the clay particles and impart a significant friction factor and hence, increase the angle of shear resistance. So, even with highly expansive materials (Bangkok clay and bentonite mixtures, in this study), the cement has a strong effect on enhancing shear stiffness, as well as increasing the apparent angle of shear resistance, $\varphi^{'}$. Furthermore, it is noted from Figure 4-35(a) that all the effective stress path values are located in the passive zone of the effective stress path plot. This is because of the low values of vertical effective stress (i.e., where the lateral effective stress is greater than the vertical effective stress), and hence, the effective stress path located in the passive zone. The values of the effective stress ratio (K_s = $\Delta\sigma_h'/\Delta\sigma_v'$) discussed earlier increase for the higher LL soil, as well as the presented amount of cement content, which means that the sample was subjected to passive failure during wetting. This investigation highlights the possibility of field passive failure for light weight structures founded at shallow depths, especially for a pavement, shallow foundation and retaining wall structures which are constructed above any expansive soil type if full lateral confinement exists.

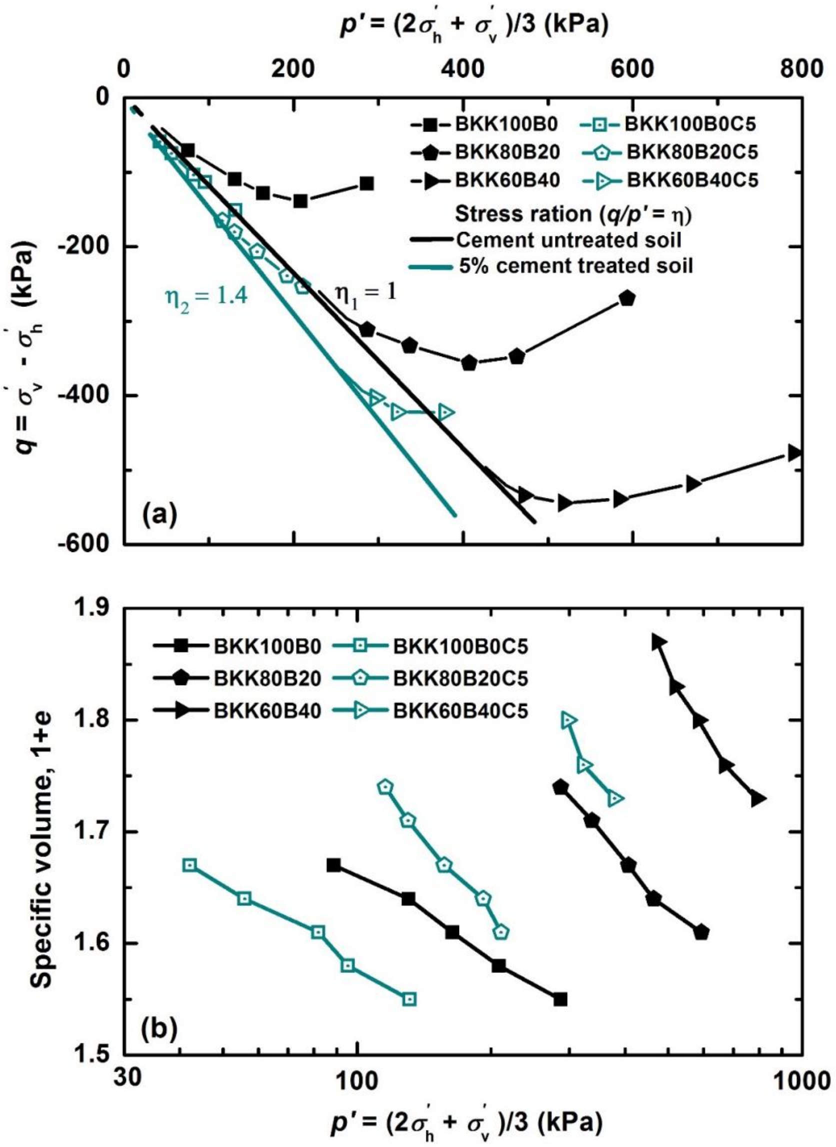


Figure 4-35: Deviator stress, q, and specific volume, 1+e, versus mean effective normal stress, p', of cement-treated and un-treated soils; (a) Effective stress path in the q-p' plane of cement un-treated soils and 5% cement-treated soils; (b) Mean effective normal versus specific volume of cement un-treated soil and 5% cement treated soils.

CHAPTER 5: DISCUSSION OF FINDINGS

5.1 Introduction

Recently, the understanding of the characteristic and behaviours of geotechnical properties of expansive soils during drying and wetting is very important for any construction site which occurring expansive soil. Therefore, a laboratory testing program (Table 3-4 and Table 3-5) were performed to investigate the geotechnical properties of expansive soil with and without cement-treated at various soil consistency limits (LL and PI) using artificially mixtures between Na-montmorillonite bentonite and non-swelling Bangkok clays. The tests were conducted on the soil specimens either by compacted at their optimum moisture content and maximum dry density or mix with water at high moisture content (i.e. close to their LL) as already described in the previous chapter (chapter 3). The results of all the findings are discussed in this chapter. As noted earlier, the present study divided in two sections, cement-untreated and cement-treated expansive soils. The first section the discussion focused on the physical and mechanical behaviour of expansive soils from low to very high expansive soils, including their correlation among the soil parameters. The correlations are established to obtain information on soil geotechnical parameters, in term of fine grain soils, which are costly and time consuming to measure directly. Those are compaction properties (optimum moisture content and maximum dry density) and CBR values, compressibility parameters (compression index, c_c, and swelling index, c_s), shrinkage and swelling strain properties (shrinkage and swelling strain and their volume change) including the swelling pressures in one-dimensional swelling are discussed. The correlation obtained in this study were also compared to other previous researchers who studied on either natural and/or artificially expansive soils and those are presented in the following sections.

5.2. Physical properties of expansive soil

5.2.1 Clay fraction

High percentage of fine particle indicates the specific surface area of clay particle and activity value with in soils (Chen, 1975; Miller, 1997; Mitchell & Soga, 2005). The Bangkok clay has 76% clay fraction (i.e. smaller than 2 μm) while the betonite's is 57% and Portland cement's is 12%. According to (Miller, 1997; Mitchell & Soga, 2005) the higher fine fraction presence in the soil mass the higher potential of shrinkage and swelling soil are obtained. The Bangkok clay has higher fine fraction than that the betonite's, however the Bangkok clay has much smaller vertical free swelling strain ($\boldsymbol{\varepsilon}_{vf}$) and areal shrinkage strain (ASS) than that the betonite's as illustrated in Figure 4-14 and Figure 4-15, respectively. It meant that even the Bangkok clay has a higher percentage of clay fraction than that the bentonite but it presented much smaller shrinkage and swelling potentials than the bentonite.

The fine fraction Bangkok clay may have little effect on soil expansion potential, depending on their mineralogical composition. According to previous researchers (Horpibulsuk et al., 2011; Surarak et al., 2012) Bangkok clay is a clay with low swelling potential exhibiting physical and engineering properties similar to those of kaolin, while Na-montmorillonite bentonite is a clay with a very high swelling potential. The clay mineral composition is much more influential than clay fraction content in dictating shrinkage and swelling potential of expansive clays. The shrinkage and swelling potential of expansive soil are primarily correlated to montmorillonite mineral content in the soil (Amer Ali Al-Rawas et al., 2005; Miller, 1997; Mitchell & Soga, 2005). Several researchers demonstrated that bentonite is a clay with a high shrinkage and swelling potential consisting of montmorillonite mineral typically 60% to 90% (Lee & Shackelford, 2005; Pusch & Yong, 2006). Kaolinite, illite and montmorillonite, in increasing order of activity values, is significant with respect to the potential expansion of soil. Compared to their equivalent montmorillonite can cause much more shrinkage and swelling potential than illite and kaolinite in soils. The Bangkok clay has very low activity value, A= 0.5, compared to the bentonite's, A= 8.0. In natural, the soil could be heterogeneous with mixtures of several clay minerals in their occurring soils. Therefore, control of the clay fraction on soil shrinkage and swelling potential seem to be strictly true only for the same clay mineral varieties. It is not strictly true that all the soil, which content higher percent of clay fraction presence higher shrinkage and swelling potential, as for example the fine fraction of Bangkok clay. Thus, the potential of shrinkage and swelling soil with clay fraction may not be consistence and it is depending on the clay mineral type as well.

5.2.2 Soil consistency limits

Soil consistency limits such as liquid limit (LL), plastic limit (PL), shrinkage limit (SL) and plasticity index (PI) caused by activity value are the indicators of the mineralogical composition of fine particle in soils. Previous investigators established the consistency limits in indicating shrinkage and swelling characteristics of expansive clays, in which the high plasticity indicates the presence high activity values of clay minerals as well as high shrinkage and swelling properties (Chen, 1975; Miller, 1997; Mitchell & Soga, 2005; Seed et al., 1962). In general, the soil contains high swelling clay mineral content, have higher liquid limits and high plasticity index (i.e. bentonite clay LL= 509% and PI= 455%, respectively) while the soil with lower swelling mineral content have lower liquid limits and plasticity index (i.e. Bangkok clay LL= 72% and PI= 41%, respectively). The mixture between the two clay fractions (bentonite and Bangkok clay) leading to decreased LL, PI and PL as the Bangkok clay increase in the mixture clays. The relationship show linearly in a semi-log proportion between bentonite and/or Bangkok clay content versus log moisture content (Figure 4-1). The relationship between LL, PI and PL with BKK presented in Figure 4-1 could be useful in term of primarily estimation the amount of none or low swelling clay to be used for obtaining a variety of soil consistency limits (LL, PI and PL) for highly expansive clay which occurring at any construction site. However, the relationship may valid for fine grain soils with similar nature to Bangkok clay that exist in different parts of the world, while for large grain size soil, the relationship which presented in Figure 4-1 may less applicable. For wider applications, the established relationship should be tested on soils with a wider range of clay fraction from different area.

5.2.3 Swelling strain characteristics

After identifying a soil as being expansive, the next most important step at any construction field is to accurately qualify the amount of volume changed when subjected to variations in moisture content. Reliable estimates of volume change is required for the selection method of treatment alternatives to minimise volume change or in the preparation of a foundation design to accommodate the anticipated volume change. Swelling under a lateral confining strain with vertical free swelling strain on the compacted mixture soils (bentonite and Bangkok clay mixtures) were performed. As expected, the higher magnitudes of vertical free swelling strain was obtained from the specimen with higher bentonite content relative to higher LL and OMC soil (Figure 4-14). The percent of vertical free swell is defined as the ratio of the increase in specimen height to its initial height before it was given free access to water. The maximum percentage of vertical swell takes place under the fully lateral restraint condition. The swelling test also conducted under a surcharge stress of 2.5 kPa in a modified compaction mold, submerging in the water for the CBR test in soaked condition. For the same soil mixture proportion with the same compaction energy and initial moisture content the swelling strain under a 2.5 kPa was significantly smaller than that the free swelling (Figure 4-7 and Figure 4-14, respectively). This is indicated that a 2.5 kPa vertical stress could significantly reduce the vertical swelling strain during wetting of the soil specimen. However, it is observed that the only 60 days of submerging the soil specimen into the water could not allow the specimen to reach its maximum swelling strain except the pure Bangkok clay (BKK100B0). This may be due to the effect of specimen size in which the specimen for the swelling under a surcharge stress was larger and thicker than the free swelling test, which related to the permeability of the soil specimen.

5.2.4 Shrinkage strain characteristic

In contrast to the swelling discussed earlier, shrinkage will occurred when the moisture content drying out. The potential of shrinkage during desiccation depend on the initial moisture content, amount of moisture lost (or residual moisture content) as well as the type of mineral contain in the soil. A comprehensive laboratory investigation on the shrinkage strain (areal shrinkage and volumetric shrinkage strains) of six different soils liquid limit were performed. The relationship between the residual moisture content and areal shrinkage strain is shown in Figure 4-16. The greater the LL soil set as their initial moisture content (w_i) is, the larger the areal shrinkage strain (ASS) occurs as well as higher volumetric shrinkage strain after reach a stable state in the air drying (Figure 4-19). The result also showed that the residual moisture content after air drying is higher for higher LL soil specimen. The higher LL soil is relatively higher swelling soil content (i.e. bentonite). During giving moisture (set at their LL as the initial state) the swelling soil particle swelled. The volume change depending on the amount of swelling soil content in the mixture soils. Therefore, the areal shrinkage strain and the volumetric shrinkage strain are depending on the swelling mineral content and the degree of residual moisture content presence in the soil. The study on the effect of compaction characteristics on shrinkage of expansive clay by Habib (Habib, 2013), showing similar results that the higher moisture content set at the initial stage, the higher volumetric shrinkage is obtained after drying.

5.3 Mechanical properties of expansive soils

5.3.1 Compaction and CBR

Maximum dry density (MDD) is commonly used as an indicator of relative compactness of soil at their optimum moisture content (OMC) with an appropriated compaction energy depend on the testing standard method for application. The study was investigated the compaction characteristic and CBR at different soil plasticity index range from 41% to 455% (PI= 41% to 455%). By using the modified compaction energy (ASTM-Standard-D1557, 2000), the compaction of the mixtures between bentonite and

Bangkok clay, the value of MDD increases (in the range of 14.2 t/m^3 to 17.0 t/m^3) with the decrease of OMC (in the range of 27.0% to 15.5%) depending on the increasing amount of non-swelling Bangkok clay content. It is observed that the above parameters are much influence by the soil plasticity index. There exists a positive relationship among the OMC and MMD to which the soil to be compacted. General results showing that the lower PI soil relatively lower swelling soil content lower OMC and higher MDD are obtained leading to higher CBR value as well.

In any particularly field before or during a construction start, soil located as their foundation either existing or made ground generally in a compacted stage, especially in road construction. The foundation or subbase of the road generally compacted close to their optimum moisture content and maximum dry density to obtain a higher bearing capacity. However during submerging in to water (i.e. in rainy season) the compacted soil foundation will absorbed more water and swell. These phenomena leading the soil lost their bearing capacity due to the soil particle swelling. These can be evidenced by looking to the results of CBR test performed on both soaked and unsoaked conditions. The soaked condition were taken a period of 65 days to simulate its possible weakness condition. The results of CBR on both un-soaked and soaked conditions show that the CBR larger for the specimen with lower PI relative to smaller OMC. The increase of CBR are well correlated to the increased of MDD Figure 4-6. In the un-soaked condition, the CBR value ranged from 9.5% (pure bentonite, PI= 455%) to 29% (pure Bangkok clay, PI= 41%), while in the soaked condition the CBR value of all the soil specimens were similar which less than 2.1%. Overall, for the cohesive soil, the reduction of PI value of the soil has much effect to the CBR value in the case of un-soaked while that in the soaked condition the decreasing of the PI of all the soil specimens does not show any significant change between the higher and lower PI soils.

A study on the effect of soaking on CBR value of sub-base soil was carried out with varying soaking period of 0, 4, 7, 14, 30 and 60 days (Jaleel, 2011). The study reported that, the decrease in soaked CBR value was pronounced in the first days of soaked CBR. The degree of decrease due to soaking is about 69% for 60 days, suggested that the CBR value decreased with increase of period soaking (Jaleel, 2011). Comparing the soaked CBR in this study (Table 4-1) to the previous study discussed earlier, the 65 days soaked CBR, the degree of decrease CBR much more than the previous study. The degree of decrease (reduction ratio, R) due to soaked CBR for all the soil specimens in this study was ranged between 93% and 95% for 65 days soaked period. The higher degree of decrease meant that the soaked CBR value close to zero than that the lower. From this result, geotechnical engineers should be aware of the importance of saturation state, long term soaking period (i.e. 65 days in this study), where the CBR value obtained in the soaked condition was very low (close to zero) compare to the un-soaked. In design of shallow foundation or road subbase, the purpose is to permit the load transfer to the sub soil with safe at a low cost. Economy is the essential item in the design of sub soil to support their super structure or pavement. The greatest possible use should be made of existing ground or locally available material. In the case of existing or available soil is a cohesive soil with high expansive, an additional barrier layer or system drainage should be installed to prevent water flow in to soil subbase or foundation. On the other hand chemical additive (i.e. cement and lime) should be applied to enhance strength and reduce the swelling properties such as swelling strain and swelling pressure of the soil. These could be preventing the reduction of the CBR value of the soil.

The relationships between MDD, OMC and CBR (unsoaked) versus PI of bentonite and Bangkok clay mixtures are showing in Figure 4-5 and Figure 4-6, among them, the correlations between OMC and CBR versus PI (Eqs. 4-3 and 4-4) are tested against the data obtained by previous researchers for various soil types including natural expansive clays as summarised in Table 5-1. Errors in predicted values against observed values, Error= (observed value - predicted value)/observed value, are presented in the same table. The errors in the predicted values of OMC and CRB based on PI compare to the observed values in published studies are within the range of -7% to +15.5% and -7.2% to +11.1%, respectively, as presented in the Table 5-1. This is acceptable accuracy in most preliminary design work, particularly given the wide range of the soil types considered here, including expansive clays and those mixed with non-expansive clays.

Table 5-1: Comparison between observed and predicted values of OMC and CBR by using previous researcher's data on various expansive soils

Soil types and author's name	Observed values		Predicted values	Errors (%)
	OMC (%)=	PI (%)=	OMC (%)= (Eq. 4-3)	
Nagaon soils, India (Talukdar, 2014)	15.1	7.9	13.6	+10.0
	15.2	7.5	13.6	+10.7
	15.8	8.4	13.6	+13.8
Expansive soils (Nayak & Christensen, 1971)	16.0	38.0	15.0	+6.2
	17.0	58.0	15.9	+6.5
Black cotton soil, India (Haricharan, Vinay Kumar, DurgaPrashanth, & AU, 2013)	17.0	24.0	14.4	+15.5
Expansive soil and cohesive non-swelling soil mixtures (Rani & Suresh, 2013)	13.5	25.8	14.5	-7.0
	14.0	10.0	13.7	+2.2
	14.5	43.0	15.2	-5.1
Black cotton soil, India (Kumar, Walia, & Bajaj, 2007)	15.7	24.5	14.4	+8.5
Expansive organic clay (Saride, Puppala, & Chikyala, 2013)	15.7	24.5	14.4	+8.5
	13.5	12	14.0	-2.0
	CBR (%)=	PI (%)=	CBR (%)= (Eq. 4-4)	Error (%)
Expansive clays induced shale (Aghamelu, Odoh, & Egboka, 2011)	31.0	29.0	30.2	2.6
	35.0	27.0	31.1	11.1
	36.0	19.0	36.2	-0.6
	41.0	13.0	42.6	-3.9

Expansive black cotton soil, India (Ogundalu, Oyekan, & Meshida, 2013)	24.0	42.0	25.7	-7.2

5.3.2 Compression and swelling indexes of expansive soil

Figure 4-8 show the e-logσ_v' curves of expansive soils (bentonite and Bangkok clay mixtures with an increment of 20%) for varying soil liquid limit starting from 72% (pure Bangkok clay) to 509% (pure bentonite). The initial and the final void rations at the applying vertical effective stress (σ_v') of the soil specimens were different for different samples having varying Bangkok clay content as swelling pressure was controlled by adding Bangkok clay content which relatively lower LL (Figure 4-8). The higher LL soil presented higher initial void ration as well as higher final void ration at the same applying vertical effective stress. Soil LL is an indicator on compression index (c_c) and swelling index (c_s). The c_c and c_s are the important parameters in predicting settlement and heave of swelling soils which relatively high cost and time consuming for doing the test. Figure 4-12 shows the relationship of c_c and c_s with LL of the expansive soils, where c_c and c_s are obtained from the loading and unloading stage present in Figure 4-8. As the LL of the soil specimen increased, c_c and c_s increased and the relationship can be represented by the equations (4-6) and (4-7), respectively. The relationship also tested using data from previous researchers for either natural or artificial soils (Table 5-2).

Comparing the predicted values (Eq.4-7) to the observed values of c_c by using data of earlier investigators (Abbasi et al., 2012; Desai, Desai, & Rao, 2009; Widodo & Ibrahim, 2012), the errors obtained are within the range of -28.9% to +23.2% as presented in the Table 5-2. The result indicated that most data the error obtained from the predicted and observed c_c values are slightly different in the predictive value of c_c using the equation obtained in this study (Eq.4-7). Suggested that more soil data from different area should be investigate. However, the correlation obtained in this study

(Eq.4-7) could use for any preliminary investigation for any naturally occurring of fine grain soil especially for the soil with high LL.

Table 5-2: Comparison between observed and predicted values of c_c by using previous researchers' data

	Observe values		Predicted values	Errors (%)
	c_c	LL	c_c (Eq.4-7)	
Pontianak soft clay (Widodo & Ibrahim, 2012)	0.279	22.6	0.271	+2.8
	0.349	25.0	0.302	+13.5
	0.511	39.0	0.484	+5.3
	0.511	46.2	0.577	-12.9
	0.663	48.0	0.601	+9.4
	0.663	48.3	0.605	8.8
	0.738	49.0	0.614	+16.8
	0.738	49.4	0.619	+16.1
Fine grain soils (Abbasi et al., 2012)	0.41	31	0.38	+7.3
	0.34	34	0.42	+23.2
	0.38	36	0.445	-17.1
Nagaraj & Srinivashnamuthy soils using Artificial neural network (Desai et al., 2009)	0.65	66	0.835	-27.8
	0.63	64	0.809	-28.2
	0.59	60	0.757	-26.6
	0.55	46	0.575	-3.3
	0.48	45	0.562	-16.7
	1.15	84	1.069	+6.7
	0.54	56	0.705	-28.9

5.4 Physical properties of cement-treated expansive soils

5.4.1 Index properties

As already mention in the chapter materials and methods (chapter 3), three soils specimens (BKK100B0, BKK80B20 and BKK60B40) were selected to represent as expansive soil from low to highly expansive, and then treated by 5 and 10% cement content. The specimen with 5 and 10% cement content that were cured for 28 days were crushed and sieve pass through the No. 40 U.S. sieve. Atterberg limit were conducted on the crushed specimens and the results showing that, the plastic limit gradually increased with a significantly decreased of liquid limit, which mainly reduced plasticity index of all the soil specimens. It is observed from the Table 4-3 that the 5% cement content lead LL of the specimens BKK100B0, BKK80B20 and BKK60B40 decreased by 28%, 36% and 55%, respectively while that the 10% cement content LL decreased by 33%, 44% and 67%, respectively. This is indicated that the cement has greater effect to reduce LL of higher LL soil than that the lower. A study on fine-grain soil with lower LL (LL= 46%) than the LL of the soil using in this study observed that the adding cement content 5 and 10% lead LL increase from 46% to 50% and 52%, respectively (Sarkar et al., 2012).

Similarly behaviour to the earlier study, (Saride et al., 2013) on lime and cement stabilised expansive organic clays (original soil has LL ranged from 28 to 59%), the adding lime and cement contents lead the LL of the soil gradually increased (LL ranged from 30% to 58% and 31% to 64% for lime and cement treated, respectively). Similar to the present study, (Mircea, Irina, & Anghel, 2014) study on cement treated high expansive soil (LL= 86%) reported that the increasing cement content lead LL decreased significantly. Overall, the cement subjected to increase LL of the treated soil for low LL soil (i.e. LL < 60%) and in contrast it lead to decreased LL for the high LL soil (i.e. LL > 72%). The magnitude of decreased in LL due to the cement treated was greater for the greater LL soil (For example in the case of Bangkok clay and bentonite, see Table 4-3). On the other hand, the plot of LL and PI on the plasticity chart presented in Figure 4-23 showing that, initially all the three untreated soil specimens (BKK100B0, BKK80B20 and BKK60B40) were in the class of CH (fat clay) and while it is treated by 5% and 10% cement contents all the soil specimen shift to MH (elastic silt), according to (ASTM-Standard-D2478, 2010).

5.4.2 Shrinkage strain characteristic

The shrinkage of expansive soil influence not only depending on the swelling clay mineral content which relatively high LL soil, but also its initial moisture content as well as curing period. As shown in Figure 4-26, the 28 day-cuing specimen presented smaller ASS than the specimen without curing. From the figure also observed that the lower cement content the effect of curing less than the higher cement content. For example, the 5% and 10% cement added to the pure Bangkok clay, ASS in the case of 28 days curing was 5% and 9% smaller than that the specimen without curing (Figure 4-26). However, the previous researchers (Wayne & David, 2004) reported that excessive amount of cement content can exacerbate shrinkage or cracking due to cement consume much more water during hydration, thus increasing drying shrinkage.

The residual moisture content and ASS during air drying varies with the time of the specimen with and without cement-treated (after 28 day-curing) are plotted in Figure 4-27 and Figure 4-28, respectively. The relationships in Figure 4-27 showing that, the ASS increase with time while the moisture content decrease. The rate of increase in the ASS and the decrease in the moisture content was gradually, and observed on the first day during air drying. A greatly rate of the increasing and decreasing were observed at day 2 to 4. This is because of the soil specimen starting a larger crack at day 2, result the contact area of the soil specimen to the air became larger and hence, higher rate of the decreasing moisture contain and increasing ASS were observed. The increasing rate of ASS drop after day 4 and continues to increase gradually to reach a stable condition at day 7 - 8, even the moisture content continue decrease gradually until reaching its stable condition at day 7 - 10.

Shrinking in expansive soils is interrelated to swelling and thus requires attention in this research. The shrinkage process due to drying may be idealised by two stages; a loss of water without desaturation from the moisture content at its LL (w_L) down to the w_s (the water content at shrinkage limit), followed by desatuartion without volume changes from the w_s down to $w_{a.d}$, which is defined as the residual water content after air-drying. The measured values for $w_{a.d}$ are presented in Table 5-3.

Table 5-3: Residual moisture content ($w_{a.d}$) and areal shrinkage strain (ASS) of cement-treated and untreated soils after room air-drying

Sample notation	ASS (after 7-10 day air-drying)	
	$w_{a.d}$ (%)	ASS (%)
BKK100B0	8	20
BKK100B0C5	7	15
BKK100B0C10	7	6
BKK80B20	10	34
BKK80B20C5	9	26
BKK80B20C10	9	10
BKK60B40	13	49
BKK60B40C5	12	46
BKK60B40C10	12	39

The soils with higher LL, when dried from the w_L, showed higher $w_{a.d}$, as shown in Table 4-3 and Table 5-3. This is because the ability to retain water after air-drying correlates well with the same ability at the LL. Hence, at a same air-drying condition (temperature and humidity), the soils with higher LL presents higher $w_{a.d}$. Under the above idealisation, the volumetric strain due to air-drying is calculated as (w_i - w_s)G_s/(1+w_iGs), where w_i is the initial water content before drying (in this test series, equal to w_L of the untreated specimen). When the strain magnitude is modest, the ASS is calculated as following:

$$ASS = \frac{2(w_i - w_s)G_s}{3(1 + w_i G_s)} \qquad (5\text{-}1)$$

The measured areal shrinkage strains (ASS) of the cement-treated and untreated soils are presented in Figure 5-1 against the ASS estimated based on the above relationship. As the equation suggests, the ASS of the untreated soils with larger w_i = w_L (i.e. larger bentonite content) is significantly larger, reaching 49% for BKK60B40C0, although the three data points for un-treated states did not exactly lie on the 1:1 line due possibly to the approximation and idealisation explained above.

When the cement was added, the ASS decreased notably. When the cement-treated mixtures were remoulded and tested for Atterberg limits, the SL significantly increased. This increase explains part of the ASS reduction via Equation (5-1). However, the effects of the cement addition seen in Figure 5-1 are not fully explained by the SL increase, as the ASS decreased much more than Equation (5-1) estimates (i.e. along the 1:1 line in Figure 5-1). The inter-particle bonding that developed in the cured cement-treated mixtures and resulting stiffness of the soil skeleton played a significant role in reducing the ASS, possibly by resisting the shrinkage and leading to early desaturation during drying.

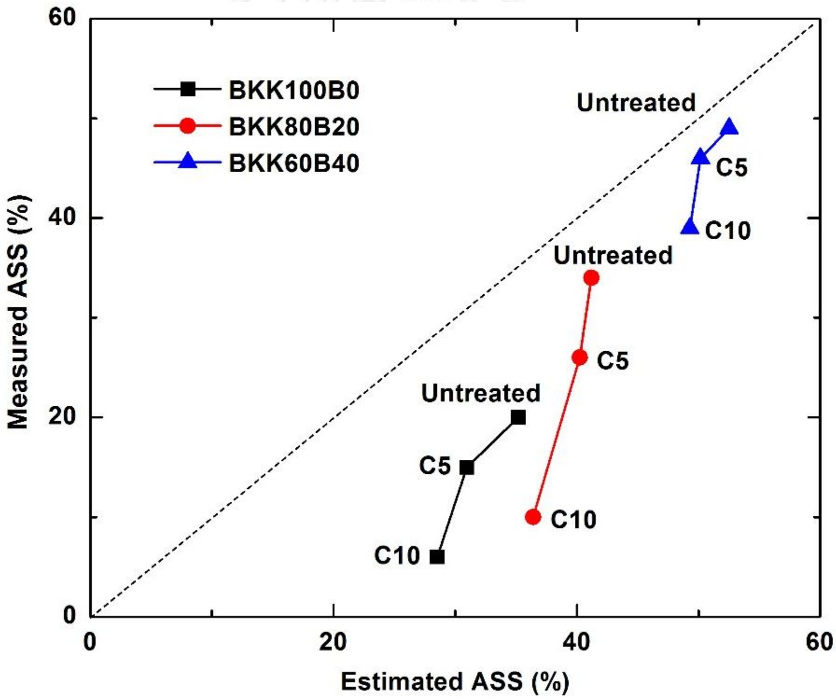


Figure 5-1: Areal shrinkage strain (ASS) of un-treated and cement-treated soils.

5.4.3 Swelling strain characteristic

From the free swelling strain test results in Figure 4-30 showing that the cement has great effect to reduce swelling of expansive soil. The first 5% cement has greater effect to reduce vertical swelling strain as well as the volumetric swelling of the expansive soil than that when another 5% cement added. The reduction of the swelling are well correlated to the greatly reduced of the LL soil while the first 5% cement added. As illustrated in Figure 4-22 showing that the first 5% added to the soil has greater influence to the LL of the soil than another 5% cement added and that could shift the clay class from CH (fat clay) to MH (elastic silt), according to the plasticity chart (ASTM-Standard-D2478, 2010). The swelling test lasted approximately 20, 35 and 45 days for the specimen BKK100B0, BKK80B20 and BKK60B40, respectively. In addition, the cement has great effect not only reduce the swelling strain, but also reduce swelling time of the soil. For example, added 5% and 10% cement to the specimen BKK100B0, the maximum swelling strain was reduced from 56% to 13% and 2% reach at 8 and 5 days, respectively. In all cases for the 5% and 10% cement-treated soil, 20 days were sufficient to ensure that the maximum swelling strain was met. The un-treated soil shows relatively rapid swelling strain with the majority of the swelling strain develop over the first 10 days (Figure 4-30a) of submerging into the water.

5.5 Mechanical properties of cement-treated expansive soils

5.5.1 Strength and stiffness

Strength test results for specimens BKK100B0, BKK80B20 and BKK60B40 with 5% and 10% cement added are shown in Figure 4-24. Figure 4-24(a) show the plots of stress-strain of the three soil specimens with and without cement added. It is noted that the soil with lower OMC and higher maximum dry density showing higher unconfined compressive strength (q_u) as well as Young's modulus (E_{50}) in both case cement-treated and un-treated as presented in Figure 4-4 and Figure 4-24(b). This characteristic is the same as that of cement stabilised low plasticity and coarse-grained soils (Horpibulsuk, Katkan, Sirilerdwattana, & Rachan, 2006), lime treated sandy clay (Consoli et al., 2014), expansive soil modified using pyroclastic dust (Ene & Okagbue, 2009), cement and lime stabilised expansive soil (Khemissa & Mahamedi, 2014), effects of organic matter on strength of compost amended expansive clay (Puppala et al., 2007), these authors reported that the increase of the stabiliser materials the unconfined compressive strength increase. However, among them the effect of organic matter on compost amended expansive soil studied by Puppala reported that the strength properties were enhanced for the compost materials was between 20-40% while the further increase, the strength properties decreased considerably. Suggested that a moderated amount of compost amendment for enhancements in expansive soil strength properties.

Figure 4-24(b) observed that the cement has much effect on the higher Bangkok clay content relatively lower OMC in which the strength increase significantly with the increase of cement content. From the figure, the first 5% cement seem has similar effect to all the soil specimen while another 5% cement added the strength was extremely increased for the lower OMC soil and that gradually for the higher OMC soil (BKK60B40). The two specimens with the lower OMC (BKK100B0 and BKK80B20) has similar behaviours to the study on cement stabilised fine grain soils (Sariosseiri & Muhunthan, 2008), which observed that cement treatment leads to significant increase in unconfined compressive strength and that markedly increased while the cement contents greater than 5%. As seen in Figure 4-24(b), the lowest unconfined compressive strength is achieved with the samples free of cement content. At the first 5% cement content, more ductile behaviour was occurred, with the post-peak stress decreasing gradually with axial strain. At another 5% cement added (10% cement content), the treated specimen becomes much more brittle, with sudden drops in post peak stress with strain. From the strength improvement factor (SIF) summarised in Table 4-4, even the cement produced lower strength for the higher OMC soil, but the strength improvement factor was higher for them. For example, the SIF of the specimens BKK100B0, BKK80B20 and BKK60B40 were increase from 1-3.3, 1-3.7 and 1-4.0 times, while 10% cement were added to the specimens, respectively. Moreover, the E_{50} increased by 3 – 5 and 4 – 6 times, while 5 and 10% cement were added to the soils, respectively. This is indicated that the cement has much effective to increase the strength and stiffness as well as decreased the failure strain even in the case of highly expansive soil.

On the other hand, reported by (AUSTROADS, 2000) on the improvement of swelling clay in road subgrades, a maximum 28 days q_u of 1 MPa (7 days UCS of 0.7 MPa) is considered to be a reasonable criterion for a modified soil to be regarded as improved. On the other hand, a guideline on soil stabilisation for soil cement stabilised in road subgrades (PCA, 1956) recommends that the value of q_u of soil-cement cured for 7 days and 28 days for soils belonging to ML or A-4 (AASHTO classification) should be in the range of 250 kPa to 500 kPa and 2,067 kPa to 6,201 kPa, respectively. Table 4-4 shows that q_u values of the specimens un-treated and treated with 5% cement and cured for 28 days are all less than the PCA requirement. However, a 10% addition of cement increased the strength up to 2,850 kPa, which fulfills the requirements of PCA, while the specimens with 60% BKK (BKK60B40C10) did not reach the range. On the other hand, (Ingles & Metcalf, 1972) recommended that the values of q_u of soil-cement road sub-base and base for light traffic should be in the range of 689 kPa to 1,378 kPa. Table 4-4 shows that all the cement-treated expansive soil specimens fulfill the requirements of soil-cement road sub-base and base for light traffic as proposed by (Ingles & Metcalf, 1972).

5.5.2 Swelling pressure and coefficient of lateral swelling pressure

The series of tests, summarised in Figure 4-31 and Figure 4-32 were conducted in order to measure the effect of controlling vertical strain in steps with a confining lateral strain to the maximum swelling pressure of the soil with and without 5% cement added during wetting. The results implies that at the zero swelling strain (no volume change) the maximum swelling pressure at a stable condition after the specimen stop sucked water, the lateral swelling pressure was larger than that the vertical. When the vertical strain permitted develop in steps (ε_h= 0%; ε_v= 2%, 4%, 6% and 8%), both the lateral and vertical swelling pressure are decreased significantly with increase in the vertical strain. It is noted that the maximum swelling pressure at the vertical direction decreases at a higher rate in comparison to the swell in lateral direction (Figure 4-34) for all the steps. In zero swelling strain (ε_h= 0% and ε_v= 0%) the ratio of lateral swelling pressure to the vertical swelling pressure (K_s) were 1.55, 1.67 and 2.0 for the specimen BKK100B0, BKK80B20 and BKK60B40, respectively. The increasing Ks are directly related to the higher swelling soil content which presented higher OMC and LL. In the case of 5% cement added the LL and PI of the soil were markedly decreased leading all the soil specimens shaft from MH (fat clay) to CH (elastic silt) as presented in Figure 4-23.

The changing of the clay class leading the vertical free swelling strain decreased significantly (Figure 4-30) and hence in the vertical controlling strain the swelling pressure much more decreased. Overall, the K_s level is greatly depending on the present of cement content and the vertical swelling strain level which permitting to develop in step in a confining lateral strain.

The $\sigma_h^{'}$ much greater than the $\sigma_v^{'}$ in both cases, with and without cement-treated, is logic when the vertical strain permitted to develop in step to the maximum vertical strain equal to 8%. This is because the vertical strain developed, the deformation to readjust the horizontal pressure was significantly subdued, preserving a large part of the initial lateral swelling pressure, which then led eventually $\sigma_h^{'}$ much larger than $\sigma_v^{'}$ values and hence, K_s increase.

The relationship between the maximum vertical and lateral swelling pressures versus LL in the case of zero swelling strain ($\boldsymbol{\varepsilon}_h$= 0%; $\boldsymbol{\varepsilon}_v$= 0%) are presented in Figure 5-2. From the figure observed that for the untreated soil (Figure 5-2a), both the vertical and lateral swelling pressures seem to decrease parallel and those decreases with decrease LL of the soil. The swelling pressures gradually decrease for the decreasing LL of the soil range from 191% to 102% when the LL lower than 102% (LL < 102%) the swelling pressure markedly decrease with the decrease LL. Comparing that behaviours to the case of 5% cement-treated soil, due to the LL of the cement-treated soil significantly decrease (Table 4-3) lead to markedly decrease the swelling pressure in both directions, vertical and lateral (Figure 4-34a). The relationship between the swelling vertical and lateral versus LL obtained from 5% cement-treated soil after 28 day-curing are plotted in Figure 5-2(b). From the figure observed that the swelling pressure in the vertical are much affected by the decrease LL of the soil than that the lateral. On the other hand, both vertical and lateral swelling pressures decrease linearly with the decrease of LL in the case of cement-treated soil.

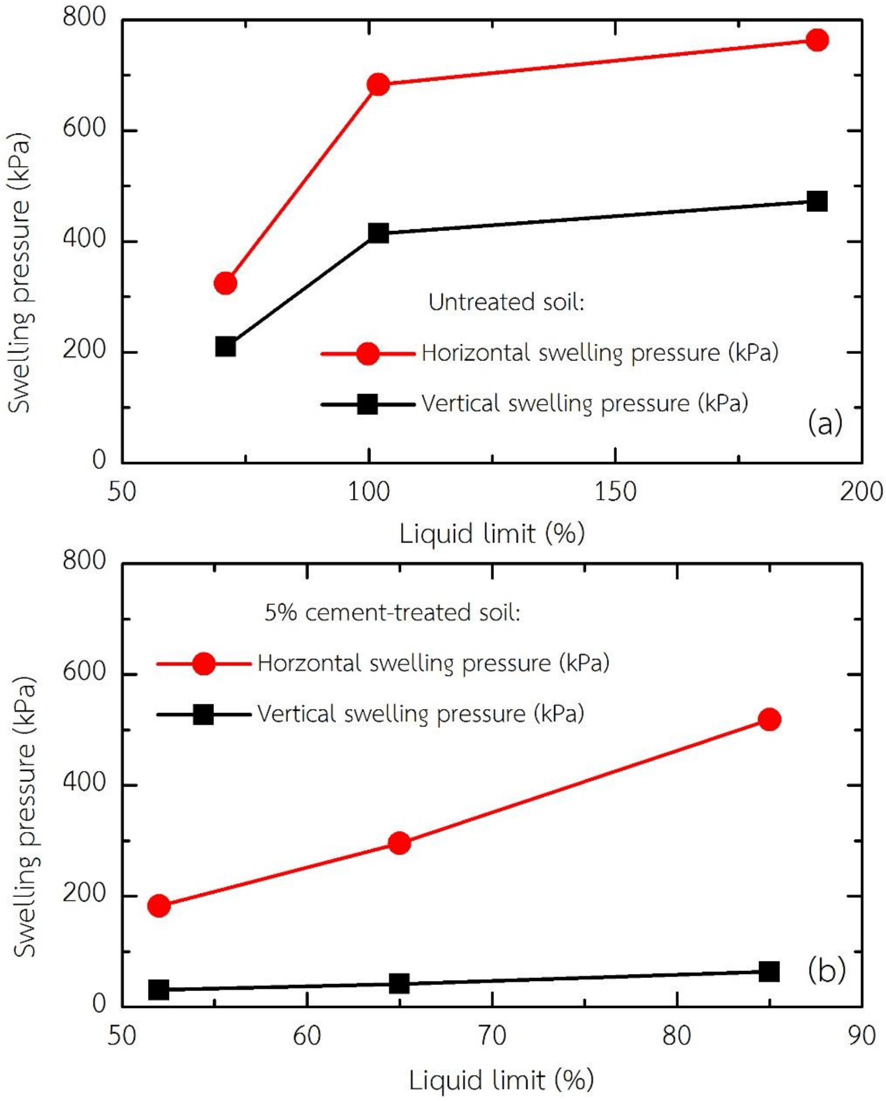


Figure 5-2: Relationship between swelling pressures versus LL of un-treated soil; (b) Relationship between swelling pressures versus LL of 5% cement-treated soil.

5.5.3 Swelling index and bulk modulus from step-loading oedometer and CSP tests

The results of conventional step-loading oedometer tests on reconstituted specimens of the same soils, described in the authors' previous paper (Ingles & Metcalf, 1972), are discussed along with the results of the confined swelling pressure (CSP) tests, reported in the previous section. For the step-loading oedometer tests, the specimens were prepared by consolidating slurried clay, not by compaction. The consolidated specimens submerged into the water were loaded and unloaded at a saturated condition (S_r = 100%), while in the CSP tests, the specimen compacted at its optimum moisture content as the initial state was allowed to freely suck the water from its bottom in the confining ring without applying load at its initial state. The maximum degree of saturation at the stable state for the CSP test (un-treated) was slightly lower than its saturated condition (92% $<S_r<$ 100%), as summarised in Table 5-4. It was observed that the specimen with higher bentonite (higher LL) had lower S_r after reaching the stable state, while 5% cement added, S_r= 100% for all the soil specimens, as calculated by $S_r = wG_s/e$ and illustrated in the Table 5-4. The maximum effective stress from both, conventional steep-loading oedometer and CSP, tests at each degree of saturation obtained after reaching the stable state are plotted in Figure 5-3. The trend of the swelling line, defined by the void ratio, *e,* versus the vertical effective stress, $\sigma_v^{'}$, of the un-treated and cement-treated specimens are compared in this figure. The swelling lines of the un-treated soils for both tests (oedometer and CSP tests) tend to be parallel to each other. The swelling lines of the CSP tests lie under both compression and swelling lines of the specimens reconstituted from slurry. This is because the initial void ratio of the compacted soils prepared for the CSP tests was smaller than the void ratio to which the reconstituted specimens were consolidated in the step-loading oedometer tests. By adding 5% cement to the soils, the slope of the swelling line from the CSP tests became gentler. The measured values of the swelling index, c_s, where $c_s = -\Delta e / \log \sigma_v^{'}$, from the CSP tests are summarized in Table 5-4. The results show that the c_s value decreased with the increase of BKK for un-treated soil. In addition, when 5% cement was added to the soils, the c_s value slightly decreased, approximately by 0.03, as compared to the un-treated soils in all the cases, which means about a 15% decrease in this coefficient for the expansive clay. The decrease may be due to the pozzolanic reaction of clay and cement, causing the resistance of soil against settlement (Ouhadi, Yong, Amiri, & Ouhadi, 2014) or swelling even for expansive clay with high liquid limits. This study indicated that the cement has a strong effect on reducing the swelling index, c_s, even for highly expansive clay.

Table 5-4: Moisture condition and swelling properties of untreated and cement-treated soils from step-loading oedometer and confined swelling pressure tests

<table>
<tr><td rowspan="2">Sample notation</td><td colspan="6">Final moisture and stress condition at vertical swelling strain, ε_v (%)</td><td colspan="3">Swelling properties</td></tr>
<tr><td>-</td><td>0</td><td>2</td><td>4</td><td>6</td><td>8</td><td>c_s</td><td>κ</td><td>K/p'</td></tr>
<tr><td colspan="10">Confine swelling pressure test</td></tr>
<tr><td rowspan="3">BKK100B0</td><td>w (%)</td><td>19.8</td><td>21.2</td><td>22.5</td><td>23.8</td><td>25.4</td><td rowspan="3">0.142</td><td rowspan="3">0.093</td><td rowspan="3">16.7</td></tr>
<tr><td>S_r (%)</td><td>95</td><td>96.3</td><td>97</td><td>97.7</td><td>99.5</td></tr>
<tr><td>p'(kPa)</td><td>286</td><td>209</td><td>164</td><td>131</td><td>75</td></tr>
<tr><td rowspan="3">BKK100B0C5</td><td>w (%)</td><td>22.4</td><td>25.2</td><td>25.5</td><td>25.8</td><td>26</td><td rowspan="3">0.115</td><td rowspan="3">0.082</td><td rowspan="3">19.0</td></tr>
<tr><td>S_r (%)</td><td>100</td><td>100</td><td>100</td><td>100</td><td>100</td></tr>
<tr><td>p'(kPa)</td><td>132</td><td>93</td><td>82</td><td>56</td><td>42</td></tr>
<tr><td rowspan="3">BKK80B20</td><td>w (%)</td><td>21.2</td><td>22.5</td><td>24.3</td><td>25.5</td><td>27</td><td rowspan="3">0.180</td><td rowspan="3">0.162</td><td rowspan="3">9.8</td></tr>
<tr><td>S_r (%)</td><td>93.5</td><td>94.3</td><td>96.9</td><td>97.1</td><td>98.3</td></tr>
<tr><td>p'(kPa)</td><td>564</td><td>463</td><td>407</td><td>337</td><td>286</td></tr>
<tr><td rowspan="3">BKK80B20C5</td><td>w (%)</td><td>23.4</td><td>24.3</td><td>25.5</td><td>26.3</td><td>27.9</td><td rowspan="3">0.150</td><td rowspan="3">0.142</td><td rowspan="3">11.5</td></tr>
<tr><td>S_r (%)</td><td>100</td><td>100</td><td>100</td><td>100</td><td>100</td></tr>
<tr><td>p'(kPa)</td><td>211</td><td>192</td><td>157</td><td>130</td><td>116</td></tr>
<tr><td rowspan="3">BKK60B40</td><td>w (%)</td><td>24.5</td><td>26</td><td>27.5</td><td>29</td><td>30.5</td><td rowspan="3">0.220</td><td rowspan="3">0.202</td><td rowspan="3">9.0</td></tr>
<tr><td>S_r (%)</td><td>92.6</td><td>93.9</td><td>95</td><td>96</td><td>97</td></tr>
<tr><td>p'(kPa)</td><td>790</td><td>669</td><td>583</td><td>518</td><td>472</td></tr>
<tr><td rowspan="3">BKK60B40C5</td><td>w (%)</td><td>26.7</td><td>28.6</td><td>29.3</td><td>-</td><td>-</td><td rowspan="3">0.185</td><td rowspan="3">0.16</td><td rowspan="3">10.5</td></tr>
<tr><td>S_r (%)</td><td>100</td><td>100</td><td>100</td><td>-</td><td>-</td></tr>
<tr><td>p'(kPa)</td><td>367</td><td>322</td><td>296</td><td>-</td><td>-</td></tr>
<tr><td colspan="10">Step-loading oedometer test (Ingles & Metcalf, 1972)</td></tr>
<tr><td>BKK100B0</td><td rowspan="3">σ'_v(kPa)</td><td colspan="5">25-1600</td><td>0.122</td><td>0.06</td><td>27.5</td></tr>
<tr><td>BKK80B20</td><td colspan="5">25-1600</td><td>0.207</td><td>0.10</td><td>18.0</td></tr>
<tr><td>BKK60B40</td><td colspan="5">25-1600</td><td>0.330</td><td>0.17</td><td>9.5</td></tr>
</table>

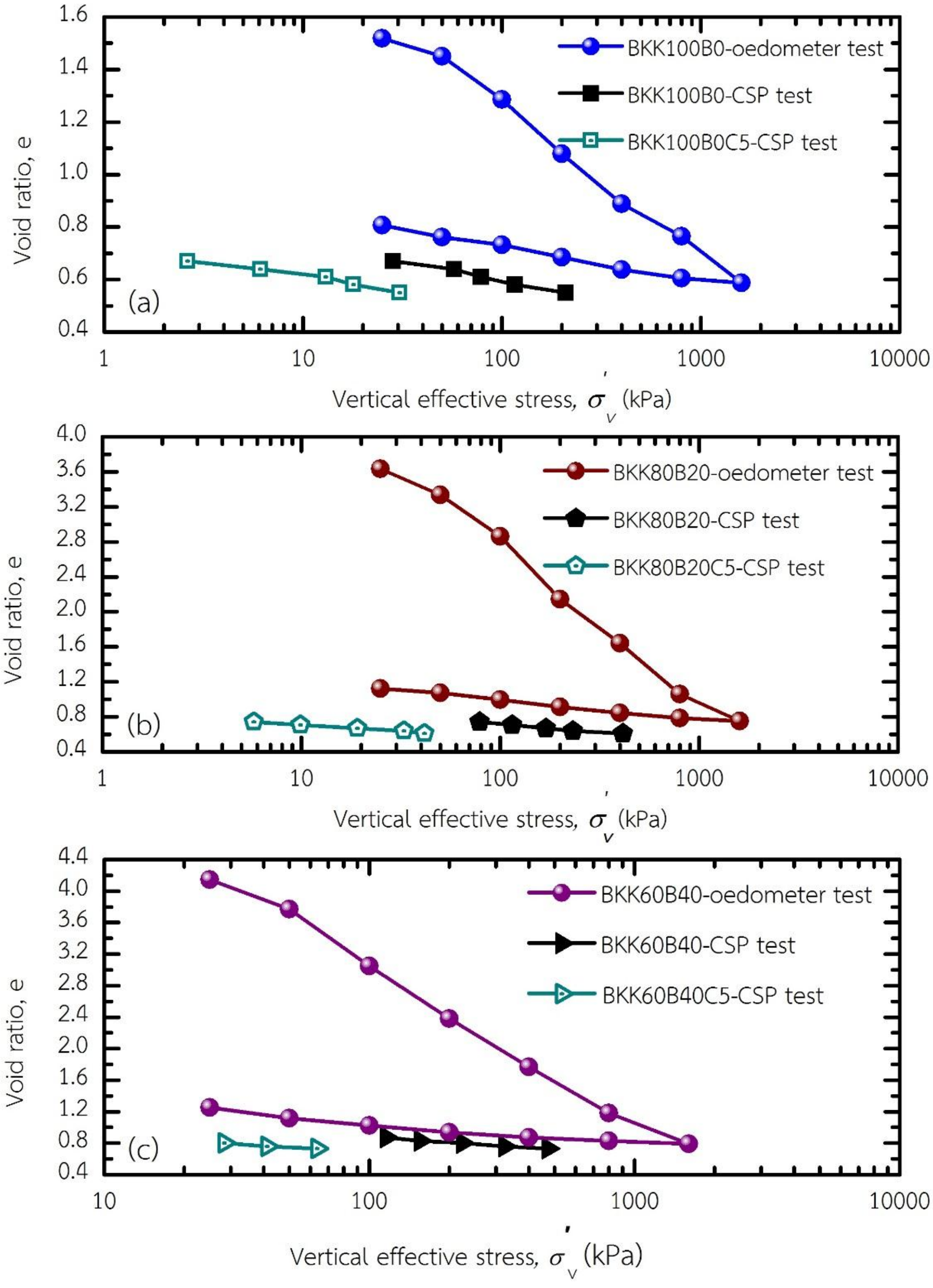


Figure 5-3: Void ratio, *e*, versus vertical effective stress, $\sigma_v^{'}$, of un-treated and 5% cement-treated soils (in conventional step-loading oedometer or confined swelling pressure tests).

From the step-loading oedometer and CSP test results, the mean slope of the swelling line, κ, was determined by $\kappa = -\frac{\Delta e}{\Delta \ln(p')}$, where $\Delta \ln(p') = \ln\left(\frac{p_{\max}'}{p_{\min}'}\right)$ over the swelling range (Figure 5-3b and Table 5-4). The mean effective stress, *p'*, for the CSP test was determined by $p' = 1/3\left(2\sigma_h^{'} + \sigma_v^{'}\right)$ while that for the oedometer test was estimated by $p' = 1/3\left(2K_s\sigma_v^{'} + \sigma_v^{'}\right)$ where K_s is an average of the lateral coefficient of earth pressure over a swelling strain range (ε_h= 0%; ε_h= 0%, 2%, 4%, 6% and 8%) taken from the results of the CSP tests with a same soil (Figure 4-34b). In this study, the *p'* values over which the swelling took place were different in each test, depending on the maximum swelling pressure of each soil specimen registered in the first step of wetting. The elastic bulk modulus, $K = p'(1+e_0)/\kappa$, of the soil specimens is proportional to *p'* when κ is constant. When *p'* is larger, as in the case of higher LL soils (larger bentonite content), a larger K value is obtained for a given κ value. Hence, in order to compare the results of the laboratory tests of all the soil specimens with and without cement treatment, a normalised bulk modulus, $K / p' = (1+e_0)/\kappa$, for both oedometer and CSP tests, was considered. The κ and K/*p'* values are summarised in Table 5-4. The results show that, for the un-treated soil the K/*p'* values decreased when the bentonite content increased for both Oedometer and CSP tests. The rates of the decrease of K/*p'* values were larger when the LL higher for un-treated soil. When 5% cement was added to the soils, K/*p'* increased compared to the un-treated soils. The K/*p'* of the higher LL (more bentonite) soils was smaller than that the lower LL (less bentonite) soils. This indicates that, for soils with higher LL, the cement had a smaller effect on improving the normalised bulk modulus than that for the soils with lower LL. The increase in the normalised bulk modulus is because the cement-treated soils were stiffer (as represented by E_{50} in Table 4-4), leading to a reduction in both vertical and lateral swelling pressure of the expansive soil during wetting.

CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The conclusions of the finding of this dissertation are divided into two main parts. Firstly, focus on the influence of non-swelling clay (Bangkok clay) on physical and mechanical properties of swelling clay (bentonite), when they are mixed together. Secondly, the investigation focus on the influence of cement on deformation characteristics and stress changes of the expansive soils (artificially mixtures of bentonite and Bangkok clay) under swelling. The vertical and lateral swelling pressures were directly measured from the experiments, then the lateral coefficient of earth pressure (K_s) and stress path were investigated. The compacted specimens were prepared for the unconfined compression, vertical free swelling strain and the confined swelling pressure tests; on the other hand, the specimen in the form of powder was uniformly mixed with water for the areal shrinkage strain test. The results obtained from this research can be concluded as following:

1. The liquid limit (LL), plastic limit (PL), and optimum moisture content (OMC) significantly decreased when the Bangkok clay content (BKK) was increased in the mixture, and those properties exhibit in a semi-log linearity against the BKK. A greatest change of the index properties such as LL value was observed when a first 20% dose of Bangkok clay was introduced to the pure bentonite.
2. The strength, maximum dry density (MDD), and CBR values increased with increasing BKK relative lower LL. The strength obtained for the reconstituted samples of the clay mixture with the BKK less than 60% were very low (q_u less than 9.2kPa against a pre-consolidation pressure of 30kPa), which would fall short of any engineering requirement. The best fit trend lines of OMC and CBR versus plasticity index (PI) have been developed. The errors in the predicted values and observed values of OMC and CBR using data of the previous studies on either natural or artificial clays are within the range of -7% to +15.5% and -7.2% to

+11.1%, respectively. This demonstrates that the correlations obtained in this study (Eqs. 4-3 and 4-4) are applicable to other soil types.

3. The compression index (c_c), the vertical coefficient of consolidation (c_v) and the swelling index (c_s) were significantly correlated to the LL and other index properties, which are in turn dependent on the swelling and non-welling clays content. An empirical relationship for compression index, c_c as function of LL established from this study was tested to predict c_c from existing studies on other soil types. The errors in the predicted values were within the range of -28.9% to +23.2% of the measured values. However, the errors presented in Table 5-2 are smaller than the rank in most cases which these errors are acceptable in preliminary prediction and design.
4. The areal shrinkage strains were strongly affected by non-swelling clay content (BKK), decreasing with increasing BKK. The initial water contents and the amount of swelling clay content are the main factors controlling the areal shrinking strain value. The clay mixtures with smaller BKK, with their higher plasticity, experienced large volume changes during swelling and shrinkage. This confirms that such a feature, well established for naturally occurring clays, also applies to a mixture of two distinctly different clays.
5. This study found that many properties of an expansive soil undergo favourable changes when mixed with a modest dose (20-40%) of non-expansive clays. When locally occurring expansive clay ground is to be improved by mixing with transported non-expansive geomaterials, the findings here are useful in exploring an optimum proportions at preliminary design stages.
6. Cement treatment has much more effective to reduced LL of the higher LL soil than that the lower. According to the plasticity chart, the 5% cement content could lead the soil specimens shift from CH (fat clay) to MH (elastic silt).
7. Higher unconfined compressive strength of the specimens is obtained by increasing the cement content and in specimens which has lower OMC relative lower water content. The cement treatment not only enhances the strength and stiffness, but also reduces the swelling and shrinkage potential of the expansive soils. For

example, from this study the 5% cement has increased the "apparent" angle of shearing resistance (φ') of the expansive soils by 11°.

8. The swelling pressures of the expansive soils were markedly decreased when cement was added. The cement has much effect to reduce in the vertical swelling pressure than in the lateral swelling pressure of the expansive soils. Therefore, due to the rapidly decreased in the vertical swelling pressure, K_s increase significantly compared to the untreated soils.
9. More attention should be paid to expansive soils in engineering works. Vertical and lateral swelling pressures are very sensitive to engineering design. In particular, for the cement treatment, any design of earth structure, especially in retaining wall, the engineer should aware the K_s value as well as the reduction of the lateral swelling pressure by considering the very rapid decrease of the vertical stress while compare that to the untreated soils. From a practical point of view, when any surplus expansive soils need to be used for construction, the findings here are guidelines a strategy in exploring an undesirable soils usable or acceptable with modest modification.

6.2 Recommendations

The results of this research provided a better understanding of the behaviour of shrinkage and swelling properties of expansive soil during drying and wetting as well as the compressibility at high initial moisture content. These results are, however, limited to a relatively few number of tests on a single soil. Further effort is still required to determine whether these findings are more generally applicable to other reconstituted and natural soils with different soil and conditions. It would also be advantageous to compare the vertical and volumetric swells measured under triaxial load condition with the volume change (which is also the vertical swell) of samples prepared under the same initial conditions of dry density and moisture content.

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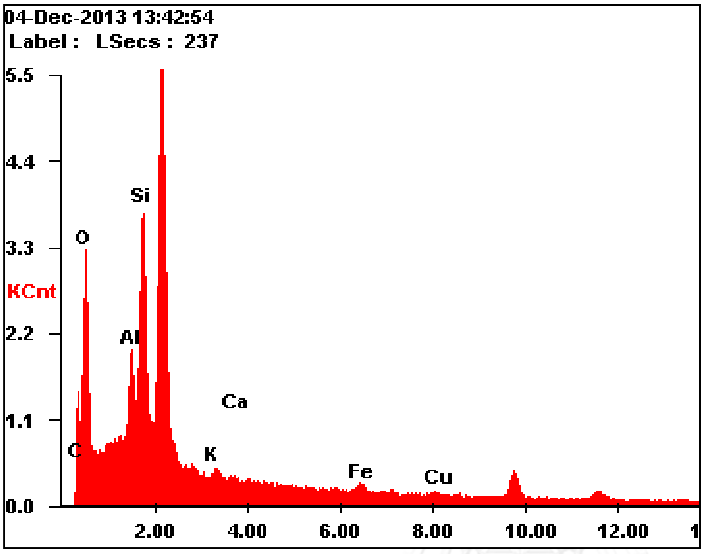
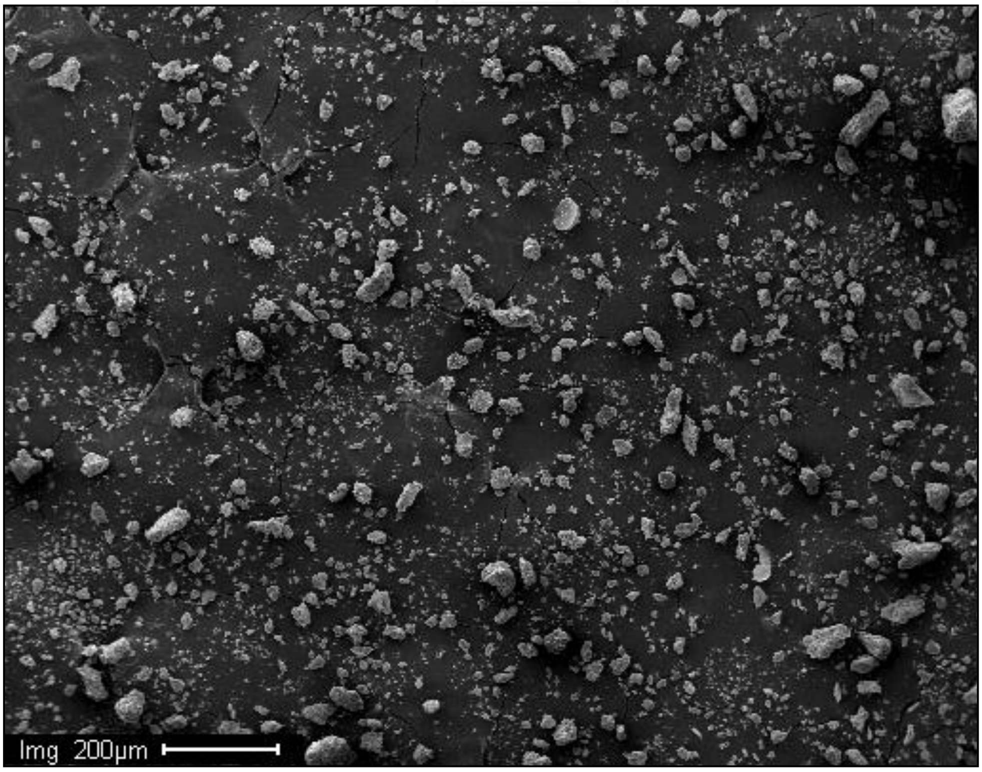
APPENDIX

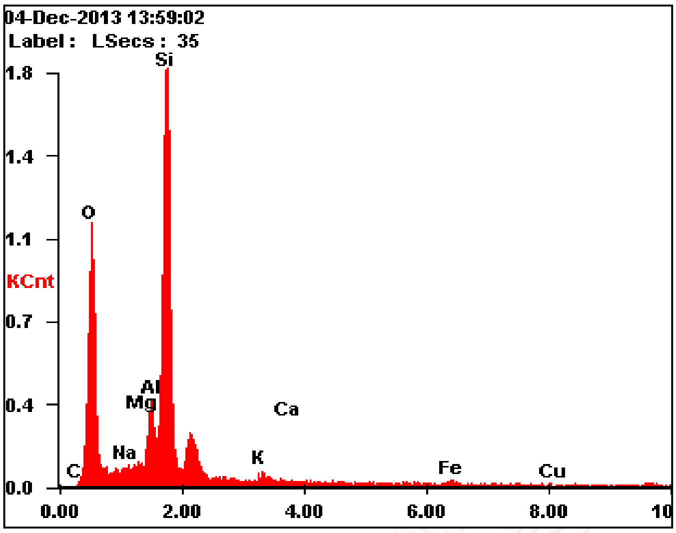
APPENDIX A: XRD & SEM ANALYSIS

1. Bangkok clay

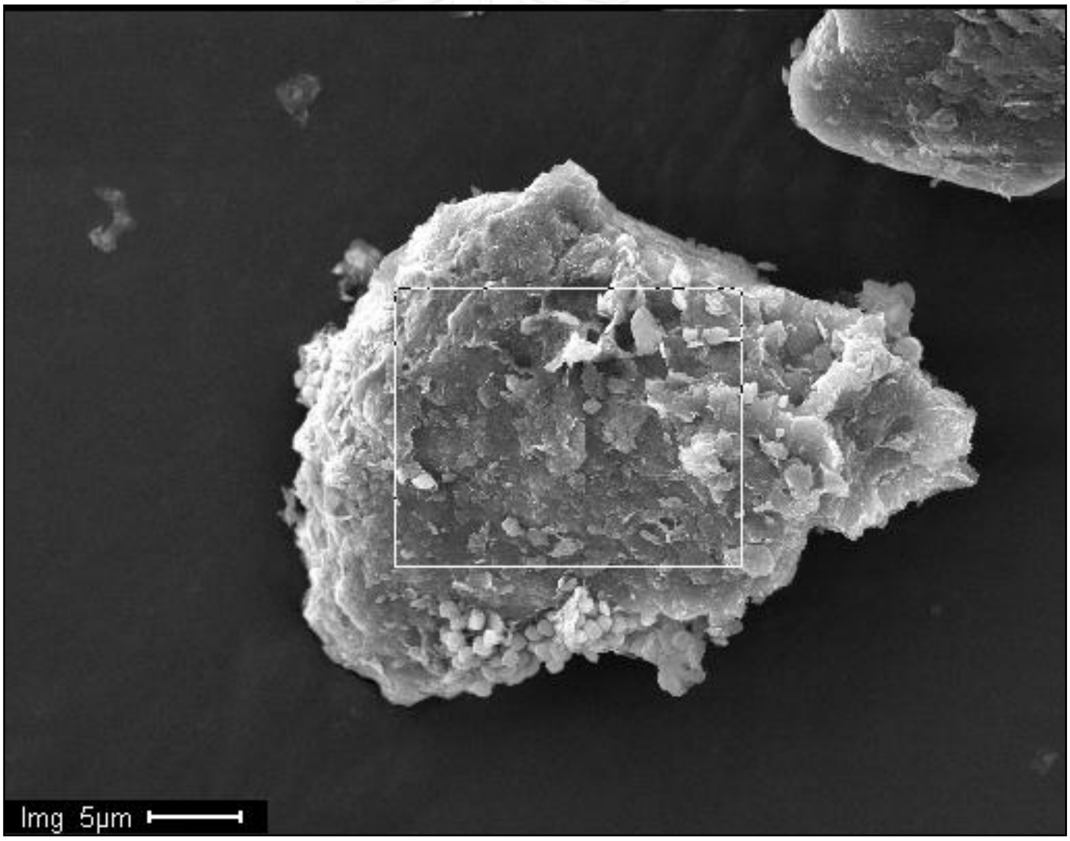


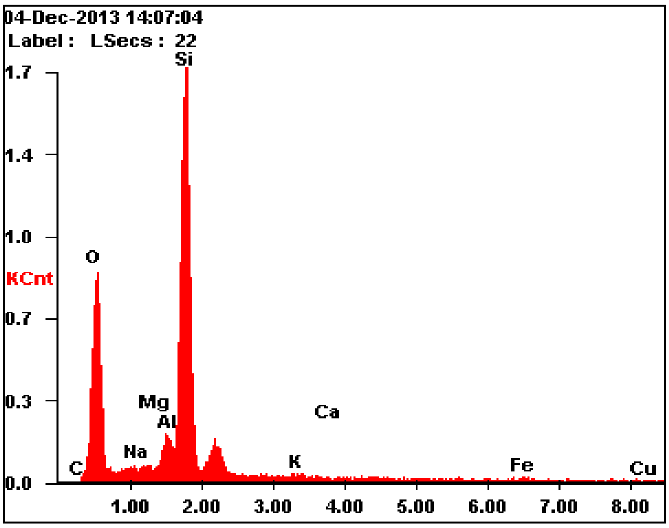
Element	Wt%	At%
C	29.34	41.38
O	39.97	42.31
Al	06.38	04.00
Si	16.70	10.07
K	00.32	00.14
Ca	00.19	00.08
Fe	03.16	00.96
Cu	03.94	01.05



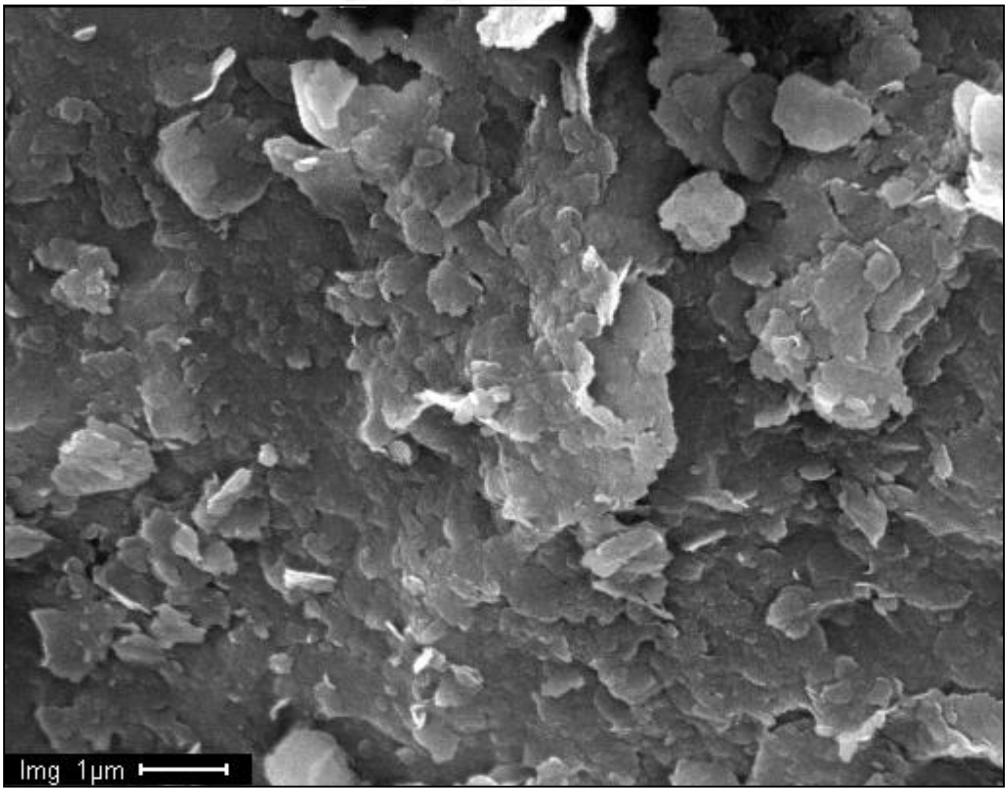


Element	Wt%	At%
C	00.00	00.00
O	46.13	61.06
Na	01.03	00.95
Mg	01.01	00.88
Al	05.96	04.68
Si	39.76	29.98
K	01.09	00.59
Ca	00.31	00.16
Fe	02.93	01.11
Cu	01.77	00.59

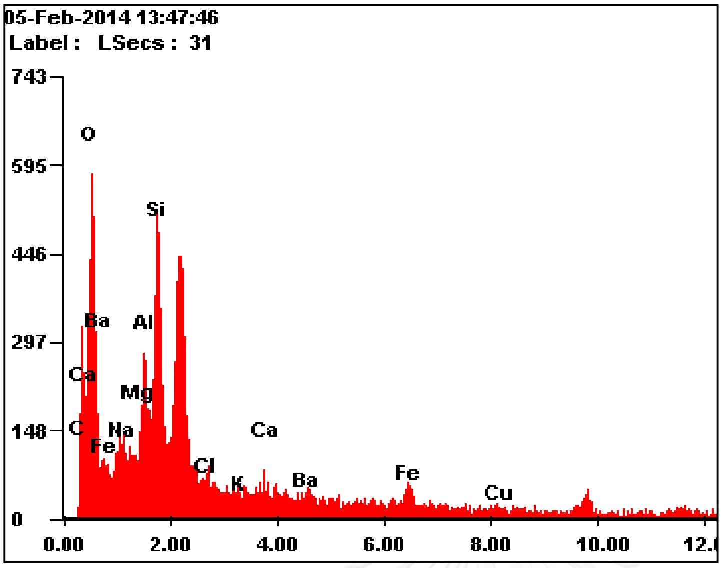




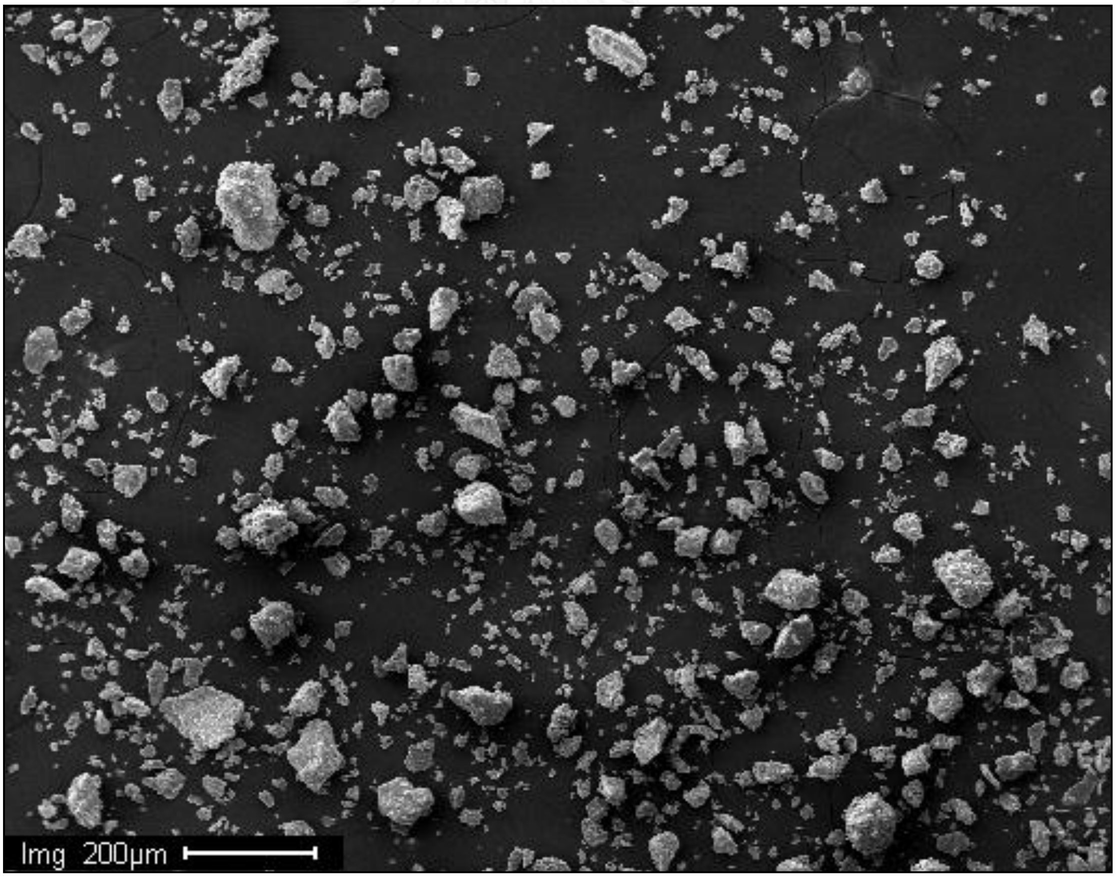
Element	Wt%	At%
C	01.41	02.46
O	43.68	57.32
Na	01.85	01.69
Mg	01.60	01.38
Al	04.33	03.37
Si	43.17	32.27
K	00.56	00.30
Ca	00.00	00.00
Fe	01.81	00.68
Cu	01.58	00.52

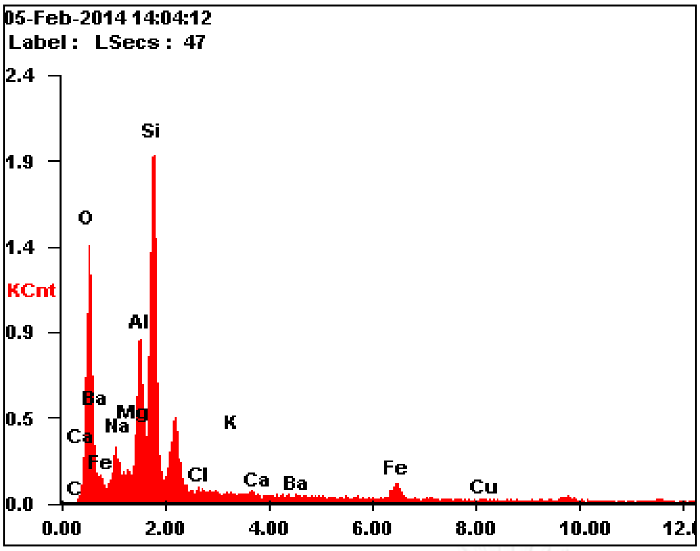


2. Bentonite

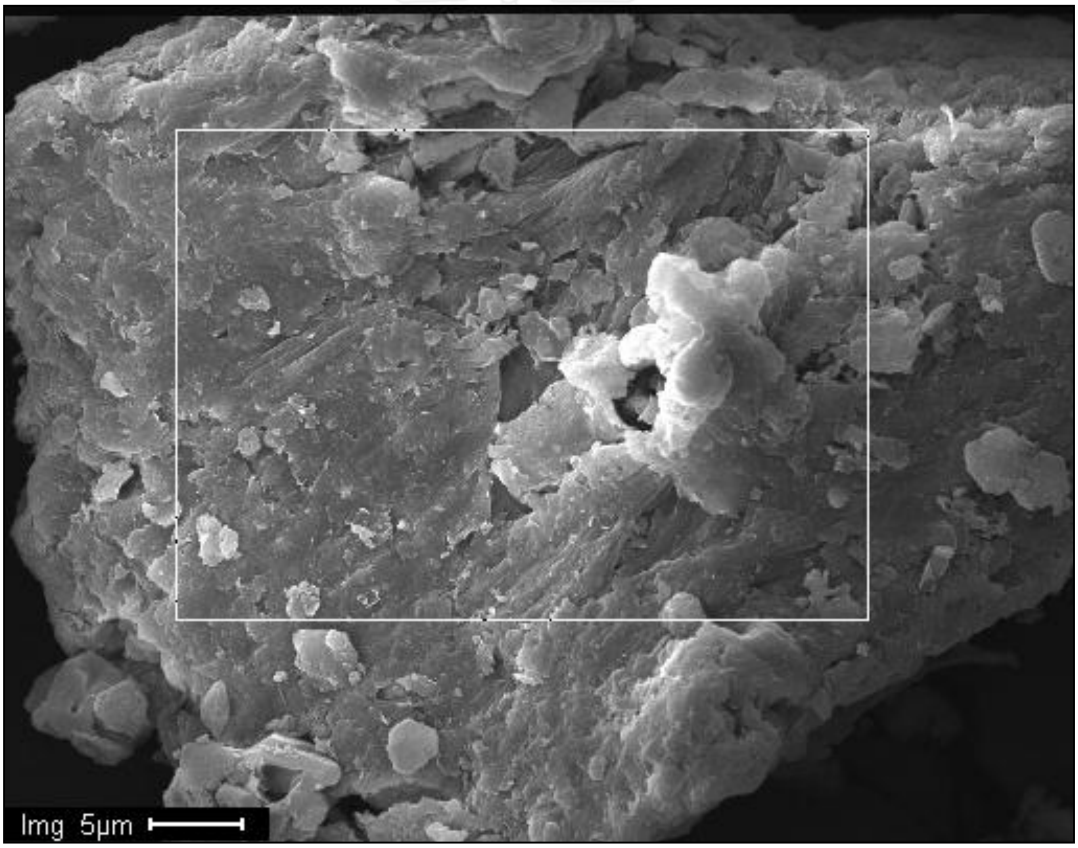


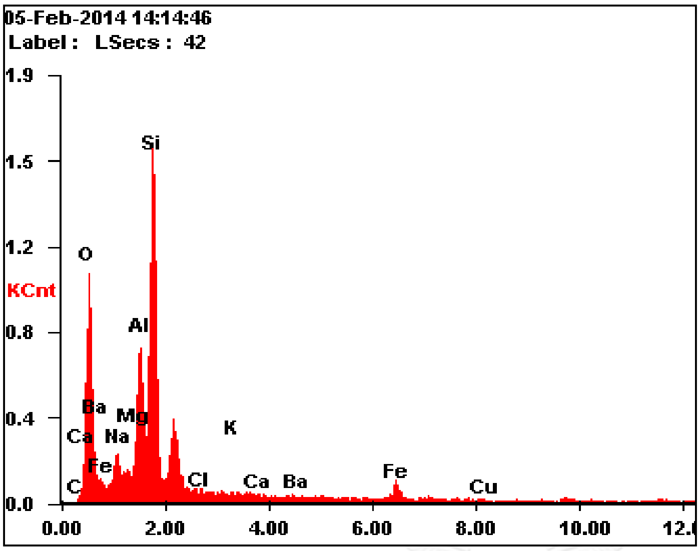
Element	Wt%	At%
C	31.45	44.93
O	35.89	38.49
Na	02.64	01.97
Mg	01.20	00.84
Al	05.26	03.35
Si	11.29	06.90
Cl	00.28	00.14
K	00.00	00.00
Ca	00.71	00.30
Ba	01.41	00.18
Fe	06.24	01.92
Cu	03.62	00.98



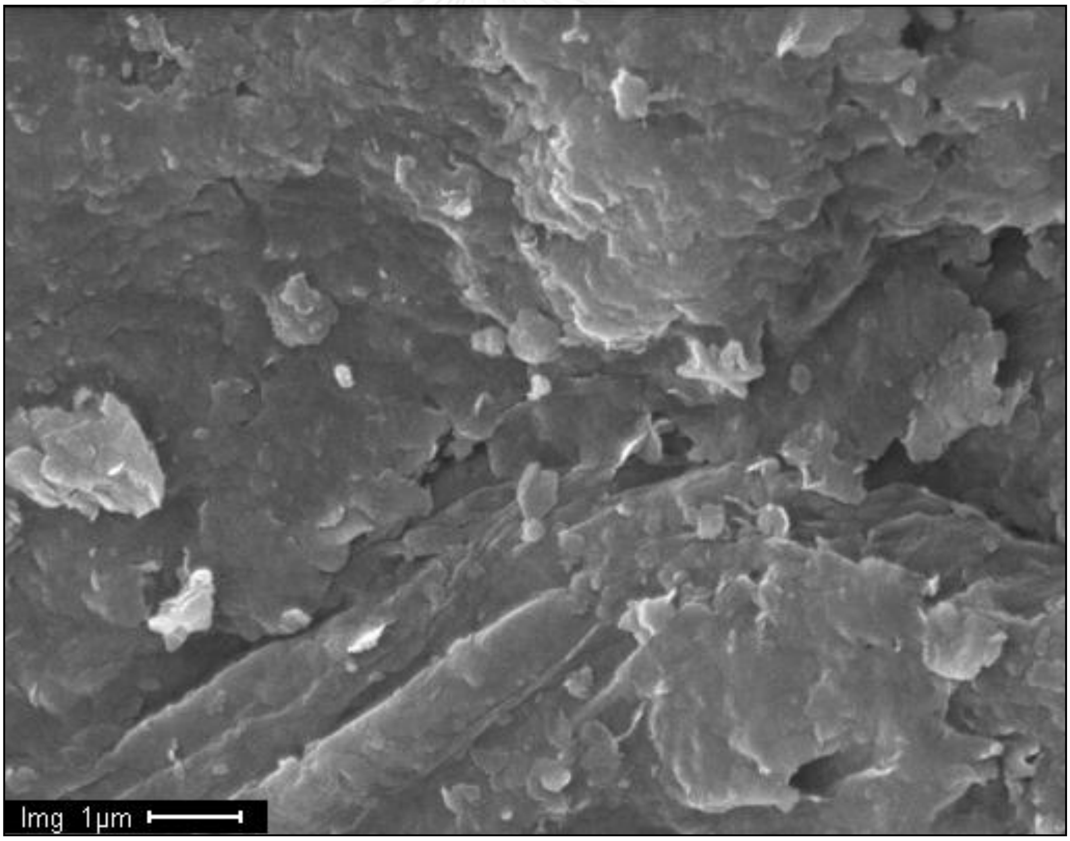


Element	Wt%	At%
C	00.00	00.00
O	38.45	54.00
Na	04.50	04.40
Mg	01.60	01.48
Al	13.08	10.89
Si	31.01	24.81
Cl	00.36	00.23
K	00.00	00.00
Ca	00.09	00.05
Ba	00.63	00.10
Fe	08.36	03.36
Cu	01.92	00.68

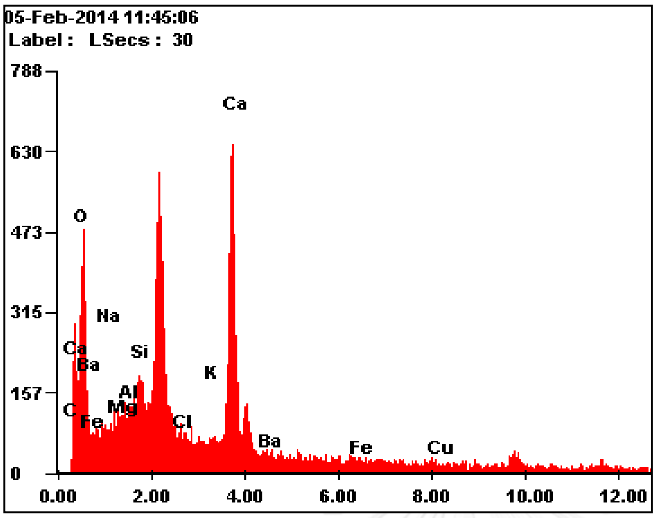




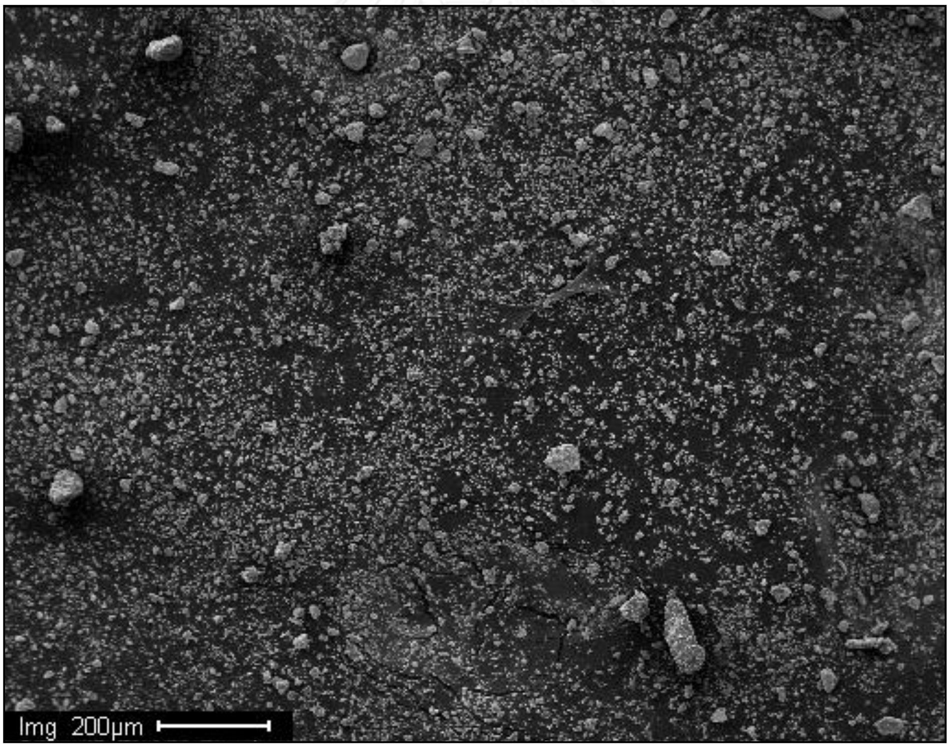
Element	Wt%	At%
C	00.00	00.00
O	36.75	52.28
Na	04.13	04.09
Mg	01.77	01.65
Al	13.45	11.34
Si	32.21	26.10
Cl	00.29	00.19
K	00.12	00.07
Ca	00.29	00.16
Ba	01.23	00.20
Fe	08.45	03.44
Cu	01.31	00.47

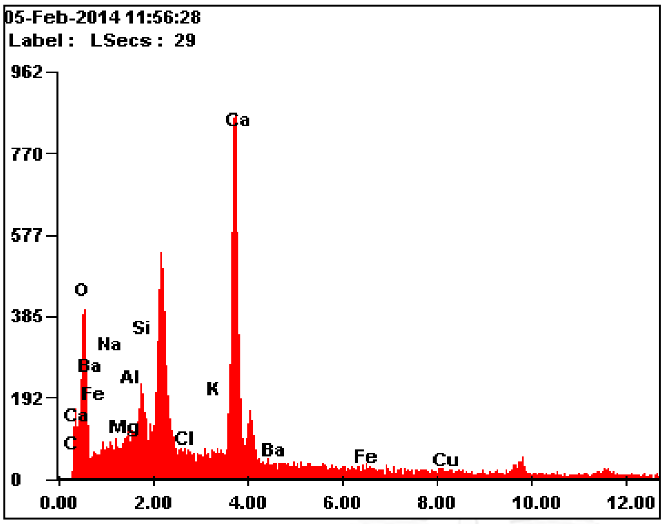


3. Cement

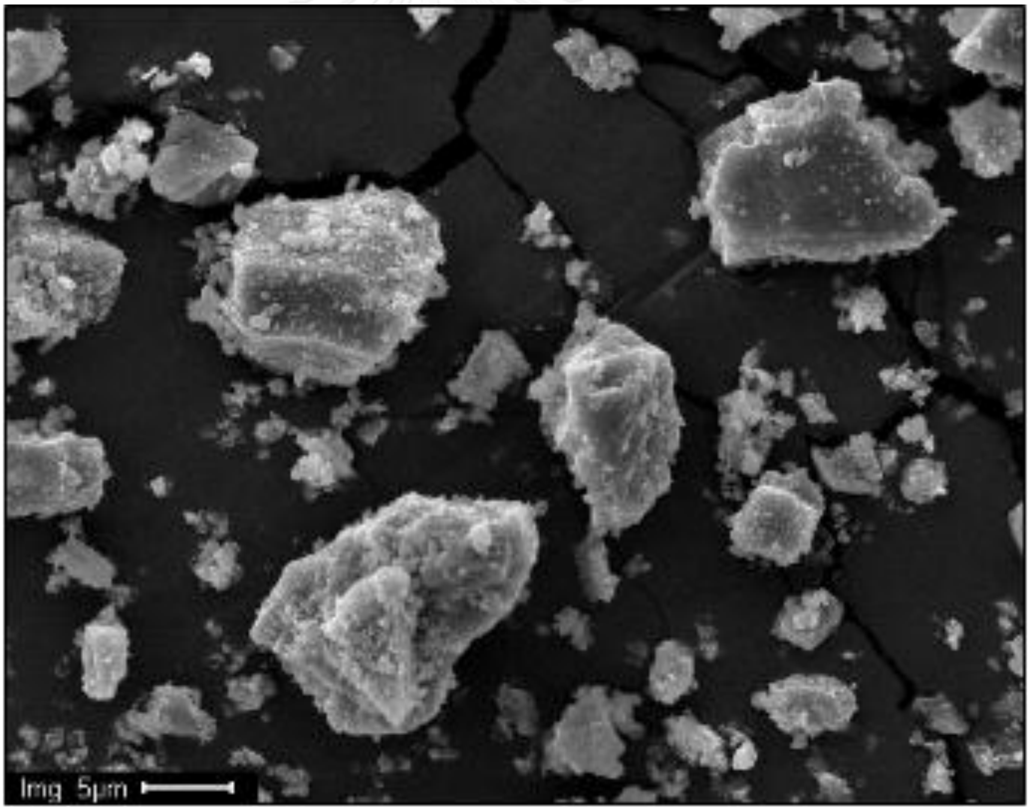


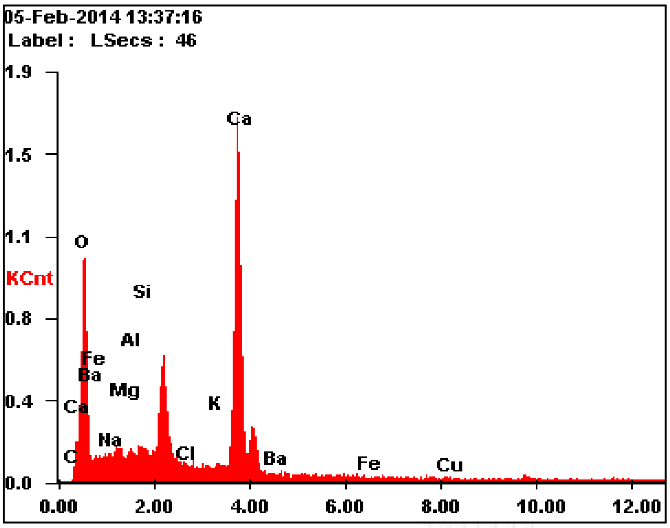
Element	Wt%	At%
C	12.47	22.21
O	35.48	47.44
Na	02.90	02.70
Mg	02.46	02.17
Al	02.97	02.36
Si	04.91	03.74
Cl	01.56	00.94
K	01.59	00.87
Ca	30.00	16.01
Ba	02.13	00.33
Fe	00.93	00.36
Cu	02.59	00.87



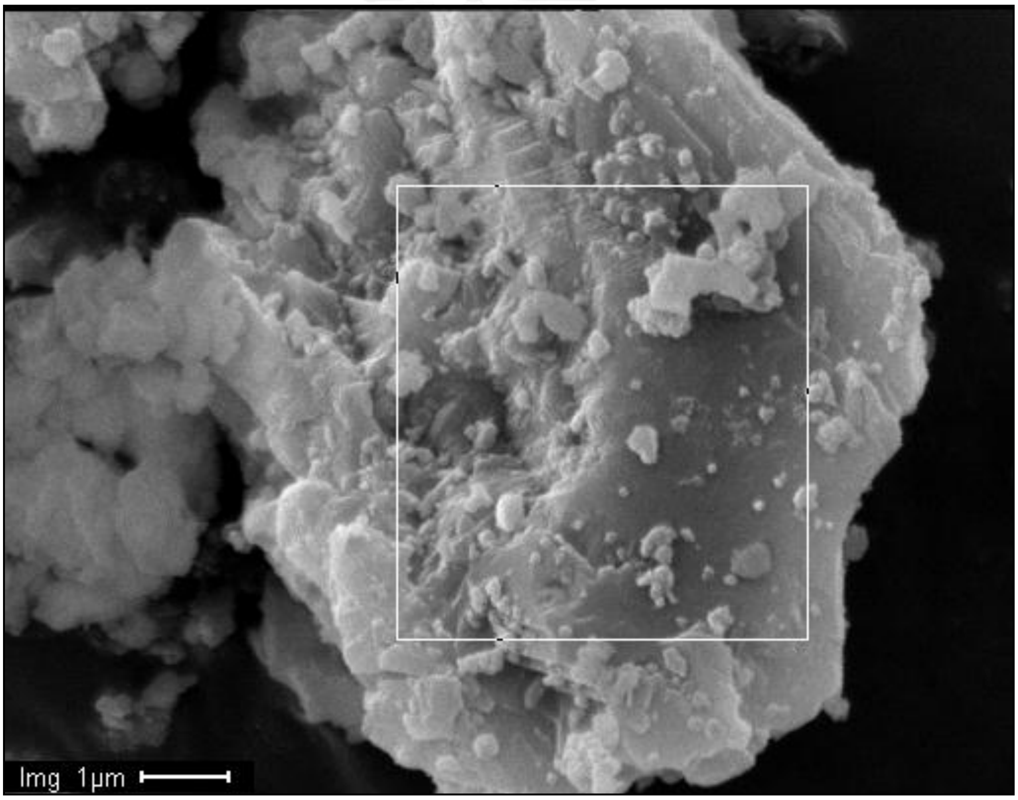


Element	Wt%	At%
C	20.39	32.94
O	36.27	43.99
Na	02.97	02.51
Mg	02.56	02.05
Al	02.88	02.07
Si	04.36	03.01
Cl	00.82	00.45
K	00.66	00.33
Ca	22.40	10.84
Ba	01.85	00.26
Fe	01.68	00.58
Cu	03.16	00.96



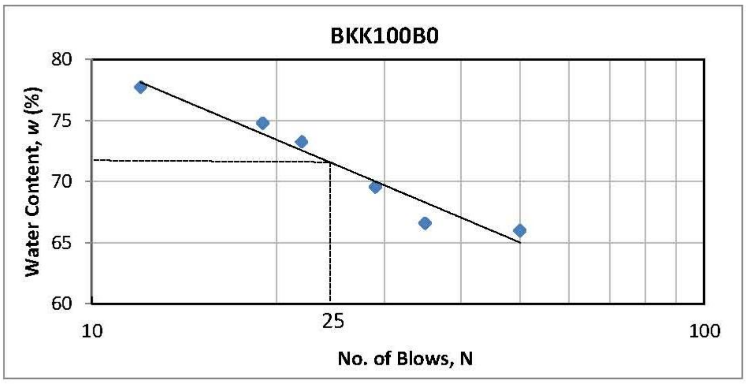


Element	Wt%	At%
C	03.05	05.39
O	53.60	71.17
Na	00.65	00.60
Mg	00.98	00.86
Al	00.78	00.61
Si	01.04	00.79
Cl	00.25	00.15
K	00.59	00.32
Ca	36.30	19.24
Ba	00.46	00.07
Fe	00.53	00.20
Cu	01.78	00.60



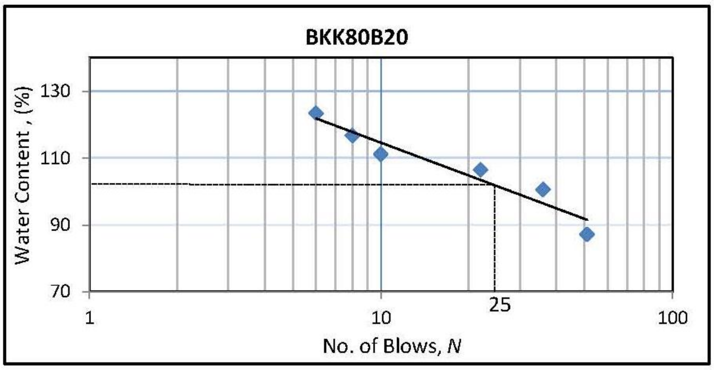
APPENDIX B: ENGINEERING PROPERTIES

1. Atterberg limits



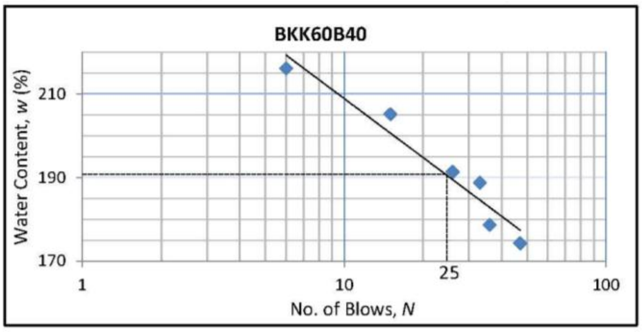
Summary

Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	Plasticity Index (%)	Liquidity Index (%)
52.09	72	31.31	13.67	40.69	0.51



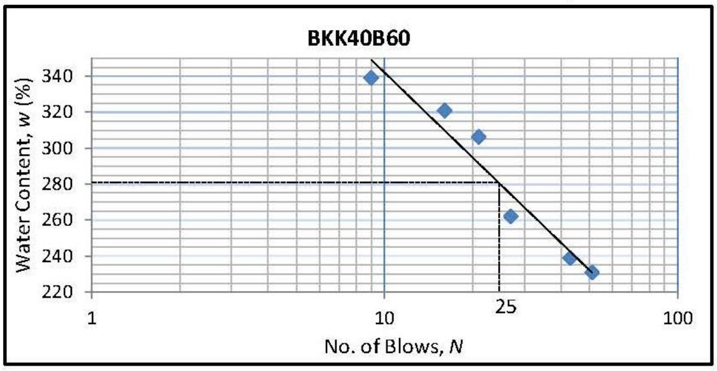
Summary

Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	Plasticity Index (%)	Liquidity Index (%)
52.09	102	37.06	16.54	64.94	0.23



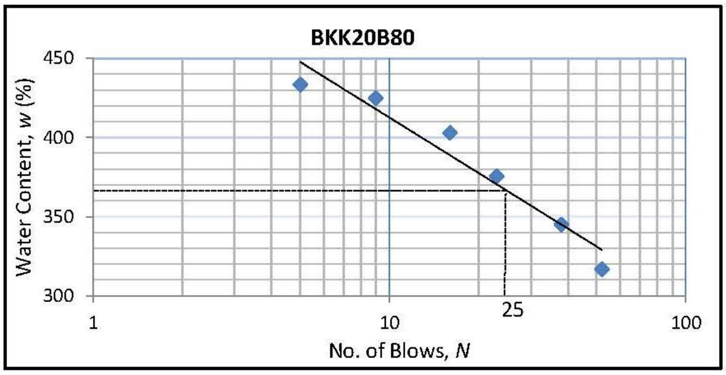
Summary

Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	Plasticity Index (%)	Liquidity Index (%)
52.09	191	39.30	11.83	151.70	0.08



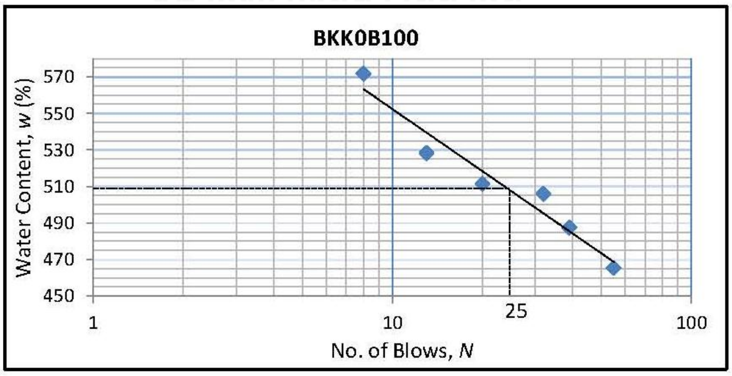
Summary

Natural Water Content, (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	Plasticity Index (%)	Liquidity Index (%)
52.09	281	42.04	12.18	238.96	0.04



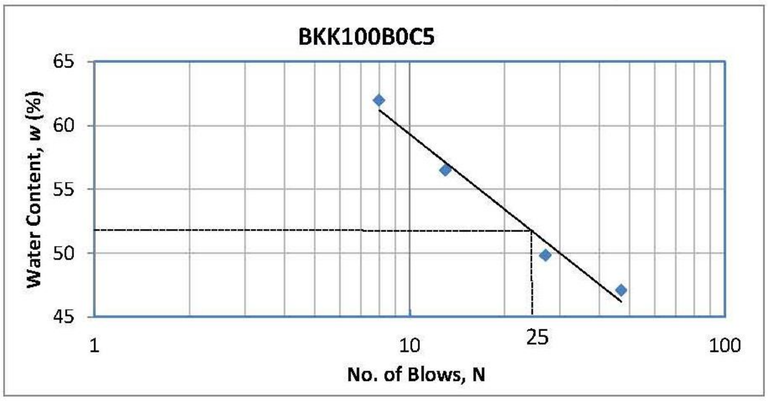
Summary

Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	Plasticity Index (%)	Liquidity Index (%)
52.09	368	49.82	22.52	318.18	0.01



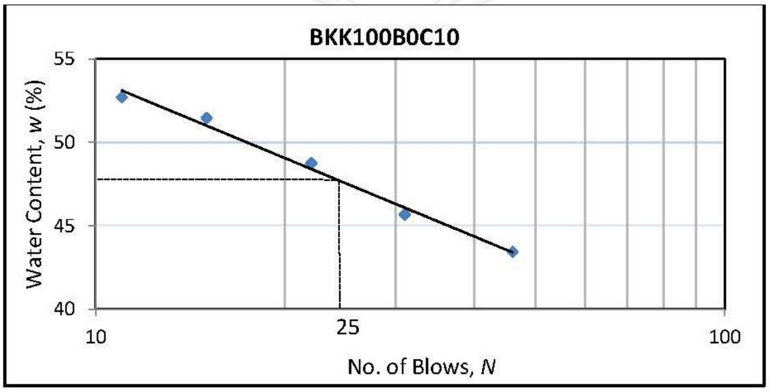
Summary

Natural Water content (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	Plasticity Index (%)	Liquidity Index (%)
52.09	509	53.89	22.61	455.11	0.00



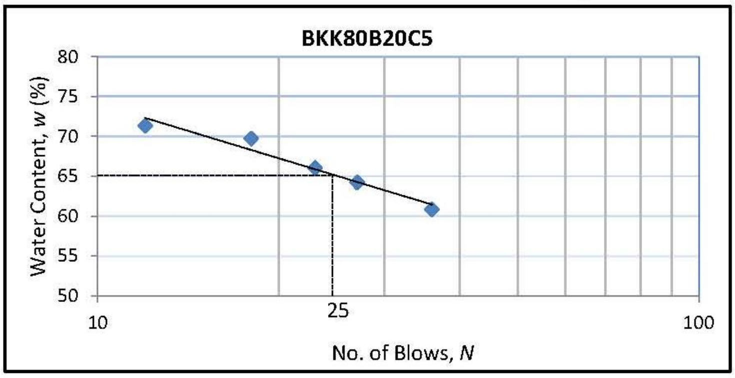
Summary

Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	Plasticity Index (%)	Liquidity Index (%)
–	52	35.25	20.58	16.75	–



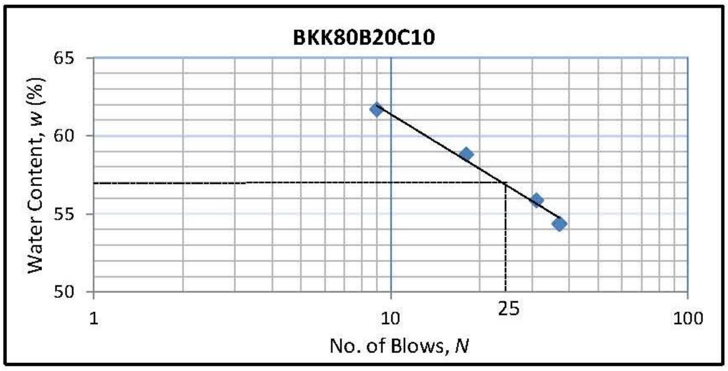
Summary

Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	Plasticity Index (%)	Liquidity Index (%)
–	48	37.32	24.71	10.68	–



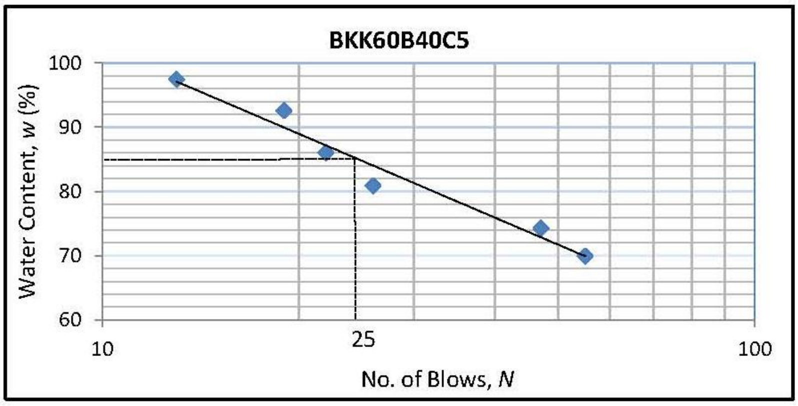
Summary

Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	Plasticity Index (%)	Liquidity Index (%)
–	65.48	42.66	17.75	22.82	–



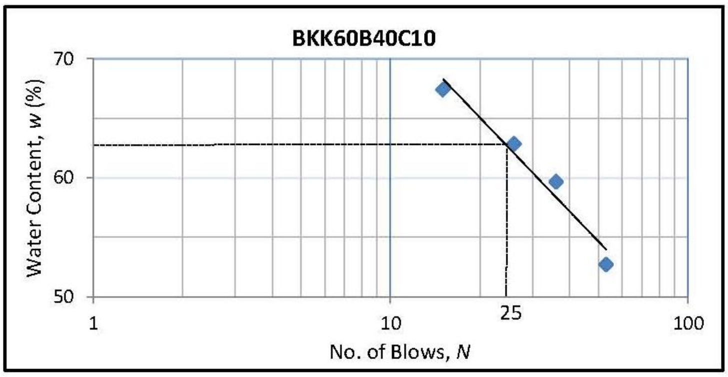
Summary

Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	Plasticity Index (%)	Liquidity Index (%)
–	57	42.71	25.62	14.29	–



Summary

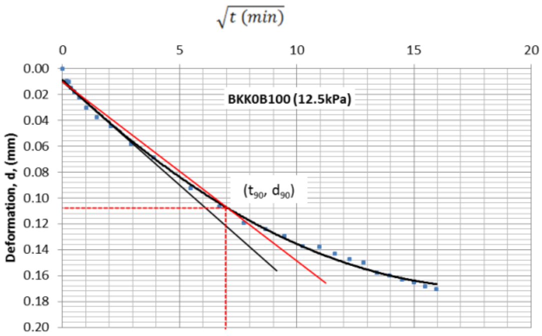
Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	Plasticity Index (%)	Liquidity Index (%)
–	85	44.75	20.12	40.25	–

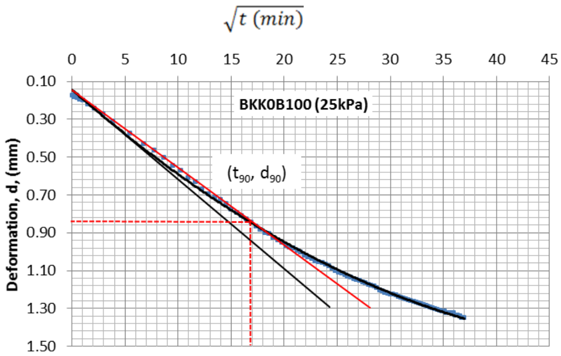


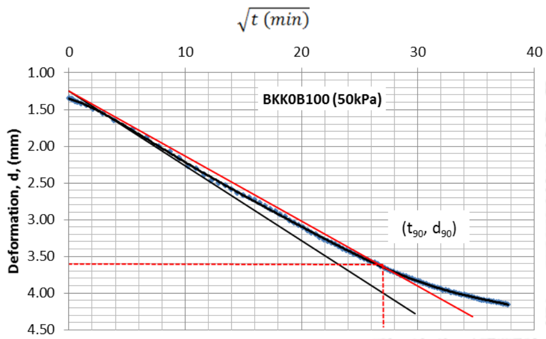
Summary

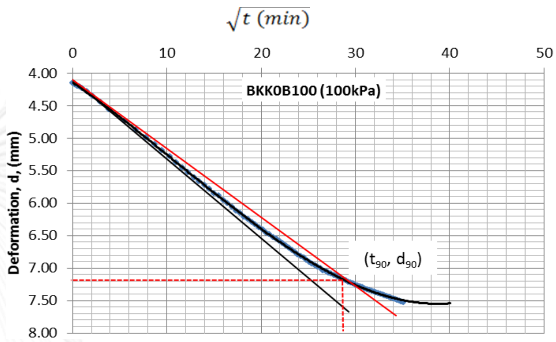
Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	Plasticity Index (%)	Liquidity Index (%)
–	63	47.86	22.94	15.14	–

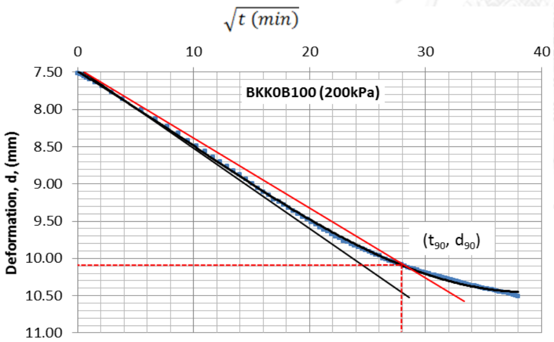
2. Consolidation

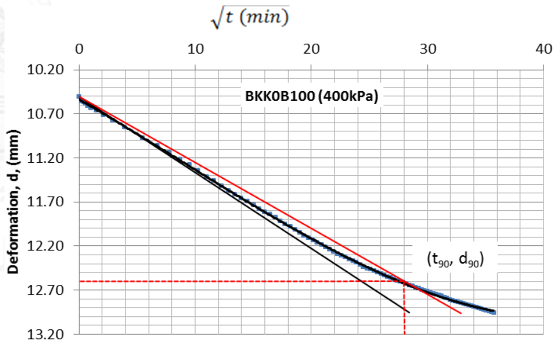


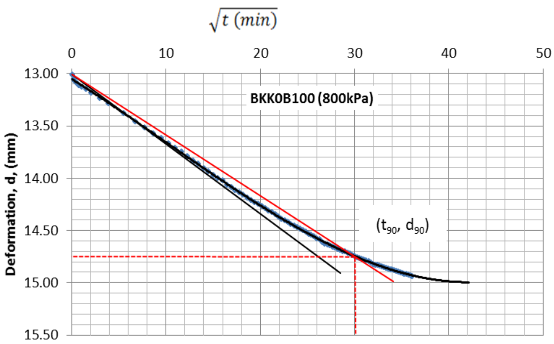


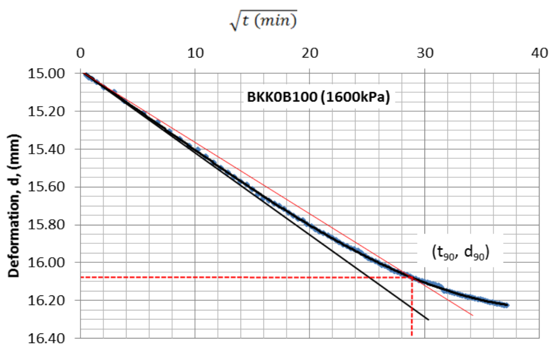


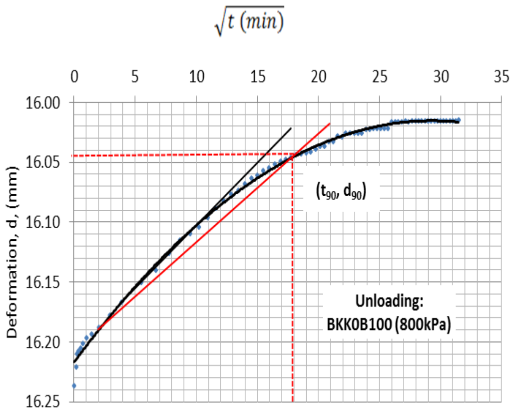




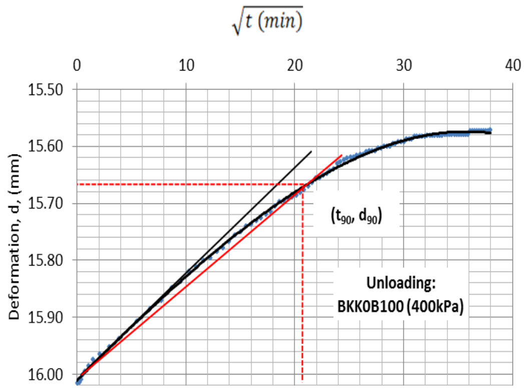




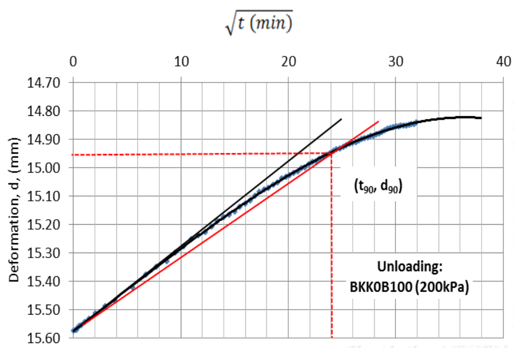




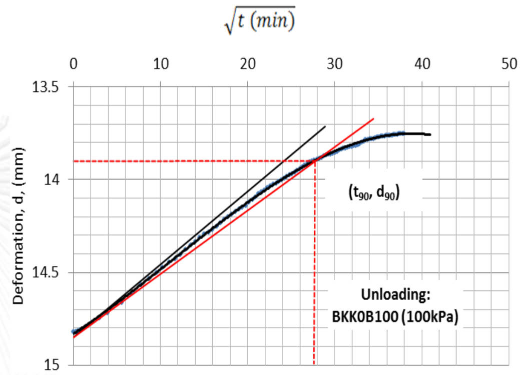
$\sqrt{t\ (min)}$
Deformation, d, (mm)
(t90, d90)
Unloading:
BKK0B100 (800kPa)



$\sqrt{t\ (min)}$
Deformation, d, (mm)
(t90, d90)
Unloading:
BKK0B100 (400kPa)



$\sqrt{t\ (min)}$
Deformation, d, (mm)
(t90, d90)
Unloading:
BKK0B100 (200kPa)



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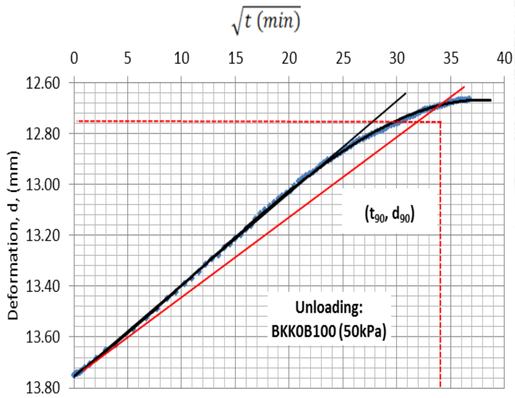
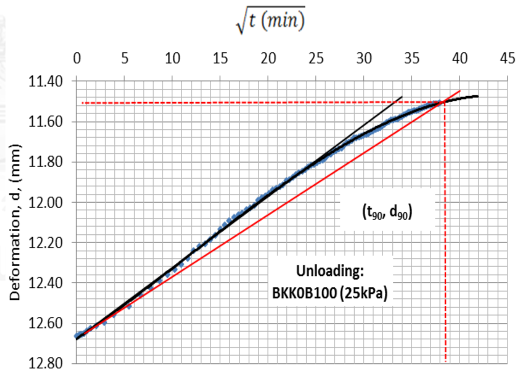
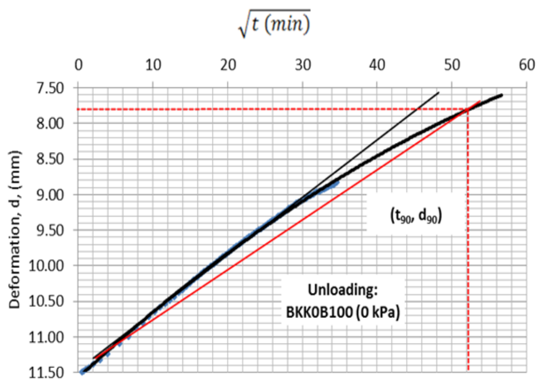
Deformation, d, (mm)
(t90, d90)
Unloading:
BKK0B100 (100kPa)



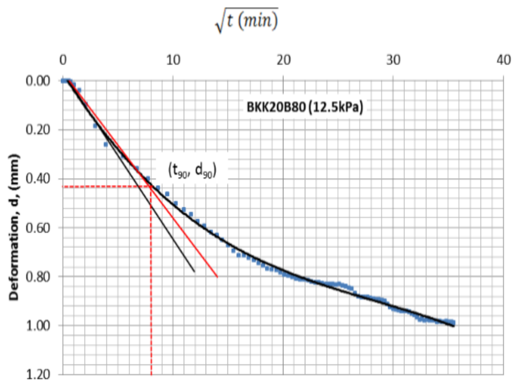
$\sqrt{t\ (min)}$
Deformation, d, (mm)
(t90, d90)
Unloading:
BKK0B100 (50kPa)



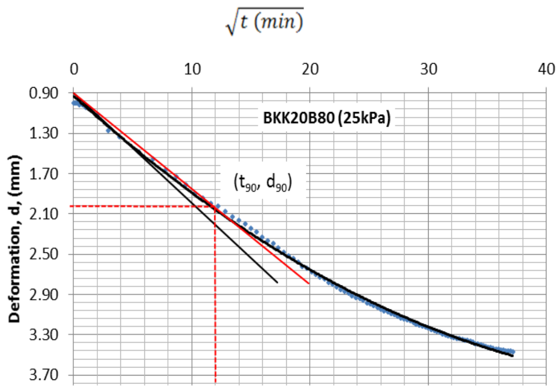
$\sqrt{t\ (min)}$
Deformation, d, (mm)
(t90, d90)
Unloading:
BKK0B100 (25kPa)



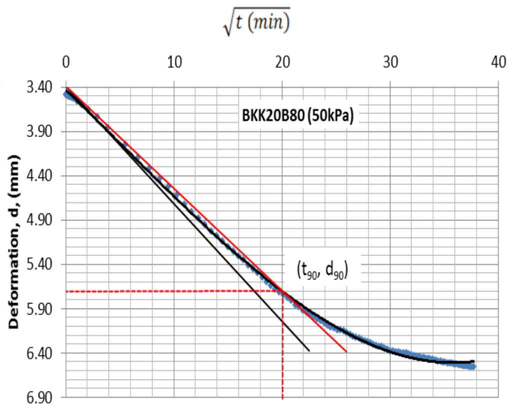
$\sqrt{t\ (min)}$
Deformation, d, (mm)
(t90, d90)
Unloading:
BKK0B100 (0 kPa)



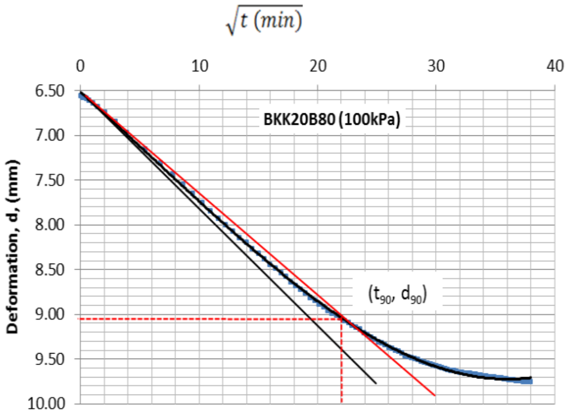
$\sqrt{t\ (min)}$
Deformation, d, (mm)
BKK20B80 (12.5kPa)
(t90, d90)



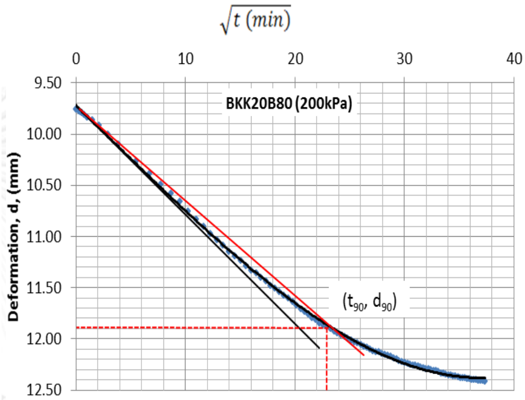
√t (min)
BKK20B80 (25kPa)
(t90, d90)
Deformation, d, (mm)



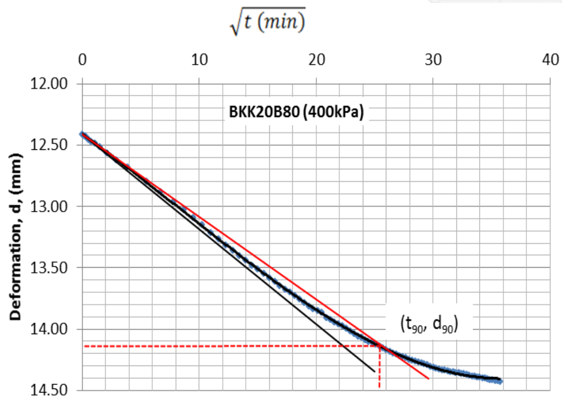
√t (min)
BKK20B80 (50kPa)
(t90, d90)
Deformation, d, (mm)



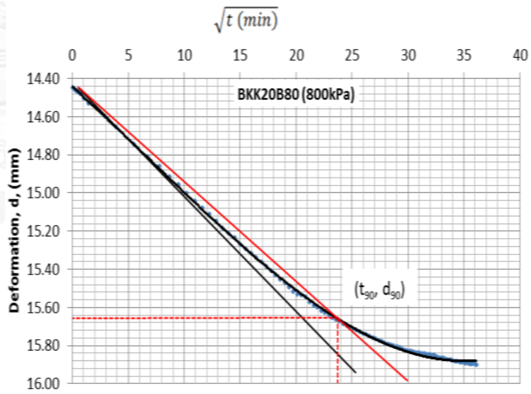
√t (min)
BKK20B80 (100kPa)
(t90, d90)
Deformation, d, (mm)



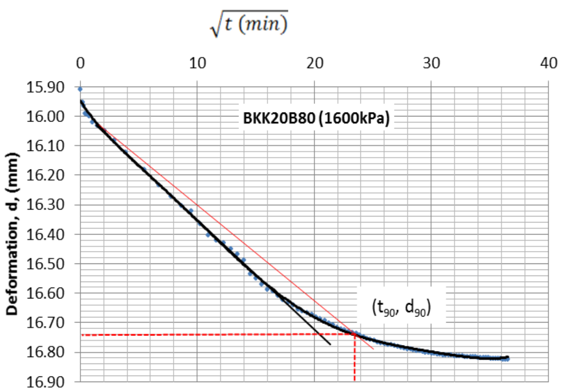
√t (min)
BKK20B80 (200kPa)
(t90, d90)
Deformation, d, (mm)



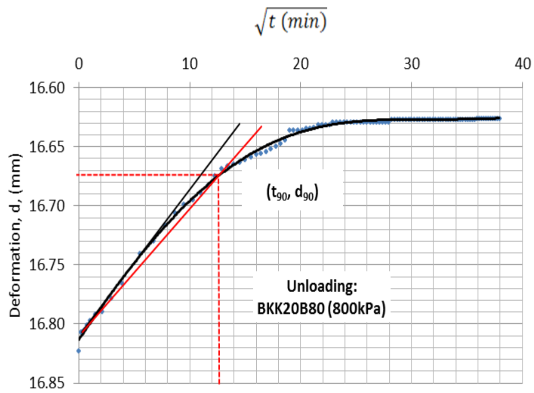
√t (min)
BKK20B80 (400kPa)
(t90, d90)
Deformation, d, (mm)



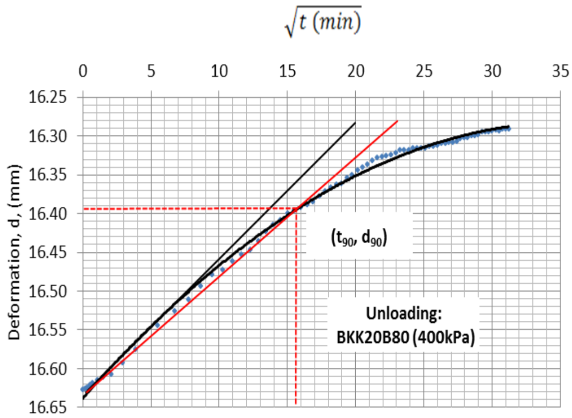
√t (min)
BKK20B80 (800kPa)
(t90, d90)
Deformation, d, (mm)

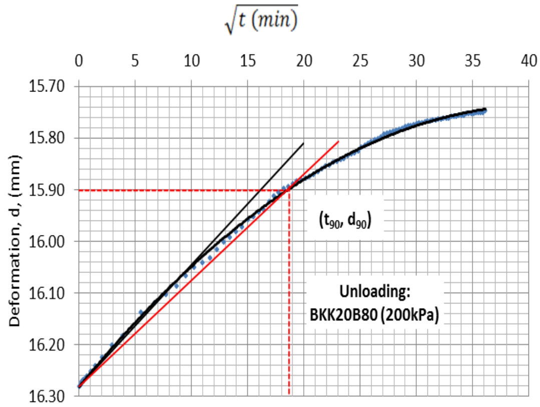


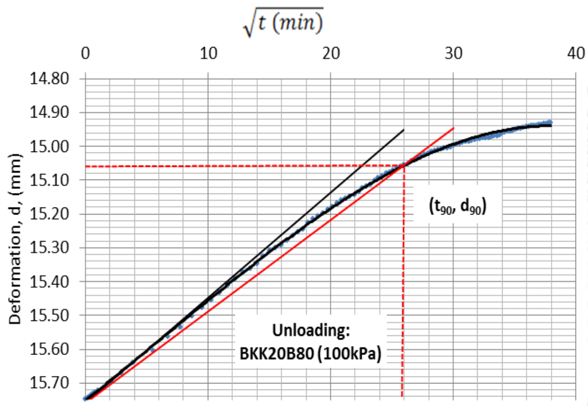
√t (min)
BKK20B80 (1600kPa)
(t90, d90)
Deformation, d, (mm)

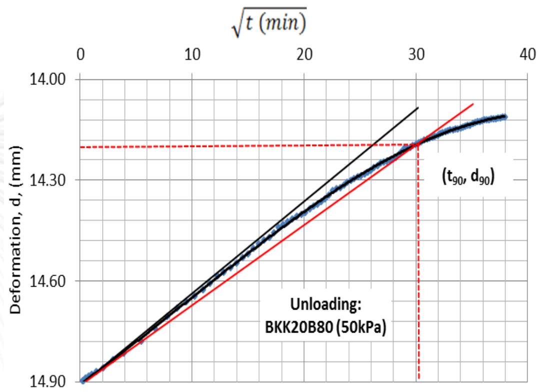


√t (min)
(t90, d90)
Unloading:
BKK20B80 (800kPa)
Deformation, d, (mm)


$\sqrt{t}\ (min)$
0 5 10 15 20 25 30 35
16.25 16.30 16.35 16.40 16.45 16.50 16.55 16.60 16.65
Deformation, d, (mm)
(t_{90}, d_{90})
Unloading:
BKK20B80 (400kPa)


$\sqrt{t}\ (min)$
0 5 10 15 20 25 30 35 40
15.70 15.80 15.90 16.00 16.10 16.20 16.30
Deformation, d, (mm)
(t_{90}, d_{90})
Unloading:
BKK20B80 (200kPa)


$\sqrt{t}\ (min)$
0 10 20 30 40
14.80 14.90 15.00 15.10 15.20 15.30 15.40 15.50 15.60 15.70
Deformation, d, (mm)
(t_{90}, d_{90})
Unloading:
BKK20B80 (100kPa)


$\sqrt{t}\ (min)$
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Deformation, d, (mm)
(t_{90}, d_{90})
Unloading:
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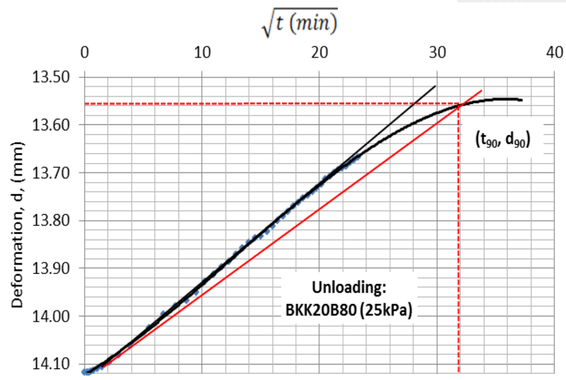
$\sqrt{t}\ (min)$
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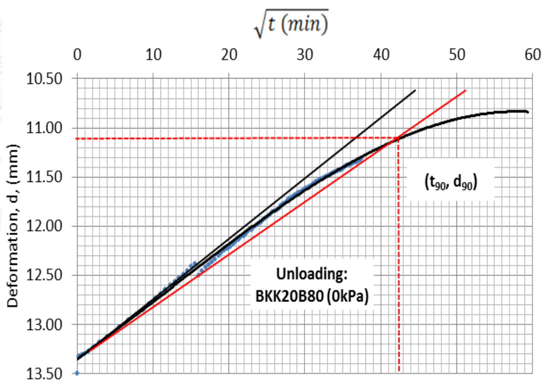
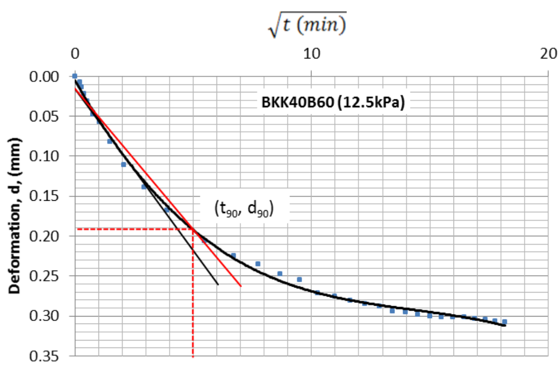
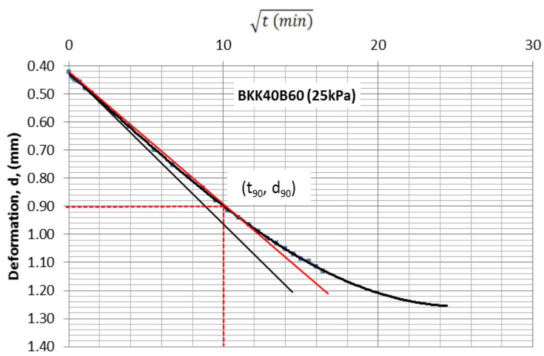
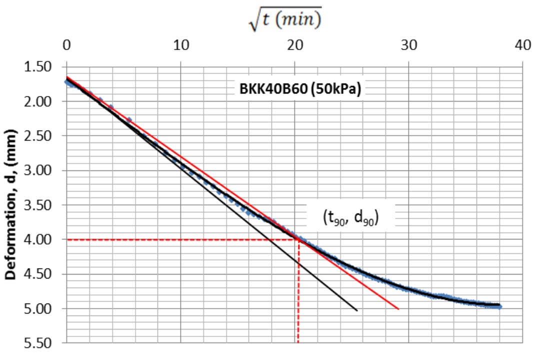
13.50 13.60 13.70 13.80 13.90 14.00 14.10
Deformation, d, (mm)
(t_{90}, d_{90})
Unloading:
BKK20B80 (25kPa)


$\sqrt{t}\ (min)$
0 10 20 30 40 50 60
10.50 11.00 11.50 12.00 12.50 13.00 13.50
Deformation, d, (mm)
(t_{90}, d_{90})
Unloading:
BKK20B80 (0kPa)


$\sqrt{t}\ (min)$
0 10 20
0.00 0.05 0.10 0.15 0.20 0.25 0.30 0.35
Deformation, d, (mm)
BKK40B60 (12.5kPa)
(t_{90}, d_{90})


$\sqrt{t}\ (min)$
0 10 20 30
0.40 0.50 0.60 0.70 0.80 0.90 1.00 1.10 1.20 1.30 1.40
Deformation, d, (mm)
BKK40B60 (25kPa)
(t_{90}, d_{90})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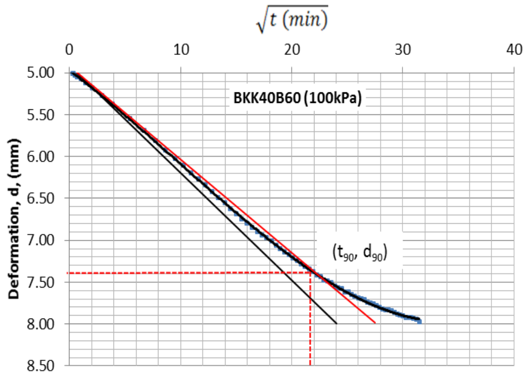
BKK40B60 (50kPa)
(t_{90}, d_{90})
Deformation, d, (mm)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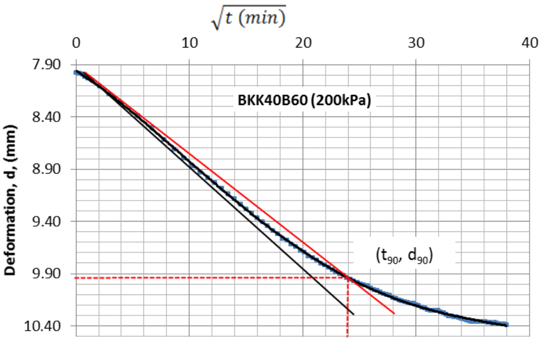
BKK40B60 (100kPa)
(t_{90}, d_{90})
Deformation, d, (mm)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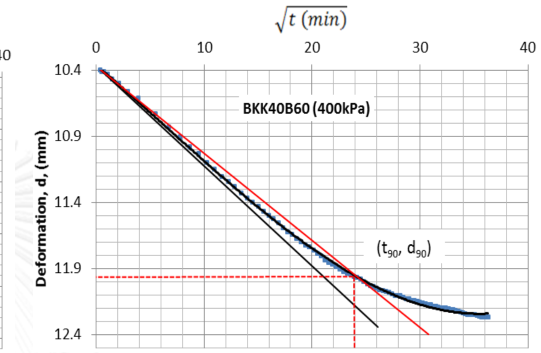
BKK40B60 (200kPa)
(t_{90}, d_{90})
Deformation, d, (mm)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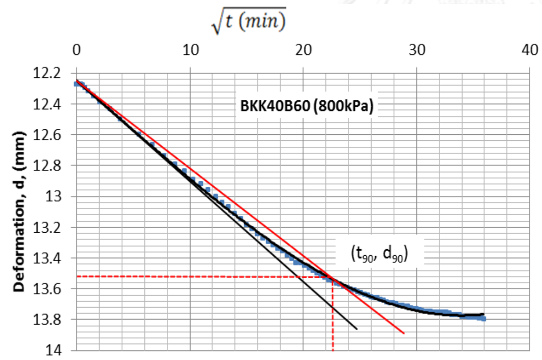
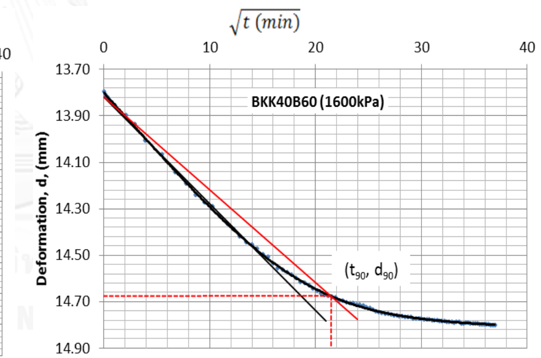
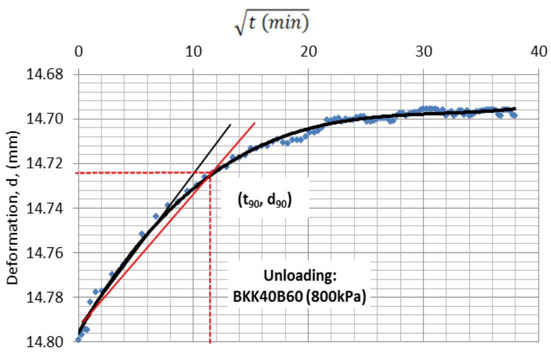
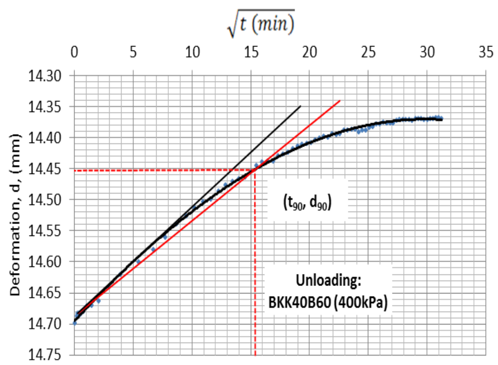
BKK40B60 (400kPa)
(t_{90}, d_{90})
Deformation, d, (mm)


$\sqrt{t\,(min)}$
BKK40B60 (800kPa)
(t_{90}, d_{90})
Deformation, d, (mm)


$\sqrt{t\,(min)}$
BKK40B60 (1600kPa)
(t_{90}, d_{90})
Deformation, d, (mm)


$\sqrt{t\,(min)}$
(t_{90}, d_{90})
Unloading:
BKK40B60 (800kPa)
Deformation, d, (mm)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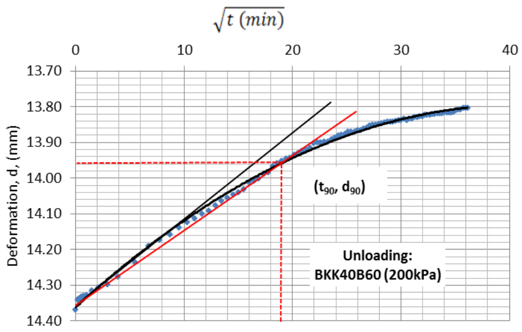
(t_{90}, d_{90})
Unloading:
BKK40B60 (400kPa)
Deformation, d, (mm)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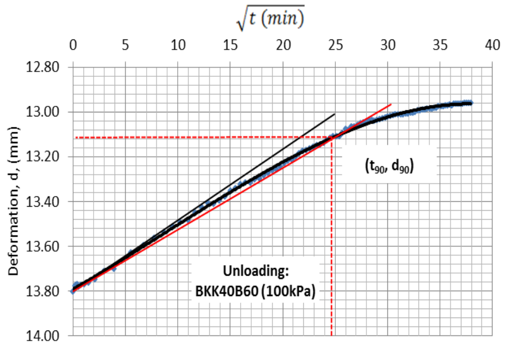
Deformation, d, (mm)
(t_{90}, d_{90})
Unloading:
BKK40B60 (200kPa)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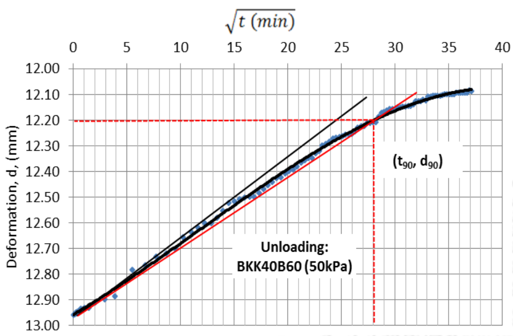
Deformation, d, (mm)
(t_{90}, d_{90})
Unloading:
BKK40B60 (100kPa)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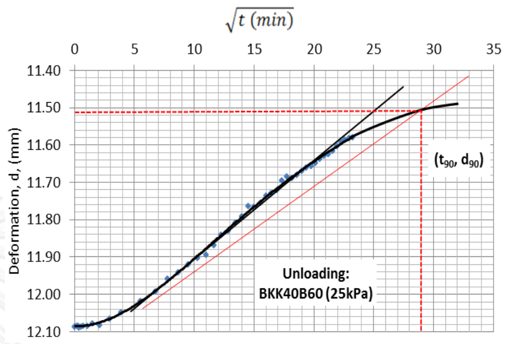
Deformation, d, (mm)
(t_{90}, d_{90})
Unloading:
BKK40B60 (50kPa)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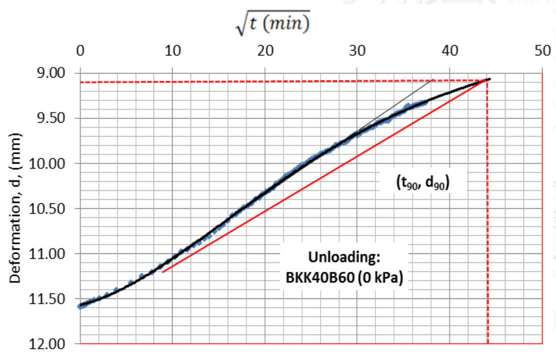
Deformation, d, (mm)
(t_{90}, d_{90})
Unloading:
BKK40B60 (25kPa)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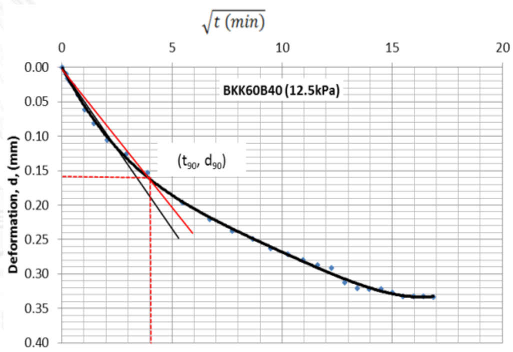
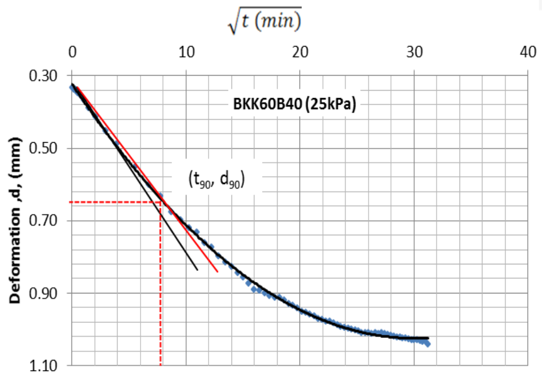
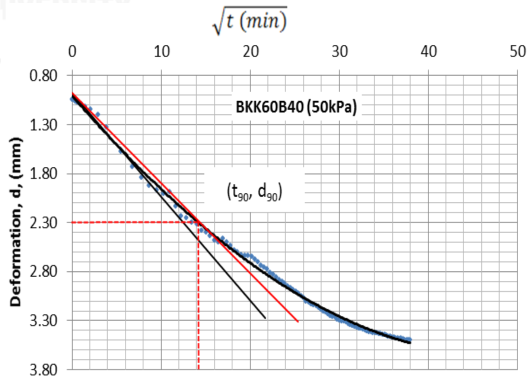
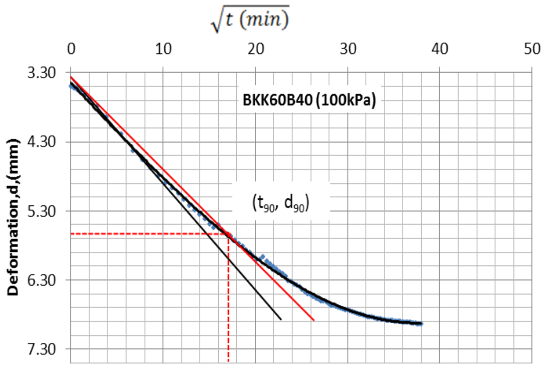
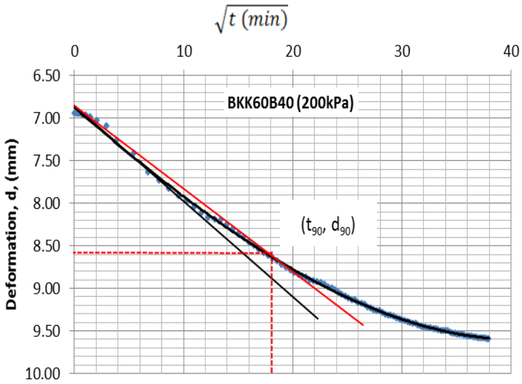
Deformation, d, (mm)
(t_{90}, d_{90})
Unloading:
BKK40B60 (0 kPa)

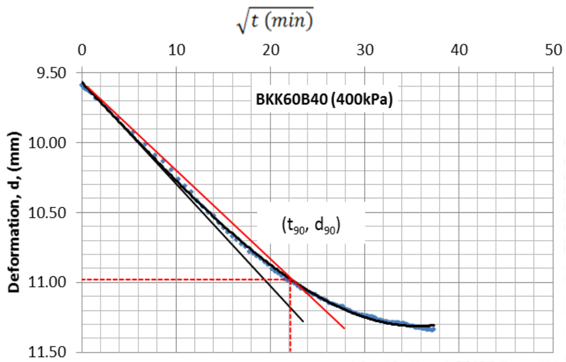

$\sqrt{t\ (min)}$
Deformation, d, (mm)
BKK60B40 (12.5kPa)
(t_{90}, d_{90})

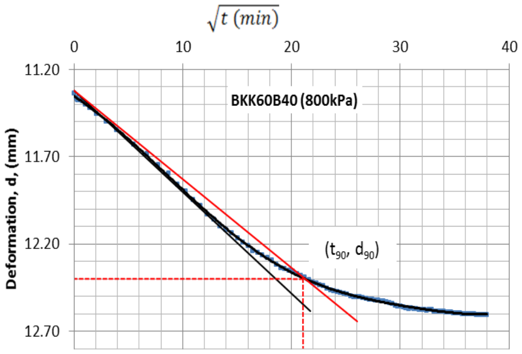

$\sqrt{t\ (min)}$
Deformation ,d, (mm)
BKK60B40 (25kPa)
(t_{90}, d_{90})

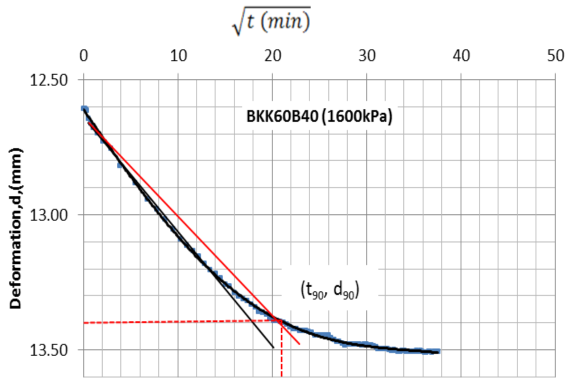

$\sqrt{t\ (min)}$
Deformation, d, (mm)
BKK60B40 (50kPa)
(t_{90}, d_{90})

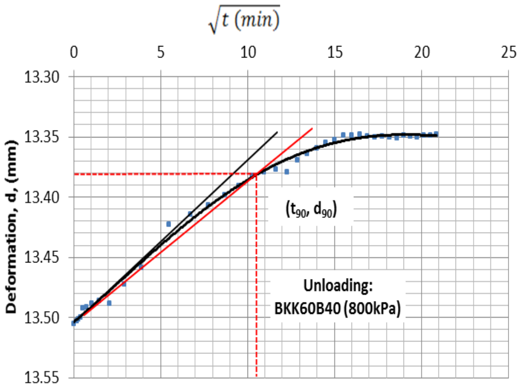

$\sqrt{t\ (min)}$
BKK60B40 (100kPa)
(t_{90}, d_{90})
Deformation,d,(mm)
0 10 20 30 40 50
3.30 4.30 5.30 6.30 7.30

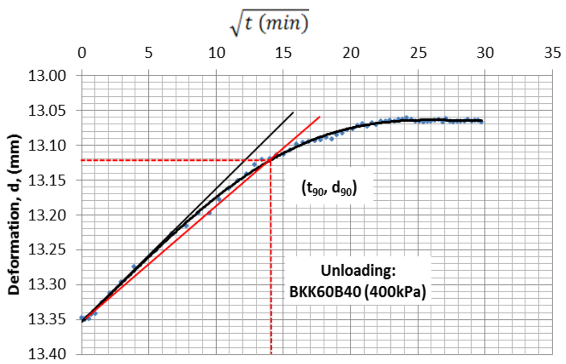

$\sqrt{t\ (min)}$
BKK60B40 (200kPa)
(t_{90}, d_{90})
Deformation, d, (mm)
0 10 20 30 40
6.50 7.00 7.50 8.00 8.50 9.00 9.50 10.00

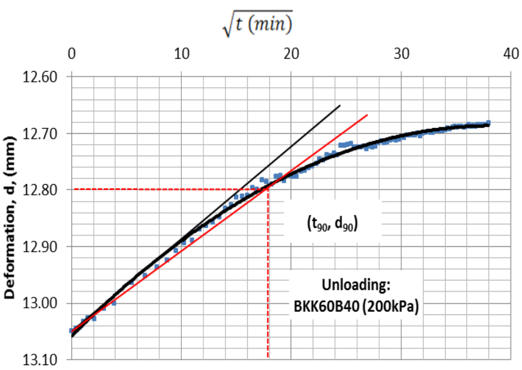

$\sqrt{t\ (min)}$
BKK60B40 (400kPa)
(t_{90}, d_{90})
Deformation, d, (mm)
0 10 20 30 40 50
9.50 10.00 10.50 11.00 11.50

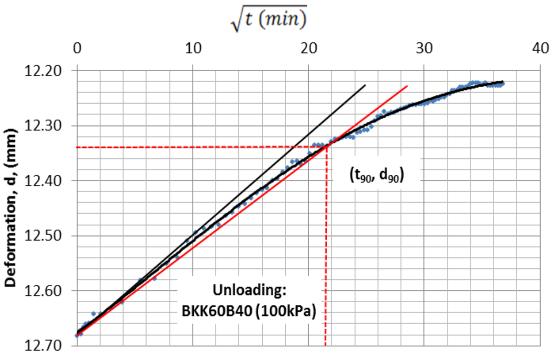

$\sqrt{t\ (min)}$
BKK60B40 (800kPa)
(t_{90}, d_{90})
Deformation, d, (mm)
0 10 20 30 40
11.20 11.70 12.20 12.70

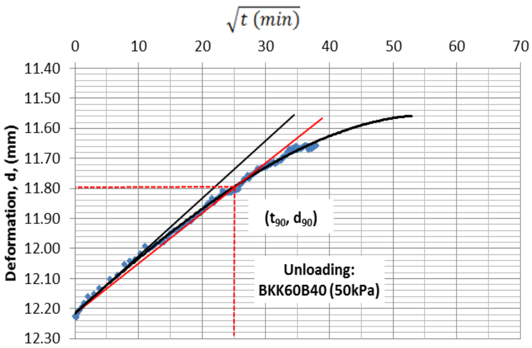

$\sqrt{t\ (min)}$
BKK60B40 (1600kPa)
(t_{90}, d_{90})
Deformation,d,(mm)
0 10 20 30 40 50
12.50 13.00 13.50

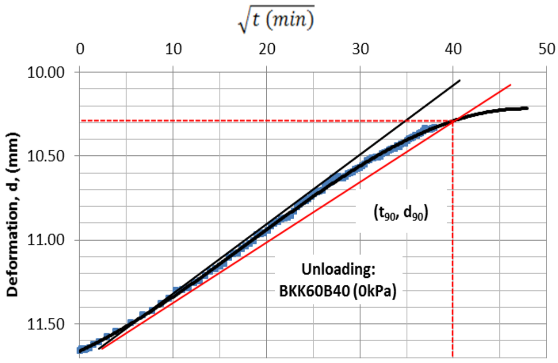

$\sqrt{t\ (min)}$
(t_{90}, d_{90})
Unloading:
BKK60B40 (800kPa)
Deformation, d, (mm)
0 5 10 15 20 25
13.30 13.35 13.40 13.45 13.50 13.55

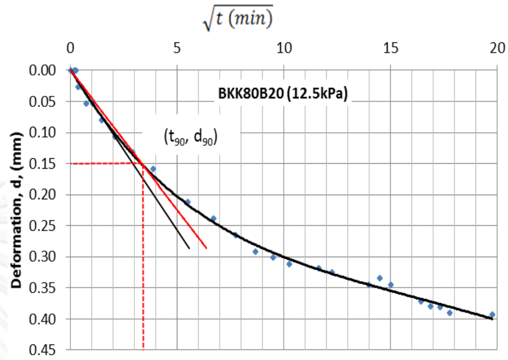

$\sqrt{t\ (min)}$
(t_{90}, d_{90})
Unloading:
BKK60B40 (400kPa)
Deformation, d, (mm)
0 5 10 15 20 25 30 35
13.00 13.05 13.10 13.15 13.20 13.25 13.30 13.35 13.40

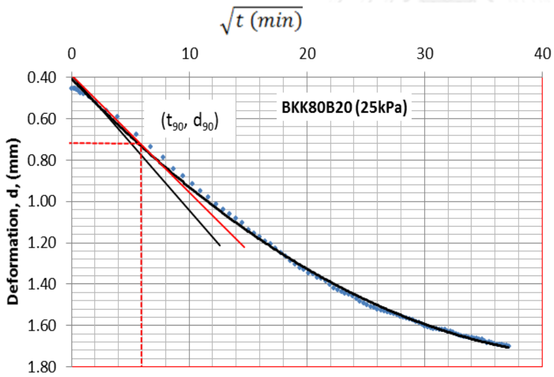

$\sqrt{t\ (min)}$
(t_{90}, d_{90})
Unloading:
BKK60B40 (200kPa)
Deformation, d, (mm)
0 10 20 30 40
12.60 12.70 12.80 12.90 13.00 13.10

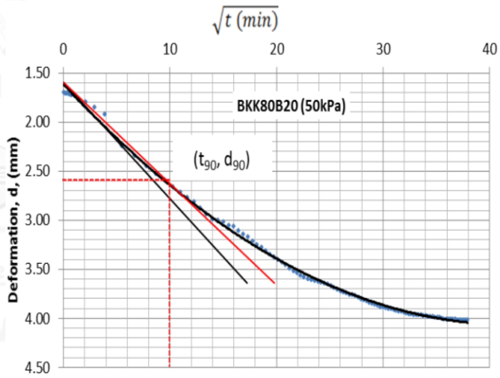

√t (min)
Deformation, d, (mm)
(t90, d90)
Unloading:
BKK60B40 (100kPa)

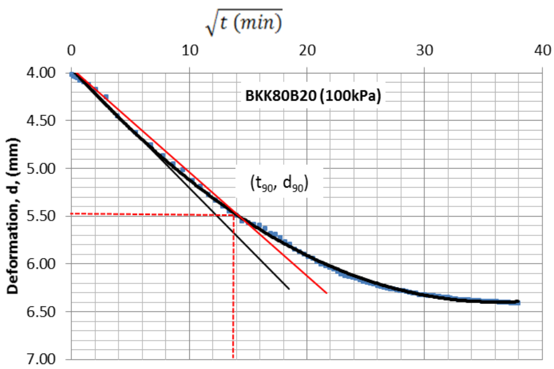

√t (min)
Deformation, d, (mm)
(t90, d90)
Unloading:
BKK60B40 (50kPa)

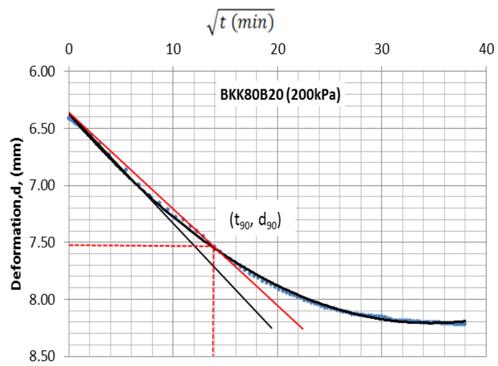

√t (min)
Deformation, d, (mm)
(t90, d90)
Unloading:
BKK60B40 (0kPa)

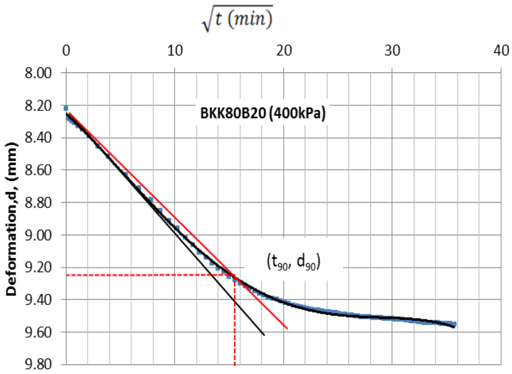

√t (min)
Deformation, d, (mm)
BKK80B20 (12.5kPa)
(t90, d90)

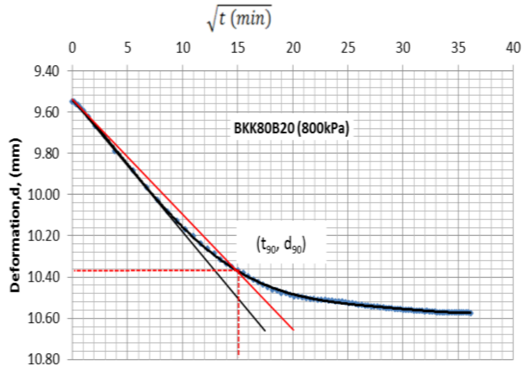

√t (min)
Deformation, d, (mm)
(t90, d90)
BKK80B20 (25kPa)

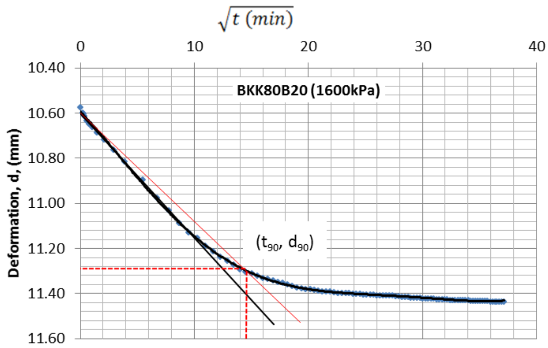

√t (min)
Deformation, d, (mm)
BKK80B20 (50kPa)
(t90, d90)

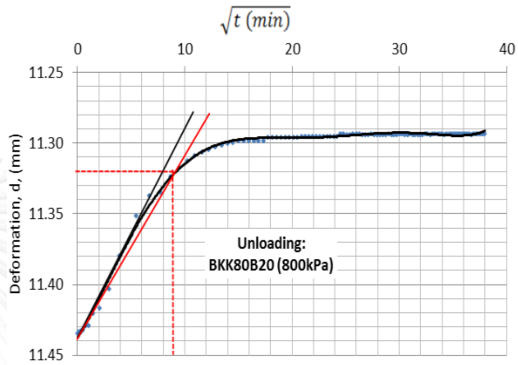

√t (min)
Deformation, d, (mm)
BKK80B20 (100kPa)
(t90, d90)

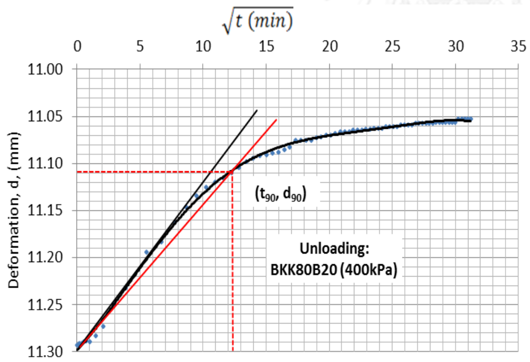

√t (min)
Deformation,d, (mm)
BKK80B20 (200kPa)
(t90, d90)

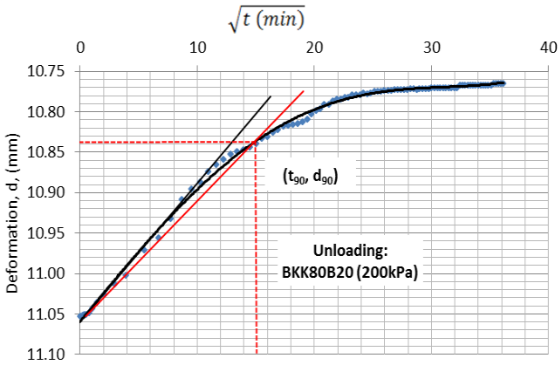

√t (min)
0 10 20 30 40
8.00 8.20 8.40 8.60 8.80 9.00 9.20 9.40 9.60 9.80
Deformation,d, (mm)
BKK80B20 (400kPa)
(t90, d90)

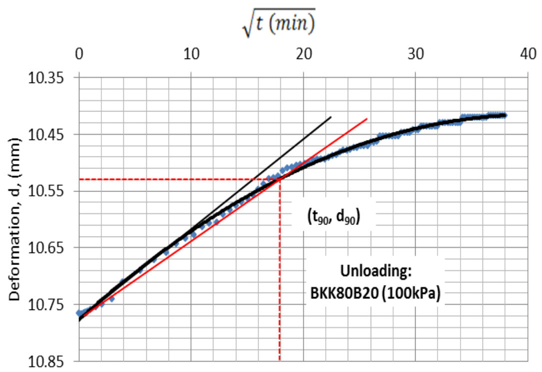

√t (min)
0 5 10 15 20 25 30 35 40
9.40 9.60 9.80 10.00 10.20 10.40 10.60 10.80
Deformation,d, (mm)
BKK80B20 (800kPa)
(t90, d90)

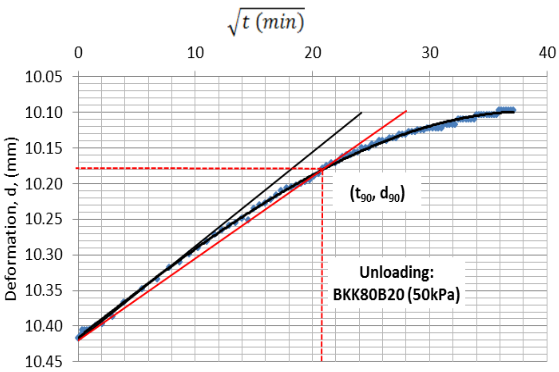

√t (min)
0 10 20 30 40
10.40 10.60 10.80 11.00 11.20 11.40 11.60
Deformation, d, (mm)
BKK80B20 (1600kPa)
(t90, d90)

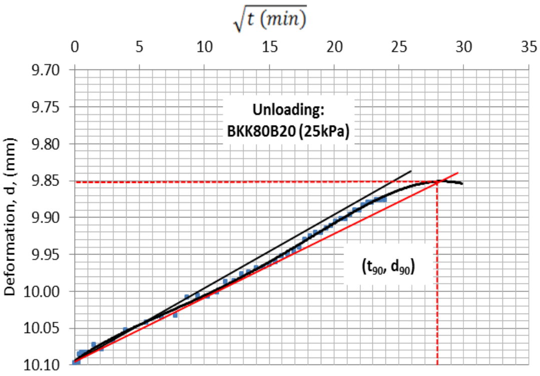

√t (min)
0 10 20 30 40
11.25 11.30 11.35 11.40 11.45
Deformation, d, (mm)
Unloading:
BKK80B20 (800kPa)

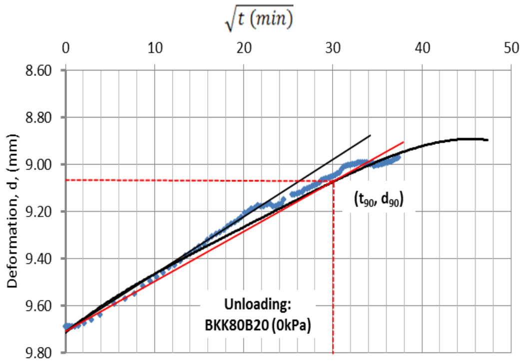

√t (min)
0 5 10 15 20 25 30 35
11.00 11.05 11.10 11.15 11.20 11.25 11.30
Deformation, d, (mm)
(t90, d90)
Unloading:
BKK80B20 (400kPa)

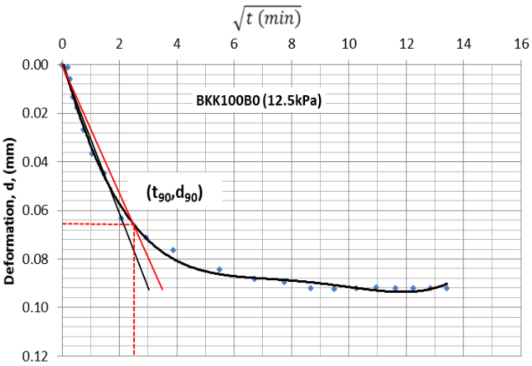

√t (min)
0 10 20 30 40
10.75 10.80 10.85 10.90 10.95 11.00 11.05 11.10
Deformation, d, (mm)
(t90, d90)
Unloading:
BKK80B20 (200kPa)

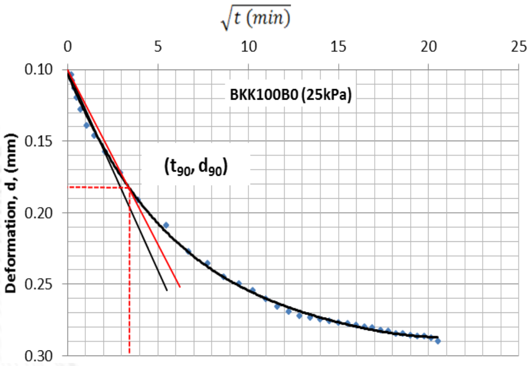

√t (min)
0 10 20 30 40
10.35 10.45 10.55 10.65 10.75 10.85
Deformation, d, (mm)
(t90, d90)
Unloading:
BKK80B20 (100kPa)

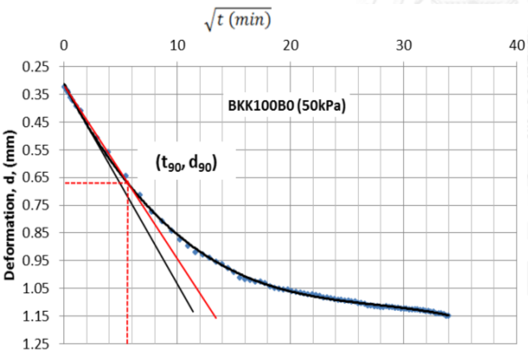

√t (min)
0 10 20 30 40
10.05 10.10 10.15 10.20 10.25 10.30 10.35 10.40 10.45
Deformation, d, (mm)
(t90, d90)
Unloading:
BKK80B20 (50kPa)

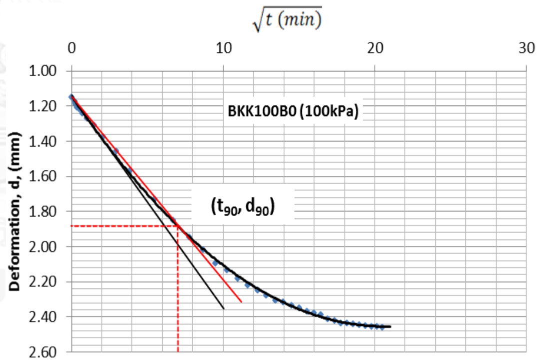

√t (min)
Deformation, d, (mm)
Unloading:
BKK80B20 (25kPa)
(t90, d90)

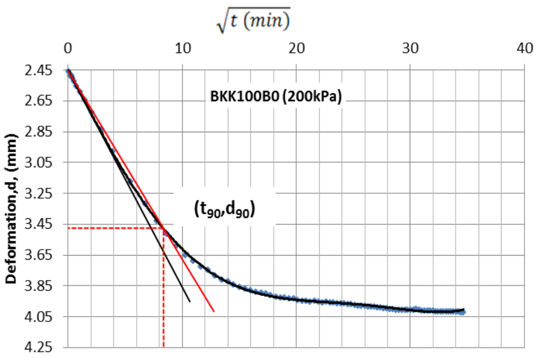

√t (min)
Deformation, d, (mm)
Unloading:
BKK80B20 (0kPa)
(t90, d90)

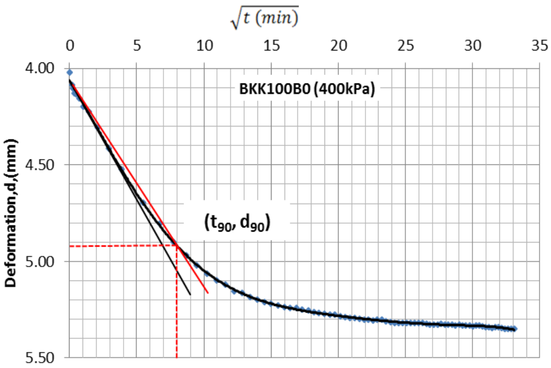

√t (min)
Deformation, d, (mm)
BKK100B0 (12.5kPa)
(t90,d90)

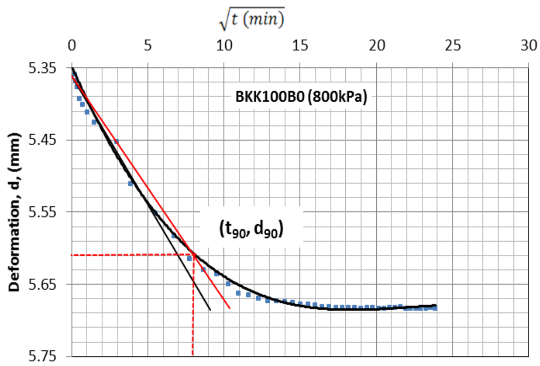

√t (min)
Deformation, d, (mm)
BKK100B0 (25kPa)
(t90, d90)

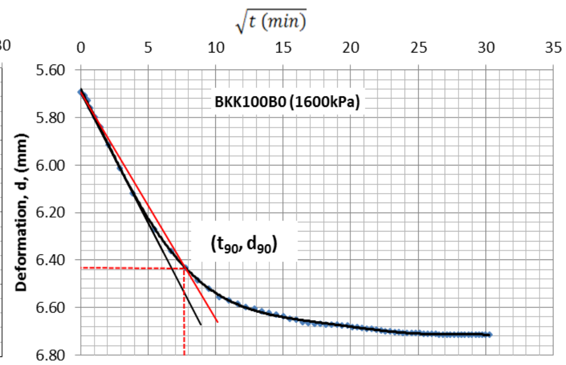

√t (min)
Deformation, d, (mm)
BKK100B0 (50kPa)
(t90, d90)

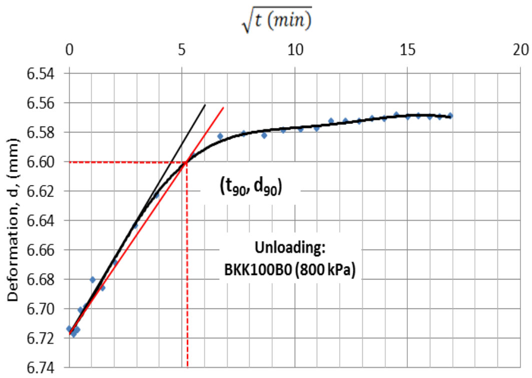

√t (min)
Deformation, d, (mm)
BKK100B0 (100kPa)
(t90, d90)

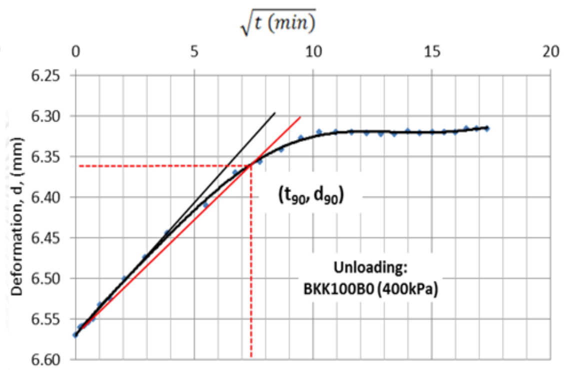

√t (min)
Deformation,d, (mm)
BKK100B0 (200kPa)
(t90,d90)

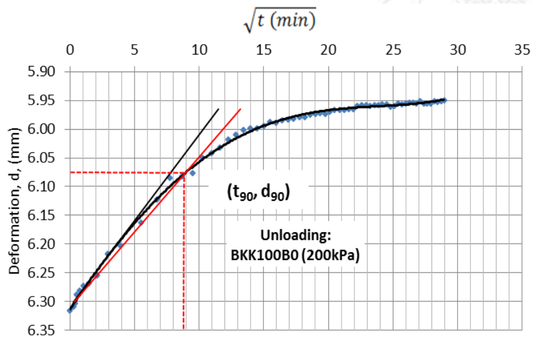

√t (min)
Deformation,d,(mm)
BKK100B0 (400kPa)
(t90, d90)

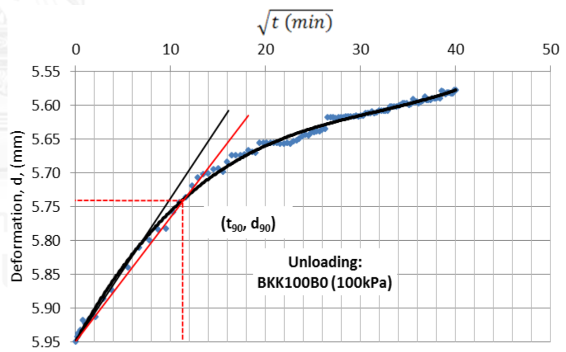

$\sqrt{t\ (min)}$
0 5 10 15 20 25 30
5.35 5.45 5.55 5.65 5.75
Deformation, d, (mm)
BKK100B0 (800kPa)
(t_{90}, d_{90})

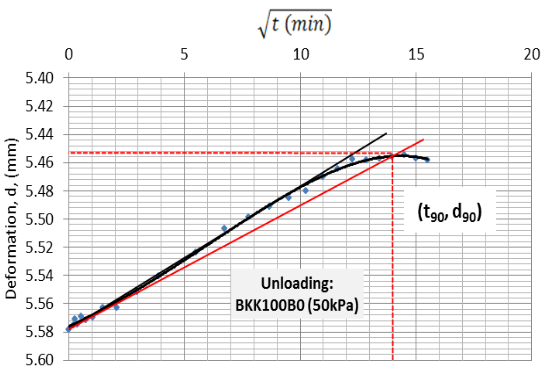

$\sqrt{t\ (min)}$
0 5 10 15 20 25 30 35
5.60 5.80 6.00 6.20 6.40 6.60 6.80
Deformation, d, (mm)
BKK100B0 (1600kPa)
(t_{90}, d_{90})

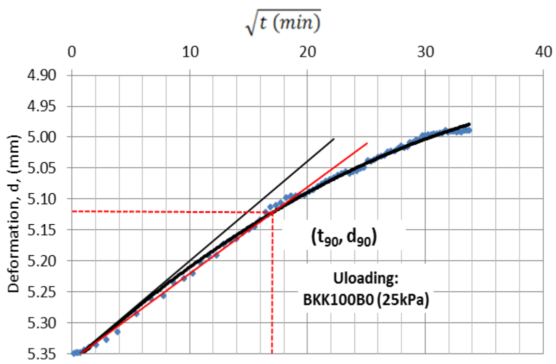

$\sqrt{t\ (min)}$
0 5 10 15 20
6.54 6.56 6.58 6.60 6.62 6.64 6.66 6.68 6.70 6.72 6.74
Deformation, d, (mm)
(t_{90}, d_{90})
Unloading:
BKK100B0 (800 kPa)


$\sqrt{t\ (min)}$
0 5 10 15 20
6.25 6.30 6.35 6.40 6.45 6.50 6.55 6.60
Deformation, d, (mm)
(t_{90}, d_{90})
Unloading:
BKK100B0 (400kPa)


$\sqrt{t\ (min)}$
0 5 10 15 20 25 30 35
5.90 5.95 6.00 6.05 6.10 6.15 6.20 6.25 6.30 6.35
Deformation, d, (mm)
(t_{90}, d_{90})
Unloading:
BKK100B0 (200kPa)


$\sqrt{t\ (min)}$
0 10 20 30 40 50
5.55 5.60 5.65 5.70 5.75 5.80 5.85 5.90 5.95
Deformation, d, (mm)
(t_{90}, d_{90})
Unloading:
BKK100B0 (100kPa)


$\sqrt{t\ (min)}$
0 5 10 15 20
5.40 5.42 5.44 5.46 5.48 5.50 5.52 5.54 5.56 5.58 5.60
Deformation, d, (mm)
(t_{90}, d_{90})
Unloading:
BKK100B0 (50kPa)


$\sqrt{t\ (min)}$
0 10 20 30 40
4.90 4.95 5.00 5.05 5.10 5.15 5.20 5.25 5.30 5.35
Deformation, d, (mm)
(t_{90}, d_{90})
Uloading:
BKK100B0 (25kPa)

VITA

The author, Mr. Sopheap Por, was born in Kampot province, Southern part of Cambodia, on November 13th, 1988. In June 2010, he graduated Bachelor’s degree from Rural Engineering department in the field of Geotechnical Engineering division at Institute of Technology of Cambodia. In October 2010, he awarded a scholarship supported by AUN/Seed-Net program (JICA) for his Master study in the field of Geological Engineering, Gadjah Mada University, Indonesia. After successfully completed his Master’s degree in October 2012, he has been gained a scholarship award in Sandwich Program under the support of AUN/Seed-Net program (JICA) for his Doctoral study in the field of Geotechnical Engineering, Department of Civil Engineering, Chulalongkorn University, under the supervision of Prof. Dr. Suched Likitlersuang. During his three-year Doctoral study, he had conducted a research work at Hokkaido University, Japan from March 2014 to October 2014 under the supervision of Associate Professor Dr. Satoshi Nishimura. He has published one international journal in Geotechnical Engineering Journal of the SEAGS & AGSSEA and another one journal is in reviewing. In additional, he has submitted one proceeding to the 20th National Convention on Civil Engineering, Chonburi, Thailand. This thesis is a partial fulfillment of requirements for the Doctoral degree in Civil Engineering department, Faculty of Engineering, Chulalongkorn University.