SHEAR STRENGTH ENHANCEMENT OF COMPACTED SOILS USING GEOPOLYMER

Soe Thiha

A Thesis Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Engineering in Geotechnology

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การเสริมสร้างกำลังรับแรงเฉือนของดินบดอัดด้วยสารจีโอพอลิเมอร์

นายโซ ทิฮา

วิทยานิพนธ์นี้เป็นส่วนหนึ่งของการศึกษาตามหลักสูตรปริญญาวิศวกรรมศาสตรมหาบัณฑิต

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SHEAR STRENGTH ENHANCEMENT OF COMPACTED SOILS USING GEOPOLYMER

Suranaree University of Technology has approved this thesis submitted in partial fulfillment of the requirements for a Master's Degree.

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โซ ทิฮา : การเสริมสร้างกำลังรับแรงเฉือนของดินบดอัดด้วยสารจีโอพอลิเมอร์ (SHEAR STRENGTH ENHANCEMENT OF COMPACTED SOILS USING GEOPOLYMER) อาจารย์ที่ปรึกษา : อาจารย์ ดร.เดโช เผือกภูมิ, 66 หน้า.

วัตถุประสงค์ของการศึกษาครั้งนี้ คือ การทดสอบในห้องปฏิบัติการเพื่อประเมินประสิทธิภาพของจีโอโพลิเมอร์ในการเสริมสร้างความแข็งแรงของวัสดุประเภทดิน จีโอโพลิเมอร์ คือ การนำเถ้าถ่าน (FA) ผสมกับโซดาไฟ (NaOH) และโซเดียมซิลิเกต (Na2SiO3) เป็นตัวกระตุ้นอัลคาไลน์ โดยอัตราส่วนของโซดาไฟ ต่อ โซเดียมซิลิเกต มีค่าเท่ากับ 1: 1 ในการศึกษาครั้งนี้ได้ใช้ดินทั้งหมดสามชนิด โดยตัวอย่างในการทดสอบถูกแบ่งออกเป็นสองกลุ่ม คือ กลุ่มตัวอย่างดินที่ผสมกับเถ้าถ่านบนพื้นฐานของจีโอโพลิเมอร์ และกลุ่มตัวอย่างดินที่ผสมกับน้ำ ในการเตรียมตัวอย่างของ FA จีโอโพลิเมอร์ สามารถดำเนินการโดยนำดินแต่ละชนิดผสมกับเถ้าถ่านบนพื้นฐานของจีโอโพลิเมอร์ 10% ของน้ำหนักดินแห้ง (FA / ดิน = 0.1) ตัวกระตุ้นอัลคาไลน์ (AL) ถูกกำหนดให้เท่ากับ 10% ของปริมาณความชื้นที่เหมาะสม (OMC) ของตัวอย่างดินโดยน้ำหนัก (AL / น้ำ = 0.1) หลังจากนั้นจึงทำการบดอัดที่ปริมาณความชื้นที่เหมาะสม แล้วจึงทดสอบหาค่ากำลังรับแรงเฉือนสูงสุดเพื่อบ่งบอกถึงค่าความแข็งหลังจากบ่มตัวที่ระยะเวลา 0 และ7 วัน ผลการศึกษาระบุว่า ปริมาณความชื้นที่เหมาะสมของดินทรายแป้งและดินตะกอนเม็ดละเอียด (sludge) ที่ผสมกับเถ้าถ่านบนพื้นฐานของจีโอโพลิเมอร์มีค่าสูงกว่ากลุ่มตัวอย่างที่ผสมกับน้ำเพียงเล็กน้อย ส่วนปริมาณความชื้นที่เหมาะสมของดินเหนียวมีค่าลดลงเพียงเล็กน้อยเมื่อผสมกับเถ้าถ่านบนพื้นฐานของจีโอโพลิเมอร์ ค่ากำลังรับแรงเฉือนของตัวอย่างดินที่ผสมกับเถ้าถ่านบนพื้นฐานของจีโอโพลิเมอร์มีแนวโน้มสูงกว่าตัวอย่างดินที่ผสมกับน้ำประมาณสองเท่า ซึ่งแสดงให้เห็นว่าเถ้าถ่านบนพื้นฐานของจีโอโพลิเมอร์สามารถเพิ่มกำลังรับแรงเฉือนของดินได้โดยการเพิ่มค่าแรงยึดติดและค่ามุมแรงเสียดทาน จากเทคนิคดังกล่าวสามารถนำไปประยุกต์ใช้ในการปรับปรุงคุณภาพดินโดยใช้จีโอโพลิเมอร์สำหรับการเสริมสร้างความแข็งแรงของเขื่อนดิน ความลาดชันของมวลดิน และมวลดินใต้ฐานรากเขื่อน

สาขาวิชา เทคโนโลยีธรณี	ลายมือชื่อนักศึกษา ________________
ปีการศึกษา 2557	ลายมือชื่ออาจารย์ที่ปรึกษา____________

SOE THIHA : SHEAR STRENGTH ENHANCEMENT OF SOILS USING GEOPOLYMER. THESIS ADVISOR : DECHO PHUEAKPHUM, Ph.D., 66 PP.

COHESION/FRICTION ANGLE/CURING PERIOD/SOIL IMPROVEMENT/GEOPOLYMER

The objective of this study is to experimentally assess the efficiency of geopolymer for strengthening of soil material. Geopolymer is a utilizing of fly-ash (FA) mixed with sodium hydroxide (NaOH) and sodium silicate (Na_2SiO_3) as alkaline activator. The ratio of NaOH and Na_2SiO_3 is 1:1. Three types of soils are used and two conditions of soil sample were prepared for testing; soil samples mixed with fly-ash based geopolymer and mixed with water. For sample with FA geopolymer, each type is mixed with FA based geopolymer of 10% of dry soil by weight (FA/soil = 0.1). Alkaline activator (AL) is fixed at 10% of optimum moisture content (OMC) of soil samples by weight (AL/water = 0.1). They were compacted at OMC state then performed the direct shear test to determine non-curing strength (at 0 day) and curing strength (at 7 days). The results indicate that OMC of silty sand and high plasticity silt (sludge) which mixed with fly-ash based geopolymer is slightly higher than those of sample mixed with water. OMC of clayey soil however is slightly decrease when mixed with fly-ash based geopolymer. Soils mixed with fly-ash based geopolymer tend to be higher state of the peak shear strength for curing sample about two times of soils mixed with water. This suggests that fly-ash based geopolymer is enhancing the shear strength of soils by increasing of cohesion and friction angle. Soil improvement

techniques using geopolymer can be applied for strengthening soil embankment, soil slope and earth dam foundation.

School of Geotechnology Student's Signature__________________

Academic Year 2014 Advisor's Signature__________________

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Soe Thiha

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

Page

TABLE OF CONTENTS (Continued)

Page

TABLE OF CONTENTS (Continued)

Page

LIST OF TABLES

LIST OF FIGURES

Figure **Page**

LIST OF FIGURES (Continued)

Figure **Page**

SYMBOLS AND ABBREVIATIONS

AL	=	Alkaline liquid activator
c_r	=	Cohesion in residual shear strength
c_p	=	Cohesion in peak shear strength
°C	=	Degree of Celcius (for temperature measurement)
E	=	Compactive effort (or) compactive energy
FA	=	Fly-ash
G0	=	Non-curing sample with geopolymer
G7	=	Curing sample with geopolymer for 7 days
Gp	=	Geopolymer
LL	=	Liquid limit of soil (Atterberg's limit test)
MDD	=	Maximum dry density (for Proctor compaction test)
OMC	=	Optimum moisture content (for Proctor compaction test)
PI	=	Plasticity index of soil (Atterberg's limit test)
PL	=	Plastic limit of soil (Atterberg's limit test)
γ_d	=	Dry density
γ_{max}	=	Maximum dry density
SG	=	Specific gravity
V_m	=	Volume of mass
w	=	Moisture content

SYMBOLS AND ABBREVIATIONS (Continued)

$\Delta\gamma_d$	=	Difference between two dry densities
ϕ_r	=	Friction angle in residual shear strength
ϕ_p	=	Friction angle in peak shear strength
σ_n	=	Normal stress
τ_f	=	Shear stress in failure state
τ	=	Shear stress
$\varnothing$	=	Diameter of cylinder

CHAPTER I

INTRODUCTION

1.1 Rationale and background

The stability of geotechnical structures (tunnel, dam, underground excavation, slope stability, etc.) depends on strength of foundation. The strength of foundation has been improved by several methods based on soil stabilization, and soil reinforcement in various applications. In foundation design, bearing capacity of soil is important to be stable for long-term period against environmental effects, which relies on shear strength of soil. A side-effect of cement production is the release of carbon dioxide (CO_2) into the atmosphere caused by the calcination of limestone and the combustion of fossil fuels during the process, and the production of one ton of cement contributes about 1 ton of CO_2 into the atmosphere. It is estimated that in producing a ton of Portland cement almost a ton of CO_2 is released to the atmosphere (Malhotra, 2002). The properties and uses of geopolymer are being explored in many scientific and industrial engineering, such as modern inorganic chemistry, physical chemistry, colloid chemistry, mineralogy, geology, and in another types of engineering technologies (Davidovits, 2011). To test the shear strength (τ) of soil, a disadvantage of ASTM (3080) standard method is a constraint of gain size of soil sample up to 4.75 mm, while the soil mass can include various grain sizes in real conditions. The three-ring testing device (Sonsakul, 2013) thus should be performed to obtain the shear test of soil in the laboratory after compaction with same mold.

The soil sample can be tested with grain sizes up to 10 mm. This study is concentrated about shear strength developed by geopolymer in three-ring mold.

1.2 Research objectives

The objective of this study is to assess the strength development of compacted soil stabilization with geopolymer in the three-ring shear testing device. Three types of soil are used and the soil samples are mixed with water, and with geopolymer. The compaction test is performed in three-ring mold. The mold will be used to test the direct shear strength in portable direct shear testing device. Four levels of curing duration are included under room temperature with the same moisture. The results from tests are compared on a same graph to figure out strength development between water and geopolymer.

1.3 Scope and limitations

The scope and limitations of the research include as follows.

1) Three types of soil will be used.
    - Silty sand from Ban Nong Bong, Maung district, Nakhonrachasima.
    - High plasticity silt (sludge) from Bang Khen water treatment plantMetropolitan Water Authority (MWA), Thailand.
    - High plasticity clay from Dankween, Nakhonrachasima.
2) The soil sample will be mixed with water and with geopolymer.
3) Geopolymer is a mixture of fly ash and alkaline liquid with equal amount of Na_2SiO_3 and NaOH.
4) Na_2SiO_3 and NaOH will be a liquid state.

5) The curing time has two stages as non-curing (for 0 day) and curing (for 7 days) under room temperature.

6) Normal stresses (σ_n) are fixed at 0.4, 0.6, 0.8 and 1 MPa.

1.4 Research methodology

The research methodology is shown in Figure 1.1 with seven steps including literature review, sample collection and preparation, basic properties test, three-ring direct shear test, resolving and comparison, discussion and conclusion, and thesis writing.

1.4.1 Literature review

Literature review will be focused to study the previous researches depending on effects of geopolymer to strength development of soils. The source of knowledge about this research is referred on text books, standard specifications, journals, technical reports and conference papers. A summary of literature review will be given in the thesis.

1.4.2 Soil collection and preparation

The soil samples in this research will be collected as three types of soil. The soil sample will be prepared to test basic properties. After basic properties tests, the selected soil at oven-dry state will be mixed with water for the three-ring direct shear tests, and the same selected soil sample with equal amount will be mixed with liquid mixture of geopolymer. Engineering properties as suggested by ASTM standard and all tests will be performed in the Geomechanics Research (GMR) laboratory of Suranaree University of Technology.

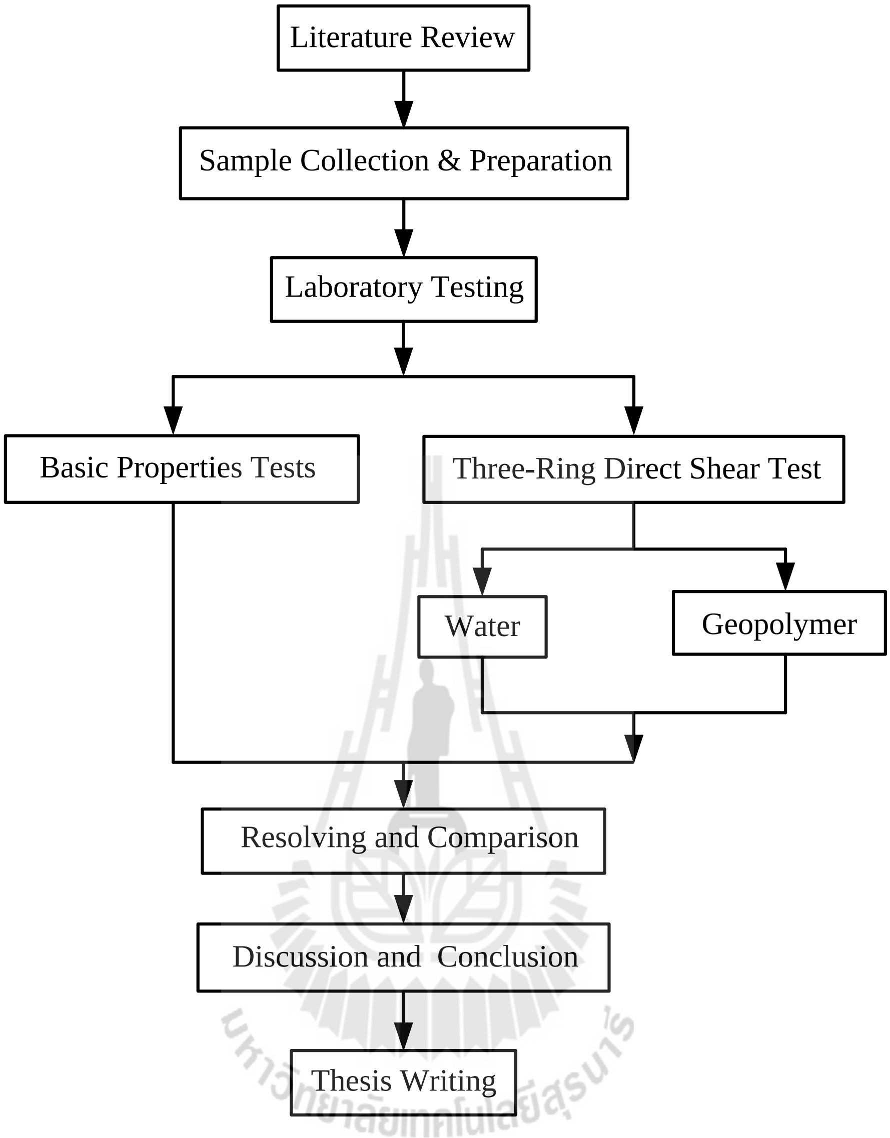


Figure 1.1 Research methodology.

1.4.3 Basic properties of soil

Basic properties of soil will be determined to know instinct behaviors of soils by classifying and then manipulate soils in actual fields. According to ASTM D4318, Atterberg's limit test will be conducted as an indicator of changes in volume and consistency when the water content changes. Specific gravity will be performed in accordance with ASTM D854. Grain size analysis test will follow ASTM D422.

Compaction test will be performed according to the ASTM D1557, using mold with diameter of 4 in (standard mold) and 4 in (three-ring mold).

1.4.4 Three-ring direct shear test

The three rings will be secured on the base plate using steel bolts and two steel clamps. The inner diameter is 10.16 cm, outer diameter is 10.76 cm, and the total height is 15.24 cm. The clamps prevent the rings from displacing during compaction. These clamps will be removed when the mold is placed into a direct shear load frame, and they can be sheared when the lateral force is applied during shear test. This means that the mold will become a shear box.

The three-ring mold requires a new shear test frame. There are two incipient shear planes of the compacted soil sample, one between the top and middle rings, and the other between the middle and bottom rings. The main components for the shear test frame are the lateral load system for pushing the middle ring, and the vertical load system for applying a constant normal stress on the compacted soil sample. The applied loads are obtained from two 20-ton hydraulic load cells, connected to separate hydraulic hand pumps. Pressure gages are used to measure the load. The shear and normal displacements are monitored by high precision dial gages.

Like in modified compaction test, the two amount of same soil blended with water only and with geopolymer, will be compacted by 10 lb. hammer and 27 blows per each of 6 layers in three-ring mold. The compacted molds will be fixed in portable direct shear device to test direct shear strength for 0 day. For 7 days, the compacted soil sample with three-ring will be kept in air-tight plastic bags. When the kept samples reach to a specific time, the mold will be performed in the portable shear testing device. Normal stresses will be 0.2, 0.4, 0.6, 0.8 and 1 MPa for the three-ring mold. Shear force is applied by a horizontal hydraulic load cell.

1.4.5 Data analysis

The data from direct shear tests will be plotted on a same graph. The illustrated data will compute peak shear strength and residual shear strength. The peak shear strength is used to calculate the cohesion and friction angle on the graph between shear strength and displacement by Mohr-Coulomb criteria. In each curing stage, the data will present two graphs for water and for geopolymer. For two curing duration (0 and 7 days), four graphs with cohesions and friction angles will be illustrated based on curing period. These four cohesions and four friction angles will plotted on a same graph between shear strength and curing period.

1.4.6 Discussion and conclusion

The thesis will be submitted at the end of research. The results of this research will be presented at an international conference or journal.

1.5 Expected results

This research will reveal the shear strength development of compacted soil with elapse time by using geopolymer. The change of cohesion and friction angle will indicate how the shear strength can be improved by using geopolymer at the same moisture content rather than by using water only. This research will be profitable in many fields, such as soil embankment construction, soil slope stability, strengthening foundation for industrial constructions and ground improvement for road constructions.

1.6 Thesis contents

Chapter I describes the objectives, rationale, and methodology of the research. **Chapter II** summarizes results of the literature review on geopolymer, compaction test, direct shear and basic properties tests. **Chapter III** describes the test materials and test methods for basic properties tests. **Chapter IV** presents the compaction test method, the performance assessment and discussion on test results. **Chapter V** shows direct shear test, test results and discussion. Conclusions and recommendations for future research are given in **Chapter VI**.

CHAPTER II

LITERATURE REVIEW

2.1 Introduction

This chapter includes theoretical background on basic property tests, compaction test, direct shear test and chemistry of geopolymer, and previous researches. The reviewing theory and background knowledge of soils improve an understanding the laboratory tests versus experimental application.

2.2 Basic properties tests

The basic properties test is known as index property test. It is essential to know soil type and how the soil manipulates for soil engineering work. Before starting construction, soil samples for field constructions will be performed basic properties tests. All of basic properties tests are following to ASTM standards.

2.3 Compaction test

Compaction is a densification of soil by removal of air with mechanical energy. The degree of compaction of a soil is measured by dry unit weight. When water is added to the soil during compaction, it acts as a softening agent on the soil particles. The soil particles slip over each other and move into a dense packed position. The dry unit weight after compaction first increases as the moisture content increases. When the moisture content is increased at the same compactive effort, the weight of the soil solids in a unit volume gradually increases.

Beyond optimum moisture content, any increase in the moisture content tends to decrease the dry unit weight. The increase water takes up the more spaces occupied by solid particles. The moisture content at the maximum dry weight is referred to as the optimum moisture content (Das, 2010). In Figure 2.1, water content is zero (w = 0) at the first state. Beyond that state, the added water content tends to increase a moist density until the optimum point in the middle of compaction curve. The condition at water content, w_1 shows clear understanding how to increase density of soil by expanding weight of soil in unit volume.

In the laboratory, Proctor (1933) proposed the compaction test for maximum dry density (MDD) and optimum moisture content (OMC). This compaction test is known as Proctor compaction test. The soil is compacted in a mold composed of upper ring and lower ring, which has inner diameter of 101.6 mm and high of 116.43

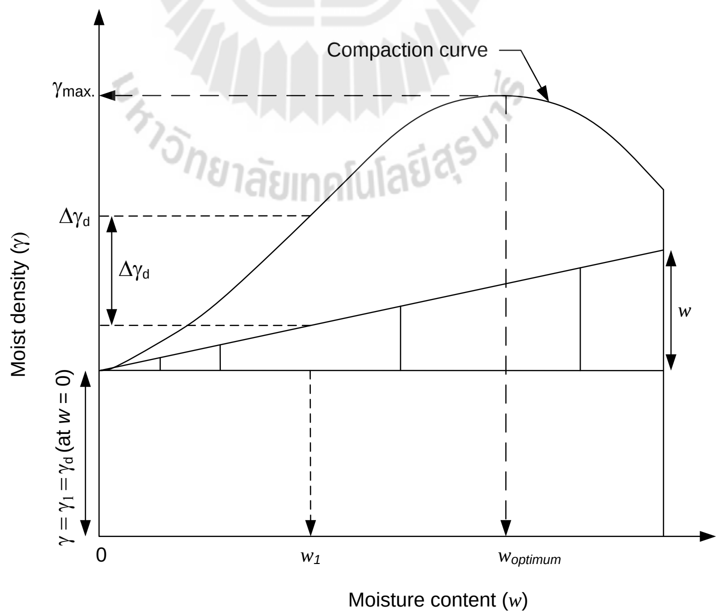


Figure 2.1 Principles of compaction (Das, 2010).

mm in the lower ring. When the soil is compacted in the mold, the compactive effort is taken from free fall high of a drop hammer. The free fall high of drop hammer high is 304.8 mm. The soil sample is compacted with various water contents in lower ring mold. After several compactions for each water contents, the moist unit weight of compacted soil (γ) is calculated as followed equation (2.1):

$$\gamma = \frac{W}{V_m} \tag{2.1}$$

where W is weight of compacted soil and V_m is volume of the mold (lower ring).

From each moist dry density, the dry density of soil can be calculated by substituting the individual moisture content of those moist dry densities in the following equation (2.2).

$$\gamma_d = \frac{\gamma}{1 + \frac{w(\%)}{100}} \tag{2.2}$$

where γ_d is the dry unit weight of soil sample and w is the individual water content of moist unit weight. After repeated compaction tests on various water contents, the compaction curve is drawn through individual water content and each dry density as shown in Figure 2.2.

Lee and Suedkamp (1972) studied compaction curves for 35 soil samples. They observed that four types of compaction curves can be found. These curves are bell shaped, one and one-half peak, double peak and old shaped as shown in Figure 2.3. The dry density of soil specimen is affected by type of soil and compaction

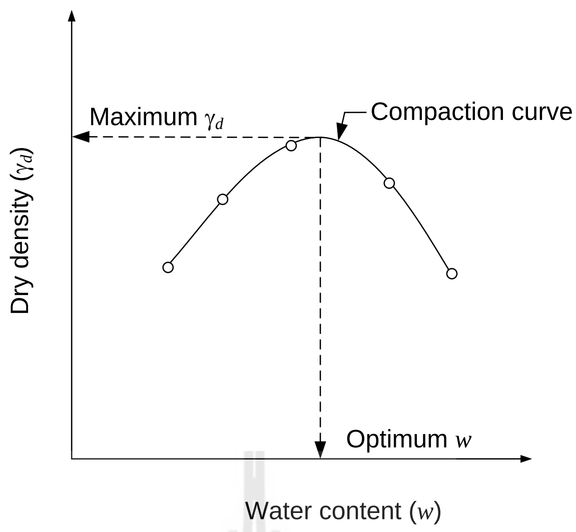


Figure 2.2 Proctor compaction test result.

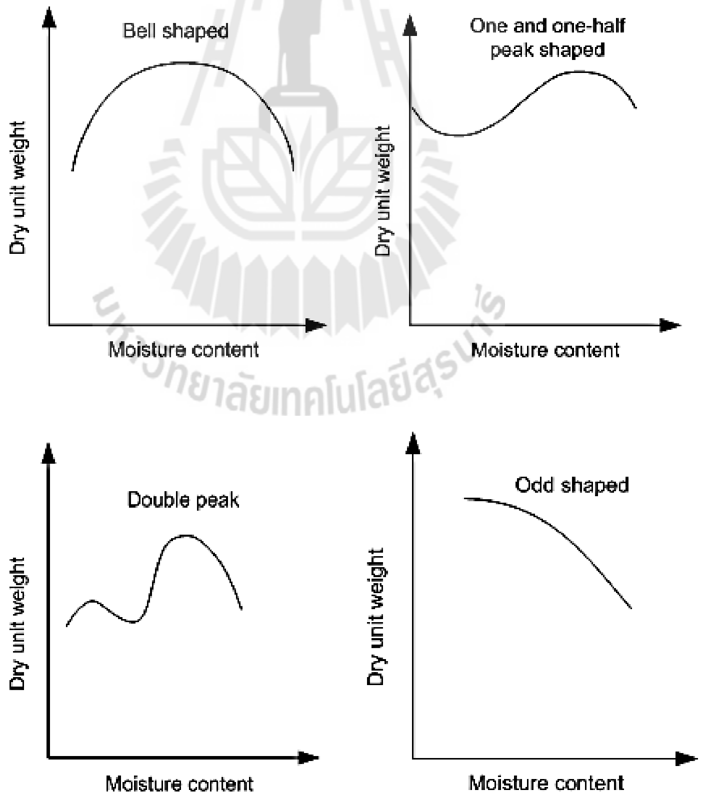


Figure 2.3 Typical curves of compaction test (Lee and Suedkamp, 1972).

energy. In general, it is higher in coarse grain soil than in fine grain soil. The compaction energy for proctor compaction test is calculated as following equation (2.3).

$$\mathrm{E} = \frac{\begin{pmatrix}\text{No. of}\\ \text{blow per}\\ \text{layer}\end{pmatrix} \mathrm{x} \begin{pmatrix}\text{No.}\\ \text{of}\\ \text{layer}\end{pmatrix} \mathrm{x} \begin{pmatrix}\text{Weight}\\ \text{of}\\ \text{Hammer}\end{pmatrix} \mathrm{x} \begin{pmatrix}\text{Height of}\\ \text{drop of}\\ \text{hammer}\end{pmatrix}}{\text{volume of mold (lower ring)}} \tag{2.3}$$

The two types of proctor compaction tests are divided into standard proctor test and modified proctor test, depending on the compactive effort (E). They are standard compaction test for compactive energy of 600 kN-m/m^3 and modified compaction test for compactive energy of 2,700 kN-m/m^3.

In this study, the soil samples will be blended two times, such as with water only and with geopolymer. The blended soil samples will be performed modified proctor test according to ASTM D1557, to know maximum dry density (MDD) and optimum moisture content (OMC).

2.4 Direct shear test

The direct shear test is the oldest and simplest form of shear test arrangement. The direct shear test apparatus is shown in Figure 2.4. The test equipment consists of a metal shear box in which the soil specimen is placed. The soil specimens may be square or circular in plan. The size of specimen generally used is about 51 mm × 51 mm or 102 mm × 102 mm across and about 25 mm high. The box is split horizontally into halves. Normal force on the specimen is applied from the top of the shear box. The normal stress on the specimens can be as great as 1050 kN/m^2 (1.05 MPa) (Das, 2010). Shear force is applied by moving one-half of shear box related to the other to cause failure in the soil specimen. Direct shear tests are repeated on similar specimens at various normal stresses. The normal stresses and the corresponding values, τ_f obtained from a number of tests are plotted on a graph from which the shear strength parameters are determined.

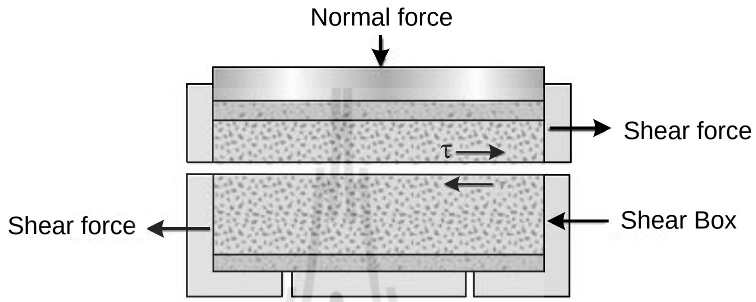


Figure 2.4 Direct shear box (Das, 2010).

The resisting shear stress increases with shear displacement until a failure shear stress which is called the peak shear strength. After failure stress is attained, the resisting shear stress gradually decreases as shear displacement increases until it finally reaches a constant value called the residual shear strength or peak shear strength as shown in Figure 2.5. When approaching to the peak and residual shear strength, test results of specimen are based on Mohr-Coulomb Criterion.

Mohr (1990) presented a theory for rupture in materials that contented that material fails because of a critical combination of normal stress and shearing stress and not from either maximum normal or shear stress alone. Thus, the functional relationship between normal stress and shear stress on a failure plane can be expressed in the following form

$$\tau_f = c + \sigma_n \tan \phi \tag{2.4}$$

The equation is so called Mohr-Coulomb failure criterion as shown in Figure 2.6.

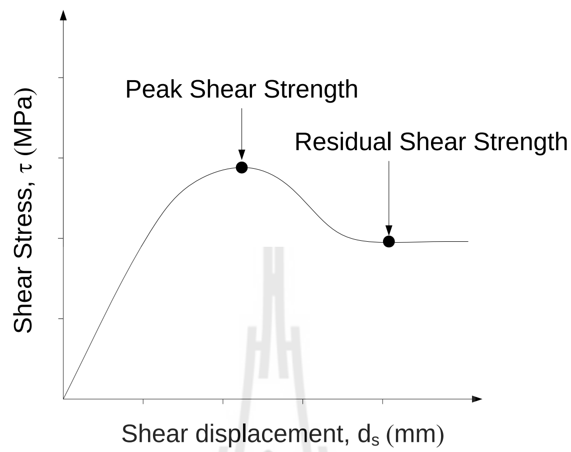


Figure 2.5 Nature of residual shear strength and peak shear strength (Das, 2010).

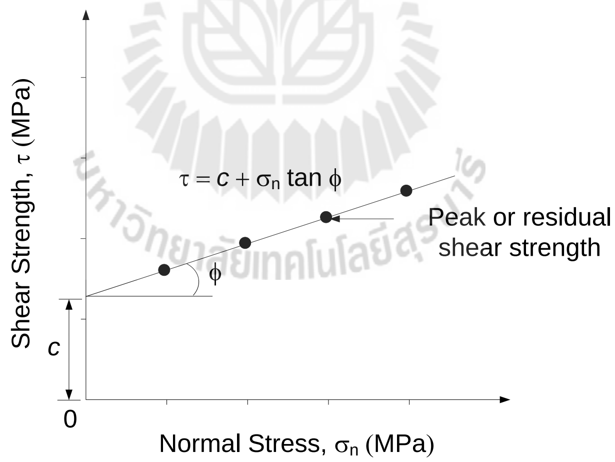


Figure 2.6 Mohr-Coulomb Criterion as a function of shear strengths and normal stresses.

2.5 Chemistry of geopolymer

In the inorganic-polymer technologies, Davidovits (1978) found "similar hydrothermal conditions which were controlling the synthesis of organic phenolic plastics on one hand, and of mineral feldspathoids and zeolites on the other hand" (Davidovits, 1991). Using mineral chemistry for development of mineral binders and mineral polymers, led to the development of amorphous to semi-crystalline three dimensional silico-aluminate structures which were termed 'geopolymers'. Poly (sialate) was suggested as the chemical designation of geopolymers based on silico-aluminates. The sialate network consists of SiO_4 and AlO_4 tetrahedra linked alternately by sharing all the oxygens. Poly(sialates) are chain and ring polymers with Si^{4+} and Al^{3+} in IV-fold coordination with oxygen and range from amorphous to semicrystalline, having an empirical formula:

$$M_n\{-(SiO_2)_z\text{-}AlO_2\}_n\,.\,wH_2O \tag{2.5}$$

where M is a cation such as potassium or sodium, 'n' is a degree of polycondensation, and 'z' is 1, 2, 3 (Wallah and Rangan, 2006). Three types of silico-aluminate structures from polymerization reactions are shown in Figure 2.7 and 2.8.

Geopolymerization is an exothermic reaction, and involves the chemical reaction of alumino-silicate oxides (Si_2O_5, Al_2O_2) with alkali polysilicates yielding polymeric Si-O-Al, as shown by the equation of polycondensation by alkali into poly(sialate-siloxo) in Figure 2.9.

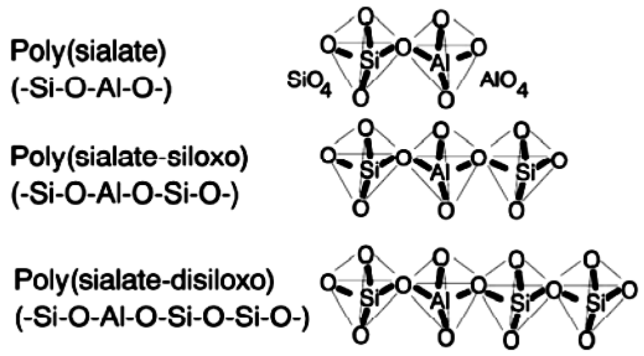


Figure 2.7 Chemical structure of Polysialates (Wallah and Rangan, 2006).

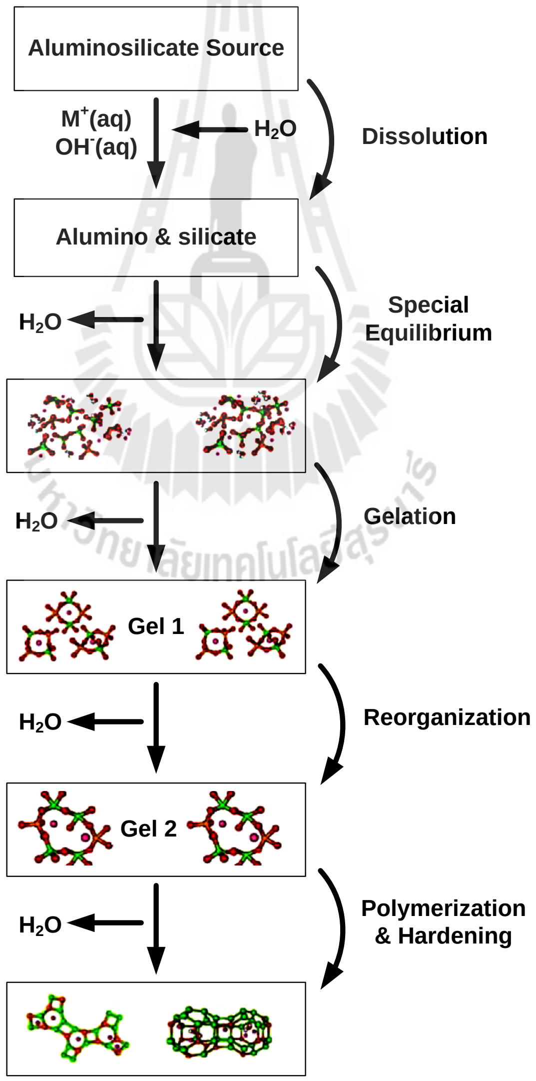


Figure 2.8 Conceptual model for geopolymerization (Duxson et al., 2007).

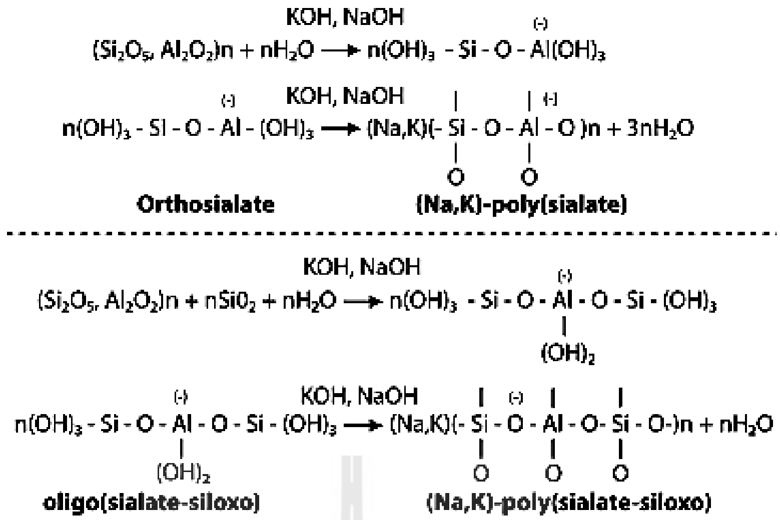


Figure 2.9 Geopolymerization reaction of an alumino-silicate (Davidovits, 1991).

The water is released during the chemical reaction in the formation of geopolymers. The water is expelled from the mixture during the curing process, indicating that water plays no role in the chemical reaction taking place, but rather provides the workability to the mixture during handling (Hardjito and Rangan, 2005).

2.6 Source materials and alkaline liquid

In the manufacture of geopolymer materials different source materials have been investigated. The main components of these materials must be silicon (Si) and aluminum (Al) in order to allow for the formation of the hardened geopolymer structure. Some of the most common forms of alumino-silicate source materials used by researchers for geopolymerization include slags, calcined clays, and coal fly ashes (Provis and Van Deventer, 2009). The source materials for geopolymers based on alumino-silicate should be rich in silicon (Si) and aluminium (Al). These could be natural minerals such as kaolinite, clays, micas, andalousite, spinel, etc whose empirical formula contains Si, Al, and oxygen (O) (Davidovits, 1988c).

Alternatively, by-product materials such as fly ash, silica fume, slag, rice-husk ash, red mud, etc could be used as source materials. The choice of the source materials for making geopolymers depends on factors such as availability, cost, and type of application and specific demand of the end users. Among the waste or by-product materials, fly ash and slag are the most potential source of geopolymers. Several studies have been reported related to the use of these source materials (Wallah and Rangan, 2006).

Another source of geopolymerization is an alkali liquid activator from soluble alkali metals that are usually Sodium or Potassium based (Wallah and Rangan, 2006). Since (1972) and Davidovits (1988c; 1988d) worked with kaolinite source material with alkalis (NaOH, KOH) to produce geopolymers. In geopolymerization, the alkali metal content of reacting minerals might have a substantial impact on strength development. This really is on the other hand with concrete manufacture which the clear presence of metals is undesirable as a result of stresses produced by alkali activation (Xu and Van Deventer, 2000b). Moreover, the mechanical properties of geopolymer cured for 7 days are not seriously affected when working with specimens with various alkali 13 compositions (Duxson et al., 2007c).

In the first study (Hardjito and Rangan, 2005) the effects of activator solution combinations were tested in the production of fly-ash based geopolymer concrete. Different amounts of sodium silicate solution were used, as well as different amounts of sodium hydroxide solution with molarities ranging from 8M to 16M. It was concluded that higher concentration (in molar units) of sodium hydroxide results in higher compressive strength and higher ratio of sodium silicate-to-sodium hydroxide ratio by mass results in higher compressive strength (Rivera, 2013).

2.7 Previous researches on shear strength test

Bagherzadeh-Khalkhali and Mirghasemi (2009) studied the effects of particle size on macro and micro mechanical behavior of coarse-grained soils, using both experimental tests and numerical simulations, on a series of both small (6cm×6cm×2cm) and large (30cm×30cm×15cm) scale direct shear tests on selected coarse-grained soils to determine the effect of stress level on the relationship between particle size and friction angle and behavior of samples. Approaches showed that the behavior of the coarse grained soil changes from strain hardening to softening during shearing as vertical stress increases. The internal friction angle reduces with increasing the stress level. Results showed that particle size greatly influences the mechanical behavior of the coarse-grained soils. The internal friction angle and the sample dilation increases with growing the particle size. An increase in the specimen scale leads to reduction of the apparent cohesion. Comparison of experimental and numerical tests reveals that the numerical simulation exaggerates the effect of particle size on the mechanical behavior.

Bergado et al. (2006) studied the laboratory tests for both index and engineering properties of the soil used as the compacted clay liner (CCL) and uniform gravel used as the protective layer between the lining system and the waste have been conducted. The soil used as the CCL has specific gravity of 2.70, liquid limit of 67%, and plastic limit of 31%, maximum dry density of 1.75 g /cm^3, and optimum moisture content of 14.5% as per standard proctor test. For soils, the failure envelope may show slight curvature, particularly under low normal stress. The shear stress versus displacement and shear stress versus normal stress curves for CCL are expected, the shear strength of the compacted clay is dependent on the applied normal stress. The internal friction angle for the compacted clay is high at low normal stress and decreases with increasing normal stress. For normal stresses below 200 kPa, the compacted clay yields a friction angle of 33 and for normal stresses above 200 kPa, the internal friction angles of the compacted clay is 19.24 degrees.

Suzuki et al. (2001) reported variation in residual strength of clay with shearing speed, 0.02 to 2.0 mm/min. They performed the ring shear test on kaolin and mudstone. The test results clarified the effect of shear displacement rate on the residual strength of soil and consolidated constant pressure ring shear tests with different shear displacement rates. The residual strengths of kaolin and mudstone were significantly influenced by the shear displacement rate. The ring shear behaviors of kaolin and mudstone are pointed out that the residual strength of a soil is notably changed by the shear displacement rate, and this tendency seems to be dependent on the physical properties of the soil. The results showed the relationships between the shear-normal stress ratio at peak and residual states, i.e. τ_p/σ_n, τ_r/σ_n and the shear displacement angle rate, θ for kaolin and mudstone, respectively. In the case of kaolin, τ_p/σ_n decrease with increasing the shear displacement angle rate above θ = 0.025 rad/min (D = 1.0 mm/min), whereas τ_p/σ_n becomes constant in a range of 0.0005 to 0.025 rad/min. In the case of mudstone, τ_p/σ_n decreases with increasing the shear displacement angle rater above θ = 0.005 rad/min (D = 0.2 mm/min).

Bhat et al. (2013) performed effect of shearing rate on residual strength of kaolin clay. The effects on the residual strength of kaolin clay due to the change in shearing rate were investigated using a torsional ring shear apparatus. In that apparatus, the specimen container has inner and outer diameters of approximately 8.0 cm and 12.0 cm, respectively, and a depth of 3.2 cm. The ratio of the outer to inner ring diameter is 1.5. The specimen is sheared through a level of 0.7 cm above the base plate. The normal load is transmitted to the specimen by the central shaft, which can be directly applied. The mechanisms are made in such a way that there is no effect of eccentricity during the application of a normal load. The shearing rate in the ring shear tests were varied in a ranging of 0.073 to 0.586 mm/min. Variation of residual strength on kaolin clay with the slow shearing rate and the mechanism behind it are discussed in this study. Test results show that hardly increases in residual strength with increase in shearing rate of kaolin clay. He concluded that the residual strength of kaolin clay is negligible with the shearing rate 0.073 mm/min to 0.162 mm/min. Hardly increase in residual strength occurred with the shearing rate varying from 0.233 mm/min to 0.586 mm/min. A change in shear mode in the shear zone may be during the change in shearing rate. This change in shear mode in shear mode could be thought as the rate effect mechanisms of residual shear strength.

Sonsakul et al. (2013) studied the performance assessment of three-ring compaction and direct shear testing device. The three-ring compaction and direct shear mold has been developed to obtain the optimum water content, dry density and shear strength of compacted soil samples. The device can shear the soil samples with grain size up to 10 mm. It can be used as a compaction mold and direct shear mold without removing the soil sample, and hence eliminating the sample disturbance.

Commercial grade bentonite is tested to verify that the three-ring mold can provide the results comparable to those obtained from the ASTM standard testing devices. Three types of soil, including clayey sand, poorly-graded sand and well-graded sand, are tested to assess the performance of the device. Their results are compared with those obtained from the ASTM standard test device. The results indicate that the shear strength, maximum dry density and optimum water content of the bentonite obtained from the three-ring mold and the ASTM standard mold are virtually identical. The three-ring mold yields the higher maximum dry density than those obtained from the standard mold. The shear strengths obtained from the three-ring mod are also higher than those from the standard shear test device. This is primarily because the three-ring mold can accommodate the soil particles up to 10 mm for the shear test, and hence resulting in higher shear strengths that are closer to the actual behavior of the soil under in-situ conditions.

2.8 Related studies on geopolymer

Davidovits (1978) coined the geopolymers which are new materials for fire and heat resistance coatings and adhesives, medicinal applications, high temperature ceramics, new binders for fire resistant fiber composites, toxic and radioactive waste encapsulation and new cement s for concrete. The properties and uses of geopolymers are being explored in many scientific and industrial disciplines: modern inorganic chemistry, physical chemistry, colloid chemistry, mineralogy, geology, and in other types of engineering process technologies. Geopolymer are part of polymer science, chemistry and technology that forms one of the major areas of materials science. Polymers are either organic material, i.e. carbon based, or inorganic polymer, for example silicon-based. The organic polymers comprise the classes of natural polymers (rubber, cellulose), synthetic organic polymers (textile fibers, plastics, films, elastomers, etc.) and natural biopolymers (biology, medicine, pharmacy). Raw materials used in the synthesis of silicon-based polymers are mainly rock-forming minerals of geological origin.

Chanprasert et al. (2014) studied the strength and microstructure of water treatment sludge-fly ash geopolymer. The compaction behavior of sludge-FA (fly ash) mixture is typical of compacted soil for a particular Na_2SiO_3/NaOH ratio (L). The unit weight of sludge-fly ash geopolymer increases with increasing L/FA until the maximum unit weight is attained at an optimum L/FA. Beyond this optimum value, the unit weight decrease as L/FA increases. The maximum unit weight is attained at L/FA ratio of approximately 1.3 for all Na_2SiO_3/NaOH ratios. At the optimum ingredient, the geopolymerization products increase with increasing heat temperature. The SEM image and XRD analysis show that the amount of geopolymerization products is more or less the same for heat temperatures of 75 ^{o}C and 85 ^{o}C. However, the loss of moisture in the sample heated at 85^{o}C results in micro-cracks and hence strength reduction. The geopolymerization process is primarily dependent upon the heat duration. Only the etching on the FA surface is found due to alkaline dissolution at early heat duration. The geopolymerization products increase with heat duration and subsequently weld clay and FA particles and fill up the pore space. The optimum heat temperature and duration for the optimum ingredient are 75^{o}C and 72 hours.

Phummiphan et al. (2014) found that the optimum liquid alkaline activators for each Na_2SiO_3: NaOH ratios are different and depend on amount of Na_2SiO_3 and NaOH solution. As the sodium hydroxide solution increases, the optimum liquid alkaline activator decreases. The optimum liquid alkaline activators for lateritic soil – FA geopolymer specimens at Na_2SiO_3/NaOH ratios of 100:0, 90:10, 80:20, and 50:50 are 19.6%, 19.13%, 18.38%, and 15.68%, respectively. The maximum compressive strength development of lateritic soil – FA geopolymer specimen is found at Na_2SiO_3: NaOH ratio of 90:10. The compressive strength increases dramatically during 7 to 28 days (max. 9.67 MPa) and then becomes almost constant after 28 days of curing. With the microstructure of lateritic soil–FA geopolymer, the formation of geopolymer in the soil is qualitatively confirmed. The FA geopolymer can improve the mechanical properties of the marginal soil. The study of marginal lateritic soil – FA geopolymer has significant impacts on pavement applications. Future work should be done to determine the appropriate proportion of the ingredient for subbase and the effective cost for soil stabilization works.

Sukmak et al. (2013) studied the strength development in clay-fly ash geopolymer, in which the fly ash (FA) replacement reduces liquid limit and dry unit weight of the clay, although the particle size distribution of FA and clay is almost the same. The compaction curves of clay–FA mixture and clay–FA geopolymer for the same clay–FA ratio are identical because the L insignificantly affects the index properties for the same FA/clay ratio. The NaOH leaches the silicon and aluminum in amorphous phase of FA and the Na_2SiO_3 acts as a binder. In this study, the liquid alkaline activator (L) is a mixture of Na_2SiO_3, consisting of 9% Na_2O and 30% SiO_2 by weight, and NaOH with a concentration of 10 molars. With this condition, the Na_2-SiO_3/NaOH ratio of 0.7 can be considered as constant for the manufacturing of clay–FA geopolymer. The optimum L/FA ratio is dependent upon only the FA replacement (FA/clay ratio). When the clay content decreases (the FA/clay decreases), the L required for the reaction are decreases. It is approximately 0.6 for FA/clay ratio of 0.3 and 0.5 for FA/clay ratio of 0.7. The relationship between strength and E/W is proposed. The optimum heat energy per weight (E/W) at the highest strength is approximately 8.50 °C h/g for FA/clay ratios of 0.3 and 0.5 and approximately 7.57 °C h/g for FA/clay ratio of 0.7. The relationship is very useful for production industry to estimate the heat temperature and duration to attain the target strength for the required weight of clay–FA geopolymer brick. The molding moisture content for the highest strength is the combination of mechanical and chemical components. The mechanical component is governed by the soil densification, where the OMC provides the densest packing. The contribution from the chemical component (geopolymerization) decreases with increasing the molding moisture content due to the reduction in L concentration. At very low moisture content, the L is not sufficient for geopolymerization reaction because it is taken by the clay particles for developing the soil structure, hence low strength is obtained. The moisture contents for the highest strength are 1.0 OMC, 0.8 OMC and 0.6 OMC for FA/clay ratios of 0.3, 0.5 and 0.7, respectively.

Chimoye (2014) also studies the strength of soft Bangkok clay improved by geopolymer from palm fuel ash. In case, the parameters are initial water content, percentage of NaOH, percentage of palm fuel ash. The water content of soft clay varies from 0.8LL, LL and 1.5LL where LL is liquid limit of soft Bangkok clay. The percentage of palm fuel ash varies from 5%, 10%, 15% and 20% of dry weight of soft clay. The initial water content is the main parameter to increase the strength, and also the NaOH and the palm fuel ash is the minor parameter to increase strength. It can be concluded that the higher the percentage of either NaOH or palm fuel ash is, the higher the strength will be. In the opposite way, the lower water content is, the higher the strength will be. When the percentage of NaOH and palm fuel ash increase, the water contents of the samples decrease. The geopolymer from palm fuel ash can be used of cement in soil stabilization.

Moayedi et al. (2011) researched the effect of sodium silicate on unconfined compressive strength of soft clay. An addition of 5 mol/L sodium silicate (Na_2SiO_3) showed the highest unconfined compressive strength (UCS) results. However the effect of chemical molarities on UCS become less and less, with longer curing time. The paper was concluded that sodium silicate still could be known as one of the best secondary additives which help to increasing the pH and make a suitable environment for other stabilizers such as cement as well as lime. Moreover, temperature and length of time curing at which curing took place had a significant influence on the amount of strength development.

CHAPTER III

SAMPLE COLLECTION, PREPARATION AND BASIC PROPERTY TESTS

3.1 Introduction

This chapter describes sample collection, sample preparation to perform laboratory tests and basic property tests. Three types of soil samples are selected, and the soil samples are stabilized with a combination of two materials which are fly ash and alkaline liquid. The homogenous mixture of fly ash and alkaline liquid is called fly ash based geopolymer (FA geopolymer). The soil samples are mixed with water, and mixed with FA geopolymer. Both compacted soil samples with water and with geopolymer are cured for 7 days and non-cured (0 day) under ambient temperature to be continued direct shear test.

3.2 Sample collection

3.2.1 Raw materials

Three types of samples are used in this study. They are silty sand from Ban Nong Bong, Maung district, Nakhon Ratchasima, sludge from Bang Khen water treatment plant, Bangkok and high plasticity clay from Dan Keen, Chock Chai district, Nakhon Ratchasima. These three types of soils represent as tested samples to approach the objective of this study, in which sludge and high plasticity clay are describing as fine grains soil and silty sand as coarse grains soil. The soils samples are collected from the layer under top soils so that the plant roots are removed away from soil samples before sieving. The roots reduce the properties of soil and engineering properties in the laboratory. After collecting these samples, they are tested to know field moisture content which is an important role of most engineering construction works. The laboratory test methods follow ASTM standards to reduce variability of test results in this study.

3.2.2 Fly ash

Fly ash (FA) is a by-product of waste materials obtained from Mae Moh Power plant in Northern Thailand. The chemical composition of fly ash is as shown in Table 3.1. FA particles are generally fine and spherical in shape. According to ASTM D618, FA is classified as class F and class C in which one difference between the Class C and F ashes is the minimum limit of 50 % of the combination of SiO_2, Al_2O_3, and Fe_2O_3 for Class C and the minimum limit of 70 % for Class F.

Table 3.1 Chemical composition of fly ash (FA) (Phummiphan, 2014).

Chemical composition (%)	Fly ash (FA)
SiO_2	36.00
Al_2O_3	16.80
Fe_2O_3	17.64
SiO_{2+} Al_2O_{3+} Fe_2O_3	70.44 (>70 Minimum)
CaO	26.73
SO_3	-
K_2O	1.83
TiO_2	0.48
MnO_2	0.15
Br_2O	-

3.2.3 Alkaline liquid

Alkaline liquid (AL) is a mixture of sodium silicate (Na_2SiO_3) and sodium hydroxide (NaOH). Both of chemicals are ready to use at liquid state. Sodium silicate is composed of 15.5% (NaOH), 32.75 % (SiO_2) and water of 51.75 % by weight, and sodium hydroxide (NaOH) is 12.5 molars in solution, which is commercial use. These two types of Na_2SiO_3 and NaOH are selected to use in this study because they are easy to find as needed amount of laboratory tests due to common local use.

3.3 Sample preparation

The three types of soil samples are prepared needs to perform laboratory tests according to ASTM standards. The laboratory tests are decided as the basic properties and engineering properties of soil samples. The collected soil samples from actual fields are prepared to conduct main laboratory tests, such as compaction test and direct shear test after the basic properties tests. The sample preparation for compaction test and direct shear test has two preparation based on water and geopolymer.

3.4 Basic property tests

The basic properties are important in all soil types, known as index properties of soil. The index properties of soil are divided as two classes of soil grain properties and soil aggregate properties. The soil grain properties are size, shape and character of soil grains. The soil aggregate properties are significant in density of cohesion-less soils. For cohesive soils, consistency and mineralogy of character of soil grains are principle. The basic properties of soils from laboratory tests are determined classification of the soil and those soils are manipulated to control field works, depended on their behaviors. This chapter reveals the basic properties of soils to index the intrinsic value of soils. The basic property tests are including as natural moisture content, specific gravity, Atterberg's limit and grain size analysis. Those test methods are performed according to ASTM standards.

3.4.1 Natural water content

For many materials, the water content is one of the most significant index properties used in establishing a correlation between soil behavior and its index properties. The natural water content (w) is measured from water content in original soil layer of field condition. This test method is a determination of the water (moisture) content in the laboratory by weight of soil and rock which are most applicable. The test method is performing according to ASTM D2216. The test specimen collected from field is dried in an oven at a temperature of 110°± 5°C to a constant mass. The loss of mass due to drying is considered to be water. The water content is calculated using the mass of water and the mass of the dry specimen.

3.4.2 Specific gravity

The test method covers the determination of the specific gravity of soil solids that pass the 4.75-mm (No. 4) sieve, and to understand a general way to find specific gravity of substance greater than one, which composes of small particles. The test method is conducted in accordance with ASTM D854. For the determination of specific gravity, the air dried soil about 50 grams is placed in a pycnometer. The proper amount of water is added into the pycnometer and stir well with glass stick. The pycnometer is heated on hot plate to deaerate to come out air bubbles dissolved in the mixture by boiling method. After heating, more water is filled into the pycnometer until reaching the standard mark on the neck of pycnometer. The temperature decreasing based on time and the weight of pycnometer are recorded. After the decreasing temperature is reached under 30 °C, the recording is stopped and all mixtures are taken out of the pycnometer to dry in the oven. To determine the specific gravity of soil solid, weight of oven dried soil is multiplied by temperature correction factor. The various specific gravity of soil solid is depended on soil type.

3.4.3 Atterberg's limit

The test is to describe the consistency of fine grained soils with various moisture contents. The consistency of soil which can be deformed is the relationship between soil deformation and rearrangement in soil grains with variable water content. Atterberg's limit test carries out plastic limit (PL), liquid limit (LL) and plasticity index (PI) of soils. The test method is following ASTM D4318. From air dried specimen, any material retained on a 425-µm (No. 40) sieve is removed and the test samples need to pass through 0.475 mm (No. 40) sieve to perform Atterberg's limit test. The soil sample is prepared as recommendation of ASTM standard. The liquid limit is prepared by set of liquid limit device. The set is including as a brass cup, grooving tool, and dropping cup machine. The soil sample is prepared in brass cup where the soil is placed a part of soil in standard cup and cut by a groove of standard dimensions. The groove width is 13-mm about ½-in and the brass cup is dropped from 10-mm by apparatus operating with a rate of two shocks per second. When the groove is closed each other of sides, the soil sample is taken to dry in the oven and the number of dropped shocks is recorded. Repeated manner is at least three times between 15 shocks and 25 shocks. The liquid limit is determined the water content at the 25 shocks to be closer the each side of groove of brass cup. The plastic limit is the water content at which the soil sample cannot be longer by deformation with a thread diameter about 3.2-mm (1/8-in) without crumbling.

3.4.4 Grain size analysis

The soil samples are performed grain size analysis to classify shape and size of soil grains according to ASTM D422. The grain size analysis test is divided into two type, wet sieve analysis for fine grain soil and dry sieve analysis for coarse grain soil. The proper soil samples will be broken up the aggregations of soil with rubber hammer in a tray until the separated grains. If the soil has the percent finer passing sieve No.4 (4.75-mm) more than 90% and passing sieve No.200 (0.075-mm) more than 20%, the test method is following wet sieve analysis and hydrometer test will be performed. The soil sample about 50 grams is placed in the 125-ml of sodium hexametaphosphate solution (40g/L) and it is stirred until the wetted condition thoroughly. The soil mixture is allowed to soak in beaker for 16 hours to 24 hours. After soaking period, the soil mixture is continued to hydrometer test procedures according ASTM standard. If the soil sample has the percent finer passing sieve No.4 (4.75-mm) not more than 90 percent and passing sieve No.200 (0.075-mm) not more than 20%, the dry sieve analysis test is performed. The unified soil classification system (USCS) follow ASTM D2487 and decides the percent finer of opening diameter into specific soil type on a semi-log graph. The test results show that three types of soil samples are silty sand as SM, sludge as MH and high plasticity clay as CH. The envelope curves of sieve analysis for all soil types are as shown in Figure 3.1 and all sieves with opening are used in accordance with ASTM recommendation as shown in Table 3.2.

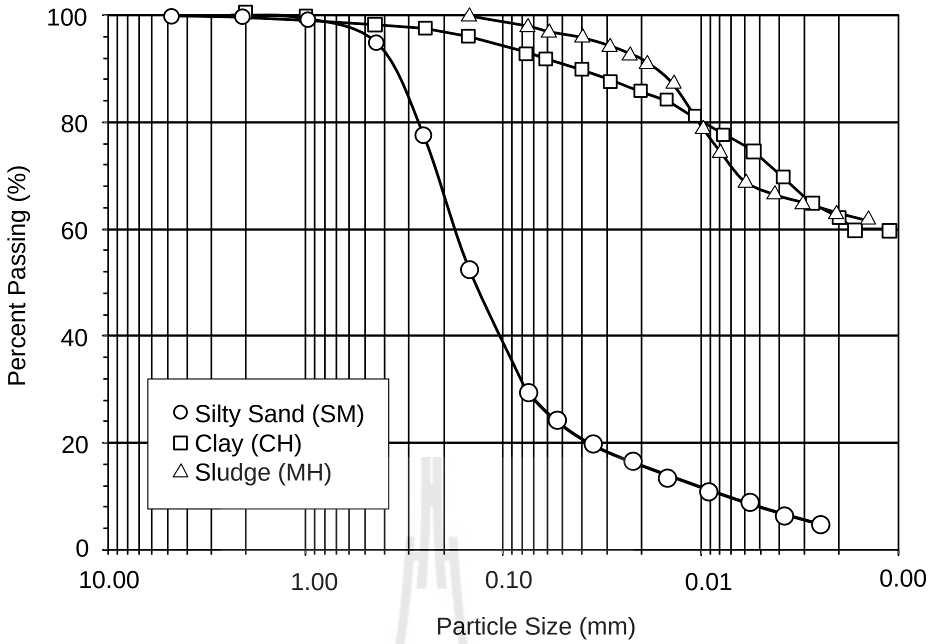


Figure 3.1 Particle size distribution curves of three types of soils.

Table 3.2 ASTM sieve arrangement with opening.

No.	Sieve	Opening (mm)
1	1/2"	12.7
2	3/8"	9.52
3	No.4	4.75
4	No.10	2.00
5	No.20	0.85
6	No.40	0.43
7	No.60	0.25
8	No.100	0.15
9	No.200	0.08
10	Pan	0.00

3.4.5 Test results

According to USCS, three types of soils are classified as silty sand, high plasticity clay and high plasticity silt. The natural moisture contents (w) are different among soil types. They are depended on weather condition. When silty sand and high plasticity clay were collected from fields, the weather is wet in late monsoon season. High plasticity silt (sludge) was kept inside rice polyethylene woven bag in the laboratory since last year. After Atterberg's limit test, silty sand is very low in PI of 0.6 percent and another two soils are 29.2 percent and 23 percent, respectively. The basic properties tests results of three soils are tabulated as shown in Table 3.3.

Table 3.3 Basic properties of three types of soils.

Locations	w (%)	SG	LL(%)	PI(%)	Soil Types
Ban Nong Bong, Muang district, Nakhon Ratchasima	3.0	2.68	12.70	0.60	Silty sand(SM)
Dan Keen, Chok Chai district, Nakhon Ratchasima	10.5	2.67	68.00	39.20	High plasticity clay (CH)
Bang Khen water treatment plant, Bangkok	5.6	2.56	55.00	23.00	High plasticity silt (MH)

CHAPTER IV

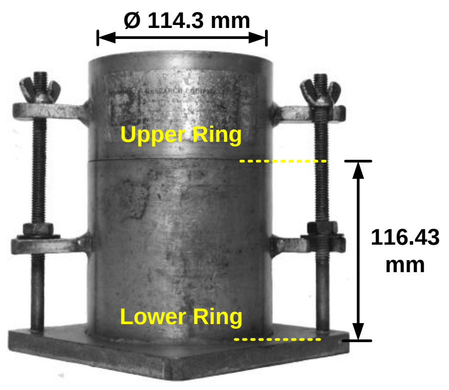
COMPACTION TEST

4.1 Introduction

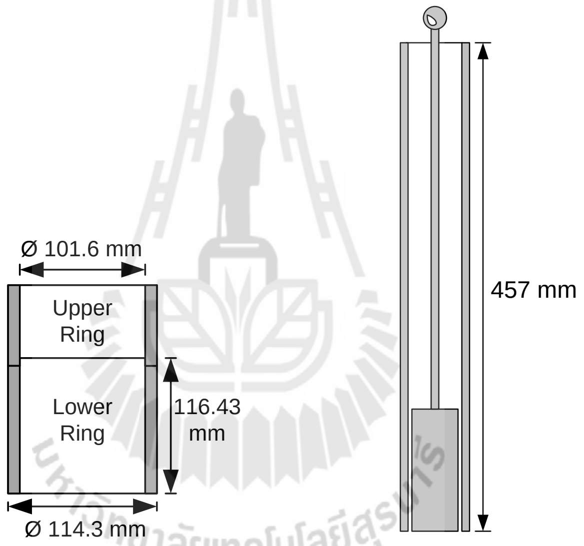
The compaction test is the most commonly use to improve the properties of soil layer in engineering fields, such as road construction, soil improvement works and earth dam construction. By compacting soil layers, the mechanical strengths of those layers are increased as high shear strength and high compressive strength against failure load than loose state. The compaction of soil appears to make closer soil grains in soil specimen to remove air. The closer soil grains get compacted and somewhat decreased air voids. The decreased air voids release undesirable settlement and increase stability of slope of embankments. The aim of compacting earth fills is to reduce settlement and permeability and to increase shear strength. Compaction is essential in many geotechnical applications (Horpibulsuk et al., 2013). In recent years, chemical stabilized soil techniques are developed in soil engineering works for compaction works. Some chemicals react with the composition of natural soils to be easy removal of air and to be closer soil grains each other. This chapter discusses about experiments in laboratory, test results of soils samples and discussion.

4.2 Experimental work

The proctor compaction test is easy to perform in the laboratory due to workability of simple equipment. Before performing the compaction test, the soil specimen is passed sieve ¾ inches or No.4 to get a standard size of specimen. The standard proctor test of two proctor tests is performed according to ASTM D698. The soil sample is compacted in a mold composed of upper ring and lower ring, which has inner diameter of 101.6 mm and high of 116.43 mm in the lower ring. When the soil is compacted in the mold, the compactive effort is 600 kN-m/m^3, which is taken from free fall high of hammer. In the lower ring of mold, the soil sample is compacted in equal three layers with 25 or 56 blows of the 5.5 lbs hammer dropped from 12-in height. The modified proctor compaction test is following ASTM D1557. The same mold is used as in the standard compaction mold, but the equal five layers are used to compact the soil specimens and the hammer weight is 10 lbs with 18 inches height (Figure 4.1). The comparisons between standard and modified compaction test are tabulated in Table 4.1. Between two proctor compaction tests, the modified compaction test is more reliable for soil engineering works because the most of construction projects are used heavy duty compacting machines in the field. The compacted soil layers in those sites are attained higher density over the maximum density of standard proctor test. In this study, the modified compaction test, thus, is employed as an essential test for the mechanical property of selected soils in case of field compaction works.



(a)



(b) (c)

Figure 4.1 Standard equipment of modified proctor compaction test (ASTM D1557) (a) standard mold (b) cross-section of standard mold (c) dropped hammer.

Table 4.1 Comparison between standard and modified proctor test (Reddy, 2002).

	Standard Proctor ASTM 698			Modified Proctor ASTM1557		
	Method A	**Method B**	**Method C**	**Method A**	**Method B**	**Method C**
Material	≤ 20% Retained on No.4 Sieve	>20% Retained on No.4 ≤ 20% Retained on 3/8"Sieve	>20% Retained on No.3/8" <30% Retained on 3/4"Sieve	≤ 20% Retained on No.4 Sieve	>20% Retained on No.4 ≤ 20% Retained on 3/8"Sieve	>20% Retained on No.3/8" <30% Retained on 3/4"Sieve
Passing sieve. No	SieveNo.4	3/8"Sieve	3/4"Sieve	SieveNo.4	3/8"Sieve	3/4"Sieve
Mold	4" Dia.	4" Dia.	6" Dia.	4" Dia.	4" Dia.	6" Dia.
Number of layer	3	3	3	5	5	5
Number of blows/La-yer	25	25	56	25	25	56

4.2.1 Soil-water

The oven dry soils under sieve No.4 are taken as the amount of 2.5 kg. The proper water content is added to the soil sample, which is depended on soil type and based on experience. For sand, the added water should be start from two percent and for high plasticity clay and sludge, it should be started from five percent of dry soil by weight. Because plastic index of coarse-grained soils are likely to be lower than 10, those optimum moisture content is lower. As an interval of moisture content for individual point on compaction curve, two percent is proper. In the other hand, the fine grain soils are higher in plasticity index and in optimum moisture content as well. The proper interval for moisture content for individual point of the fine grain soils is five percent. The samples then are blended with water for about ten minutes to be homogenous mixture. After blending, the mixture of soil and water is continued as a conventional procedure of the compaction test. The mixture is compacted in the standard mold for equal five layers with 25 blows. The hammer weight is 10 lbs and the free fall height is 457 mm. The compaction energy is 2,700 kN-m/m^3. After compacting soil sample in the mold with equal three layers, the upper ring is removed from the mold and the lower ring is trimmed and weight. The small amount of soil from the lower ring is taken to measure moisture content by drying in the oven for 16 to 24 hours. Those moisture contents are applied in the equation which can calculate the dry density or dry unit weight. All standard requirements and standard procedures are as follow in ASTM D1557. For various water contents, the mixing task is repeated at least five times to figure out optimum moisture content related with maximum water content through the whole compaction curve.

4.2.2 Soil-fly ash geopolymer

Likewise in soil-water mixture, the oven dry sample under sieve No.4 is used. The proper amount of oven dry soil is about 2.5 kg to mix with fly ash based geopolymer. Fly ash is taken as a ratio of 0.1 by weight of dry soil (FA/soil = 0.1) and blended with the dry soil in a mixer for about five minutes. Liquid alkaline activator (LA) and water with ratio of 0.1 (LA/water = 0.1) is added into the mixture of dry soil and FA. Then, they are blended for about ten minutes in a mixer to be a homogenous mixture again. The compaction procedures are same to previous task in soil-water mixture. The moisture content with LA/water = 0.1 is started from two percent for silty sand and five percent for sludge and high plasticity clay which have high plasticity index. Until two individual points beyond optimum moisture content, the compaction test is repeated again and again. The samples mixture is compacted in each layer of equal five layers. After compaction test, the soil samples are kept in oven to calculate moisture contents and dry densities to draw a compaction curve. The samples preparation process for test samples is as shown in Figure 4.2.

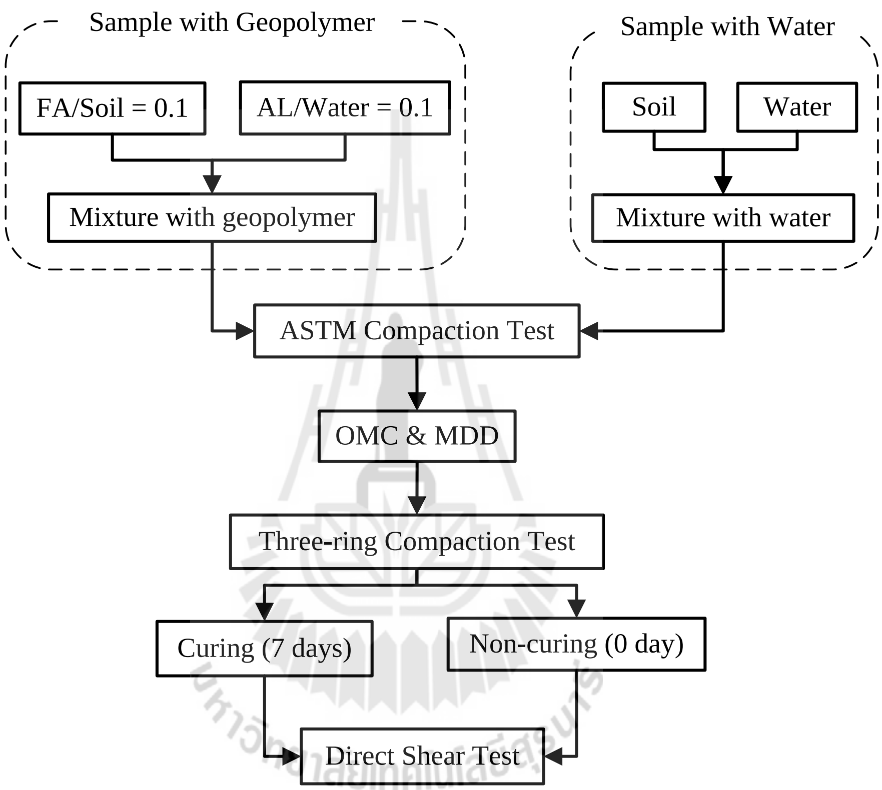
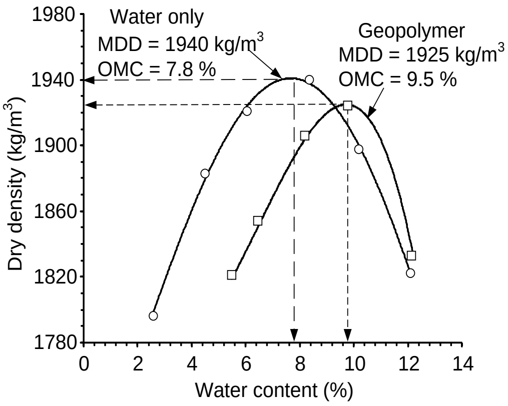


Figure 4.2 Mixed process for laboratory tests.

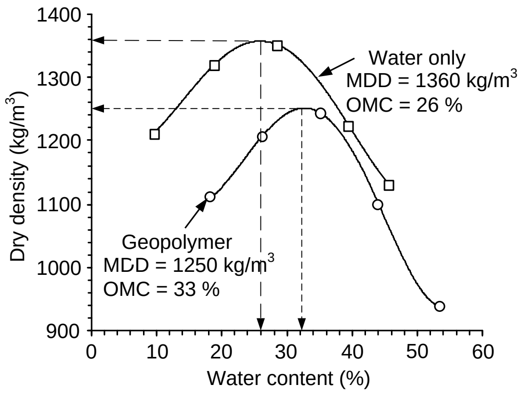
4.3 Test results and discussion

The moisture contents from the oven dry soil samples after compaction test are used to calculate dry densities in individual moisture contents. These moisture contents and dry densities are illustrated in a same graph. X axis is assumed as moisture contents and Y axis is assumed as dry densities. Then the compaction curve is drawn through the individual points as shown in Figure 4.3. In compaction graphs, the peak point on each graph is the maximum for dry density and the optimum point for moisture content. All of compaction test results are shown in Table 4.1.

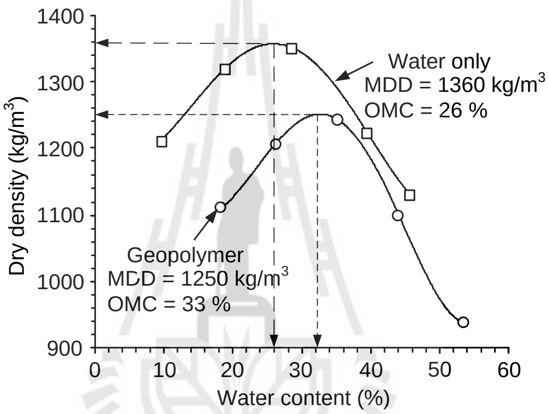
According each soil types, the compaction curves are different in optimum moisture content (OMC) and maximum dry density (MDD) (Table 4.2). Moreover the different mixing between water and FA geopolymer gives the different result. In silty sand (SM) with water, the optimum moisture content is 7.8 percent and the maximum dry density is 1,940 kg/m^3. When the silty sand is mixed with FA geopolymer, the optimum moisture content is higher than sample with water as 9.5 percent. But the maximum dry density is lower than sample with water as 1,925 kg/m^3. In sludge soil (MH), the behavior of compaction is similar to silty sand. The optimum moisture content of sample with water is higher than sample with FA geopolymer. When the sample is mixed with water, the optimum moisture content is 26 percent and the maximum dry density is 1360 kg/m^3. But the optimum moisture content with FA geopolymer is 33 percent and the maximum dry density is 1,250 kg/m^3. For high plasticity clay (CH), the compaction behavior is not following previous two soil samples. The optimum moisture content and the maximum dry density of sample with FA geopolymer are higher than sample with water. While the optimum moisture content of sample with FA geopolymer 21 percent and the



(a) Silty sand



(b) High plasticity clay



(c) Sludge

Figure 4.3 Typical compaction curves.

Tabel 4.2 Compaction parameters of three types of soils.

Soil Type	Compaction Characteristic	OMC (%)	MDD (kg/m^3)
SM	Water	7.8	1,940
	Geopolymer	9.5	1,925
CH	Water	21.0	1,634
	Geopolymer	19.0	1,573
MH	Water	26.0	1,360
	Geopolymer	32.0	1,250

maximum dry density is 1634 kg/m^3, the optimum moisture content and the maximum dry density of sample with water are 19 percent and 1,571 kg/m^3.

Theory of soil mechanics generally indicates that moisture contents of a soil sample beyond OMC decrease the dry density of that soil under the same compaction energy. Both of silty sand (SM) and high plasticity silt (MH) agree with this concept, except of high plasticity clay (CH). OMC of silty sand and high plasticity silt which mixed with FA geopolymer is slightly higher than those of sample mixed with tap water. Otherwise, dry densities of these soils are decreased. When high plasticity clay is mixed with FA based geopolymer, the dry densities are decreased but OMC is decreased against the theory of soil mechanics concept as well.

CHAPTER V

DIRECT SHEAR TEST

5.1 Introduction

The shear strength is a challenging aspect of soil engineering works as soil slope stability in dam construction (tailing dam and earth fill dam) and retaining structure in foundation. To be stable the soil slope, shear strength is essential to resist overburden pressure of soil strata. Cohesion and friction are main parameters for shear strength of soil. For determination of shear strength of soil, direct shear test is the oldest and simplest form in the laboratory. The direct shear box test is a conceptually simple test that apparently was used for soil testing as early as 1776 by Coulomb (Lambe and Whitman, 1969) and was featured prominently by French engineer Alexandre Collin in 1846 (Skempton, 1984). It is used to measure the shear strength of soil because time taken for testing is fast and sample preparation is easy. It moreover can approach the assessing shear strength of remolded, intact and reconstituted soil specimens.

In this chapter, the shear strength of soil is measured by using three-ring direct shear test device. The results point out the enhancement through the compaction and geopolymer. The shear stresses of compacted soils are developed by using geopolymer based on non-curing and curing condition. This reveals changes of cohesion and friction angle that indicate how the shear strength can be improved by using geopolymer at the same moisture content rather than by using water only.

All test specimens are conducted in the laboratory under ambient temperature which is likely to simulate the natural field condition.

5.2 Experimental works

5.2.1 Three-ring direct shear device

The three ring mold consists of four rings, such as top ring, middle ring, bottom ring and last ring like a collar of mold. Among them, the top, middle and bottom rings are essential to measure the dry density (Figure 5.1). The inner diameter is 10.16 cm, outer diameter is 10.76 cm and combined height is 15.24 cm. The three rings are secured on the base plate using steel bolts and two steel clamps. The soil sample in three-ring mold is dynamically compacted with a release of steel hammer weight of 10 pounds. Energy of compaction is the same with ASTM modified proctor test. Between three-ring mold and ASTM standard mold, the MDDs and OMCs obtained are very similar.

The three-ring mold requires a new shear test frame. Since there are two incipient shear planes of compacted soil samples, one between the top and middle rings and the other lateral load system for pushing the middle ring, and the vertical load system for applying a constant normal stress on the compacted soil sample (Sonsakul et al., 2013). According to the energy equation (2.3), the number of blows is 27 from dropped hammer in each layer of equal six layers to be same with the energy of ASTM modified compaction test (2,700 kN-m/m^3). The three-ring shear device is as shown in Figure 5.2. The lateral load (shear force) and vertical load (normal load) are gained from 20 tons hydraulic loads cell connected to hydraulic hand pumps. To measure the horizontal displacement and vertical volume change, the

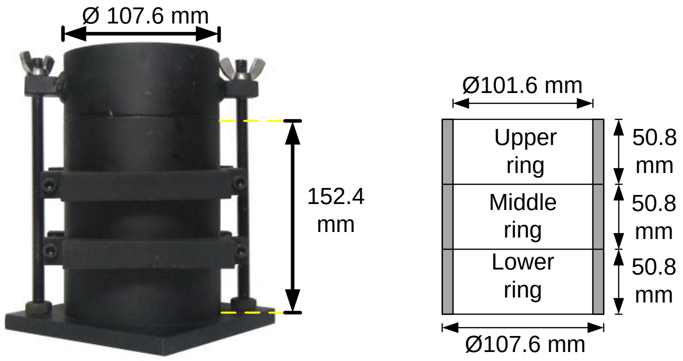


Figure 5.1 Three-ring mold.

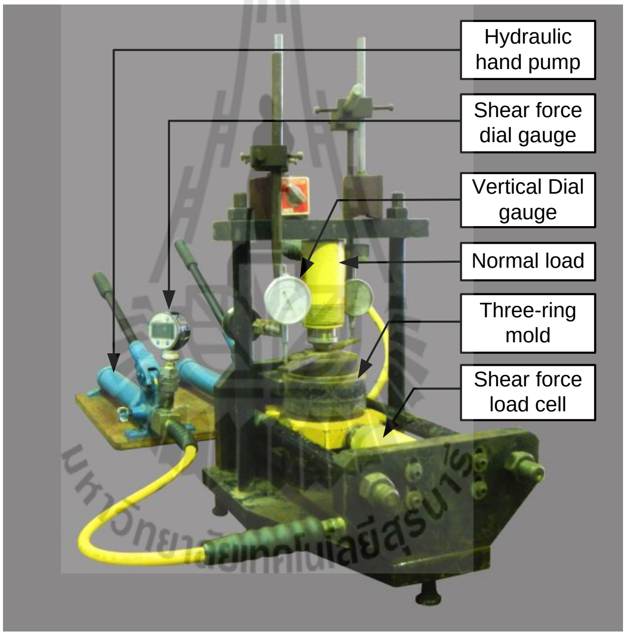
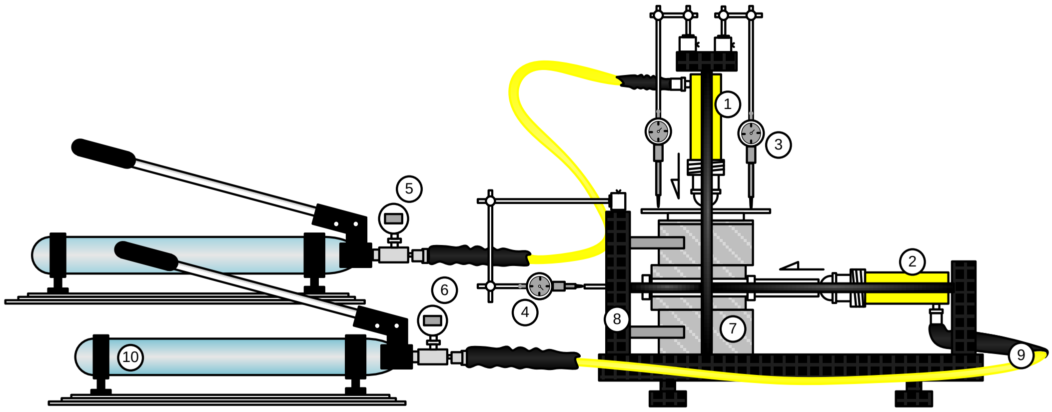


Figure 5.2 Three-ring direct shear device.

precise dial gauges are attached to the frame of device. The vertical normal load and the horizontal shear force are controlled by the movements of arms from precise dial gauges fixed in the frame of device.

The soil sample is compacted in three-ring mold and continuously the mold with removing the sample is continuously set up in direct shear test device as shown in Figure 5.3. The main important facts of three-ring direct shear device are



(1) Hydraulic load cell for normal force
(2) Hydraulic load cell for shear force
(3) Precise dial gauge for vertical deformation
(4) Precise dial gauge for shear displacement
(5) Digital dial gauge for normal force measurement
(6) Digital dial gauge for shear force measurement
(7) Three-ring mold
(8) Direct shear frame
(9) Hydraulic pipe between hand pump and load cell
(10) 20-tons hydraulic hand pump

Figure 5.3 Side view of three-ring direct shear testing device.

able to perform direct shear strength of the compacted soils with sample disturbance and the three-ring direct shear device allows testing the soil samples with the maximum grain size up to 10 mm.

5.2.2 Verified performance in soil-water sample

The sample preparation of soils is similar to ASTM compaction test. The oven dry soil sample is taken as an amount of 2.5 kg after passing sieve No.(4). The soil sample is mixed with water content at OMC state of ASTM modified compaction test. The homogenous mixture is compacted in three-ring mold with 27 blows from dropped hammer in each layer of six equal layers. The compactive energy is the same with ASTM modified compaction test (2,700 kN-m/m^3). The weight of compacted sample then is measured on digital balance to check whether the dry density is same to MDD of ASTM modified compaction test. After compaction, the last ring (collar) is removed, and the mold combined of top, middle and bottom rings is trimmed to obtain the definite mold volume and mold weight.

The compacted soil samples are based on curing period and non-curing period. For non-curing period (0 days), the compacted soil sample in three-ring mold is immediately set up in the three-ring direct shear testing device. The shear displacement is continuously taken place. For curing period (7 days), the compacted soil sample in three-ring mold is kept in air tight plastic bags not to occur moisture change. The sample is cured for 7 days under ambient temperature (27-30°C). When reaching at seventh day, the sample is fixed in shear device to take place shear displacement.

Beyond curing state, subsequently, the compacted soil with three-ring molds is set up in direct shear device. To take place shear displacement, the normal forces are applied from the vertical hydraulic load cell connected to 20-ton hydraulic hand pump. While the normal stresses are 0.4, 0.6, 0.8 and 1.0 MPa, the shear stresses are taken place from horizontal load cell connected to hydraulic hand pump. The shear stresses are manually controlled by a normal rate (approximately 1 mm/min). The shear rate (1 mm/min) is significant to be different outcome in shear displacements of various soil samples. The shear displacements can be obvious in horizontal precise dial gauge. Beyond shear displacement of 8 cm, three-ring shear test is stopped for every soil samples because the shear stresses are generally occurred at a constant value after that displacement. After shearing the samples in three-ring mold, the small amount of soil sample from the shearing mold is taken to measure the moisture content at the shearing state of those soils. All of every soil samples are recorded for optimum moisture contents and maximum dry densities, and they are compared to ASTM modified compaction test (Tables 5.1 and 5.2).

5.2.3 Verified performance in soil-fly ash geopolymer sample

The mixing procedure of soil samples with FA geopolymer is similar to the procedure of soil samples with water. The proper amount of soil sample for compaction test is 2.5 kg as in previous sample preparations. Fly ash is added into the soil sample and mixed together. Into the mixture of soil and fly ash, the alkaline liquid activator composed of NaOH and Na_2SiO_3 is poured. The amount of alkaline liquid activator is according to the mix ratio obtained from ASTM modified compaction test. The mixture is blended for fifteenth mixtures to be homogenous. The homogenous mixture is compacted in three-ring molds as in previous compaction test of the sample with water. For non-curing period (0 days), the compacted soil sample is set up immediately in direct shear testing device. For curing period (7 days), the compacted soil sample with three-ring mold is kept in air-tight plastic bags under ambient temperature (27-30°C).

Therefore, the non-curing mold with soil sample is immediately fixed in shear testing device to conduct shear test. The procedures of three-ring direct shear test are same to previous procedures in soil-water samples. For curing mold with soil sample, the mold with compacted soil sample is set up in device to shear the samples according to the previous procedures, and the shear displacement, shear rate and normal force are the same. The optimum moisture contents and maximum dry densities of every sample with FA geopolymer are similar to the value in ASTM modified compaction test. These values are shown in Tables 5.1 and 5.2, compared to ASTM standard test.

Table 5.1 Variations of moistures content of direct shear test specimens in three-ring molds.

Soil Type	Curing Time (days)	Tested Samples with	OMC (%)	Moisture Content (%)			
				0.4 MPa	0.6 MPa	0.8 MPa	1.0 MPa
SM	0	Water	7.8	7.5	7.6	7.6	7.6
		GP	9.5	9.4	9.5	9.5	9.3
	7	Water	7.8	7.6	7.7	7.5	7.6
		GP	9.5	9.3	9.2	9.1	9.3
CH	0	Water	21	19.8	18.1	19.0	18.4
		GP	19	19.3	19.6	18.8	18.9
	7	Water	21	22.6	22.5	22.0	21.6
		GP	19	18.7	18.8	18.5	18.6
MH	0	Water	26	27.9	27.6	28.4	27.3
		GP	32	32.1	32.0	32.2	31.6
	7	Water	26	28.6	28.4	28.6	28.5
		GP	32	32.1	31.9	31.7	31.6

Table 5.2 Variations of maximum dry densities of three-ring direct shear test specimens in three-ring molds.

Soil Type	Curing Time (days)	Tested Samples with	MDD (kg/m^3)	Density (kg/m^3)			
				0.4 MPa	**0.6 MPa**	**0.8 MPa**	**1.0 MPa**
SM	0	Water	1,940	1,937	1,941	1,940	1,941
		GP	1,925	1,926	1,925	1,925	1,924
	7	Water	1,940	1,940	1,943	1,939	1,939
		GP	1,925	1,926	1,926	1,924	1,926
CH	0	Water	1,634	1,653	1,635	1,615	1,650
		GP	1,573	1,589	1,607	1,614	1,619
	7	Water	1,634	1,625	1,635	1,636	1,628
		GP	1,573	1,598	1,597	1,592	1,655
MH	0	Water	1,360	1,360	1,360	1,360	1,359
		GP	1,250	1,249	1,250	1,250	1,250
	7	Water	1,360	1,360	1,360	1,359	1,359
		GP	1,250	1,249	1,249	1,250	1,250

5.3 Test results and discussion

The shear test data are plotted on a graph with relationship of shear stress and shear displacement. Under different four normal loads, peak shear stress and residual shear stress are attained. In silty sand and high plasticity silt, the shear stresses of non-curing state are slightly different between water and geopolymer (Figure 5.4(a) and 5.4(c)). The shear stresses of curing condition are almost double different (Figure 5.4(b) and 5.4(d)). The shear stresses of high plasticity silt are shown in (Figure 5.5(a) – 5.5(d)). Likewise in high plasticity clay, the behaviors of shear stresses are same as previous two soils (Figure 5.6(a) – 5.6(d)). The shearing features of three types of soils depended on curing or non-curing state are illustrated in Figure 5.7.

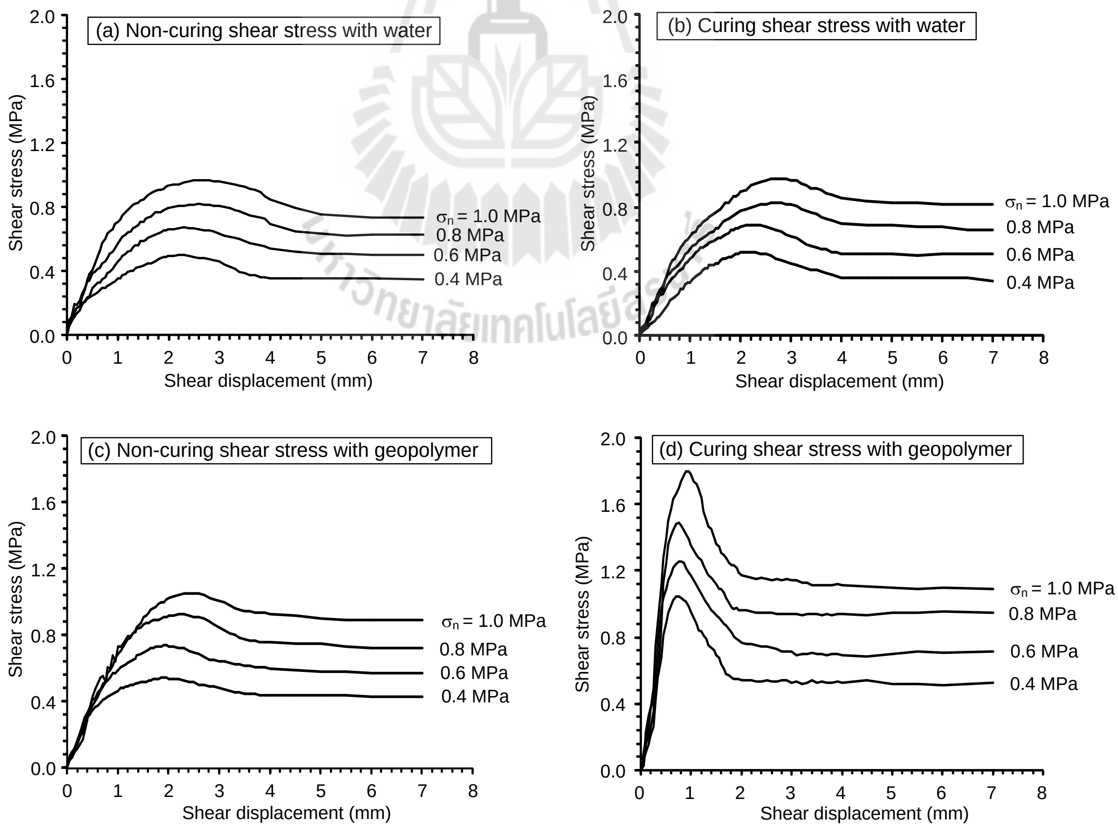


Figure 5.4 Shear stresses related with shear displacement obtained from silty sand.

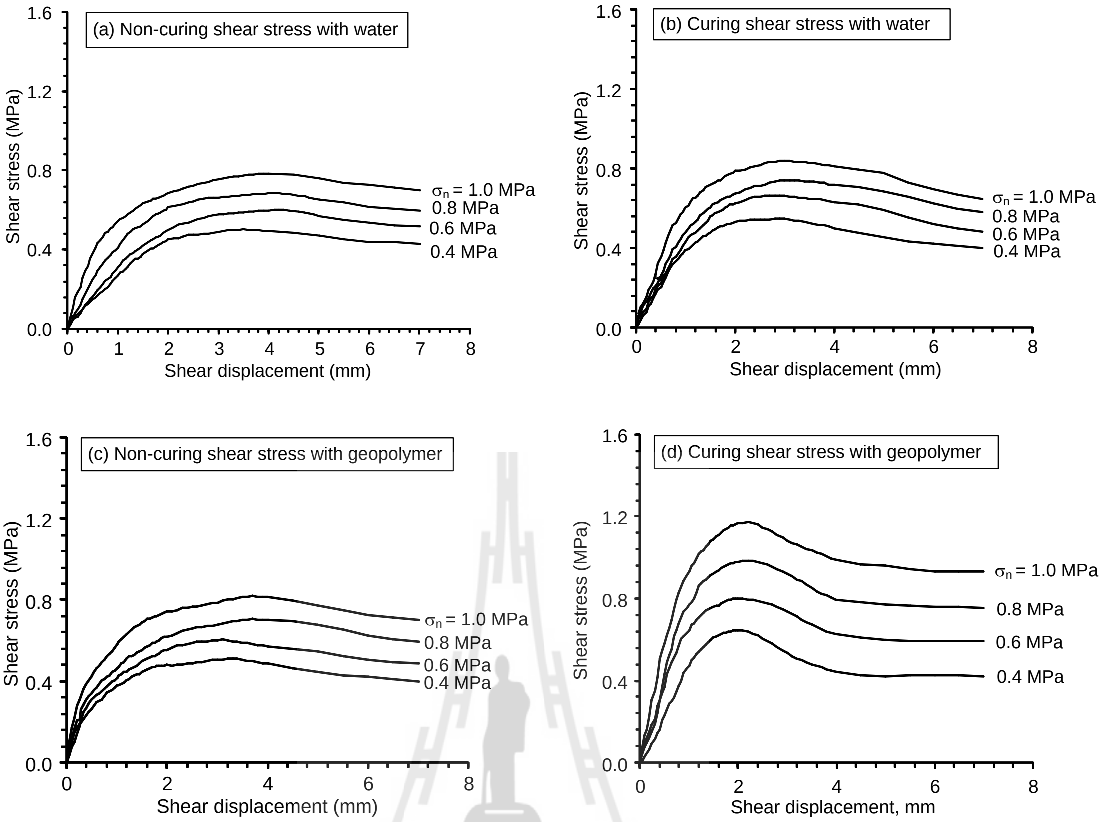


Figure 5.5 Shear stresses related with shear displacement obtained from sludge.

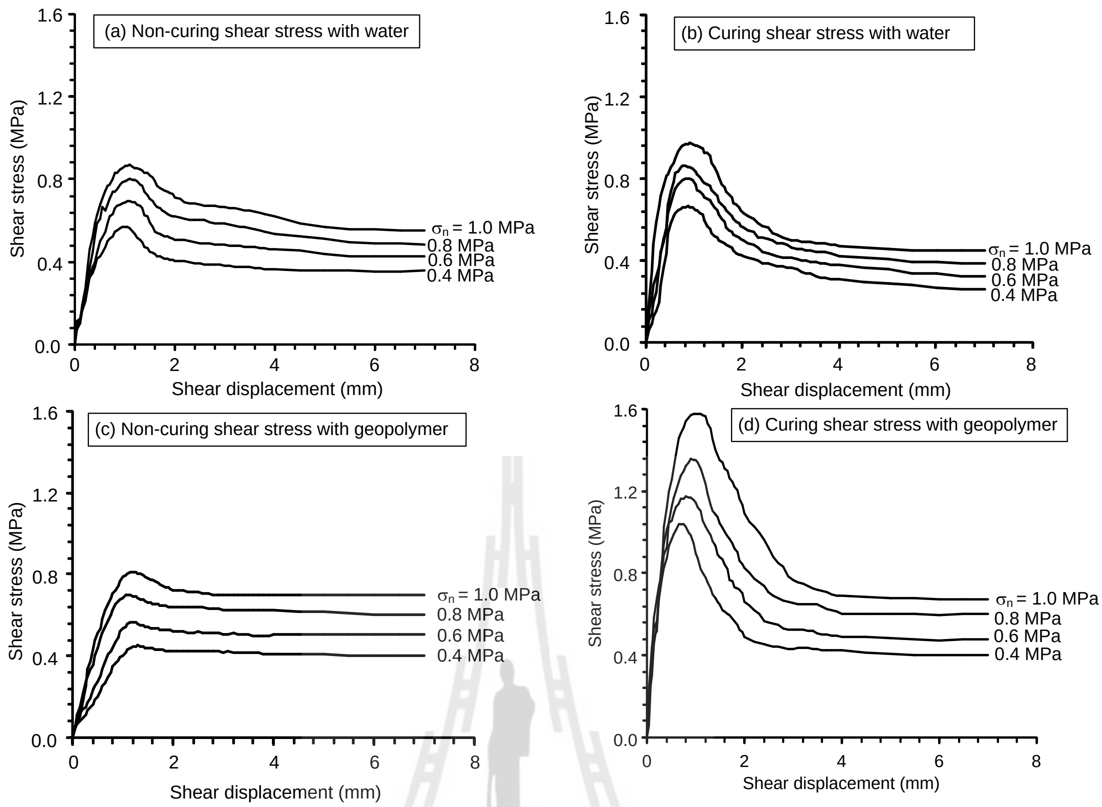
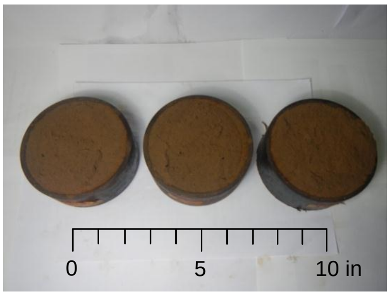
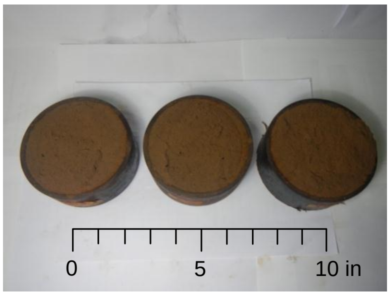
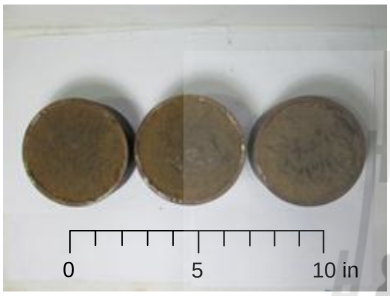


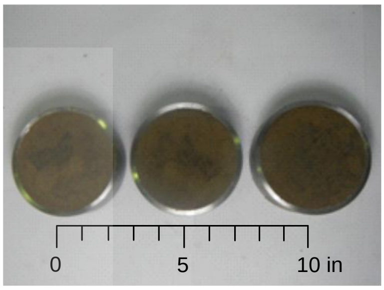
Figure 5.6 Shear stresses related with shear displacement obtained from high plasticity clay.



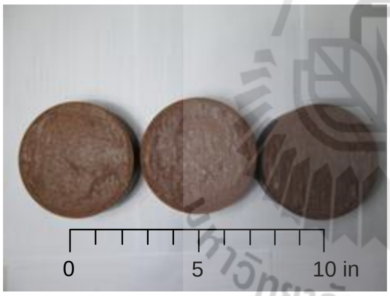


(a) Non-curing state of silty sand (b) Curing state of silty sand





(a) Non-curing state of sludge (b) Curing state of sludge



(a) Non-curing state of high plasticity clay (b) Curing state of high plasticity clay

Figure 5.7 Shearing features of soil specimens in three-ring device through non-curing and curing period.

The peak shear stresses are related with shear strength and normal stress. According to the graphs, peak and residual shear strength vary with the curing period. Silty sand and high plasticity silt are same in behaviors. All of peak and residual strengths are display as the curing period increases the shear strength of compacted samples with geopolymer, expect of clay sample in cured residual shear strength (Figure 5.8). In addition, the shear strength parameters of each soil samples are arranged in Table 5.3. The test results show that the manners of peak and residual shear strength are depended on soil type. For silty sand, peak shear strength of samples are reached at around 2.5 mm, except of 7 days cured sample with geopolymer occurred around 1 mm. For high plasticity silt, peak shear strengths are occurred within 3 to 4 mm until 7 days strength of sample with geopolymer occurred around 2 mm. In high plasticity clay, the shear strengths are diverse from previous two soil samples. The peak shear strengths are taken place around 1 mm.

For all soil types, the internal friction angle of peak and residual strength are merely different between curing periods when those samples are mixed with water. But when those samples are mixed with FA based geopolymer, the friction angles are started to increase since non-curing state. After curing for 7 days, the friction angle is almost double increased. All of cohesions in residual strength state are similarly increased with respect to internal friction angles of those soils. The cohesions of peak strength are not stable and flocculated.

The strain softening implies on three soils samples that silty sand and high plasticity silt sample behave as ductile behavior until curing of soil sample mixed with FA based geopolymer. After curing for 7 days, the strain softening with FA based geopolymer are found as brittle behavior. In high plasticity clay, the strain softening is occurred as already brittle condition before cured strain softening for 7 days and it proves higher brittle behavior after curing for 7 days.

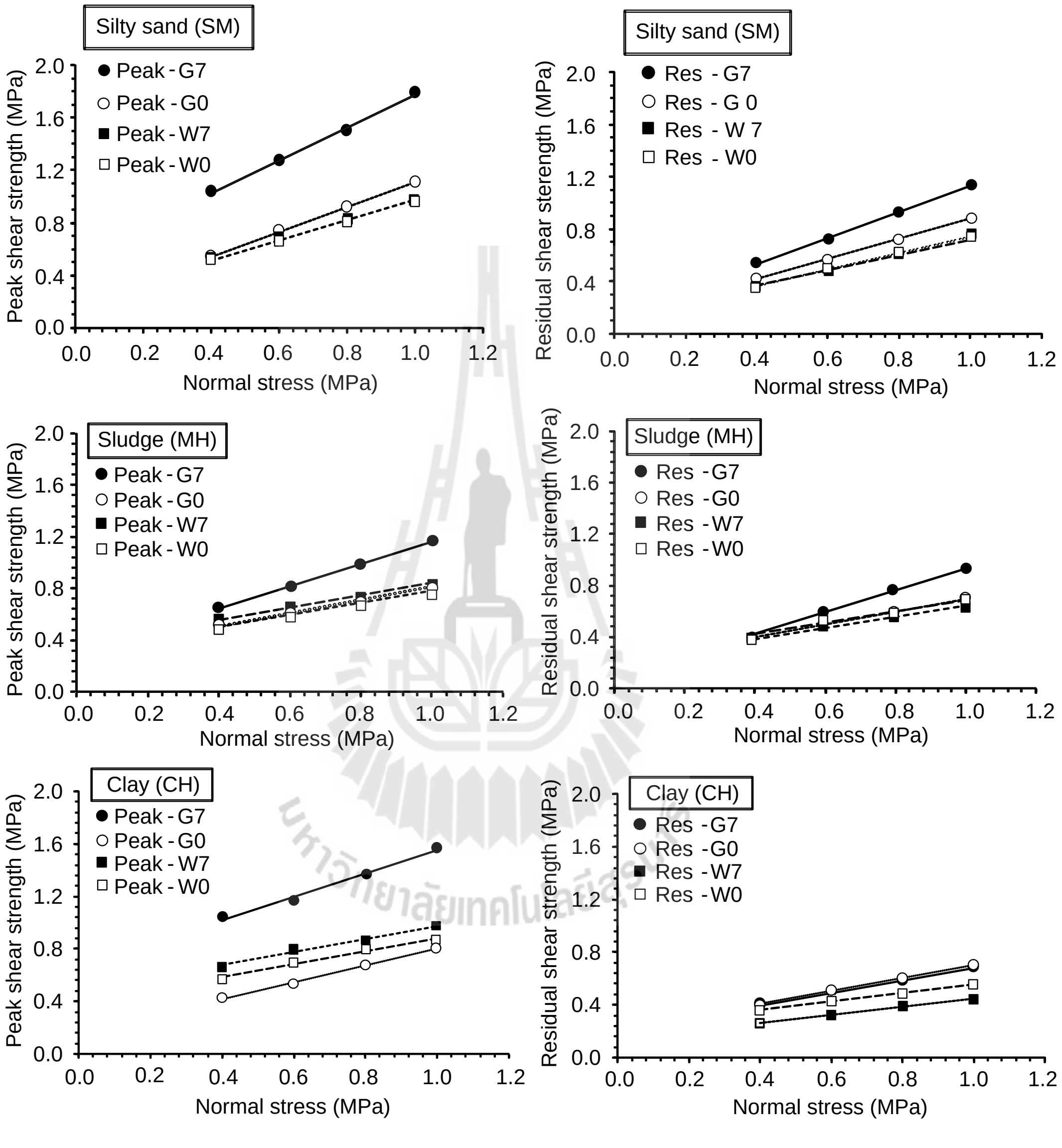


Figure 5.8 Shear enhancements on types of soil (G7 = 7 days curing sample with geopolymer, G0 = non-curing sample with geopolymer, W7 = 7 days curing sample with water, W0 = non-curing sample with water).

Table 5.3 Shear parameters of three soil types between non-curing (0 day) and curing (7 days) conditions.

Soil Type	**Curing Time (days)**	**Tested Samples with**	**Peak**		**Residual**	
			c_p (MPa)	**ϕ_p (°)**	**c_r (MPa)**	**ϕ_r (°)**
SM	0	Water	0.20	37.8	0.11	32.5
		GP	0.22	43.3	0.12	37.3
	7	Water	0.23	36.7	0.11	32.4
		GP	0.59	51.4	0.13	45.1
CH	0	Water	0.38	26.5	0.23	18.0
		GP	0.17	32.4	0.21	26.2
	7	Water	0.48	26.2	0.14	17.4
		GP	0.66	41.8	0.24	25.2
MH	0	Water	0.32	25.0	0.25	23.8
		GP	0.30	27.2	0.19	26.8
	7	Water	0.37	25.4	0.21	23.3
		GP	0.29	41.3	0.08	40.5

CHAPTER VI

CONCLUSIONS AND RECOMMENDATION FOR FUTURE STUDY

6.1 Conclusions

The compaction test, three-ring direct shear test have been performed in laboratory to assess the enhancing shear strength of compacted soils with geopolymer. The study is of importance when the local soils need to be strengthening to meet the design requirements in field conditions, such as soil embankment, soil slope and earth dam foundation. In laboratory, the soil samples based on three types (silty sand, sludge and high plasticity clay) are compacted in three-ring mold before shearing process in direct shear device. Because three-ring mold has an advantage that the soils can immediately be sheared after compaction without sample disturbance and without removal of soils from the mold. The test specimens are based on curing (7 days) and non-curing (0 day) state under ambient temperature (27°C to 30°C). The ambient temperature is feasible as field conditions. For curing state, the specimens in three-ring mold are kept in air tight plastic bags to control the moisture lost. For non-curing state, the specimens are immediately sheared in three-ring direct shear testing device. Curing and non-curing are to approach the strength after construction. Otherwise, the most of chemicals need time interval to take place chemical reaction inside their phase together with raw materials like local soils. While shearing, the horizontal displacement rate (shear displacement rate) is reasonable in 1 mm/min to be obvious strain softening of typical soils in shear behaviors. All specimens have been sheared with dry state without submerging into water.

The three-ring compaction results are similar to ASTM standard test like in previous research (Sonsakul et al., 2013). The modified compaction test is easy to perform in laboratory for the optimum moisture contents and the maximum dry density of soils. Then, the soil specimens are compacted in three-ring mold to continue direct shear test. For silty sand and sludge, the soils samples with fly ash based geopolymer are increase in the optimum moisture content and decrease in the maximum dry. In contrast, high plasticity clay soil with fly ash based geopolymer is acting different in decreasing of optimum moisture content corresponding with decreasing of maximum dry density. It is likely to be that fine particles of fly ash come inside of soil grains and closer between soil grains with more lubricating agent, alkaline liquid. They absorb more water, and then the moisture content is increase than in state of mixing with water. The compaction result points out that fly ash based geopolymer cannot improve the maximum dry density of soils.

The result of three-ring direct shear tests gives higher strengths in shearing when the soil samples are mixed with geopolymer and those higher strengths increase more in all soil types through curing state. After compacting with fly ash based geopolymer, the soils are attained a harden state through time period as long as chemical reaction occurs between soil grains and geopolymer molecules. In fields, when the selected soils are instantly mixed with chemical substances, the blending process should not be longer to save time consuming of project. In laboratory, the short time interval (almost 15 minutes) of mixing process of soil samples with geopolymer reflects the advantages on field condition that in-situ mixing process can be performed as fast as possible. Moreover, the more laboratory strengths based on curing period under ambient temperature (27°C to 30°C) also point out that the field strength can also be attained after construction because of chemical reaction under ambient temperature in actual condition. Although the clay soil is normally low internal friction angle, the compacted condition with geopolymer makes a higher internal friction angle. Indeed, the compaction can increase the shear strength of soil mechanically. The soils mixed with geopolymer transform to more brittle behavior in strain softening. This notes that fly ash based geopolymer enhances the shear strength of soils by increasing of cohesion and friction angle. Soil improvement techniques using geopolymer can be applied for strengthening the soil embankment, soil slope and earth dam foundation.

6.2 Recommendations for future study

The further studies for future are suggested as follows:

1. The more soil specimens should be used to experimentally perform three-ring direct shear test with various shearing rates. The results would identify more approaching the enhancement of compacted soils using geopolymer.
2. The various ratios of geopolymer and raw materials might be performed under high ambient temperature (>30°C) and more curing periods (>7 days). The results might be helpful for strengthening soil properties in tropical field conditions.

3. The soil specimens may be under wet-dry cycle process according to ASTM standard before shearing the sample in direct shear device. The results could benefit in assessment of the weather effect on soils properties beyond construction sate.
4. Microscopic studies as SEM and XRD may be employed when shearing for soil specimens with geopolymer. These techniques would clarify the geopolymerization process related with shear strength of soils.

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