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กากแคลเซียมคาร์ไบด์และถ่านลอย



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มหาวิทยาลัยเทคโนโลยีสุรนารี

ปีการศึกษา 2555

**MECHANICAL PROPERTIES OF CALCIUM CARBIDE
RESIDUE - FLY ASH STABILIZED SILTY CLAY**



Apichit Kumpala

**A Thesis Submitted in Partial Fulfillment of the Requirements for the
Degree of Doctor of Philosophy of Engineering in Civil Engineering**

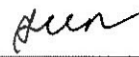
Suranaree University of Technology

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Suranaree University of Technology has approved this thesis submitted in partial fulfillment of the requirements for the Degree of Doctor of Philosophy.

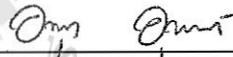
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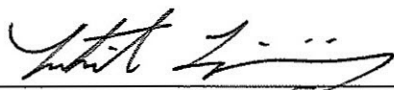
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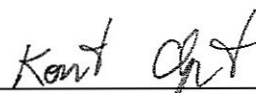
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อภิชาติ คำภาหาล้า : คุณสมบัติทางกลของดินเหนียวปนดินตะกอนปรับปรุงด้วยกาก
แคลเซียมคาร์ไบด์และเถ้าลอย (MECHANICAL PROPERTIES OF CALCIUM
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วิทยานิพนธ์นี้มีจุดประสงค์ที่จะทำการศึกษาลักษณะการควบคุมคุณสมบัติทางวิศวกรรม
ของดินเหนียวปนดินตะกอนที่มีปัญหา โดยใช้กากแคลเซียมคาร์ไบด์ (CCR) และเถ้าลอย (FA) เพื่อ
เป็นแนวทางในการกำหนดสัดส่วนการผสมที่ดีที่สุดสำหรับการปรับปรุงดินเหนียวและศึกษา
ความเป็นไปได้ในการนำดินเหนียวปนดินตะกอนผสมกากแคลเซียมคาร์ไบด์มาใช้เป็นวัสดุ
หมุนเวียน (recycled materials) ในงานโครงสร้างชั้นทางกากแคลเซียมคาร์ไบด์เป็นวัสดุขยะที่เหลือ
จากปฏิกิริยาเคมีในการผลิตก๊าซอะเซทิลีน (C_2H_2) กระบวนการผลิตก๊าซอะเซทิลีน ใช้
แคลเซียมคาร์ไบด์ (CaC_2) ซึ่งมีสภาพเป็นของแข็งทำปฏิกิริยากับน้ำและเกิดก๊าซอะเซทิลีน (C_2H_2)
และแคลเซียมไฮดรอกไซด์ ($Ca(OH)_2$) หลังจากกระบวนการผลิตกากแคลเซียมคาร์ไบด์จะถูกแช่ใน
น้ำ กากแคลเซียมคาร์ไบด์ประกอบไปด้วยแคลเซียมไฮดรอกไซด์เป็นหลัก องค์ประกอบทางเคมี
ของกากแคลเซียมคาร์ไบด์คล้ายกับปูนขาว เถ้าลอยเป็นหนึ่งในวัสดุปอซโซลานที่ได้จากการเผา
ถ่านหินเพื่อเป็นพลังงานในการผลิตกระแสไฟฟ้า ผลการศึกษาพบว่า การผสมเหนียวปนดินตะกอน
ด้วยกากแคลเซียมคาร์ไบด์จะช่วยลดความเป็นพลาสติกปริมาณน้ำที่เหมาะสมของการบดอัดมีค่า
เพิ่มขึ้นตามการลดลงของหน่วยน้ำหนักแห้งสูงสุด CCR- fixation point ซึ่งประมาณได้จากการ
ทดสอบดัชนีพลาสติกเป็นตัวแปรที่เหมาะสมต่อการกำหนดอัตราส่วนผสม ปริมาณน้ำที่เหมาะสม
ในการบดอัดให้กำลังอัดและความทนทานสูงสุดและให้การบวมตัวขยายตัวและการยุบตัวต่ำสุด
การพัฒนากำลังของดินเหนียวปนดินตะกอนผสมกากแคลเซียมคาร์ไบด์แบ่งออกเป็น 3 โซน ได้แก่
active zone, inert zone และ deterioration zone ใน inert zone และ deterioration zone เมื่อผสมเถ้า
ลอยในปริมาณที่เหมาะสม แคลเซียมไฮดรอกไซด์ส่วนเกิน (free lime) ที่มีอยู่อย่างหนาแน่นในกาก
แคลเซียมคาร์ไบด์จะทำปฏิกิริยาปอซโซลานกับซิลิกาและอลูมินาในเถ้าลอย มีผลทำให้เกิดการ
พัฒนากำลังและความคงทนอย่างมีนัยยะสำคัญด้วยเหตุนี้เองเถ้าลอยจึงจัดเป็นวัสดุที่ปรับปรุงกำลัง
และความทนทานของดินเหนียวผสมกากแคลเซียมคาร์ไบด์ ปริมาณที่เหมาะสมที่สุดของเถ้าลอย
ประมาณร้อยละ 20 ของน้ำหนักดินแห้ง ความคงทนมีความสัมพันธ์โดยตรงกับกำลังอัดแกนเดียว
ในสถานะเริ่มต้น (ก่อนกระบวนการเปียกสลับแห้ง) ดังนั้น กำลังอัดแกนเดียวของดินภายใต้
กระบวนการเปียกสลับแห้งจึงสามารถประมาณได้จากกำลังอัดแกนเดียวในสถานะเริ่มต้น
ประโยชน์ที่ได้จากการวิจัยนี้ทำให้สามารถประมาณกำลังอัดของดินตัวอย่างที่ต้องออกแบบ เพื่อให้

มีกำลังอัดเพียงพอในขณะที่ใช้งานผลจากวิเคราะห์ด้วยภาพถ่ายขยายกำลังสูงและการวิเคราะห์ขนาด
คลื่นแสดงให้เห็นว่าอนุภาคของดินเหนียวปนดินตะกอนผสมกากแคลเซียมคาร์ไบด์ปั้นใหม่มีขนาด
ใหญ่กว่าอนุภาคของกากแคลเซียมคาร์ไบด์และดินเหนียวปนดินตะกอนเดิม เนื่องจากผลของ
ปฏิกิริยาปอซโซลาน การหดตัวเชิงเส้นและอัตราส่วนการบวมตัวอิสระของวัสดุหมุนเวียนมีค่า
ลดลงจากเดิมอย่างเห็นได้ชัดเจน ที่พลังงานการบดอัดและอัตราส่วนของกากแคลเซียมคาร์ไบด์
เดียวกัน ค่าหน่วยน้ำหนักดินแห้งของวัสดุหมุนเวียนมีค่าต่ำกว่าตัวอย่างดินที่ปรับปรุงด้วยกาก
แคลเซียมคาร์ไบด์ ผลของพฤติกรรมนี้เกิดจากความแข็งแรงที่ผิวของอนุภาคเม็ดดินต่อต้านอิทธิพล
ของการบดอัด ผลจากการพัฒนากำลังและการลดลงของอัตราส่วนช่องว่างกับเวลา เป็นที่ยืนยันว่า
ปฏิกิริยาปอซโซลานยังคงพัฒนาอย่างต่อเนื่องหลังจากการปั้นใหม่ ลักษณะนี้แสดงเป็นนัยยะว่า
ปฏิกิริยาปอซโซลานส่วนใหญ่เกิดขึ้นบนผิวของอนุภาคของดินเหนียวและกากแคลเซียมคาร์ไบด์
การปั้นใหม่ของดินเหนียวปนดินตะกอนผสมกากแคลเซียมคาร์ไบด์เป็นการทำให้แรงยึดเหนี่ยว
ระหว่างอนุภาคเม็ดดินจากพันธะเชื่อมประสานที่เกิดจากปฏิกิริยาปอซโซลานระหว่างอนุภาคของ
ดินเหนียวและกากแคลเซียมคาร์ไบด์แตกและแยกออกจากกัน อนุภาคของดินเหนียวที่แตกออกและ
กากแคลเซียมคาร์ไบด์ที่เหลืออยู่สามารถทำปฏิกิริยาต่ออย่างเมื่อทำการผสมน้ำและบดอัดใหม่ ผล
จากการวิจัยนี้สนับสนุนความเป็นไปได้ในการปรับปรุงโครงสร้างชั้นทางด้วยกากแคลเซียมคาร์ไบด์
และเถ้าลอย รวมทั้งวัสดุหมุนเวียนจากดินเหนียวที่ปรับปรุงด้วยกากแคลเซียมคาร์ไบด์

สาขาวิชาวิศวกรรมโยธา

ปีการศึกษา 2555

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APICHIT KUMPALA : MECHANICAL PROPERTIES OF CALCIUM
CARBIDE RESIDUE - FLY ASH STABILIZED SILTY CLAY. THESIS
ADVISOR : PROF. SUKSUN HORPIBULSUK, Ph.D., P.E., 148 PP.

ENGINEERING PROPERTIES/POZZOLANIC REACTION/SOIL
STABILIZATION/ WETTING-DRYING CYCLES / RECYCLED MATERIALS

This thesis aims at investigating the mechanism controlling the engineering properties of a problematic soil by calcium carbide residue (CCR) and fly ash (FA), to determine the optimal mix proportion for the other clayey soils stabilized and to examine the possibility of using the recycled CCR-FA stabilized clay as fill and pavement materials. The CCR is a by-product of acetylene (C_2H_2) production process through the hydrolysis of calcium carbide (CaC_2). The CCR is generated as an aqueous slurry and is composed essentially of calcium hydroxide ($Ca(OH)_2$) with minor parts of calcium carbonate ($CaCO_3$), unreacted carbon and silicates. The chemical composition of the CCR is similar to those of the hydrated lime. Fly ash (FA) is one of the waste pozzolanic materials extracted from flue gases of a furnace fired with coal of an electric power plant. Its generation is far in excess of utilization. This thesis presents engineering properties of the CCR stabilized silty clay and the recycled CCR stabilized silty clay to ascertain its performance in pavement base and subbase applications. The input of CCR, which reduces the plasticity index of the clay, increases the optimum water content, OWC and decreases the maximum dry unit weight, $\gamma_{d, \max}$ of the stabilized clay. The CCR fixation point simply obtained from the index test is proved as a practical indicator for determining the CCR content to obtain the required engineering properties at a particular molding water content. For a

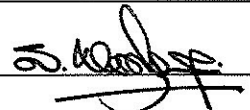
particular CCR content, the optimum water content is the most appropriate in terms of strength, and durability. The strength improvement is classified into three zones: active, inert and deterioration. In the inert and deterioration zones, the input fly ash at optimal content reacts with the excess $\text{Ca}(\text{OH})_2$ from the CCR and hence a significant improvement of the strength and durability. The optimal FA content is found at about 20%. The strength analysis shows that the durability is directly related to the unsoaked strength (prior to the w-d cycles). Consequently, the relationship between the w-d cycle strength and unsoaked strength is proposed. It is useful for quick determination of the unsoaked strength for mix design to attain the target strength at the design service life. Scanning electron microscopic images show that the recycled particles are larger than the CCR and clay particles due to the attached pozzolanic products. The large grains reduce linear shrinkage and free swell ratio of the recycled material. For the same compaction energy and CCR content, the unit weight of the recycled material is lower than that of the stabilized clay because the harder attached pozzolanic products resist the compaction. The strength development and the reduction in void ratio with time confirm that the pozzolanic reaction still prevails even after remolding. The remolding of stabilized clay breaks down the cementitious bonds between the CCR-clay clusters and the unreacted CCR and clay particles in the clusters are then free to interact with water. The research outcome reinforces the possibility of using the recycled CCR stabilized clay as fill and pavement materials.

School of Civil Engineering

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Student's Signature _____

Advisor's Signature _____



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Seven years ago, I have entered the School of Civil Engineering, Suranaree University of Technology, which led to a big change in my life. I have pursued my Master's and Ph.D. studies in the Field of Geotechnical Engineering. It was my great opportunity to work under the supervision of Professor Dr. Suksun Horpibulsuk for both Master's and Ph.D. studies. I would like to express my deepest gratitude and appreciation for Professor Dr. Suksun Horpibulsuk for his invaluable guidance, encouragement and enthusiasm throughout my studies at the Suranaree University of Technology.

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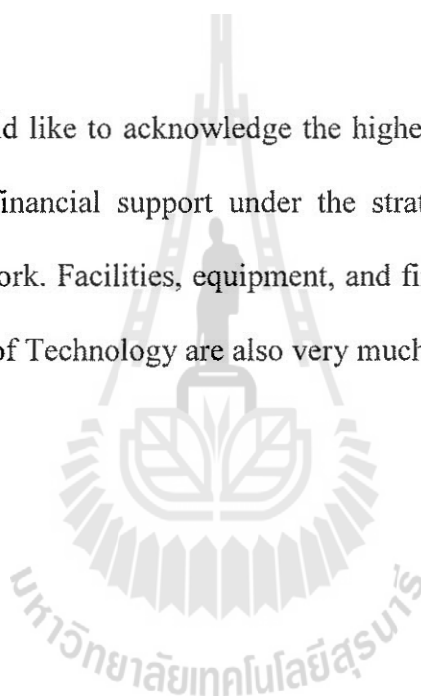


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CHAPTER I

INTRODUCTION

1.1 Rationale of the Study

The soil in northeast Thailand generally consists of two layers. The upper soil layer (varying from 1.0-3.0 m thickness) is wind-blown and deposited over several decades. It is silty clay with low to moderate strength. This upper soil is a problematic soil, which is sensitive to change in water content. Its collapse behavior due to wetting is illustrated by Kohgo et al., 1997 and Kohgo and Horpibulsuk, 1999. The soil in the dry state has high bearing capacity and low compressibility. The bearing capacity significantly decreases as the water content increases, which causes several problems for foundation and pavement structures (Horpibulsuk et al., 2008). The lower layer is a residual soil, weathered from claystone, consisting of clay, silt and sand.

The chemical stabilization is one of the extensively used techniques to improve the engineering properties of a problematic soil. The resistance to compression and consequent strength development increase with increasing curing time. Portland cement is commonly used for this stabilization. Effects of some influential factors i.e., water content, cement content, curing time and compaction energy on the microstructure and engineering characteristics of cement stabilized soils have been extensively researched (Terashi et al., 1979; Tatsuoka and Kobayashi, 1983; Kamon and Bergado, 1992; Nagaraj et al., 1997; Yin and Lai, 1998; Consoli et al., 2000; Horpibulsuk, 2001; Horpibulsuk and Miura, 2001; Miura et al., 2001;

Consoli et al., 2000; Horpibulsuk, 2001; Horpibulsuk et al., 2004a; Horpibulsuk et al., 2006; Horpibulsuk et al., 2010a; Horpibulsuk and Miura, 2001; Horpibulsuk et al., 2004b; Horpibulsuk et al., 2003; Horpibulsuk et al., 2005; Horpibulsuk et al., 2012b; Horpibulsuk et al., 2011a; Horpibulsuk et al., 2010b; Horpibulsuk et al., 2009; Horpibulsuk et al., 2011b; Horpibulsuk et al., 2011c; Horpibulsuk and Raksachon, 2010; Kamon and Bergado, 1992; Kasama et al., 2007; Miura et al., 2001; Nagaraj et al., 1997; Suebsuk et al., 2010; Suebsuk et al., 2011; Tatsuoka and Kobayashi, 1983; Terashi et al., 1979; Yin and Lai, 1998 and others). However, the cement used in stabilization engineering properties of clay has a drawback. It results in shrinkage crack due to the inherently high shrinkage and brittleness of soil cement. Also, a high unit cost and energy intensive process of Portland cement are the driving forces for the alternative cementitious additives. More importantly, the cement manufacturing process emits CO₂ into the atmosphere which accounts for 5% of the total CO₂ released into the air.

For clayey soils, which contain high natural pozzolanic materials, the Ca(OH)₂ rich materials can be used to produce a moderately high strength geo-material (Bell, 1996; Diamond and Kinter, 1966; Horpibulsuk et al., 2012a; Horpibulsuk et al., 2012b). The cementing property is identified as a pozzolanic reaction. The engineering characteristics of Ca(OH)₂ rich materials stabilized clay are attributed to three basic reactions (Bell, 1996; Diamond and Kinter, 1965; Diamond and Kinter, 1966; Herrin and Mitchell, 1961) which are cation exchange, flocculation and aggregation, and pozzolanic reaction. The addition of Ca(OH)₂ rich material to soil supplies an excess of Ca²⁺ and the cation exchange will occur with Ca²⁺ replacing dissimilar cations from the exchange complex of the soil. This causes flocculation and

aggregation of the clay fraction. The clay particles clump together into larger sized aggregates. The influence of cation exchange, flocculation and aggregation are primarily responsible for the change in plasticity and shrinkage (Horpibulsuk et al., 2012a; Thompson, 1966). The shear strength of the stabilized soil gradually increases with time mainly due to the pozzolanic reaction. Calcium hydroxide can react with the silicates and aluminates (natural pozzolanic materials) in the clay (<0.002 mm particle) to form cementitious materials, consisting of calcium silicates and aluminate hydrates (Diamond and Kinter, 1965; Hunter, 1988).

For economic and environmental impact, some waste Ca(OH)_2 rich materials can be utilized together with natural pozzolanic materials in clay to develop a cementitious material. Calcium Carbide Residue (CCR) is a by-product of acetylene (C_2H_2) production process through the hydrolysis of calcium carbide (CaC_2). Its production is described in the following equation (Ramasamy and Periathamby, 2002):



From Eq. (1.1), it is seen that 64 g of calcium carbide (CaC_2) provides 26 g of acetylene gas (C_2H_2) and 74 g of CCR. The CCR is generated as an aqueous slurry and is composed essentially of calcium hydroxide (Ca(OH)_2) with minor parts of calcium carbonate (CaCO_3), unreacted carbon and silicates (Hologado et al., 1992; Ramasamy and Periathamby, 2002). The characteristics of the sludge are influenced by processing parameters during acetylene fabrication. Although not being classified as dangerous/hazardous, its management and disposal require special caution because the high base sludge ($\text{pH} > 12$) can also contain metals (Mg, Br, Sr, Cd, Cu, Pb, Fe,

Mn, Ni and Zn) (Hologado et al., 1992; Ramasamy and Periathamby, 2002). Presently, the demand of CaC_2 for producing acetylene gas in Thailand is 18,500 tons/year. This provides 21,500 tons/year of calcium carbide residue and the demand is continuously increasing each year. Application of the CCR as construction, building and pavement materials includes soil stabilization, pozzolan activation, asphaltic paving mixes and concrete (Al-Khaja et al., 1992; Consoli et al., 2001; Jaturapitakkul and Roongreung, 2003; Ramasamy and Periathamby, 2002). Horpibulsuk et al. (2012a) investigated the strength development in the CCR stabilized clay. They showed that the plasticity index of the stabilized clay decreases with the CCR content up to a certain content and then becomes almost constant. This certain content is designated as the CCR fixation point. It is useful as an indicator to determine the optimal CCR content for strength improvement. Even though there are some available researches on the soil stabilized with the CCR (Horpibulsuk et al., 2012a), they focused on the strength development. These studies are not sufficient for earth structure and pavement design. Some other engineering properties such as swelling and collapse behaviors and bearing capacity and durability characteristic are required.

1.2 Research objective

This thesis aims at investigating the mechanism controlling the engineering properties of the CCR stabilized silty clay, to determine the optimal mix proportion for stabilizing the other clayey soils, and to investigate the possibility of using recycled CCR stabilized clay as fill and pavement materials. The engineering properties studied are unconfined compressive strengths under unsoaked and soaked

conditions and wetting and drying cycles, water absorption, volume change and swelling and collapsible behavior.

This research has been undertaken with the following objectives:

1. To understand the mechanical properties of CCR stabilized silty clay.
2. To understand the influence of w-d cycles on the strength of CCR stabilized silty clay and evaluate the advantages of the input fly ash (FA) on the durability improvement against the w-d cycles.
3. To understand the basic and engineering properties of the recycled CCR stabilized silty clay.

1.3 Organization of the dissertation

This thesis contains six chapters, with emphasis on three main parts. The first part studies the effect of influential factors such as water content, CCR content and curing time on the physical and engineering properties of the clay stabilized with the CCR. The second part illustrates the advantages of the input FA on the durability improvement against the wetting-drying cycles (w-d cycles). Finally, the study focuses on the engineering properties of the reclaimed CCR stabilized silty clay for pavement remedy applications such as pavement recycling technique.

This thesis divides according to the following outline:

Chapter I is the introduction part describing purposes and scope of the study.

Chapter II is the literature review of the mechanisms, properties, influential factors, development, application, etc. on the lime stabilized soils.

Chapter III presents the effect of influential factors such as water content, CCR content and curing time on the physical and engineering properties of the clay stabilized with the CCR.

Chapter IV presents the reduction of strength of the CCR stabilized silty clay due to the influence of w-d cycles and evaluates the advantage of the input FA on the durability improvement against the wetting-drying cycles (w-d cycles).

Chapter V presents the engineering properties of the reclaimed CCR stabilized clay for pavement remedy applications such as pavement recycling technique. The basic properties such as the index, the particle size, the linear shrinkage, the free swelling, the compaction and the compressive strength of reclaimed CCR stabilized clay are investigated.

Separate conclusions are drawn in each chapter and overall conclusions are summarized in **Chapter VI**.

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CHAPTER II

REVIEW OF THE LITERATURE

2.1 General

Soil or ground improvement techniques in geotechnical engineering means modification of soil properties such as increasing in soil shear strength, reducing in soil compressibility and soil permeability (Bergado et al., 1996). As tabulated in Table 2.1, the ground improvement techniques can be classified broadly into 4 categories, namely: reinforcement, admixture or grouting, compaction or densification, and dewatering.

The applicability of these techniques depends on soil types. Various ground improvement methods have been tested to provide various solutions to existing problems on a problematic soil that may influence their utilization. Soil improvement techniques are not limited to portions below ground, but also include soil improvement of embankment fill above ground by earth reinforcement. The combined above and below ground improvement may yield optimum advantages in the solution of problems in infrastructure construction on a problematic soil.

Table 2.1 Applicability of ground improvement for different soil type (Kamon and Bergado, 1991)

Improvement mechanism	Reinforcement	Admixture or grouting	Compaction	Dewatering
Improving period	Depending on the life of the material	Relatively short term	Long term	Long term
Organic soil	X	X		X
Volcanic clay soil	X	X		X
High plastic soil	X	X		X
Low plastic soil	X	X	X	X
Silty soil	X	X	X	X
Sandy soil	X	X	X	
Gravel soil	X	X	X	
Improved state of soil	Interaction between soil and material (no change in soil stage)	Cementation (change in soil stage)	High density by decreasing void ratio (change in soil stage)	High density by decreasing void ratio (change in soil stage)

2.2 Silty Clay in Northeast Thailand

The soil in northeast Thailand generally consists of two layers. The upper soil layer (varying from 1.0-3.0 m thickness) is wind-blown and deposited over several decades. It is silty clay with low to moderate strength. This upper soil is a problematic soil, which is sensitive to change in water content. Its collapse behavior due to wetting is illustrated by Kohgo et al., 1997 and Kohgo and Horpibulsuk, 1999. The soil in the dry state has high bearing capacity and low compressibility. The bearing capacity significantly decreases as the water content increases, which causes several problems for foundation and pavement structures (Horpibulsuk et al., 2008). The lower layer is a residual soil, weathered from claystone, consisting of clay, silt and sand.

The previous researches studied on this soil, such as Horpibulsuk et al., (2008) investigated a case history on the failure of Suranivet 9, a student dormitory, Suranaree University of Technology. They found that, the severe damage in the building was due to the differential settlement, which was caused by the variation in thickness of the upper soil (clayey sand). Part of the building was underlain by SUT silty clay (high bearing capacity) and part by the upper sand (low bearing capacity). Based on the SHANSEP technique and the direct shear test results on undisturbed and remolded SUT silty clay samples, the relationship between the normalized shear strength and the OCR is presented as $S_u / \sigma'_{vc} = 0.278OCR^{0.8}$, where $1 < OCR < 32$. The relationship between the undrained shear strength and the standard penetration number obtained by the back analysis of the pile load test result is $S_u = N/1.5$, where $29 < N < 68$. These two relations were successfully used to design the underpinned piles. Based on the triaxial compression and pile load test results, the yield surface of SUT silty clay can be practically defined by the elliptical expression. Drucker Prager

failure is suitable for the prediction of the ultimate load and the load-settlement curve of the micro-pile. In the other words, modified cam clay model is the appropriate soil model.

Horpibulsuk et al. (2009) observed the role of fly ash on the strength and microstructure development in the blended cement admixed clay. The study found that the flocculation of clay particles due to the cation exchange process is controlled by cement content, regardless of fly ash content. It results in the increases in dry unit weight with insignificant change in liquid limit. Hence, OWCs of stabilized and unstabilized silty clay (low swelling clay) are practically the same. The surfaces of fly ash in the blended cement stabilized clay as still smooth for different curing times and fineness, suggesting that pozzolanic reaction is minimal. Fly ash is considered as a dispersing material in the blended cement stabilized clay. This is different from the application of fly ash as a pozzolanic material in concrete structure in which $\text{Ca}(\text{OH})_2$ from hydration is much enough to be consumed for pozzolanic reaction. From the microstructural investigation, it is concluded that the role of fly ash as a non-interacting material is to disperse the cement-clay clusters with large pore space into smaller clusters with smaller pore space. The dispersing effect by fly ash increases the reactive surface, and hence the increase in degree of hydration as clearly illustrated by the increase in the induced $\text{Ca}(\text{OH})_2$ with replacement ration and fineness.

Horpibulsuk et al. (2010) analyzed the strength development in cement-stabilized silty clay based on microstructural considerations. A qualitative and quantitative study on the microstructure is carried out using a scanning electron microscope, mercury intrusion pore size distribution measurements, and thermal gravity analysis. Three influential factors in this investigation are water content,

curing time, and cement content. Cement stabilization improves the soil structure by increasing inter-cluster cementation bonding and reducing the pore space. As the cement content increases for given water content, three zones of improvement are observed: active, inert and deterioration zones. The active zone is the most effective for stabilization where the cementitious products increase with cement content and fill the pore space. In the active zone, the effective mixing state is achieved when the water content is 1.2 times the optimum water content. In this state, the strength is the greatest because of the highest quantity of cementitious products. In the short stabilization period, the volume of large pores (larger than $0.1\mu\text{m}$) increases because of the input of coarser particles (unhydrated cement particles) while the volume of small pores (smaller than $0.1\mu\text{m}$) decreases because of the solidification of the cement gel (hydrated cement). With time, the large pores are filled with the cementitious products; thus, the small pore volume increases, and the total pore volume decreases. This causes the strength development over time.

For clayey soils, which contain high natural pozzolanic materials, the calcium hydroxide [$\text{Ca}(\text{OH})_2$] rich materials can be used to produce a moderately high strength geo-material. The engineering characteristics of $\text{Ca}(\text{OH})_2$ rich materials stabilized clay are attributed to three basic reactions (Herrin and Mitchell, 1961) which are cation exchange, flocculation and aggregation, and pozzolanic reaction. The influence of cation exchange, flocculation and aggregation are primarily responsible for the change in plasticity and shrinkage (Thompson, 1966 and Horpibulsuk et al., 2012a). The shear strength of the stabilized soil gradually increases with time mainly due to the pozzolanic reaction. Calcium hydroxide can react with the silicates and aluminates (natural pozzolanic materials) in the clay ($<0.002\text{ mm}$ particle) to form cementitious

materials, consisting of calcium silicates and aluminate hydrates (Diamond and Kinter, 1965; Hunter, 1988).

2.3 Calcium Carbide Residue

Calcium Carbide Residue (CCR) is a by-product of acetylene (C_2H_2) production process through the hydrolysis of calcium carbide (CaC_2). Its production is described in the following equation (Ramasamy and Periathamby, 2002):



From Eq.(1), it is seen that 64 g of calcium carbide (CaC_2) provides 26 g of acetylene gas (C_2H_2) and 74 g of CCR. The CCR is generated as an aqueous slurry and is composed essentially of calcium hydroxide ($Ca(OH)_2$) with minor parts of calcium carbonate ($CaCO_3$), unreacted carbon and silicates (Hologado et al., 1992; Ramasamy and Periathamby, 2002). The characteristics of the sludge are influenced by processing parameters during acetylene fabrication. Although not being classified as dangerous/hazardous, its management and disposal require special caution because the high base sludge ($pH > 12$) can also contain metals (Mg, Br, Sr, Cd, Cu, Pb, Fe, Mn, Ni and Zn) (Hologado et al., 1992; Ramasamy and Periathamby, 2002). Presently, the demand of CaC_2 for producing acetylene gas in Thailand is 18,500 tons/year. This provides 21,500 tons/year of calcium carbide residue and the demand is continuously increasing each year.

Application of the CCR as construction, building and pavement materials includes soil stabilization, pozzolan activation, asphaltic paving mixes and concrete (Consoli et al., 2001; Ramasamy and Periathamby, 2002; Al-Khaja et al., 1992;

Jaturapitakkul and Roongreung, 2003). Horpibulsuk et al. (2012) investigated the strength development in the CCR stabilized clay. They showed that the plasticity index of the stabilized clay decreases with the CCR content up to a certain content and then becomes almost constant. This certain content is designated as the CCR fixation point. It is useful as an indicator to determine the optimal CCR content for strength improvement.

2.4 Fundamental Concepts of Lime Stabilized

Lime is used extensively to change the engineering properties of fine-grained soils and the fine grained fractions of more granular soils. It is most effective in treating plastic clays capable of holding large amounts of water. The particles of such clays have highly negative-charged surfaces that attract free cations (i.e., positive-charged ions) and water dipoles. As a result, a highly diffused water layer shown in Figure 2.1 forms around the clay particles, thereby separating the particles and causing the clay to become weak and unstable. The extent to which this occurs depends on the amount of water present and the morphology and mineralogy of the clay (Little, 1987). The addition of lime to a fine-grained soil in the presence of water initiates several reactions. The two primary reactions, cation exchange and flocculation agglomeration, take place rapidly and produce immediate improvements in soil plasticity, workability, uncured strength, and load deformation properties.

Depending on the characteristics of the soil being treated, a pozzolanic reaction may also occur, resulting in the formation of various cementing agents that further increase mixture strength and durability. Pozzolanic reactions are time and temperature dependent. Therefore, given appropriate temperatures, strength

development is gradual but continuous for long periods of time. Temperatures less than 55 to 60°F (13 to 16°C) retard the reaction, while higher temperatures accelerate the reaction. A fourth reaction which may occur in the lime is carbonation, in which lime reacts with atmospheric carbon dioxide to form a relatively insoluble carbonate. This chemical reaction is detrimental to the stabilization process. It can be avoided by properly expedited and sequenced construction procedures that avoid prolonged exposure to the air and/or rainfall.

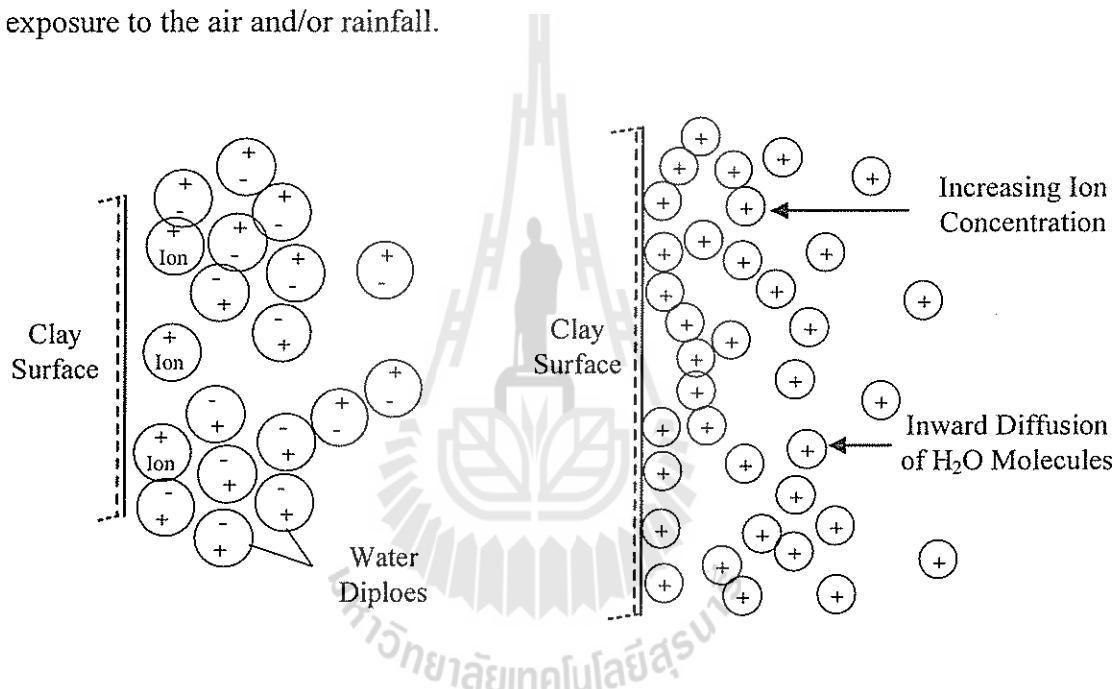


Figure 2.1 Formation of a diffused water layer around clay particle (Little, 1987).

2.4.1 Cation Exchange and Flocculation–Agglomeration

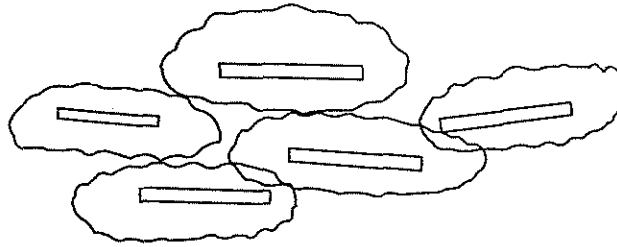
Practically all fine-grained soils display cation exchange and flocculation - agglomeration reactions when treated with lime in the presence of water. The reactions occur quite rapidly when the soil and lime are intimately mixed.

Assuming equal concentrations, the general order of replaceability of the common cations is given by the Lyotropic series, $\text{Na}^+ < \text{K}^+ \ll \text{Mg}^{++} < \text{Ca}^{++}$ (Grim, 1953). In general, higher valence cations replace those of lower valence, and larger

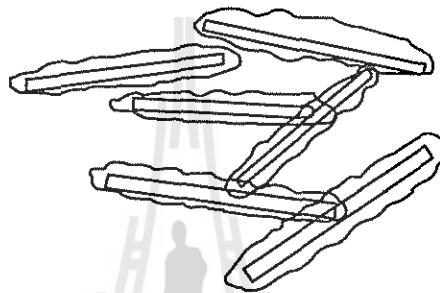
cations replace smaller cations of the same valence. The addition of lime to a soil in a sufficient quantity supplies an excess Ca^{++} , which replaces the weaker metallic cations from the exchange complex of the soil. The exchange of cations causes a reduction in the size of the diffused water layer, thereby allowing clay particles to approach each other more closely, or flocculate (Herrin and Mitchell, 1961). The negativity of the clay particles is increased. Free Ca^{++} ions satisfy this increased negative charge, thereby strengthening the electrostatic forces of flocculation. This phenomenon is illustrated in Figure 2.2.

Flocculation and agglomeration produce an apparent change in texture, with the clay particles “clumping” together into larger-sized “aggregates” (Diamond and Kinter, 1965; Terrel et al., 1979). The flocculation and agglomeration are caused by the increased electrolyte content of the pore water and as a result of ion exchange by the clay to the calcium form. The net result of cation exchange and flocculation - agglomeration is soil modification (Little, 1987):

- i Substantial reduction and stabilization of the adsorbed water layer.
- ii Increased internal friction among the agglomerates and greater aggregate shear strength.
- iii Much greater workability due to the textural change from plastic clay to friable, sand like material.



(a) Parallel arrangement of clay particles with hydrated water layers



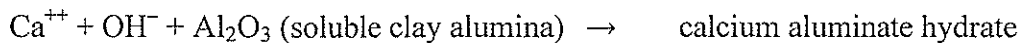
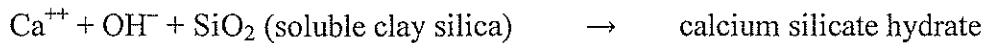
(b) Edge-to-face attraction induced by thin water layer which allows attractive forces to dominate.

Figure 2.2 The reason for the textural change is due to the phenomenon of cation exchange followed by flocculation and agglomeration of the clay particles.

2.4.2 Pozzolanic Reaction

The reactions between lime, water, soil silica, and alumina that form various cementing-type materials are referred to as pozzolanic reactions. The cementing products are calcium silicate hydrates and calcium aluminate hydrates, which are the same as hydrates formed during the hydration of Portland cement (Diamond and Kinter, 1965; Croft, 1964; Terrel et al., 1979). Although a wide variety

of hydrate forms can be obtained, the basic pozzolanic reaction is illustrated in the following equations:



Possible sources of silica and alumina in typical fine-grained soils include clay minerals, quartz, feldspars, micas, and other similar silicate or aluminosilicate minerals, either crystalline or amorphous in nature. The clay minerals and amorphous materials are the important sources in most soils. When a significant quantity of lime is added to a soil, the pH of the soil–lime mixture is elevated to approximately 12.4 (Croft, 1964; Grim, 1953). The solubilities of silica and alumina are greatly increased at these elevated pH levels. Thus, as long as enough residual calcium from the lime remains in the system and the pH remains high enough to maintain solubility, the pozzolanic reaction will continue (Little, 1987). The extent to which the soil–lime pozzolanic reaction proceeds is influenced primarily by natural soil properties. Soils exhibiting a strength increase of greater than 50 lb/in² (345 kPa) after a 28 days of curing period at 73°F (23°C) are considered “reactive,” whereas those with strength increases lower than 50 lb/in² (345 kPa) are deemed non-reactive. Several soil properties and characteristics influence the lime reactivity of a soil, which include the following: soil pH, organic carbon content, natural drainage, excessive quantities of exchangeable sodium, clay mineralogy, and degree of weathering,

presence of carbonates, extractable iron, silica-sesquioxide ratio, and silica-alumina ratio (Ruenkraitersa, 1982).

2.4.3 Carbonation

Lime carbonation is an undesirable reaction that may also occur in soil–lime mixtures. In this reaction, lime reacts with carbon dioxide to form calcium carbonate instead of the cementitious CAHs and CSHs.

2.5 Engineering Properties and Characteristics of Soil - Lime Mixtures : Laboratory Results

Since the beneficial effects of lime stabilization are the result of various reactions between the fines portion of the soil and lime, fine-grained soils, such as clay and silty clay, respond most favorably. A minimum clay content of approximately 10 percent and a plasticity index (*PI*) greater than 10 are desirable, although benefits have been noted for lower *PI* silty soils containing less clay.

For low *PI* sands and non-plastic soils, a pozzolan additive is needed to produce the necessary lime-silica reaction. Fly ash, volcanic ash, and expanded shale fines are examples of pozzolans that have been successfully incorporated.

The effects of lime treatment or stabilization on pertinent soil properties can be classified as immediate and long-term. Immediate modification effects are achieved without curing and are of interest primarily during the construction stage. They are attributed to the cation exchange and flocculation–agglomeration reactions that take place when lime is mixed with the soil. Long-term stabilization effects take place during and after curing, and are important from a strength and durability

standpoint. While these effects are generated to an extent by cation exchange and flocculation–agglomeration, they are primarily the result of pozzolanic strength gain.

2.5.1 Subgrade Soil Modification

The treatment of subgrade soils with lime facilitates construction work in three major ways. First, the addition of lime to the soil decreases the liquid limit and increases the plastic limit, which results in a significant reduction in the *PI*. This reduction in *PI* means a marked increase in workability (Thompson, 1966) which, in turn, expedites manipulation and placement of the treated soil. The manner in which lime influences the plasticity characteristics is illustrated in Figure 2.3

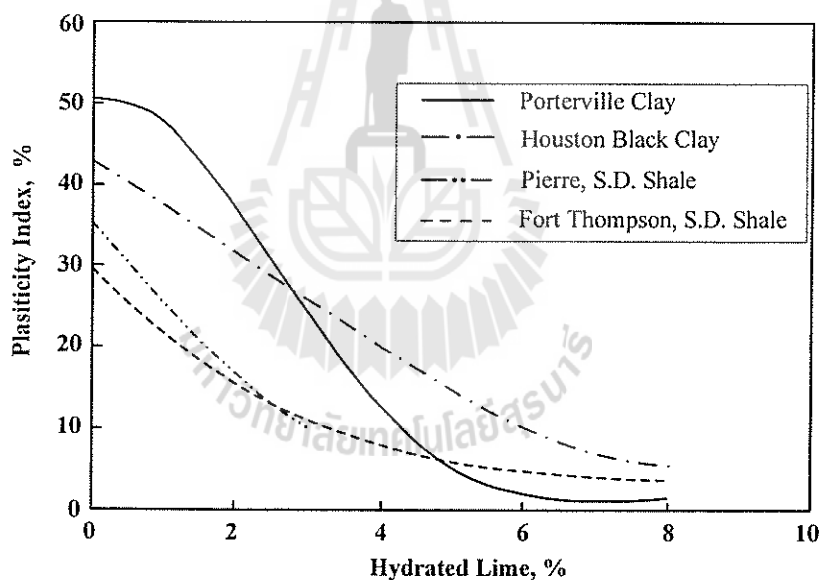


Figure 2.3 Influence of lime on Plasticity index, *PI* (Holtz, 1969).

The second way in which construction is facilitated is through a change in the moisture - density relationship of the soil, as a result of more and more lime being added. The change reflects the new nature of the soil and is marked by a

decrease in the maximum dry density and increase in the optimum moisture content (Hausamann, 1990). Figure 2.4 illustrates this phenomenon for a clay-loam soil.

Whereas prior to the application of lime, a soil in a fairly saturated state would not be capable of reaching maximum density, the same soil with lime may require additional moisture to achieve the new maximum density. The excess water is expended in the soil-lime reactions, thereby creating a less mud-prone construction platform.

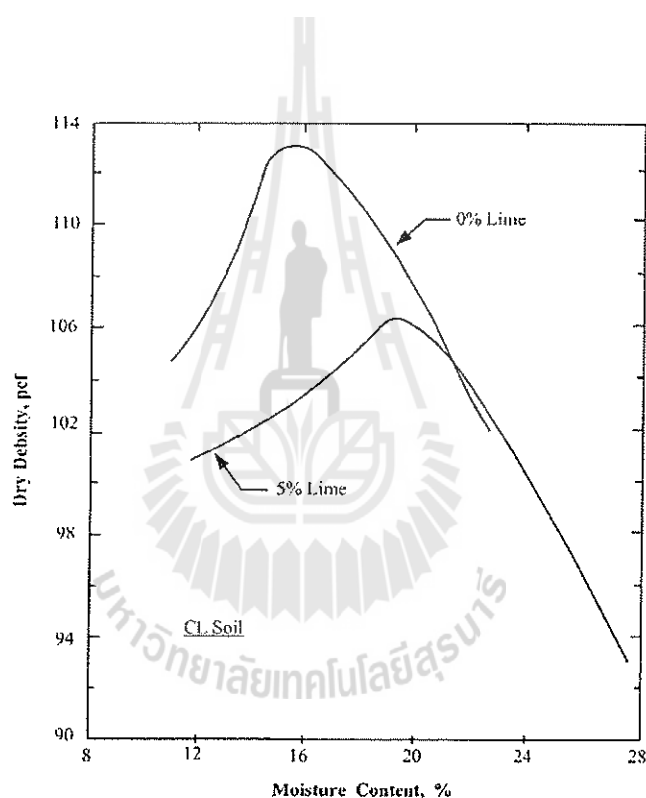


Figure 2.4 The shift in maximum dry density and optimum moisture content is evidence of the physical changes that occur (immediately) during lime treatment (Terrel et al., 1979).

The third major way in which construction is facilitated by the addition of lime to a soil is through immediate increases in soil strength and deformation properties. These improvements, which are largely the result of the flocculated

particle structure, increase the mobility of wheeled vehicles involved in construction operations and they help provide a stable working platform for all construction equipment. Examples of the immediate increases in three specific strength parameters unconfined compression strength, cone index, and the California Bearing Ratio (CBR) are shown in Figure 2.5. As indicated by this figure, the strength increases in some cases can be much more than two fold.

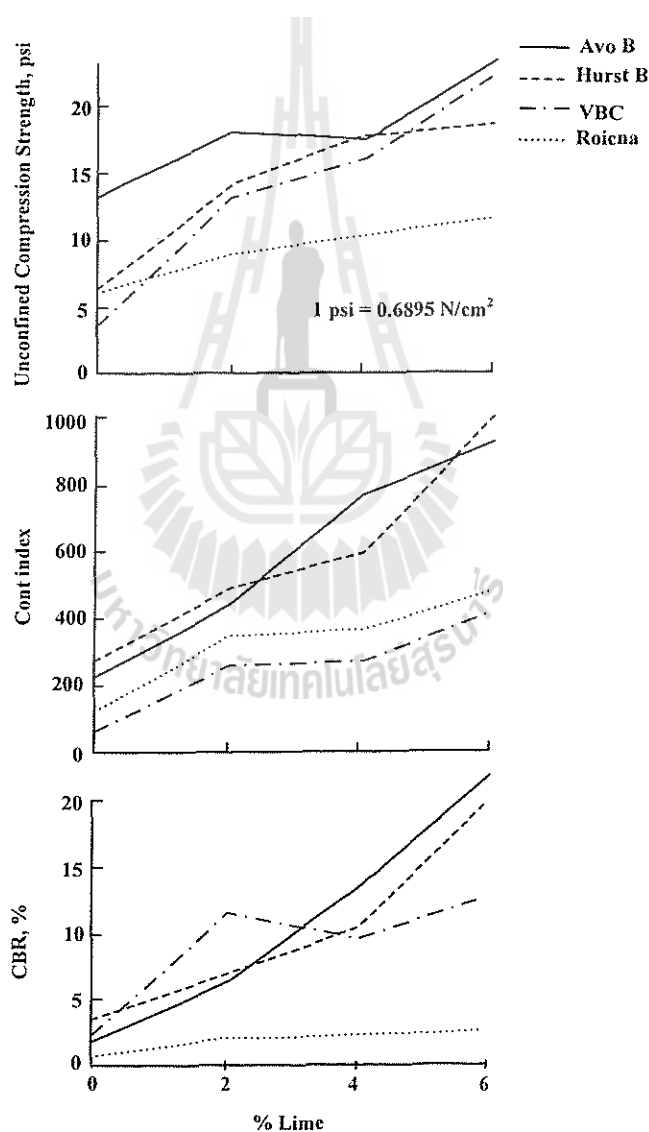


Figure 2.5 Immediate effects of lime treatment on soil strength
(Transportation Research Board, 1987).

2.5.2 Treatment of Expansive Soil

Soil swell potential and swelling pressure are normally significantly reduced by lime treatment (Little, 1987). In fact, the reduction in *PI* associated with virtually all fine-grained soils upon the addition of lime is a significant indication of the reduction of swell potential due to lime stabilization. One particular relationship developed between *PI* and swell potential is illustrated in Figure 2.6. In this plot, the percent swell is defined as volume change incurred in the soil as the moisture content increases from optimum moisture level to saturation level.

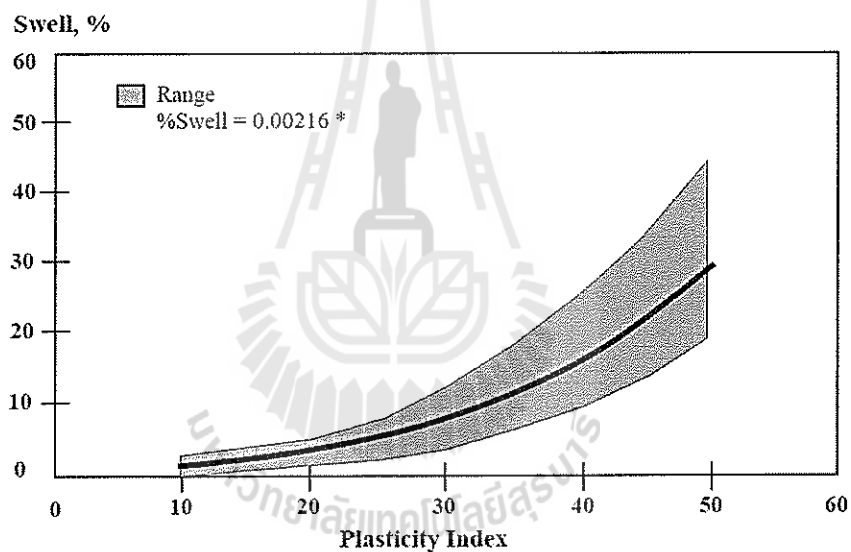


Figure 2.6 Swell potential as a function of plasticity index (Little, 1995).

CBR swell values of lime-treated soils vary, but it is not uncommon to decrease swell to less than 0.1 percent, compared to values of 7 to 8 percent for untreated soil. Typical expansive pressures are shown in Figure 2.7 (Transportation Research Board, 1987).

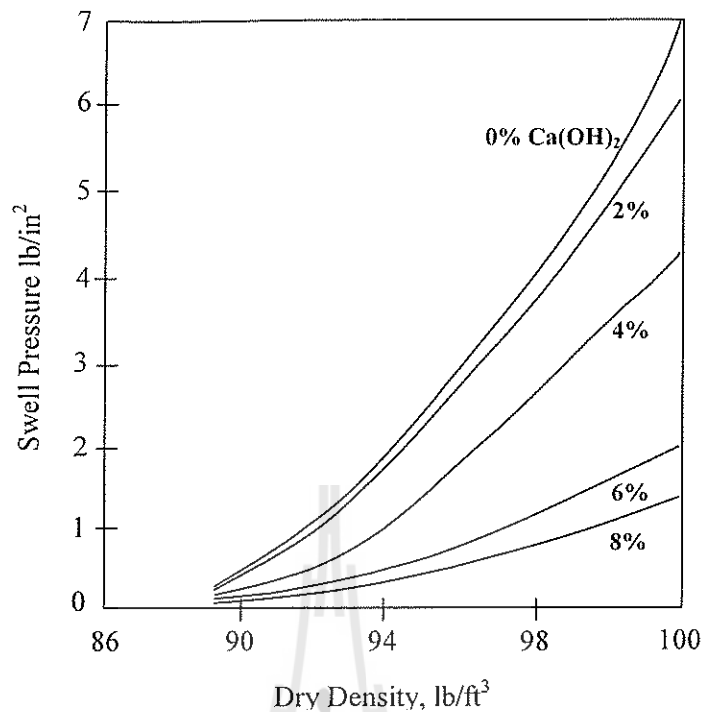


Figure 2.7 Swell pressure–density relationships for lime-treated Porterville clay
(Transportation Research Board, 1987).

2.5.3 Pavement Performance

The material properties of both lime-stabilized soils and lime-stabilized aggregates, as related to their impact on overall pavement performance, can be divided into four categories (Little, 1999):

2.5.3.1 Strength

The most obvious improvement in a lime-reactive soil or aggregate is strength gain over time. The various strength parameters impacted by the pozzolanic reactions include unconfined compressive strength, tensile strength, flexural strength, and CBR. Examples of long-term strength increases for various lime-treated soils can be seen in Figure 2.8. The strength changes which is affected by

lime percentage and time can be seen. Long-term field performance of lime-stabilized pavements has also been validated (Kentucky Transportation Center, 2002)

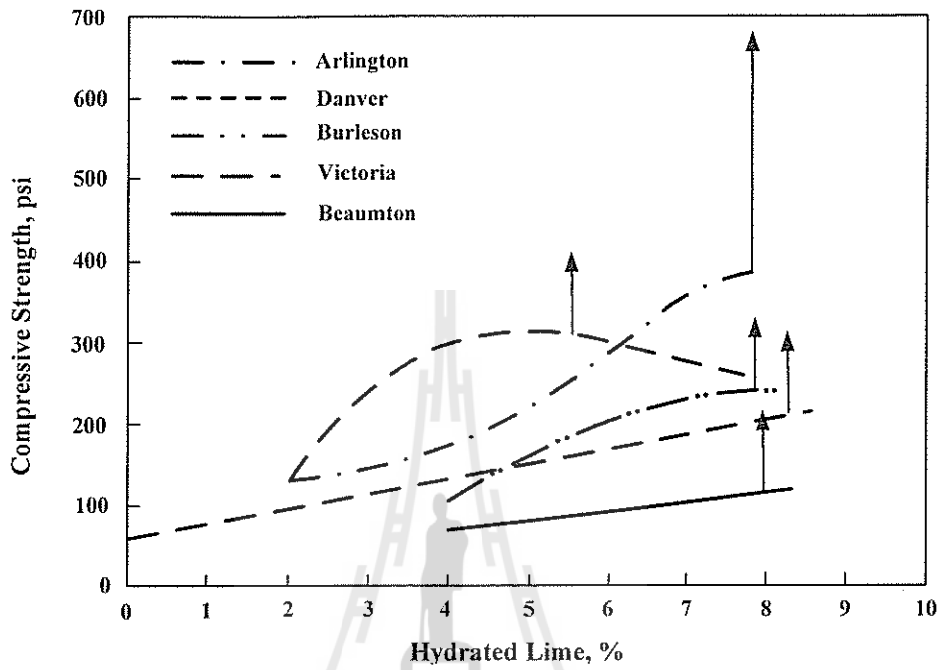


Figure 2.8 Long-term strength gain effects of various lime-treated soils in TX & CO (Little, 1995).

2.5.3.2 Stiffness

Concurrent with the strengthening of a soil brought about by pozzolanic reactions, are changes in the stress-strain relationship of the material (Little, 1999). Lime-stabilized soils fail at much higher deviator stresses than their non-stabilized counterparts, and at a much lower strain (typically about 1 percent strain for the stabilized mixture versus about 3 percent for the nonstabilized material). Materials tested in the laboratory (repeated-load triaxial and indirect tensile tests) and in the field (impulse deflection testing, vibrational testing) both confirm significant increases over time in the resilient properties of lime-treated materials. For instance,

as shown in Figure 10, as the curing time for a lime-stabilized clay increases, the compressive stress at failure increases dramatically and the strain at failure correspondingly decreases.

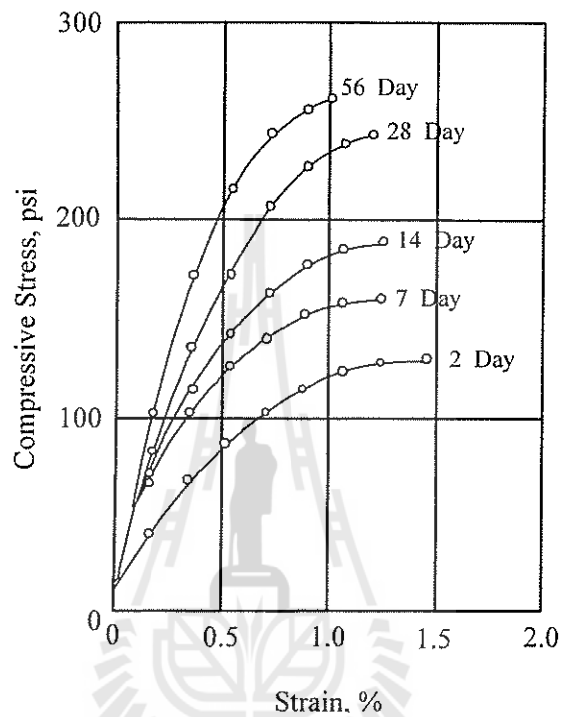


Figure 2.9 Change in stress-strain characteristics over time for a reactive, lime-stabilized soil (Goose Lake clay) (Little, 1995)

2.5.3.3 Fracture and fatigue

Flexural fatigue strength is related to the number of loads that can be carried by a material at a given stress level, and it is an important consideration in the evaluation of lime-soil and lime-aggregate mixtures. The strength-gain effects produced by pozzolanic reactions are often substantial for reactive soils. The response curves of various cured lime-soil mixtures in Figure 2.10 are analogous to those obtained for some cement treated aggregates. The strengths of these mixtures at 5

million stress applications varied from 41 to 66 percent of the ultimate flexural strength.

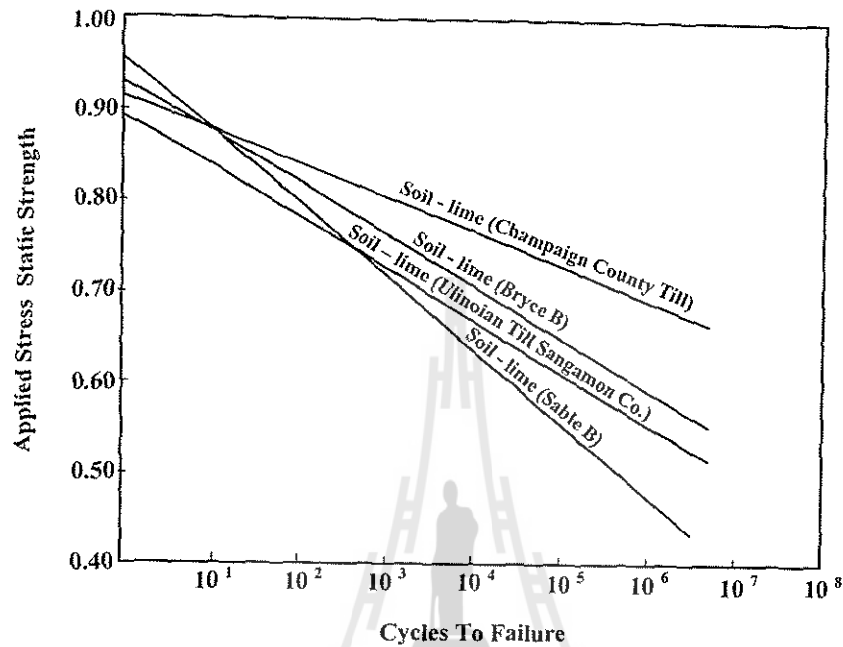


Figure 2.10 Flexural fatigue response curves for various lime-stabilized soils in Illinois (Swanson and Thompson, 1967).

2.5.3.4 Durability

The ability of lime-stabilized materials to resist the detrimental effects of moisture and freeze-thaw cycling over time has been evaluated in several ways, in both the laboratory (e.g., soaking in conjunction with strength/stiffness tests, cyclic freeze-thaw tests) and the field. The results of these evaluations have often shown only slight detrimental effects of environment on the levels of strength/stiffness produced by the addition of lime. As an example, an Illinois study found that the ratio of soaked to unsoaked compressive strength of lime-soil mixtures is quite high, at approximately 0.7 to 0.85 (Thompson, 1970). The soaked specimens

seldom achieved 100 percent saturation and, in most cases, the degree of saturation was in the range of 90 to 95 percent.

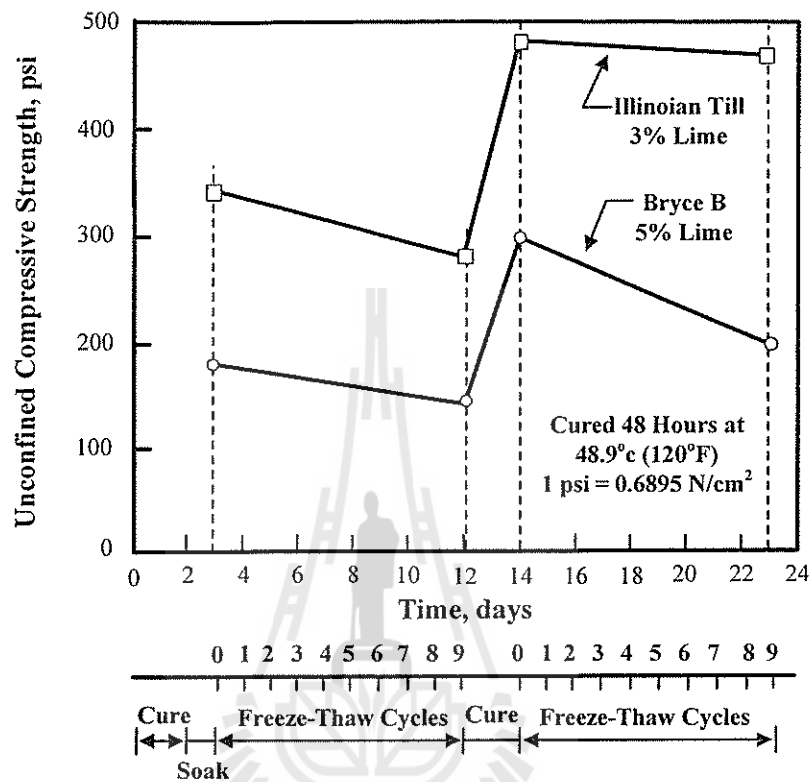


Figure 2.11 If adequate lime is available, pozzolanic reactions will continue to occur under favorable conditions (Thompson and Dempsey, 1969)

2.6 Summary

The soil in northeast Thailand is problematic, which is sensitive to the change in water content. In the dry state, it has high bearing capacity and low compressibility. The bearing capacity significantly decreases as the water content increases, which causes several problems for the foundation and pavement structure. Because this soil contains high natural pozzolanic materials, the calcium hydroxide $[\text{Ca}(\text{OH})_2]$ rich materials can be used to produce a moderately high strength geo-material. The

engineering characteristics of $\text{Ca}(\text{OH})_2$ rich materials stabilized clay are attributed to three basic reactions: cation exchange, flocculation and aggregation, and pozzolanic reaction. The influence of cation exchange, flocculation and aggregation are primarily responsible for the change in plasticity and shrinkage. The shear strength of the stabilized soil gradually increases with time mainly due to the pozzolanic reaction.

The CCR is a by-product of acetylene (C_2H_2) production process through the hydrolysis of calcium carbide (CaC_2). The CCR is generated as an aqueous slurry and is composed essentially of calcium hydroxide ($\text{Ca}(\text{OH})_2$) with minor parts of calcium carbonate (CaCO_3), and unreacted carbon and silicates. The chemical composition of the CCR is similar to that of the hydrated lime. Consequence, the CCR is possibly used to improve the engineering properties of the silty clay. Applications of the CCR to civil engineering, construction and infrastructure works include soil stabilization, pozzolan activation, asphaltic paving mixes and concrete, etc.

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CHAPTER III

ENGINEERING PROPERTIES OF CALCIUM CARBIDE RESIDUE STABILIZED SILTY CLAY

3.1 Introduction

Reuse of waste materials for pavement applications is of great international interest. The urgent need for reuse of waste materials is driven mainly by environmental considerations, due to the increased scarcity of natural resources and the increasing cost of land fill in most countries (Arulrajah et al., 2011a). The previous researches included recycled crushed brick (Arulrajah et al., 2011b; Debieb and Kenai, 2008; Poon and Chan, 2006), recycled crushed glass (Disfani et al., 2011; Grubb et al., 2006; Wartman et al., 2004), recycled concrete aggregate (Courard et al., 2010; Tam and Tam, 2007), other types of construction, demolition and excavation wastes (Arulrajah et al., 2011a; Bennert et al., 2000; Herrador et al., 2012; Khalaf and DeVenney, 2005) and waste biosolids (Arulrajah et al., 2011c). Besides, the improvement of engineering properties of waste materials by the cementing agent is an alternative means (Hoyos et al., 2011; Puppala et al., 2011; Taha et al., 2002).

Portland cement is commonly used for the stabilization. Effects of some influential factors i.e., water content, cement content, curing time and compaction energy on the microstructure and engineering characteristics of cement stabilized soils have been extensively researched (Consoli et al., 2000; Horpibulsuk et al., 2004a; Horpibulsuk et al., 2006; Horpibulsuk et al., 2010a; Horpibulsuk and Miura, 2001; Horpibulsuk et al., 2004b; Horpibulsuk et al., 2003; Horpibulsuk et al., 2005; Horpibulsuk et al., 2011a; Horpibulsuk et al., 2010b; Horpibulsuk et al., 2009b; Horpibulsuk et al., 2011b; Horpibulsuk et al., 2011c; Kamon and Bergado, 1992; Kasama et al., 2007; Miura et al., 2001; Nagaraj et al., 1997; Suebsuk et al., 2010; Suebsuk et al., 2011; Tatsuoka and Kobayashi, 1983; Terashi et al., 1979; Yin and Lai, 1998 and others). A high unit cost and energy intensive process of Portland cement are the driving forces for the alternative cementitious additives.

For clayey soils, which contain high natural pozzolanic materials, the calcium hydroxide [$\text{Ca}(\text{OH})_2$] rich materials can be used to produce a moderately high strength geo-material. The engineering characteristics of $\text{Ca}(\text{OH})_2$ rich materials stabilized clay are attributed to three basic reactions (Herrin and Mitchell, 1961) which are cation exchange, flocculation and aggregation, and pozzolanic reaction. The influence of cation exchange, flocculation and aggregation are primarily responsible for the change in plasticity and shrinkage (Horpibulsuk et al., 2012a; Thompson, 1966). The shear strength of the stabilized soil gradually increases with time mainly due to the pozzolanic reaction. Calcium hydroxide can react with the silicates and aluminates (natural pozzolanic materials) in the clay (<0.002 mm particle) to form cementitious materials, consisting of calcium silicates and aluminate hydrates (Diamond and Kinter, 1965; Hunter, 1988).

For economic and environmental impact, some waste Ca(OH)_2 rich materials can be utilized together with natural pozzolanic materials in clay to develop a cementitious material. Calcium Carbide Residue (CCR) is a by-product of acetylene (C_2H_2) production process through the hydrolysis of calcium carbide (CaC_2). Its production is described in the following equation (Ramasamy and Periathamby, 2002):



From Eq.(3.1), it is seen that 64 g of calcium carbide (CaC_2) provides 26 g of acetylene gas (C_2H_2) and 74 g of CCR. The CCR is generated as an aqueous slurry and is composed essentially of calcium hydroxide (Ca(OH)_2) with minor parts of calcium carbonate (CaCO_3), unreacted carbon and silicates (Hologado et al., 1992; Ramasamy and Periathamby, 2002). The characteristics of the sludge are influenced by processing parameters during acetylene fabrication. Although not being classified as dangerous/hazardous, its management and disposal require special caution because the high base sludge ($\text{pH} > 12$) can also contain metals (Mg, Br, Sr, Cd, Cu, Pb, Fe, Mn, Ni and Zn) (Hologado et al., 1992; Ramasamy and Periathamby, 2002). Presently, the demand of CaC_2 for producing acetylene gas in Thailand is 18,500 tons/year. This provides 21,500 tons/year of calcium carbide residue and the demand is continuously increasing each year.

Application of the CCR as construction, building and pavement materials includes soil stabilization, pozzolan activation, asphaltic paving mixes and concrete (Al-Khaja et al., 1992; Consoli et al., 2001; Jaturapitakkul and Roongreung, 2003; Ramasamy and Periathamby, 2002). Horpibulsuk et al. (2012) investigated the

strength development in the CCR stabilized clay. They showed that the plasticity index of the stabilized clay decreases with the CCR content up to a certain content and then becomes almost constant. This certain content is designated as the CCR fixation point. It is useful as an indicator to determine the optimal CCR content for strength improvement. Even though there are some available researches on the soil stabilized with the CCR (Horpibulsuk et al., 2012b), they focused on the strength development. These studies are not sufficient for earth structure and pavement design. Some other engineering properties such as swelling and collapse behaviors and bearing capacity are required.

This research aims to study the effect of influential factors such as water content, CCR content and curing time on the physical and engineering properties of a problematic silty clay stabilized with the CCR. The engineering properties involved are soaked and unsoaked strength, swelling and collapse behavior, water absorption, volume change due to water immersion and unsoaked and soaked bearing capacity. This study is to ascertain the performance in fill and pavement base and subbase applications. The CCR fixation point is used to analyze the change in engineering properties with the CCR content. This research outcome is useful as fundamental for facilitating the engineering decision on the mix proportion of soil, water and CCR. The knowledge gained from this outcome can be taken to explain and analyze the improvement of engineering properties and to determine the optimal mix proportion for the other clayey soils stabilized with different Ca(OH)_2 rich material.

3.2 Laboratory investigation

3.2.1 Soil Sample

The soil sample is a problematic silty clay collected from the Suranaree University of Technology campus in Nakhon Ratchasima province, Thailand at a depth of 3 meters. This soil is an inland clay, which is sensitive to change of water content (Horpibulsuk et al., 2008b). The soil contains high fine particle content, similar to many marine soft clayey soils such as Bangkok clay (Horpibulsuk et al., 2007), Singapore clay (Arulrajah and Bo, 2008a; Arulrajah and Bo, 2008b; Chu et al., 2009a; Chu et al., 2009b) and Ariake clay (Modmoltin, 2002; Ohtsubo et al., 2000) etc. The fine particles play significant problems on the swelling and collapse upon drying and wetting for inland clays (Horpibulsuk et al., 2008b; Kohgo et al., 1993) and on bearing capacity and consolidation for soft marine clays (Arulrajah et al., 2007; Arulrajah et al., 2006; Bo et al., 2012; Bo et al., 2011). The chemical stabilization can improve these unfavorable engineering characteristics. Thus, the research outcome provides useful information for both marine and inland clays.

The grain size distribution of the silty clay (Figure 3.1) shows that the clay is composed of 2% sand, 43% silt and 55% clay. The specific gravity is 2.72. The liquid and plastic limits are approximately 55% and 27%, respectively. Based on the Unified Soil Classification System (USCS), the clay is classified as high plasticity (CH). During sampling, the groundwater had disappeared. The natural water content was 10 percent. The free swell test proposed by Prakash and Sridharan (Prakash and Sridharan, 2004) shows that the clay is classified as low swelling with a free swell ratio (FSR) of 1.6. The Cation Exchange Capacity, CEC is 27.6 meq/100g. SEM

photo of the clay shown in Figure 3.2a indicates that its particles are irregular in shape. The chemical composition of the clay is shown in Table 3.1. A XRD-pattern of the clay (Figure 3.1) shows that main chemical composition is SiO_2 . The sum of SiO_2 , Al_2O_3 and Fe_2O_3 is 60.54%, which is considered as high for pozzolanic reaction.

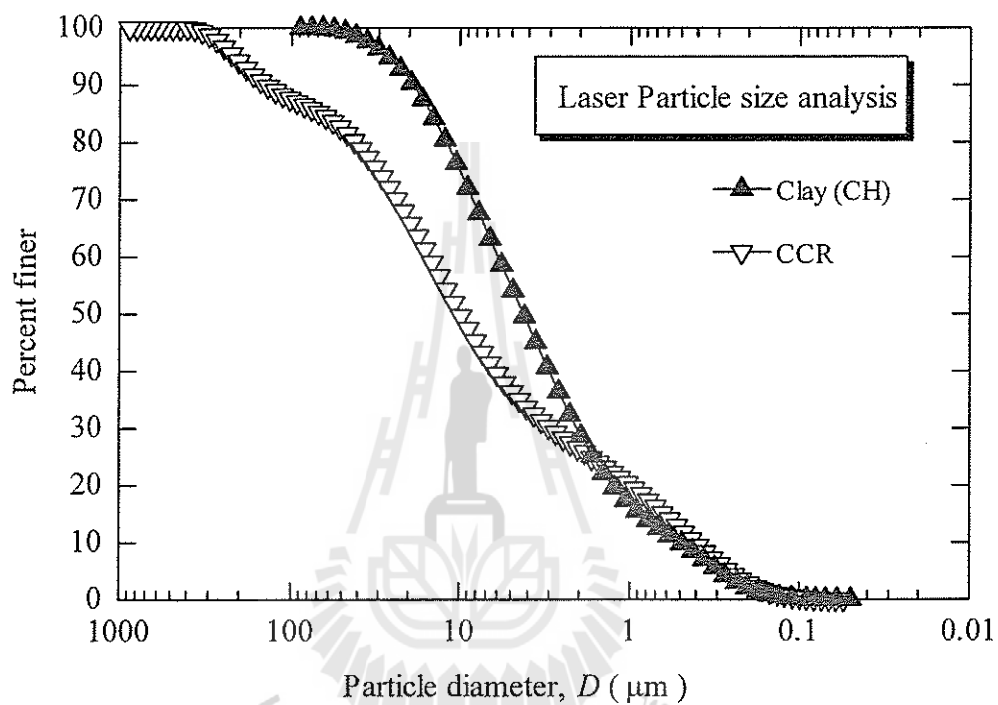
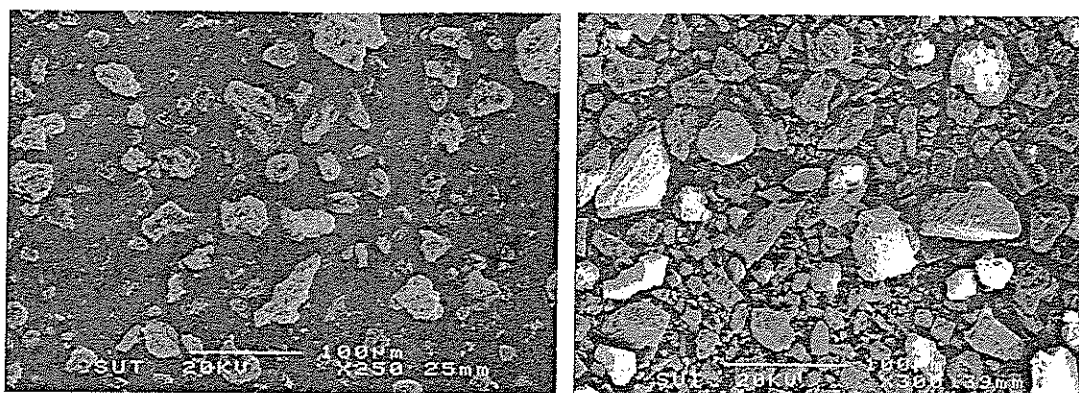


Figure 3.1 Grain size distribution of the silty clay and the CCR.



(a) Clay

(b) CCR

Figure 3.2 SEM photos of the silty clay and the CCR.**Table 3.1** Chemical properties of the clay, the CCR, and hydrated lime.

Chemical composition (%)	Clay	CCR	Hydrated lime
CaO	26.15	70.78	90.13
SiO ₂	20.10	6.49	1.29
Al ₂ O ₃	7.55	2.55	0.24
Fe ₂ O ₃	32.89	3.25	0.49
MgO	0.47	0.69	0.22
SO ₃	4.92	0.66	0.45
Na ₂ O	ND	ND	ND
K ₂ O	3.17	7.93	3.3
LOI	3.44	1.35	1.21

3.2.2 Calcium Carbide Residue

Calcium Carbide Residue (CCR) was obtained from the Sai 5 Gas Product Co., Ltd. The CCR was oven-dried at 200°C for 3 hours and ground in a Los Angeles abrasion machine. The CCR was passed through a sieve No. 40 (425 μm). The specific gravity is 2.25. Table 3.1 shows the chemical composition of the CCR compared with that of a hydrated lime. The chemical composition shows the CaO contents of 90.13% and 70.78% for the hydrated lime and the CCR, respectively. The CCR contains pozzolanic materials (SiO_2 , Al_2O_3 and Fe_2O_3) of about 12.3% while the hydrated lime contains very few of about 2%. The XRD pattern of the CCR is similar to that of the hydrated lime, showing the Ca(OH)_2 as a main composition (*vide* Figure 3.3). The Ca(OH)_2 contents are about 96.5% and 76.7% for the hydrated lime and the CCR, respectively. The high Ca(OH)_2 and CaO contents of the CCR indicate that it can react with pozzolanic materials and produce a cementitious material.

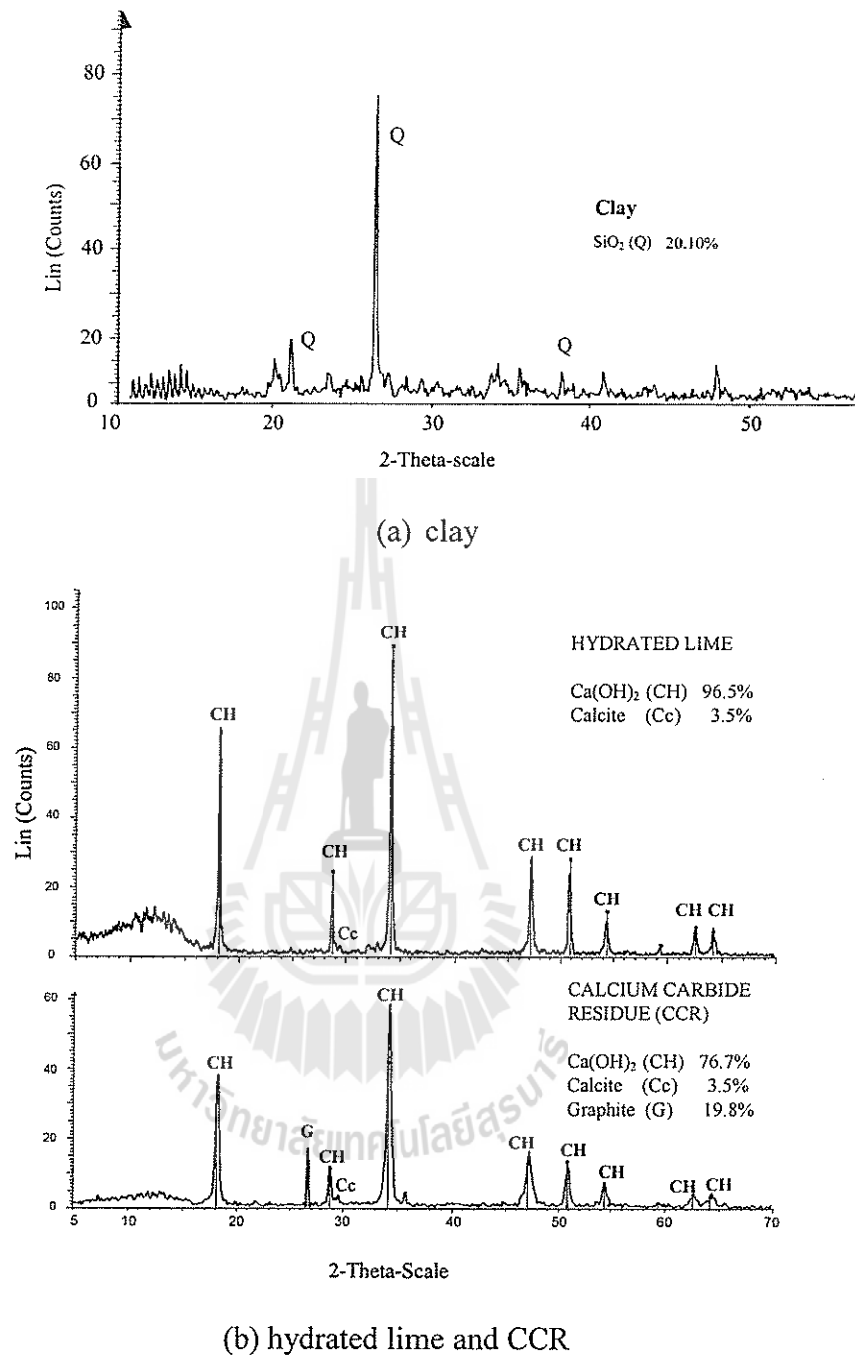


Figure 3.3 XRD pattern of (a) the clay and (b) the hydrated lime and the CCR.

The grain size distribution of the CCR compared with that of the clay is shown in Figure 3.1. The curves were obtained from the laser particle size analysis. The D_{50} of the CCR and the clay are 0.01 and 0.006 mm, respectively. SEM photo of

the CCR is also shown in Figure 3.2. From the grain size distribution and the SEM photos, it is found that the clay particles are smaller than the CCR particles.

3.2.3 Methodology

The clay was passed through a 16-mm sieve to remove coarser particles. It was air-dried for at least three days and then the water content was adjusted for laboratory tests. The index and linear shrinkage tests on the CCR stabilized samples were carried out immediately after thorough mixing. To evaluate the mechanical properties of the CCR stabilized clay, a series of tests, such as unconfined compression, oedometer, water immersion and California bearing ratio (CBR) tests were carried out. All the index and mechanical tests were carried out according to the American Society of Testing and Materials (ASTM) standards. The compaction of the natural clay and the CCR stabilized clay was carried out using a 100 mm standard mold under modified Proctor energy (ASTM D 1557). The optimum water content, OWC , and maximum dry unit weight, $\gamma_{d,max}$ of the compacted clay under the modified Proctor energy are 19% and 17.1 kN/m³, respectively. Generally, the standard Proctor energy is employed for the fill applications of the unstabilized clay. The modified Proctor energy was performed on this stabilized clay to study its engineering properties for pavement base and subbase.

The linear shrinkage is the decrease in length of a soil sample in a shrinkage mold (semi cylindrical troughs 250 mm and 25 mm diameter, with brazed on square ends) when oven dried starting with a water content at the liquid limit. The percentage linear shrinkage (LS) of the specimen is calculated from:

$$LS(\%) = \frac{L_s}{L} \times 100 \quad (3.2)$$

Where L_s is the longitudinal shrinkage of the specimen and L is the length of the mold.

The unconfined compressive strengths were measured on cylindrical samples, which were compacted on both dry and wet sides of optimum. After 24 hours of compaction, the stabilized samples were dismantled from the mold, wrapped in vinyl bags and stored in a humidity chamber of constant temperature ($25 \pm 2^\circ\text{C}$). Unconfined compression (UC) test was run on the samples after 1, 7, 14 and 28 days of curing. The rate of vertical displacement was fixed at 1 mm/min. The tests were performed under soaked and unsoaked conditions. Soaked condition is to simulate the attack of complex wet weather or rainy weather, where large amounts of water filter into the samples. After a desired curing, the standard cylindrical samples were immersed in deionized water for 2 hours in a room at the constant temperature of $25 \pm 2^\circ\text{C}$. The vertical swell and perimeter were measured to determine the volume change prior to the UC test.

The vertical swell and CBR tests were carried out in the standard one dimensional CBR mold. The samples were prepared using the modified Proctor compaction energy, and a pre-loading pressure was imposed on the compacted samples. The samples were allowed to swell under the condition of 4-day water immersion, and readings of dial gauge were periodically recorded by a displacement transducer. The percent swell of samples was calculated as $\Delta h/h_i$, where Δh is the change in height of sample and h_i is the initial height of sample.

The collapse tests were performed in the oedometer ring (60 mm in diameter and 20 mm in height) based on the ASTM D 5333-03. The load increment applied on the sample was twice larger than the previous load. The duration of each load was 24 hours. The collapsible behavior was recorded at an applied vertical stress of 200 kPa under water immersion for 24 hours. The collapse index is determined in the form:

$$\%C = \frac{100\Delta e}{1 + e_0} \quad (3.3)$$

Where Δe is the change in void ratio upon wetting after 24 hours and e_0 is the initial void ratio (before collapse).

The testing program is summarized in Table 3.2. For all tests at each curing time and combination of water content and CCR content, at least five samples were tested under the same condition to check for consistency of the test. In most cases, the results under the same testing condition were reproducible with low mean standard deviation, SD ($SD/\bar{x} < 10\%$, where \bar{x} is mean strength value). The unsoaked and soaked strengths of the lime stabilized clay were also measured and compared with those of CCR stabilized clay to illustrate the advantage of the CCR stabilization. The hydrated lime content was varied from 0 to 30% and the water content was the optimum lime point (7%). The unconfined compression tests were performed on the samples after 7, 14 and 28 days of curing.

Table 3.2 Summary of the testing program.

Test	Stabilizer	Water content, w	Curing time (days)	Replacement ratio (%)
Index properties	CCR	-	-	0, 3, 5, 7, 10, 15, 20
Linear shrinkage	CCR	$w_L = 55\%$	-	0, 3, 5, 7, 10, 20, 30
Compaction	CCR	-	-	0,3,5,7,10
Unconfined compression (soaked and unsoaked conditions)	CCR	$OWC = 19\%$	7	0, 3, 5, 7, 10, 12, 15, 20, 25, 30
Unconfined compression (soaked and unsoaked conditions)	CCR, Hydrated lime	$0.80OWC,$ $0.90OWC,$ $OWC,$ $1.10OWC,$ $1.20OWC$	7, 14, 28	CCR fixation point = 7
California bearing ratio (CBR)	CCR	$0.90OWC,$ $0.95OWC,$ $OWC,$ $1.05OWC,$ $1.10OWC$	1, 7, 14, 28	CCR fixation point = 7
Swell pressure	CCR	$OWC = 19\%$	1, 7, 14, 28	0, 3, 5, 7,10, 15, 20, 30
Collapse	CCR	$OWC = 19\%$	28	0, 3, 5, 7, 10, 20, 30
Water absorption and Volume change	CCR	$OWC = 19\%$	7, 14, 28	CCR fixation point = 7

The triaxial test on the CCR stabilized clay is not within the scope of this study because this paper aims to understand the fundamental behavior of the CCR stabilized clay as a pavement material, which is generally subjected to low confining stress under soaked and unsoaked conditions. The triaxial test results of this material

is so far very few and necessary for further investigation. It is useful for the development of a suitable constitutive model.

3.3 Test results

Figure 3.4 shows the effect of CCR content on the index properties of the CCR stabilized clay. As the CCR content increases, the plastic limit, *PL* of the stabilized clay significantly increases while the liquid limit, *LL* tends to change with small magnitude, resulting in a decrease in the plasticity index, *PI*. However, when the CCR content is greater than 7%, the change in *PI* is minimal. The CCR content of 7% is thus designated as “CCR fixation point” (Horpibulsuk et al., 2012). The decrease in *PI* indicates the flocculation and the coagulate aggregation of the clay particles, which are caused by the absorption of Ca^{2+} ions.

Figure 3.5 illustrates the shrinkage characteristics of the CCR stabilized clay. The natural clay inherently exhibits high linear shrinkage (about 17%). The soil stabilization by the CCR reduces significantly the linear shrinkage. By adding 7% CCR, the linear shrinkage reduces up to about 4 times of the original value. Initially, the linear shrinkage reduces remarkably with the CCR content. The reduction is insignificant when the CCR content is greater than the CCR fixation point. The CCR fixation point is the transitional point that the dramatic change in linear shrinkage limit turns to the insignificant change.

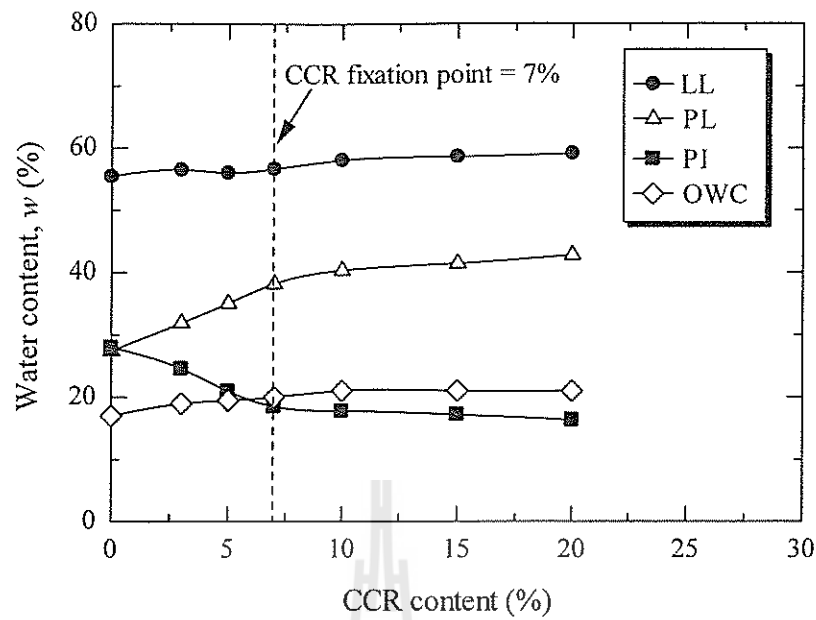


Figure 3.4 Index properties of the CCR stabilized clay.

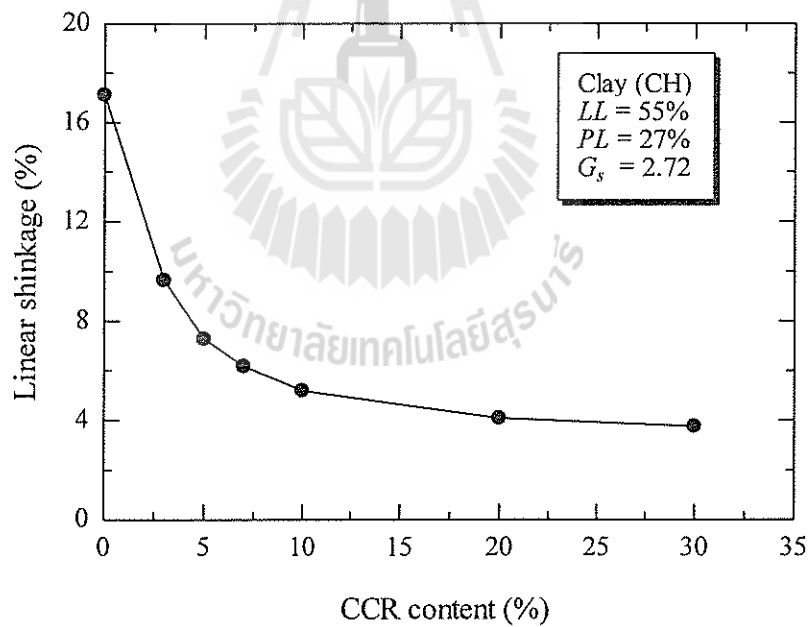


Figure 3.5 Influence of the CCR content on the linear shrinkage.

Figure 3.6 shows the compaction curves of the clay and the CCR stabilized clay. The *OWC* tends to increase as the CCR content increases up to the CCR fixation

point. The reduction in maximum dry unit weight is associated with the increase in *OWC*. The reduction in the maximum dry unit weight could be due to the lower specific gravity of CCR and an immediate formation of cementitious products, which reduce the compatibility (Lees et al., 1982). The compaction characteristics (*OWC*, $\gamma_{d,max}$) are practically constant when the CCR content is higher than the CCR fixation point.

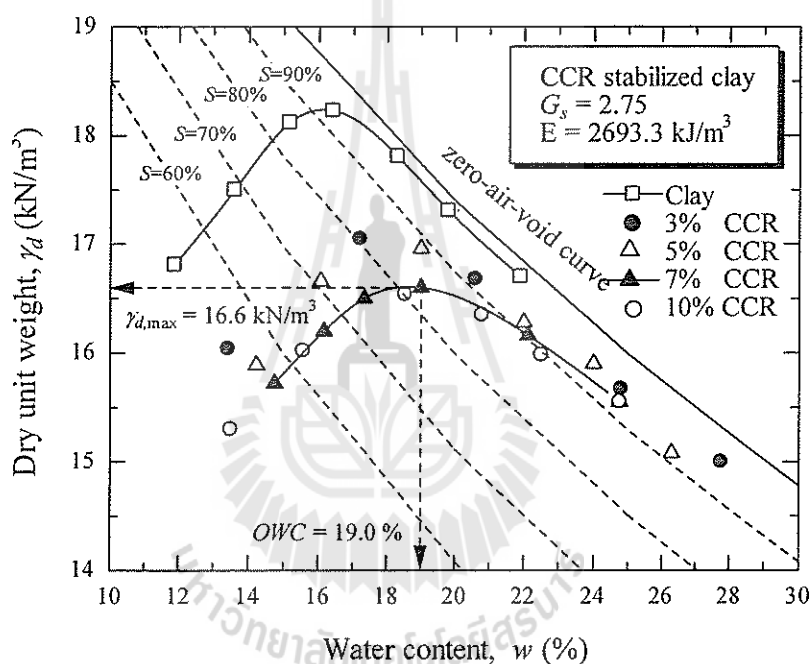


Figure 3.6 Compaction curves of the silty clay and the CCR stabilized clay.

The strength development with the CCR contents of the stabilized clay at the *OWC* (maximum dry unit weight) under unsoaked and soaked conditions is shown in Figure 3.7. The stabilizer contents were designed to be the same for both conditions. The compaction enhances the interparticle forces (effective stress). Consequently, the unsoaked strength of the compacted clay (without CCR) is very high with the compressive strength of 824 kPa. However, the water absorption induces the swell

pressure (reduces the effective stress). The soaked strength becomes null after soaking for few minutes. This significant strength reduction causes several damages of earth structures and pavements in the northeast Thailand during raining. This unpreferable characteristic can be improved by the CCR as shown by the high soaked strength of the CCR stabilized clay. The strength improvement of the stabilized clay under soaked and unsoaked conditions is classified into three zones (*vide* Figure 3.7).

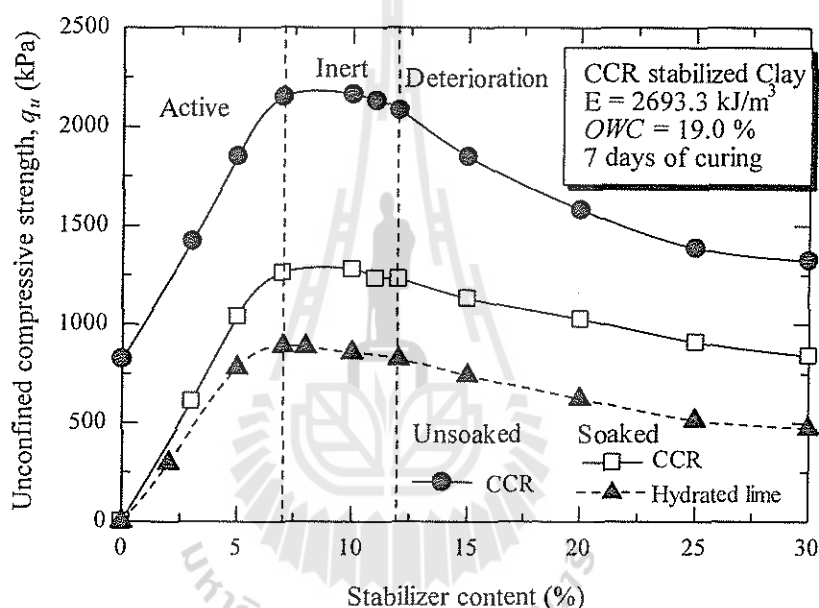
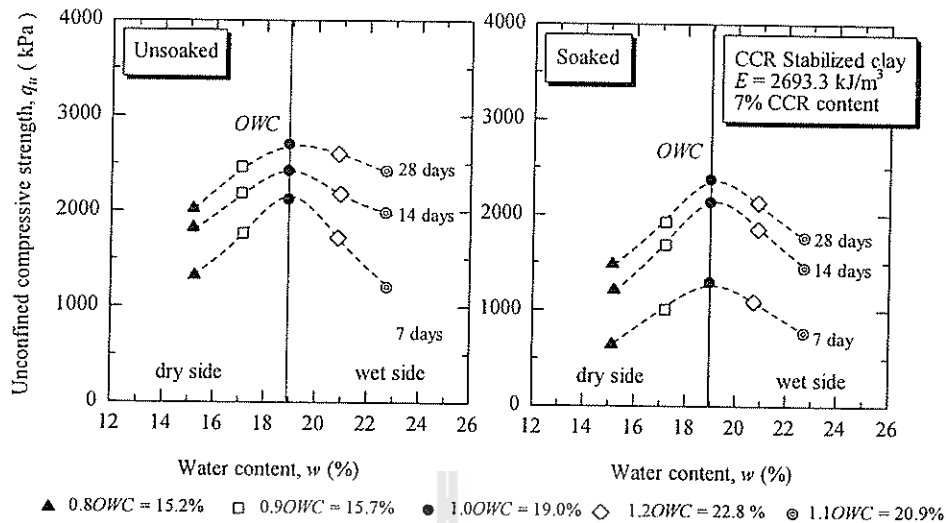


Figure 3.7 Improvement zones of the CCR and lime stabilized samples.

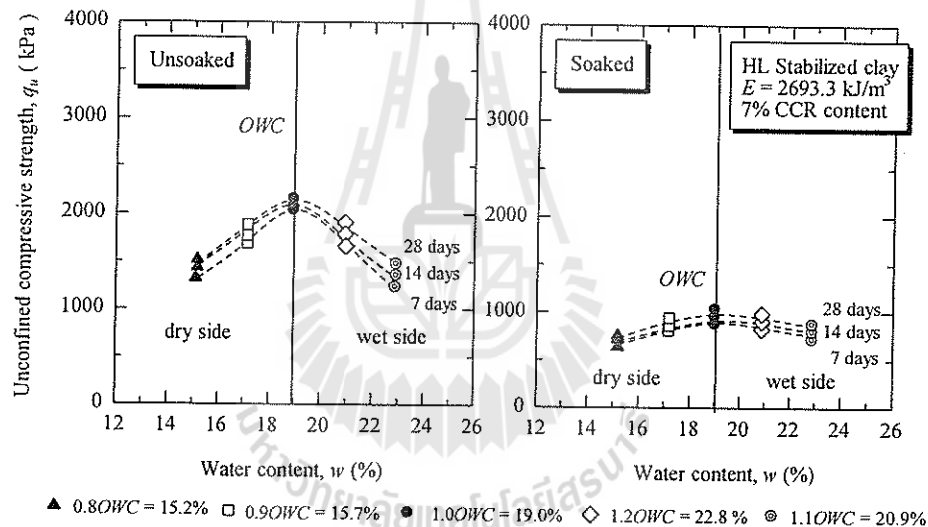
As the CCR content increases, the strength significantly increases. This zone is designated as the active zone. Beyond this zone, the strength development slows down. The incremental gradient becomes nearly zero and does not make any further significant improvement. This zone is referred to as the inert zone (CCR content = 7-12%). The strength decrease appears when the CCR content is higher than 12%. This zone is identified as the deterioration zone. Even though the same pattern of strength development is found for both unsoaked and soaked conditions, the unsoaked strength

is higher. It is interesting that the optimal CCR content providing the highest unsoaked and soaked strengths is 7%, which is the CCR fixation point. The soaked strength of the hydrated lime stabilized clay is also presented in the figure. For all the stabilizer contents, the soaked strengths of the CCR stabilized clay are higher than those of the lime stabilized clay.

Figure 3.8 shows the strength versus water content relationships of the CCR and hydrated lime stabilized samples at the same stabilizer content of 7% under unsoaked and soaked conditions. The maximum unsoaked and soaked strengths of the CCR stabilized clay for different curing times are at about *OWC* (*vide* Figure 3.8a). For the same dry unit weight, both the unsoaked and soaked strengths of the samples compacted on the wet side are slightly higher than those on the dry side. The unsoaked and soaked strengths at different water contents increase with increasing the curing time. For the lime stabilized clay, the relationship between the strength and the water content shows the same pattern as the CCR stabilized clay. But the strength development with time of the lime stabilized clay is insignificant after 7 days of curing. The advantage of the CCR over the hydrated lime is depicted by comparing Figures 3.8a to 3.8b. The early strength (7 day strength) of the lime stabilized clay is slightly lower than the CCR stabilized clay for both unsoaked and soaked conditions and different water contents. But the CCR stabilized clay exhibits remarkably higher strengths for longer curing time because of the insignificant strength development with time of the lime stabilized clay.



(a) CCR stabilized clay



(b) Hydrated lime stabilized clay

Figure 3.8 Compressive strength under unsoaked and soaked conditions of (a) the CCR stabilized clay and (b) the hydrated lime stabilized clay.

Figure 3.9 shows the relationship between the CBR and the water content of the 7% CCR samples under unsoaked and soaked conditions for different curing times. Under both conditions, the maximum CBR values of the CCR stabilized clay are at about *OWC*. The unsoaked CBR values of the samples compacted on the dry

side of optimum are slightly higher than those on the wet side, but the soaked CBR values are lower.

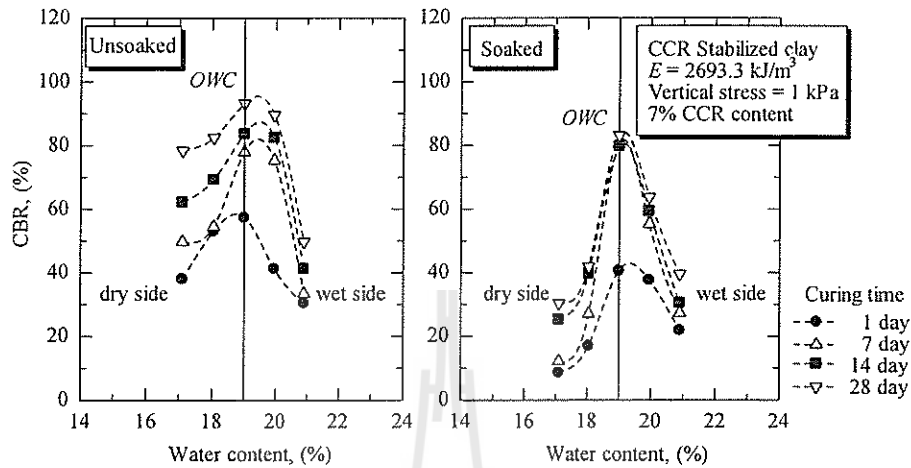


Figure 3.9 Relationship between CBR and water content of CCR stabilized clay.

Figure 3.10 shows the vertical swell versus the CCR content relationship of the CCR stabilized clay at 7 days of curing. The vertical swell of the compacted clay (without CCR) strongly depends on the state of water content. The vertical swell of compacted clay is about 6% at the optimum water content (maximum dry unit weight). The soil stabilization by CCR greatly reduces the vertical swell due to the increase of the cementation bonding among the clay particles. For all water contents, the vertical swell decreases as the CCR content increases. The CCR stabilized samples on the dry side of optimum exhibit highest vertical swell and those compacted at *OWC* yields the lowest vertical swell. This characteristic is the same as that of the compacted clay but the CCR stabilized clay possesses much lower vertical swell. The optimal CCR content for improving the swell behavior ranges from 5% to 10%. The effect of the state of water content is significant when the CCR content is

less than 5%. The vertical swell is less than 1% and insignificantly changed with the CCR content when the CCR content is higher than 5%.

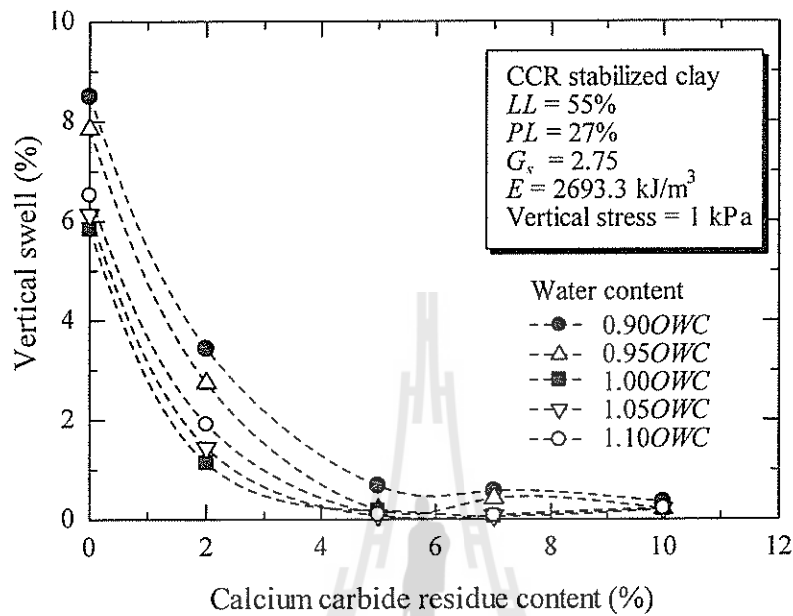


Figure 3.10 Vertical swell of the CCR stabilized clay at different water contents.

Figure 3.11 shows the effect of curing time on the vertical swell of the 7% CCR samples. At very short curing time (1 day), the strength is essentially dependent upon the state of water content. The stabilized samples compacted at *OWC* exhibit the lowest vertical swell. For longer curing times (greater than 7 days), the vertical swell is less than 0.5% for the samples compacted on the dry side of optimum and is less than 0.2% for the samples compacted at *OWC* and on the wet side of optimum. For a given water content, the vertical swell insignificantly changes when the curing time is longer than 7 days.

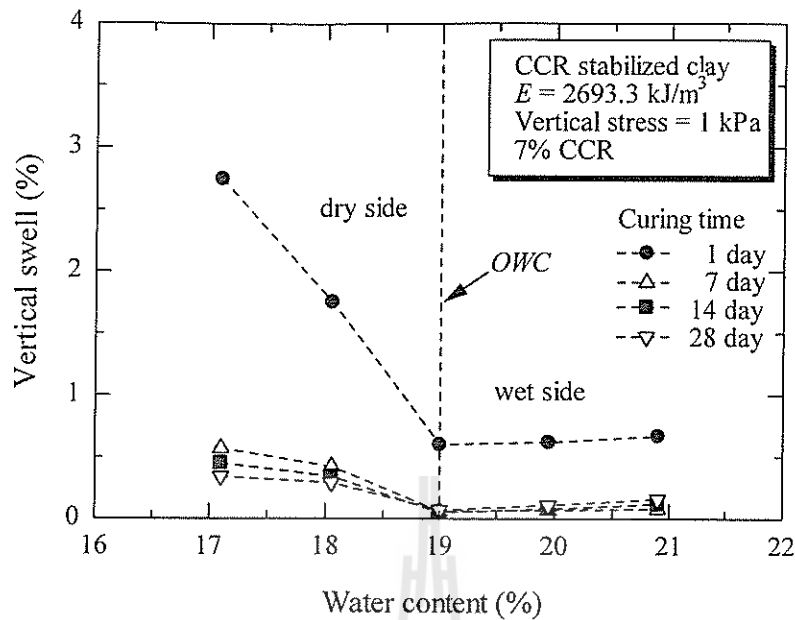


Figure 3.11 Vertical swell versus water content relationships of the CCR stabilized clay for different curing times.

Figure 3.12 shows the vertical swell and the swell pressure versus the CCR content relationships of the stabilized samples at *OWC*. The samples were placed in oedometer rings and allowed to swell under water immersion. The swell pressure is the pressure applied on the samples until the vertical swell becomes null. It is found that the vertical swell of the compacted clay (without CCR) is about 6% (almost the same value as obtained from the soaked CBR test) and the swell pressure is 70 kPa. The CCR significantly reduces the vertical swell and the swell pressure. The vertical swell and the swell pressure of the CCR stabilized clay decrease as the CCR content increases up to the CCR fixation point. The input CCR in the inert zone (CCR content = 7%-12%) yields insignificantly change in the vertical swell and the swell pressure. The vertical swell and the swell pressure increase when the CCR content is in the deterioration zone (CCR content > 12%). The relationship between the swell pressure

and the vertical swell for different curing times and CCR contents is shown in Figure 3.13, which is well represented by the exponential function.

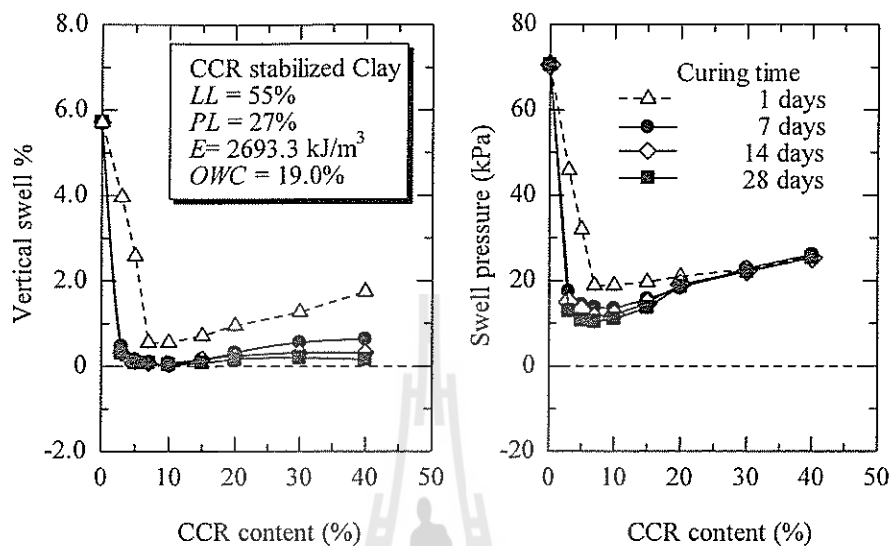


Figure 3.12 Relationship between vertical swell and swell pressure versus CCR content of the CCR stabilized clay.

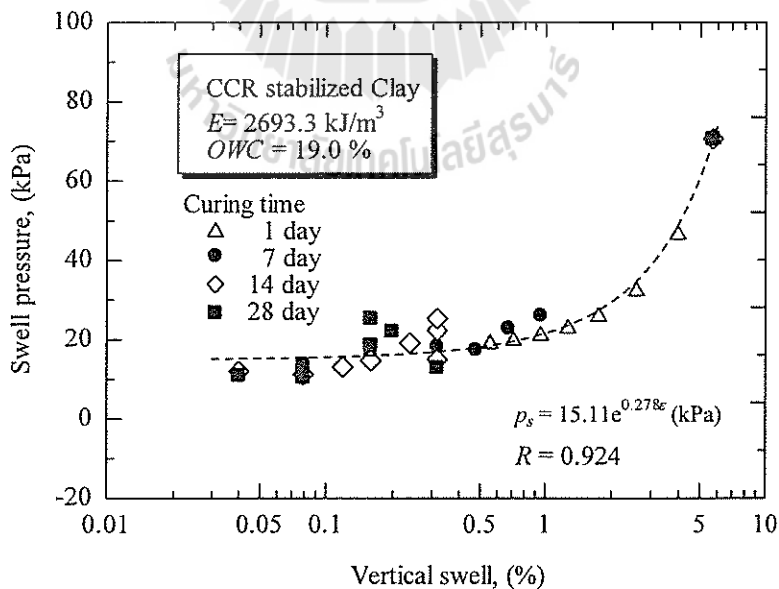


Figure 3.13 Relationship between swell pressure and vertical swell strain of the CCR stabilized clay.

Figures 3.14 and 3.15 show the collapse behavior of the CCR stabilized clay at different CCR contents after 28 days of curing. The initial void ratios were determined from the known volume, water content and specific gravity of the stabilized clay. The compression curves for the stabilized clay with the CCR contents greater than the fixation point (especially 20% and 30% CCR) possess the high void ratio due to the high swell pressure and exhibit the low resistance to compression (*vide* Figure 3.14). The collapse strain of the compacted sample is 8.4% and then classified as the moderately severe (*vide* Figure 3.15). The collapse behavior is significantly improved by the CCR. The collapse index significantly reduces as the CCR content increases up to 7% (CCR fixation point) and then increases. The optimal CCR content to improve the collapse behavior is thus the CCR fixation point. This content yields the collapse index of about 2, which is the upper boundary for the slight zone.

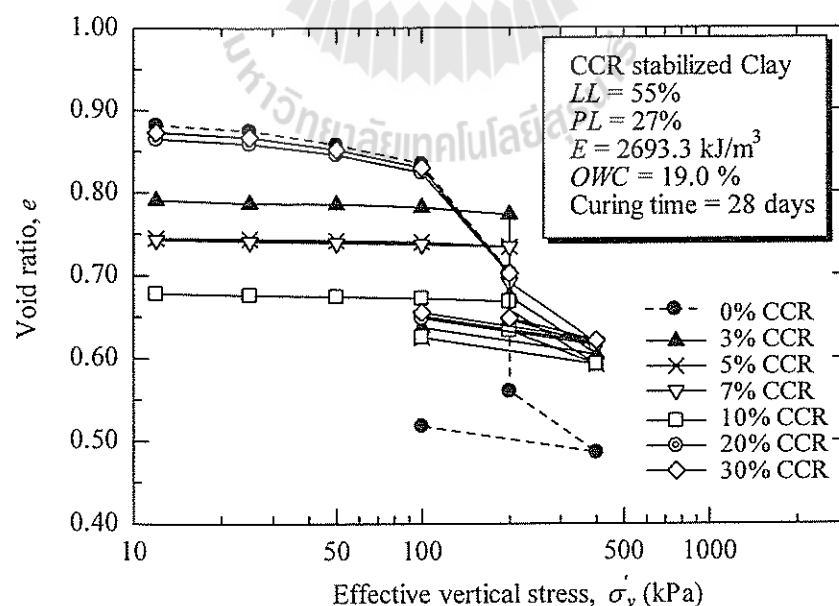


Figure 3.14 Collapse behavior of the CCR stabilized clay under K_0 -condition.

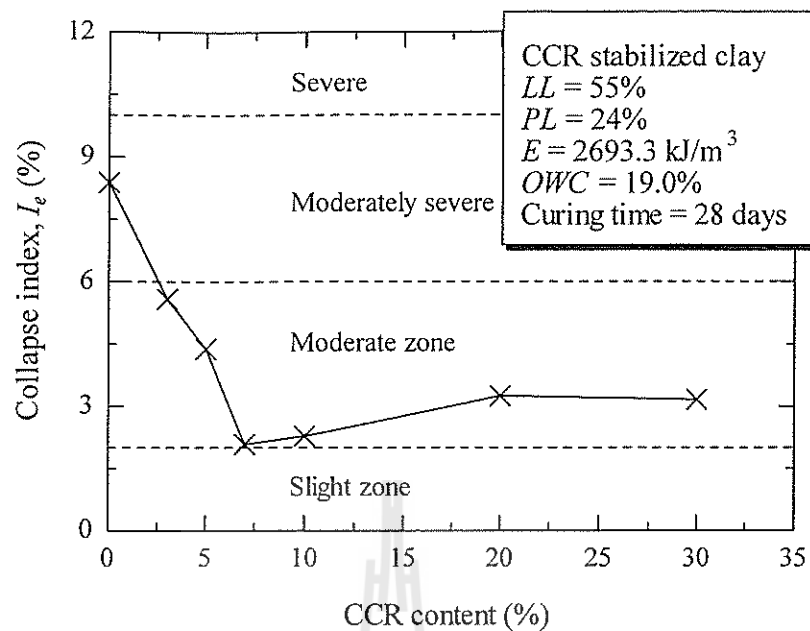


Figure 3.15 Collapse index and CCR content relationship of the CCR stabilized clay.

3.4 Analysis and discussion

The input of CCR causes the cation exchange and soil flocculation, resulting in the reduction in the soil plasticity index. The reduction in plasticity index is insignificant when the CCR content is greater than the CCR fixation point. Because the compaction curves and the compaction characteristics (OWC and $\gamma_{d,max}$) of the silty clay are essentially dependent upon the plasticity index (Gurtug and Sridharan, 2002; Horpibulsuk et al., 2008a; Horpibulsuk et al., 2009a; Nagaraj et al., 2006). The compaction curves and the compaction characteristics of the stabilized clay change with the input of CCR up to the CCR fixation point.

For a particular state of water content, the unsoaked strength and bearing capacity development are essentially dependent upon the CCR content. It is illustrated in this paper that the CCR fixation point is the optimal CCR content in terms of strength, bearing capacity and swell and collapse behaviors. To examine the validity of the CCR fixation point as an indicator for the other $\text{Ca}(\text{OH})_2$ stabilized clays, some available test results were taken and reanalyzed. The results were from Bell (1996) for lime stabilized montmorillonite and kaolin and from Lasedj and Al-Mukhtar (2008) for lime stabilized expansive clay. All the results (Figures 3.16 to 3.18) show that the fixation point is directly related to the maximum strength of the stabilized clay. The ASTM D 6276 suggests a method to determine the optimal lime content based on the pH test. The optimal lime content results in a soil-lime pH of 12.4. Beyond this content, the free lime remains in the soil-lime mixture. The determination of the optimal CCR content (CCR fixation point) by the index test is a simple alternative means. The CCR content far in excess of the CCR fixation point causes the strength reduction. This strength reduction is also found in concrete containing high free lime content. The increase in amount of the free lime with the CCR content when the CCR content is greater than the CCR fixation point is depicted by the thermal gravity (TG) analysis (Table 3.3).

Table 3.3 TG analysis of consumption Ca(OH)_2 of CCR stabilized clay.

CCR content	The Ca(OH)_2 content.		Consumed Ca(OH)_2
	7 days of curing	28 days of curing	
30%	9.20	8.26	0.94
20%	8.56	7.53	1.03
10%	7.92	6.80	1.12
9%	7.87	6.73	1.14
8%	7.79	6.70	1.09
7%	7.65	6.54	1.11
5%	7.48	6.79	0.69
3%	7.18	6.65	0.53
2%	6.21	6.12	0.09
1%	6.19	6.13	0.06

Remark:

The data for CCR contents of 1 to 10% were from Horpibulsuk et al., (2011a).

The results were from Horpibulsuk et al. (2012) and the present work. The testing method can be referred to Horpibulsuk et al. (Horpibulsuk et al., 2012b; Horpibulsuk et al., 2011a; Horpibulsuk et al., 2010b; Horpibulsuk et al., 2009b; Horpibulsuk et al., 2011b). The result shows that although the Ca(OH)_2 contents increase with the CCR content, the consumed Ca(OH)_2 between 7 and 28 days remains essentially constant in the inert and the deterioration zones. Even though it is not coincidentally proved that the CCR fixation point is the most suitable for all soil properties at a particular state of water content, this study illustrates that the unsoaked and soaked strength, collapse and swelling behavior, water absorption, volume change due to immersion and unsoaked and soaked bearing capacity, which are the essential

parameters for pavement design, are significantly dependent upon the CCR fixation point.

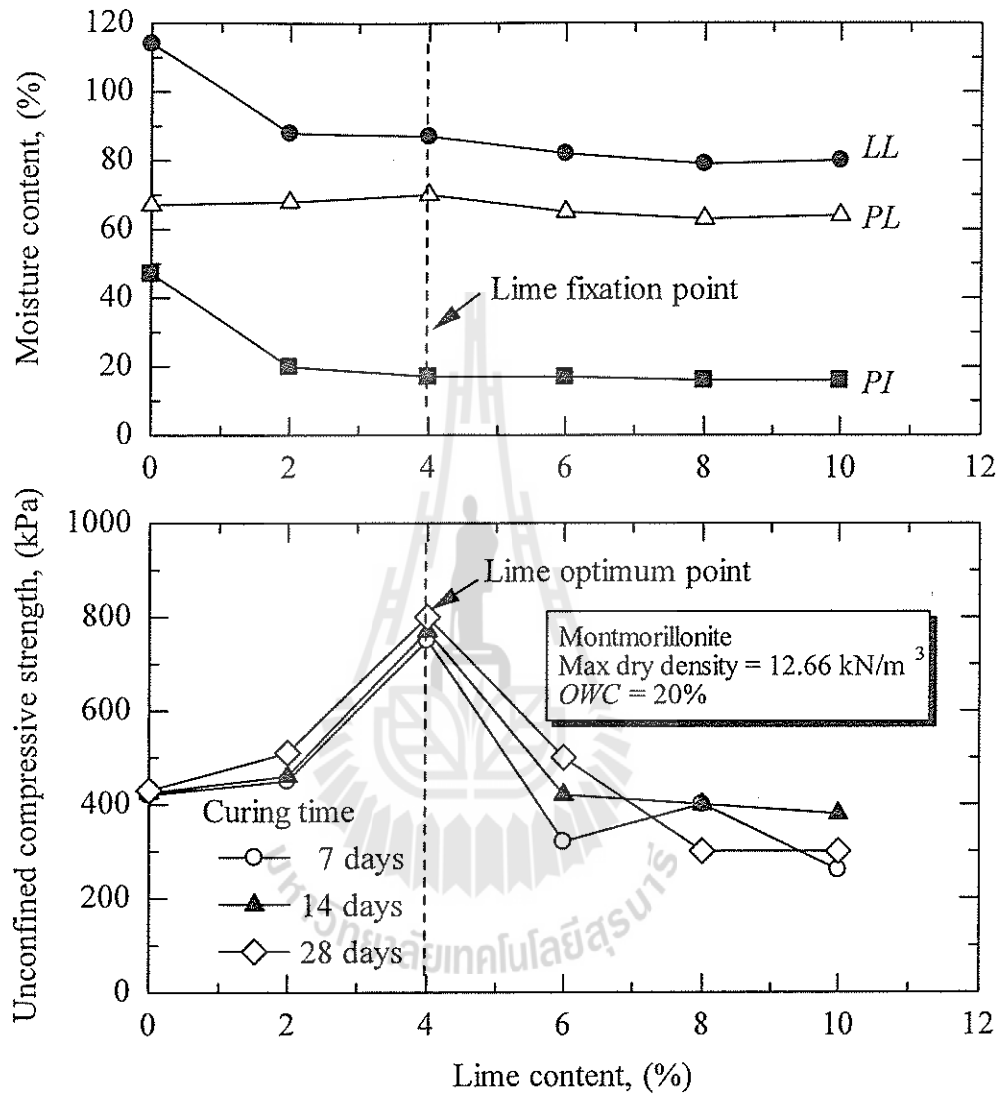


Figure 3.16 Effect of lime content on consistency limits and strength for stabilized montmorillonite (Bell, 1996)

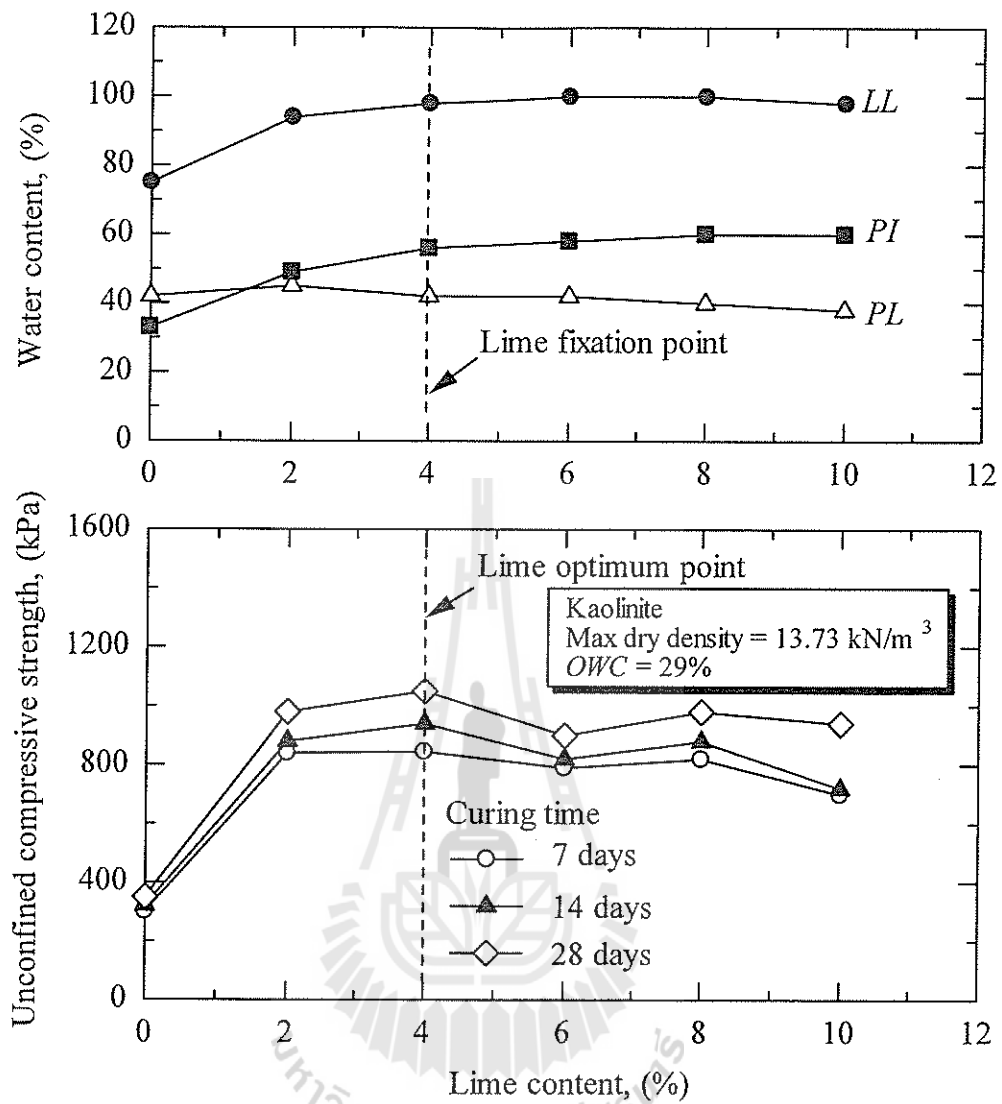


Figure 3.17 Effect of lime content on consistency limits and strength for stabilized kaolinite (Bell, 1996).

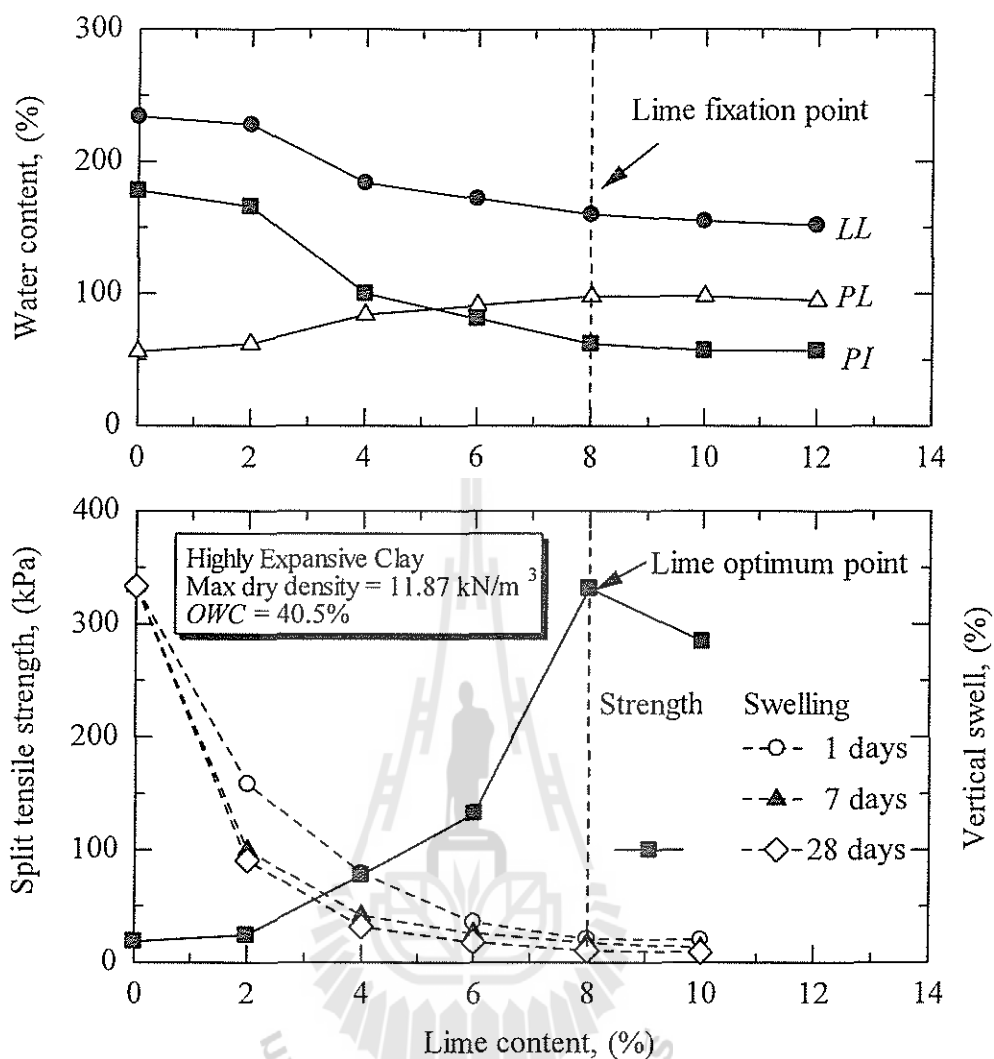


Figure 3.18 Effect of lime content on consistency limits and strength for stabilized expansive clay (Lasledj A. and AL-Mukhtar M., 2008)

The soaked engineering properties of the CCR stabilized clay is dependent upon the state of water content (which governs the densification, chemical reaction and water absorption). The water content at *OWC* provides the highest density and cementation bond strength as indicated by the highest unsoaked strength and bearing capacity. The lower water content is not enough for pozzolanic reaction while the higher water content causes the high water/stabilizer ratio, *W/S*. The higher the *W/S*,

the lower the strength (Horpibulsuk et al., 2006; Horpibulsuk et al., 2003; Horpibulsuk et al., 2011a; Horpibulsuk et al., 2011b; Horpibulsuk et al., 2011c). The increase in the swell pressure and the volume change is caused by the water absorption (Figures 3.19 and 3.20). The volume of the samples after mixing decreases (shrinks) with curing time due to the pozzolanic reaction (Figure 3.20). The mixing water content at 1.2OWC shows the highest shrinkage percentage while the mixing water content at 0.8OWC shows the lowest shrinkage percentage. Over the time of the immersion, the samples expand due to the increase in the water absorption. The optimum water content exhibits the lowest water absorption and volume change while the 0.8OWC (dry side) exhibits the highest water absorption and volume change. Consequently, the samples compacted on the dry side of optimum possess the lowest soaked strength and CBR (*vide* Figures 3.8 and 3.9). It is of interest that the water absorption capacity is insignificantly changed with the curing time. The immersion for 2 hours is considered as appropriate (insignificant change in the water absorption after 2 hours). This is in agreement with the specification by the American Association of State Highway and Transportation Officials (AASHTO).

In addition to the economic and environmental advantages, the soil stabilization by the CCR yields higher strength than that by the hydrated lime. This is possibly because the CCR contains more pozzolanic materials (SiO_2 , Al_2O_3 and Fe_2O_3) of about 12.3%. Figure 3.21 shows the comparison of the strength development in the CCR and the hydrated lime pastes. Both pastes were made up at the same condition (water content of 30% and dry unit weight of 10.1 kN/m^3). It is clearly shown that the strength of the hydrated lime paste is relatively low and insignificantly increases with curing time while the strength of the CCR paste

essentially increases with curing time. Since the CCR itself can produce the pozzolanic reaction with water, the soil improvement by the CCR is more effective than that by the hydrated lime.

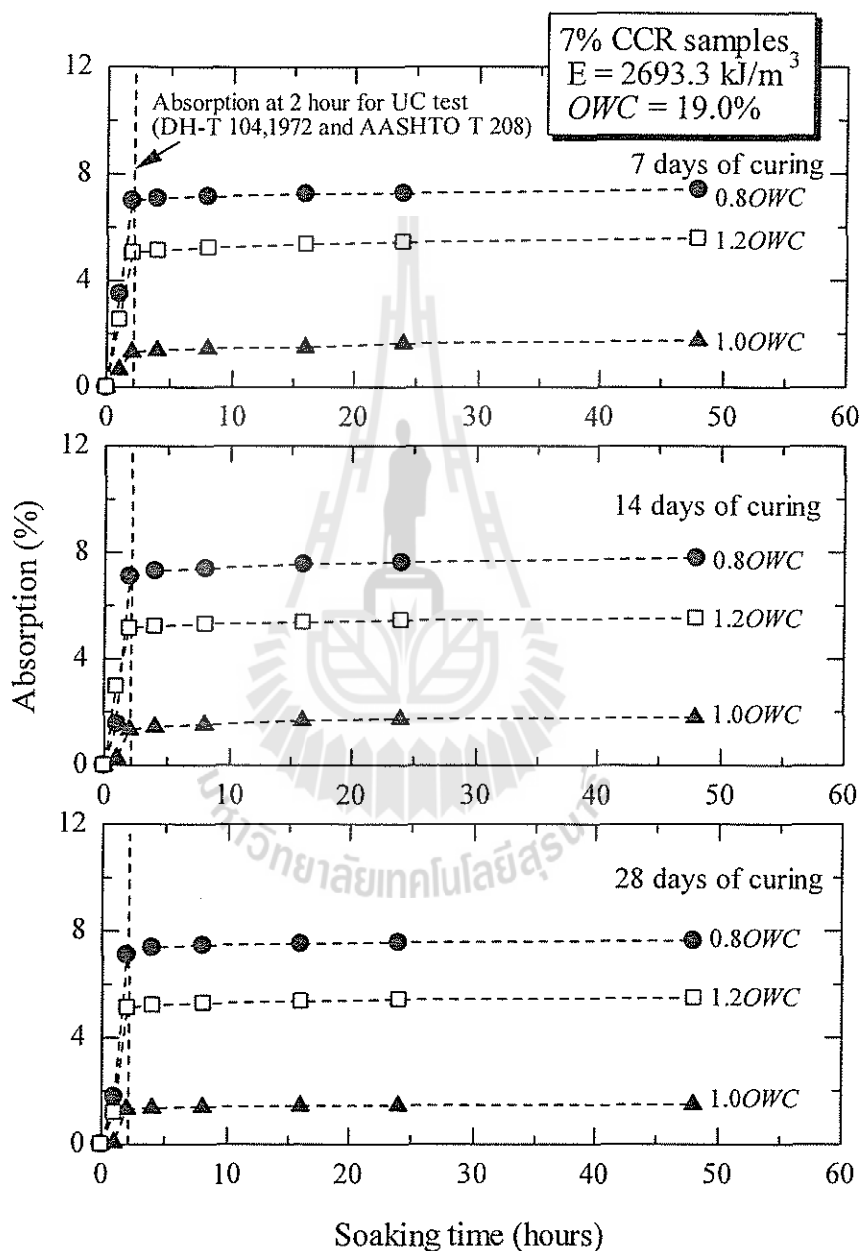


Figure 3.19 Absorption characteristics of the CCR stabilized clay.

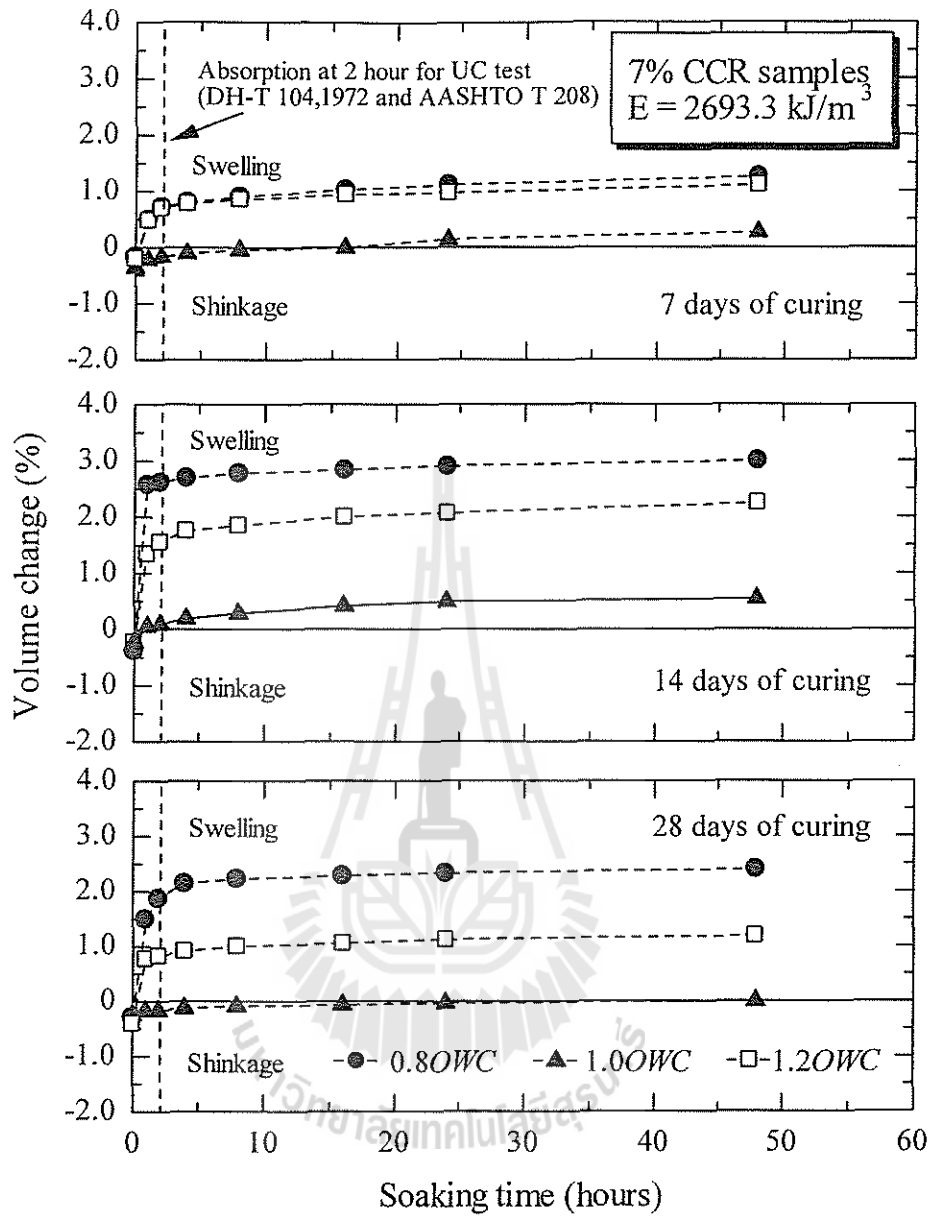


Figure 3.20 Volume change of the CCR stabilized clay under soaked condition.

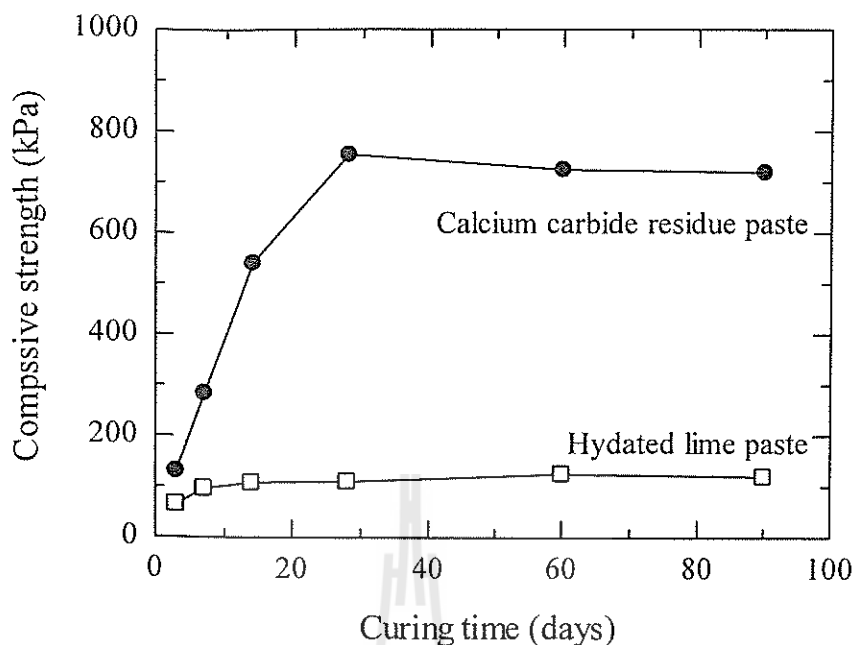


Figure 3.21 Compressive strength development of CCR paste and hydrated lime paste.

3.5 Conclusions

This paper presents the engineering properties of the problematic silty clay stabilized with the calcium carbide residue to ascertain its performance in pavement base and subbase applications. The calcium carbide residue can be regarded as an alternative cementing agent. The following conclusions can be drawn:

1. The chemical composition and the XRD-pattern of the CCR are similar to those of the hydrate lime. The CCR contains very high amount of Ca(OH)_2 (about 76.7%). Because the silty clay contains the large amount of natural pozzolanic materials (SiO_2 , Al_2O_3 and Fe_2O_3) of 60.5%, the Ca(OH)_2 from the CCR can react with natural pozzolanic materials, resulting in the improvement of the engineering properties of the CCR stabilized silty clay. This stabilized clay can be considered as a pavement geomaterial.

2. The soil improvement by the CCR can be classified into three zones: active, inert and deterioration. The CCR fixation point, which is simply obtained from the laboratory test can be used to identify the active zone. The CCR fixation point indicates the capacity of the clay to absorb Ca^{2+} ions and react with $\text{Ca}(\text{OH})_2$. The applicability of the fixation point is verified by the available test results of the clayey soils stabilized with the $\text{Ca}(\text{OH})_2$ rich material.
3. Because the compaction curves and compaction characteristics of the clayey soils are governed by the plasticity index, they are changed with the CCR content. The *OWC* of the stabilized clay increases as the CCR content and insignificantly alters when the CCR content is higher than the CCR fixation point where the plasticity index becomes essentially constant.
4. The CCR enhances the chemical bonding among the clay particles. The *OWC* is the suitable mixing state, providing the best engineering properties (highest strength and bearing capacity and lowest water absorption, swelling and collapse). The lower water content is not enough for pozzolanic reaction while the higher water content causes the high water/stabilizer ratio, W/S.
5. The water absorption induces the swell pressure. The samples compacted on the dry side of optimum shows higher water absorption potential than those on the wet side due to lower degree of saturation. Consequently, the samples compacted on the dry side exhibits poorest engineering properties.

6. The soil stabilization by the CCR yields higher strength than that by the hydrated lime. This is possibly because the CCR contains more pozzolanic materials (SiO_2 , Al_2O_3 and Fe_2O_3) of about 12.3%. Thus, this stabilization is useful in terms of engineering, environmental and economic viewpoints.

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CHAPTER IV

INFLUENCE OF WET-DRY CYCLES ON COMPRESSIVE STRENGTH OF CALCIUM CARBIDE RESIDUE-FLY ASH STABILIZED CLAY

4.1 Introduction

The chemical stabilization is one of the extensively used soil improvement techniques. The resistance to compression and consequent strength development increase with increasing curing time. Portland cement is commonly used for this stabilization. A high unit cost and energy intensive process of Portland cement are, however, the driving forces for the alternative cementitious additives. More importantly, the cement manufacturing process emits CO₂ into the atmosphere which accounts for 5% of the total CO₂ released into the air.

For clayey soils, which contain high natural pozzolanic materials, the Ca(OH)₂ rich materials can be used as alternative cementing agent to produce moderately high strength geomaterials. The cementing property is identified as a pozzolanic reaction. The engineering characteristics of Ca(OH)₂ rich materials stabilized clay are attributed to three basic reactions (Herrin and Mitchell, 1961), which are cation exchange, flocculation and aggregation, and pozzolanic reaction. The addition of Ca(OH)₂ rich material to clayey soil supplies an excess of Ca²⁺ and the cation exchange will occur with Ca²⁺ replacing dissimilar cations from the exchange

complex of the soil. This causes flocculation and aggregation of the clay fraction. The clay particles clump together into larger sized aggregates. The influence of cation exchange, flocculation and aggregation are primarily responsible for the change in plasticity and shrinkage (Horpibulsuk et al., 2012b). The shear strength of the stabilized soil gradually increases with time mainly due to the pozzolanic reaction. Calcium hydroxide can react with the silicates and aluminates (natural pozzolanic materials) in the clay (<0.002 mm particle) to form cementitious materials, consisting of calcium silicates and aluminates hydrates (Diamond and Kinter, 1965; Hunter, 1988).

Calcium Carbide Residue (CCR) is a by-product of acetylene production process, which contains mainly calcium hydroxide, Ca(OH)_2 . It was proved as a waste cementing agent for soil stabilization (Horpibulsuk et al., 2012a; Horpibulsuk et al., 2012b; Kumpala and Horpibulsuk, 2012a; Kumpala and Horpibulsuk, 2012b). Its production is described in the following equation:



From Eq.(4.1), it is seen that 64 g of calcium carbide (CaC_2) provides 26 g of acetylene gas (C_2H_2) and 74 g of CCR in terms of Ca(OH)_2 . Presently, the demand of CaC_2 for producing acetylene gas in Thailand is 18,500 tons/year. This provides 21,500 tons/year of calcium carbide residue and the demand is continuously increasing each year. Due to its high base, the CCR was hardly utilized in any work and all of it went to disposal area as slurry form. After being sun-dried for a few days, the slurry form is changed to dry form.

Horpibulsuk et al. (2012a, 2012b) Kumpala and Horpibulsuk (2012a) investigated the engineering properties of the CCR stabilized clay to ascertain the performance in the fill and pavement applications. They proved that the CCR stabilization is more effective than the lime stabilization in terms of engineering, economic and environmental viewpoints. The strength improvement for a particular curing time is classified into three zones: active, inert and deterioration (*vide* Figure 4.1). In the active zone, the strength increases remarkably with increasing the CCR content. All the input Ca(OH)_2 is consumed by the natural pozzolanic material in the soil for pozzolanic reaction. This active zone can be determined from the CCR fixation point, which is simply obtained from the index test. The CCR fixation is defined as the CCR content that the plasticity index of the CCR-clay mixture insignificantly changes with the input of CCR. The strength development in the inert zone slows down; the incremental gradient becomes nearly zero and does not make any further significant improvement. This is because the natural pozzolanic materials in the clay are not sufficient to react with excess Ca(OH)_2 from the CCR. The strength decrease appears when the CCR content is in the deterioration zone. The free lime causes the unsoundness and hence the strength reduction.

Because the Ca(OH)_2 is abundant in the inert and deterioration zones, the input of pozzolanic materials (CCR replacement) in these two zones further enhances the strength development (Horpibulsuk and Phetchuay, 2012; Horpibulsuk et al., 2012). Fly ash (FA) is one of the waste pozzolanic materials extracted from flue gases of a furnace fired with coal of an electric power plant. Its generation is far in excess of utilization. It can be used together with Portland cement for the construction of geotechnical and geo-environmental infrastructures (Cokca, 1997; Sharma, 1996;

Kawasaki et al., 1981 and Horpibulsuk et al., 2011a; Horpibulsuk et al., 2010; Horpibulsuk et al., 2009; Horpibulsuk et al., 2011b). Horpibulsuk et al. (2012a) revealed that the input of FA in the inert zone is the most effective in terms of strength development. The strength remarkably increases with FA content and the optimal FA content is about 22% of dry weight of soil.

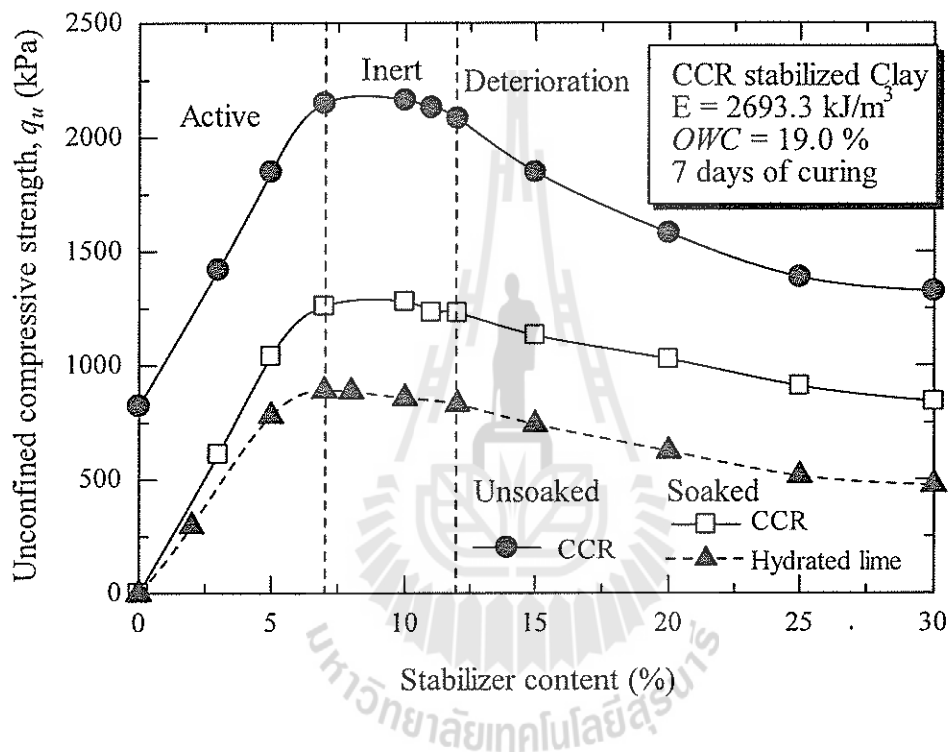


Figure 4.1 Improvement zones (Kumpala and Horpibulsuk, 2012a).

Even though there are available researches on the soil stabilized with the CCR and fly ash (FA), these studies do not consider the durability characteristic against the wetting – drying cycles (w-d cycles), which is a main aspect for pavement design. The w-d cycles cause tension crack and surface crack, which damage the stabilized pavement structure. The investigation of the life time of the stabilized materials via wetting and drying test is thus significant. This research aims to investigate the influence of w-d cycles on the compressive strength of the CCR-FA stabilized clay to

ascertain its performance in the pavement application. The wet-dry cycle strengths were compared with the recommended values for the cement stabilized pavement material by the American Concrete Institute, ACI and the U.S. Army Corps of Engineers (ACI, 1990; U.S. Army Corps of Engineers, 2004). The role of FA on the strength and durability development is analyzed and discussed. Based on the analysis of test results, an empirical relationship between w-d cycles strengths and unsoaked strength is proposed. This equation is useful for quick estimation of the unsoaked strength to attain the target strength at the design service life. Finally, a rational laboratory mix design method for CCR-FA stabilized clay is suggested.

4.2 Materials and methods

4.2.1 Soil Sample

The soil sample is a silty clay collected from the Suranaree University of Technology campus in Nakhon Ratchasima province, Thailand at a depth of 3 meters. This soil is an inland clay, which is sensitive to change of water content (Horpibulsuk et al., 2008). The grain size distribution (Figure 4.2) shows that the clay is composed of 2% sand, 43% silt and 55% clay. The specific gravity is 2.72. The liquid and plastic limits are approximately 55% and 27%, respectively. Based on the Unified Soil Classification System (USCS), the clay is classified as high plasticity (CH). During sampling, the groundwater had disappeared. The natural water content was 10 percent. The free swell test proposed by Prakash and Sridharan (2004) shows that the clay is classified as low swelling with a free swell ratio (FSR) of 1.6. The Cation Exchange Capacity, CEC is 27.6 meq/100g. Scanning electron microscope (SEM) image of the clay shown in Figure 4.3 indicates that its particles are irregular in shape.

The chemical composition of the clay is shown in Table 4.1. A x-ray diffraction (XRD) pattern of the clay (Figure 4.4) shows that main chemical composition is SiO_2 . The sum of SiO_2 , Al_2O_3 and Fe_2O_3 is 60.54%, which is considered as high for pozzolanic reaction.

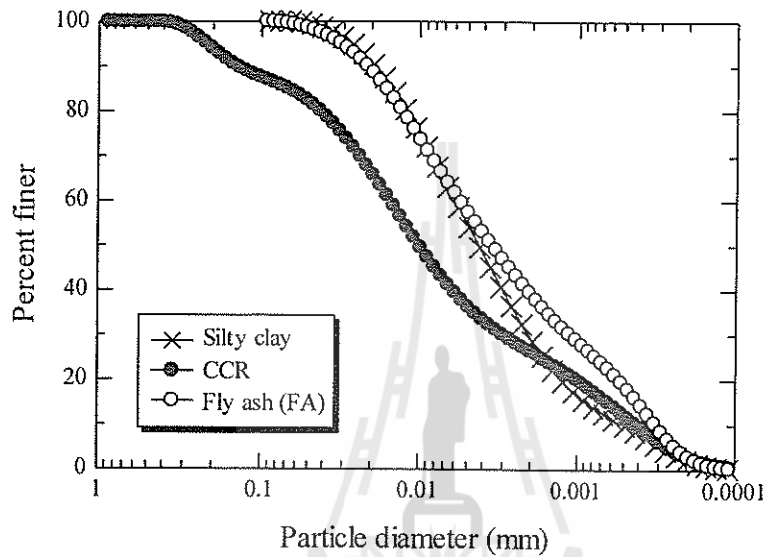


Figure 4.2 Grain size distribution of the clay, the CCR and the FA.

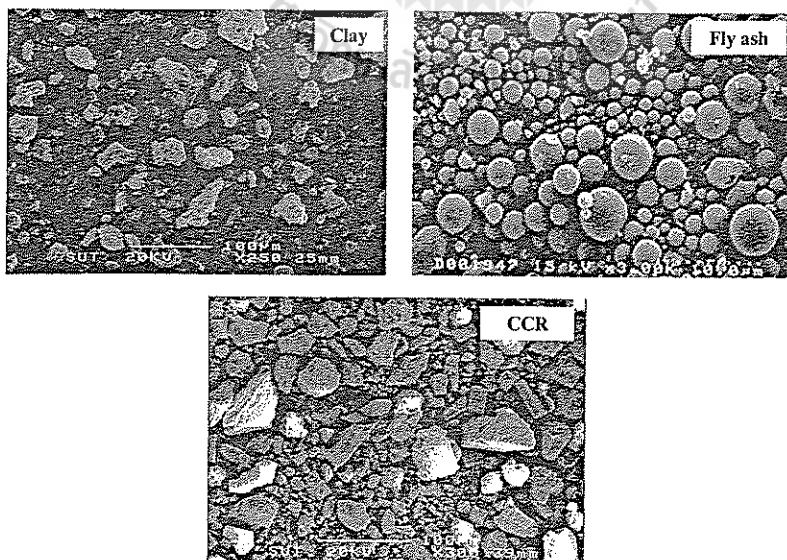
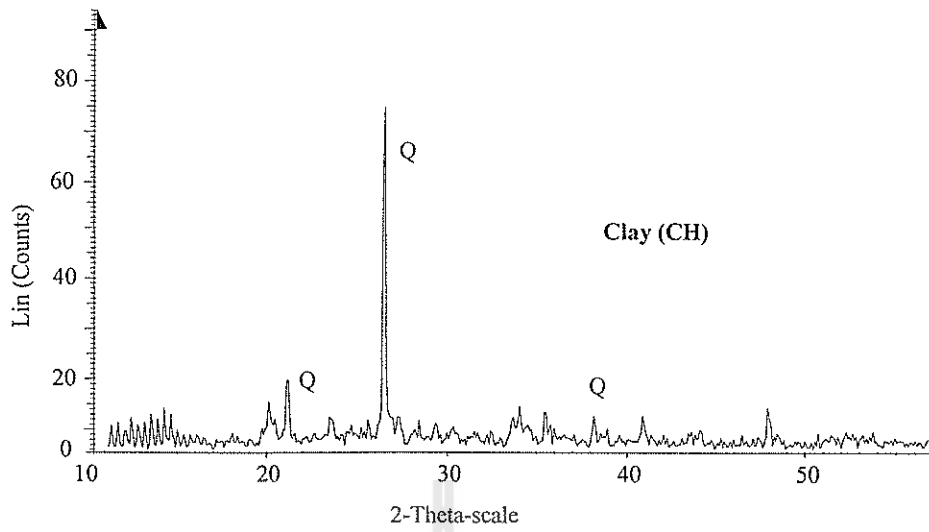
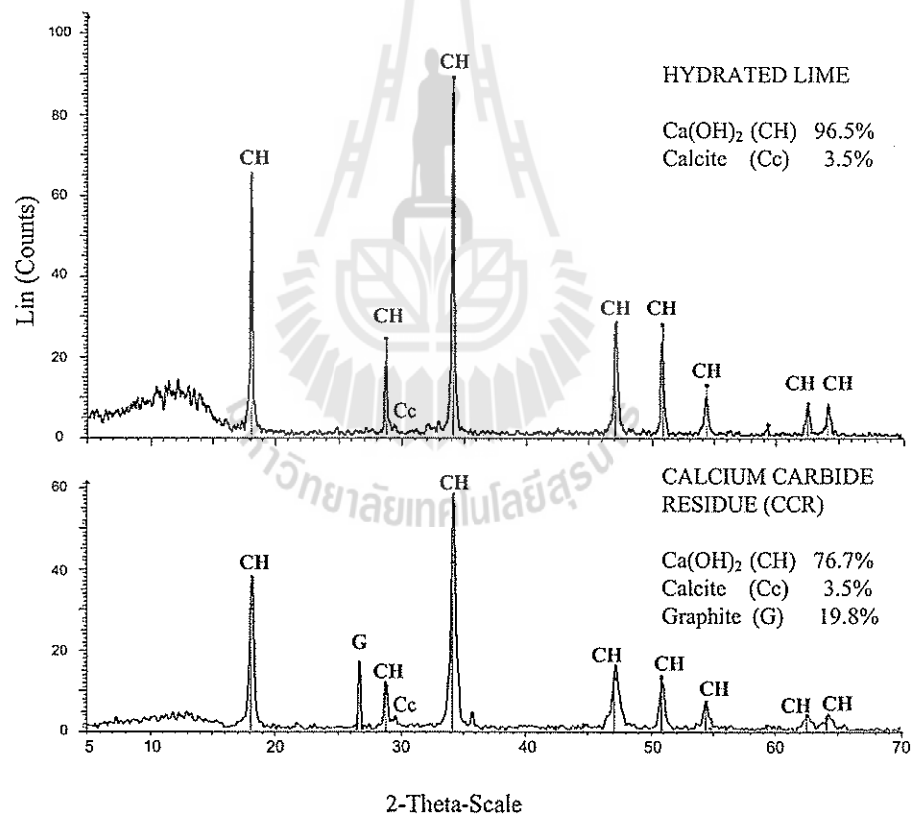


Figure 4.3 SEM photos of the clay, the fly ash and the CCR.



(a) Clay



(b) CCR and Hydrated lime

Figure 4.4 XRD pattern of (a) the clay and (b) the hydrated lime and the CCR and (c) the fly ash.

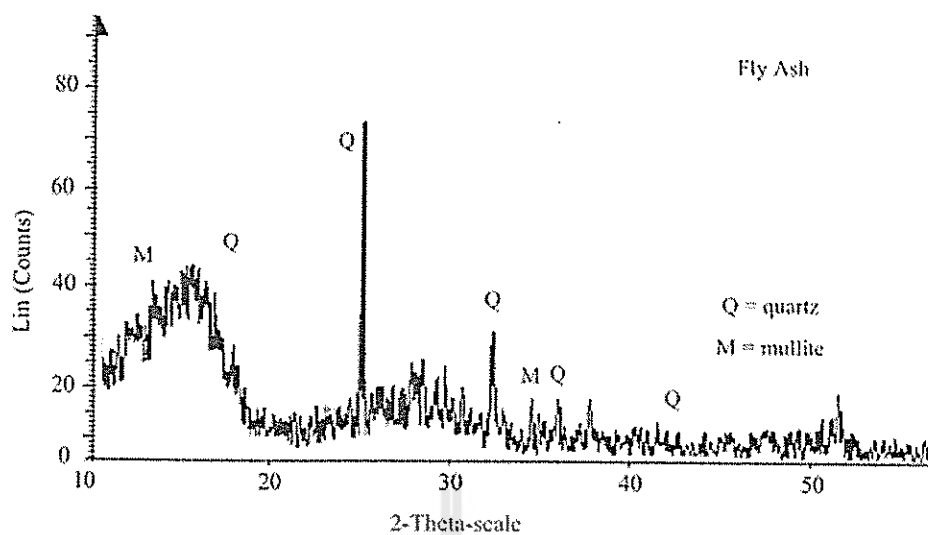


Figure 4.4 XRD pattern of (a) the clay and (b) the hydrated lime and the CCR and (c) the fly ash.

Table 4.1 Chemical properties of the clay, the CCR, the hydrated lime, and the fly ash

Chemical composition	Clay (%)	CCR (%)	Hydrated lime (%)	Fly ash (%)
SiO ₂	20.10	6.49	1.289	49.04
Al ₂ O ₃	7.55	2.55	0.238	37.91
Fe ₂ O ₃	32.89	3.25	0.489	2.75
CaO	26.15	70.78	90.129	1.03
MgO	0.47	0.70	0.223	0.39
SO ₃	4.92	0.66	0.866	0.18
Na ₂ O	ND	ND	N.D.	0.38
K ₂ O	3.17	7.94	3.303	0.52
LOI	3.44	1.35	1.21	4.70

4.2.2 Binder

Calcium Carbide Residue (CCR) from the Sai 5 Gas Product Co., Ltd and the fly ash (FA) from the Mae Moh power plant in the north of Thailand were used in this study. The CCR was oven-dried at 100°C for 24 hours and ground in a Los Angeles abrasion machine. Both the CCR and FA were passed through sieve No. 40 (425 μm). The specific gravity values are 2.32 and 2.39, respectively. Table 4.1 shows the chemical composition of both FA and CCR compared with that of a hydrated lime. Total amount of the major components SiO_2 , Al_2O_3 and Fe_2O_3 in FA are 81.48%. It is thus classified as class F fly ash in accordance with ASTM C 618. The chemical composition (Table 4.1) shows the CaO contents of 90.13% and 70.78% for the hydrated lime and the CCR, respectively. This result is in agreement with the XRD pattern (*vide* Figure 4.4). The XRD pattern of the CCR is similar to that of the hydrated lime, showing the $\text{Ca}(\text{OH})_2$ as a main composition. The $\text{Ca}(\text{OH})_2$ contents are about 96.5% and 76.7% for the hydrated lime and the CCR, respectively. The high $\text{Ca}(\text{OH})_2$ and CaO contents of CCR indicate that it can react with pozzolanic material and produce a cementitious material. The grain size distribution of the FA and the CCR compared with that of the silty clay is shown in Figure 4.2. The curves were obtained from the laser particle size analysis. The D_{50} of the FA and the CCR are 0.0035 and 0.01 mm, respectively. Scanning electron microscopic (SEM) images of the FA and the CCR are also shown in Figure 4.3. From the grain size distribution and the SEM images, it is found that the silty clay and FA particles are much smaller than the CCR particles. The CCR is irregular in shape while the FA is spherical.

4.2.3 Methodology

The clay was passed through a 16 mm sieve to remove coarser particles. It was air-dried and then the water content was adjusted for laboratory compaction test. All the index and mechanical tests on the silty clay and stabilized clay were carried out according to the American Standard for Testing and Materials (ASTM). The compaction of the unstabilized clay, the CCR stabilized clay and the FA-CCR stabilized clay was carried out under the modified Proctor energy (ASTM D 1557). The optimum water content, OWC , and the maximum dry unit weight, $\gamma_{d,max}$ of the compacted clay were 19% and 17.1 kN/m³, respectively. The CCR contents used for the stabilization were 5, 7 and 12%, which are the representatives of the active and inert zones, respectively (*vide* Figure 4.1). The FA contents were from 0 to 40% by weight of dry soil.

After 24 hours of compaction, the stabilized samples were dismantled from the mold, wrapped in vinyl bags and stored in a humidity chamber of constant temperature ($25\pm 2^\circ\text{C}$). The water absorption and the unconfined compressive strengths were measured on the CCR-FA stabilized samples, which were compacted on both dry and wet sides of the optimum water content. Following is a summary of the method of cyclic wetting and drying test (ASTM D 559). The samples were submerged in tap water at room temperature for 5 hours. They were then dried in the oven at a temperature of 70°C for 48 hours and air-dried at room temperature for at least 3 hours. This process is referred to as 1 w-d cycle. After attaining the target w-d cycles, the standard cylindrical samples were immersed in deionized water for 2 hours in a room at the constant temperature of $25\pm 2^\circ\text{C}$. Unconfined compression (UC) tests

were then run with a rate of vertical displacement of 1 mm/min. The 1, 3 and 6 w-d cycles were considered in this study.

The testing program is summarized in Table 4.2. For all tests at each curing time and combination of water content, CCR content and FA content, at least five samples were tested under the same condition to check for consistency of the test. In most cases, the results under the same testing condition were reproducible with a low mean standard deviation, SD ($SD/\bar{x} < 10\%$, where \bar{x} is the mean strength value).

The growth of cementitious products in the CCR-FA stabilized samples was illustrated by the scanning electron microscope (SEM). The SEM samples were frozen at -195°C by immersion in liquid nitrogen for 5 minutes and evacuated at a pressure of 0.5 Pa at -40°C for 5 days (Horpibulsuk et al., 2010; Horpibulsuk and Raksachon, 2010; Miura et al., 1999). All samples were coated with gold before SEM (JOEL JSM-6400) analysis.

Table 4.2 Summary of the testing program.

Test	Binder	Water content (%)	Curing time (days)	CCR/FA content (%)
UC Test (influence of w-d cycles)	CCR	$OWC = 19$	7, 14, 28	5, 7, 12
Compaction	CCR and FA	-	-	CCR = 10, 20 FA = 0, 6, 12, 18, 24
Water absorption	CCR and FA	$0.80OWC$, OWC , $1.20OWC$,	7	CCR fixation point = 7 FA = 10, 15, 20, 30,40
UC Test (strength development with FA)	CCR and FA	$0.80OWC$, OWC , $1.20OWC$,	7	CCR fixation point = 7 FA = 0, 3, 5, 7, 10, 15, 20, 25,30
UC Test (strength development with FA under w-d cycles)	CCR and FA	$OWC = 19$	7, 14, 28	CCR at active zone = 5 CCR at Inert zone = 10, 12 FA = 0, 3, 5, 7, 10, 15, 20, 25, 30

4.3 Test results

Figure 4.5 shows compaction curves of the 10% and 20% CCR stabilized samples with various FA contents. The maximum dry unit weight decreases with the CCR content because the specific gravity of the CCR is much lower than the clay.

The input of FA increases the compactability of the stabilized clay because the FA particles are smaller than the CCR particles (*vide* Figure 4.2 and 4.3). The smaller sized and round shape enables them to fill up the pore space between the clay and CCR particles. Consequently, the maximum dry unit weight of the CCR stabilized clay increases with the input FA. The decrease in the optimum water content is associated with the increase in the maximum dry unit weight.

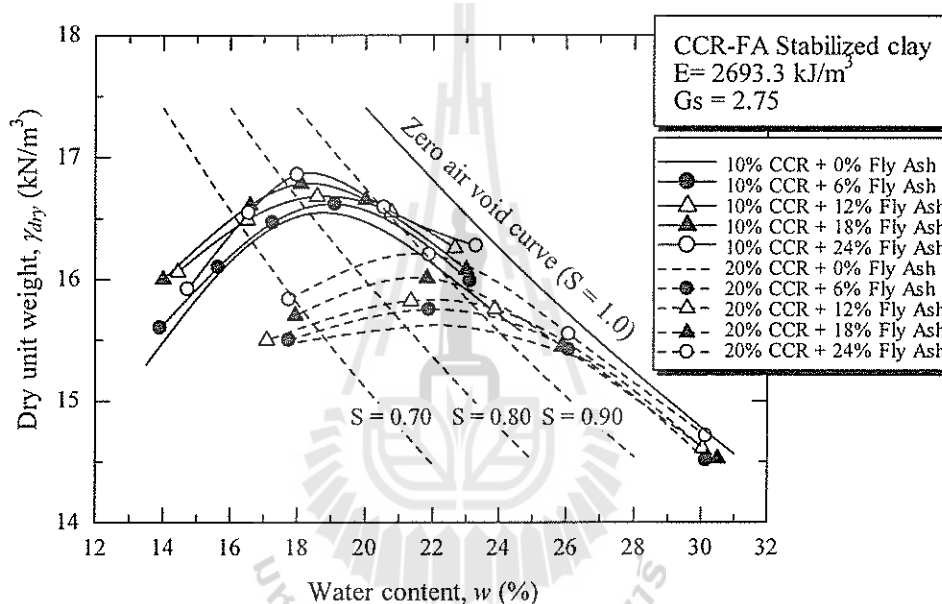


Figure 4.5 Influence of fly ash on the compaction curves of CCR stabilized clay.

Figure 4.6 shows the influence of w - d cycles on the unconfined compressive strengths of the 5%, 7%, and 12% CCR samples cured for 7, 14 and 28 days. For all the CCR contents, the compressive strength of the CCR stabilized samples without w - d cycle increases with increasing CCR content and curing time. The strength of 7% and 12% CCR samples are essentially the same because the CCR contents are in the inert zone where the natural pozzolanic reaction in the clay is not sufficient to react with $\text{Ca}(\text{OH})_2$ in the CCR. Compared with the recommended strengths by the ACI

and the U.S. Army Corps of Engineers, the CCR stabilized clay without w-d cycles passes the recommendation. Both institutes recommend the 7-day and 28-day strengths not lower than 1723 and 2068 kPa, respectively. However, the strengths reduce significantly with number of cycles. All samples cured for 7 days cannot pass the recommendation after being subjected to only 1st cycle. The same is found for the 5% CCR sample cured for 28 days. The 7% and 12% CCR samples cure for 28 days can resist up to the 2nd w-d cycles. In other words, the service life of this material is short and its durability against w-d cycles must be improved. The 7-day criterion is more critical than the 28-day one because the rate of strength development with time of this material is relatively high and close to that of the cement stabilized clay (Horpibulsuk et al., 2012a).

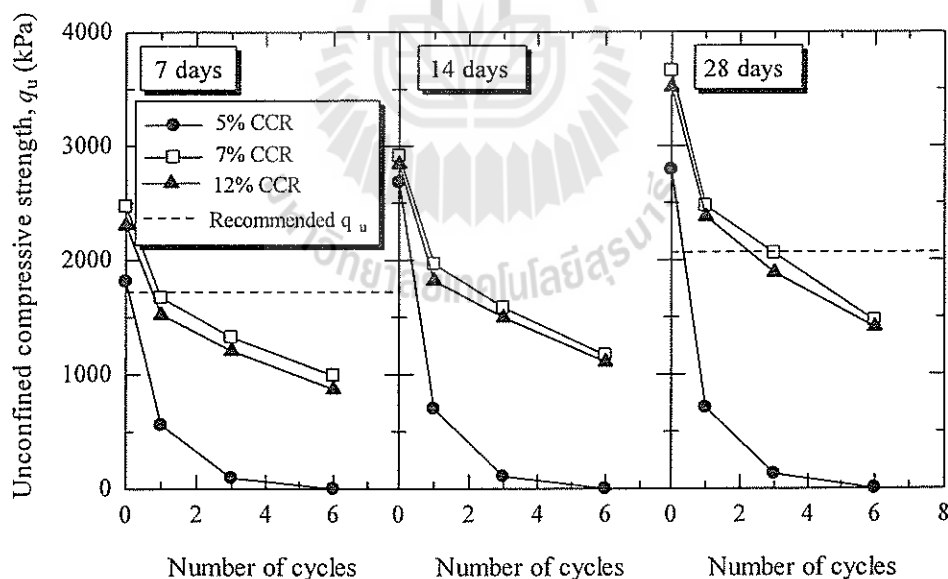


Figure 4.6 Influence of wetting-drying cycles on the compressive strength of CCR stabilized clay.

The previous researches (Horpibulsuk et al., 2012b; Kumpala and Horpibulsuk, 2012a) showed that the highest strength for a particular clay stabilized with CCR is found at CCR fixation point (7% for this tested soil) (*vide* Figure 4.1). A way to further enhance the strength of the CCR stabilized clay is to input the pozzolanic materials such as fly ash and biomass ash in the inert zone (Horpibulsuk et al., 2012a). Figure 4.7 shows the influence of the input FA on the strength development of the 7% CCR samples. The samples were compacted on the dry side (0.8*OWC*), at the optimum water content (*OWC*), and on the wet side (1.2*OWC*) at 7 days of curing. For all water contents tested, the compressive strength increases with increasing FA up to a certain FA content. The water content at *OWC* provides the highest strengths for all FA contents and followed by 0.8*OWC* and 1.2*OWC*, respectively. The maximum compressive strength of all samples was at about 20% FA and beyond this FA content, the compressive strength shows a declining trend. This result is in agreement with that reported by Horpibulsuk et al. (2012b).

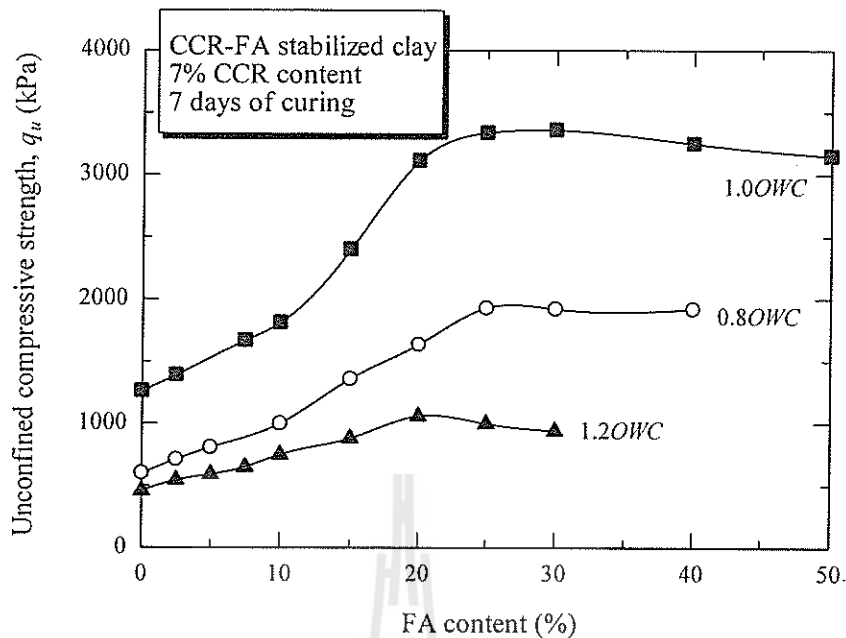


Figure 4.7 Influence of fly ash on the development on CCR stabilized clay.

It is now to examine the influence of input FA on the w-d cycle strengths. Since the *OWC* provides the highest strength, this water content was selected for the examination. Figures 4.8 and 4.9 show the strength development in the CCR-FA stabilized samples compacted at *OWC* under 1, 3, and 6 w-d cycles for various FA contents. The advantage of the input FA is clearly seen; the w-d cycle strengths increase remarkably with the increase in FA content for all curing times tested. Compared with the unsoaked strength, the highest strengths for different w-d cycles are at about 20% FA and the strength development is the similar pattern. This result indicates that the w-d cycle strength is dependent upon the unsoaked (initial) strength. It is also noted that for the same input of FA, the 12% CCR samples show slightly higher strength than the 7% CCR samples. In other words, the CCR fixation is the most appropriate content for strength and durability improvement by FA.

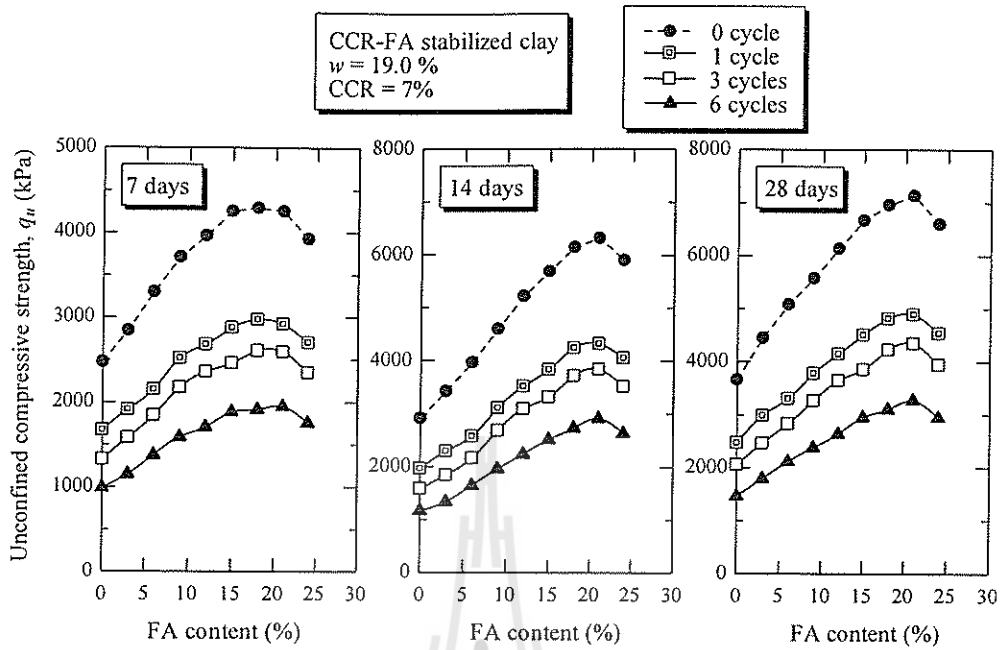


Figure 4.8 Influence of fly ash on the compressive strength of 7% CCR stabilized clay under wetting-drying cycles.

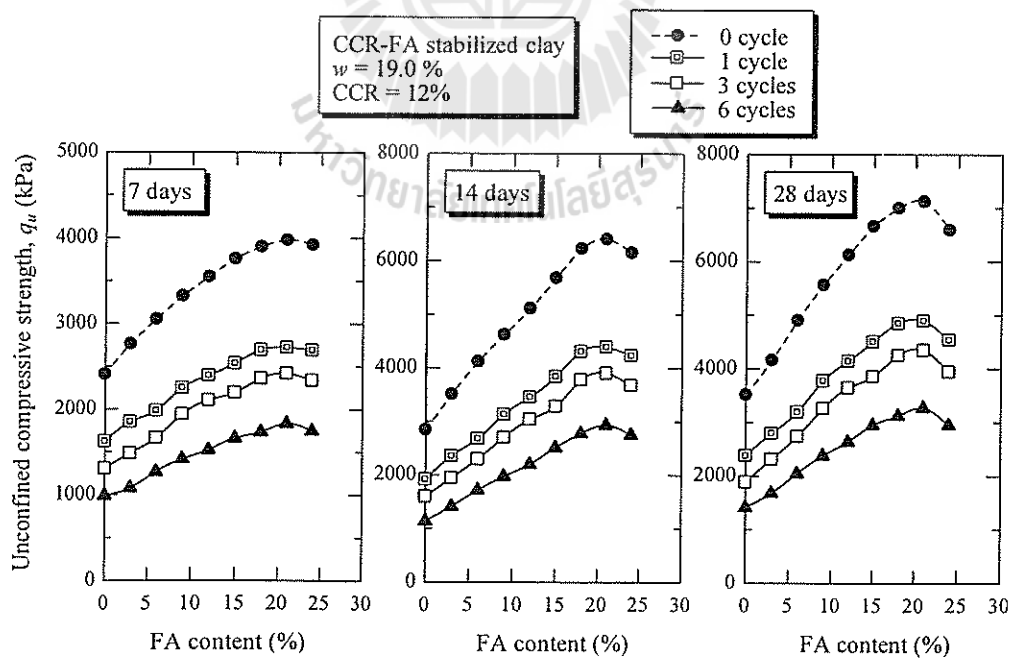
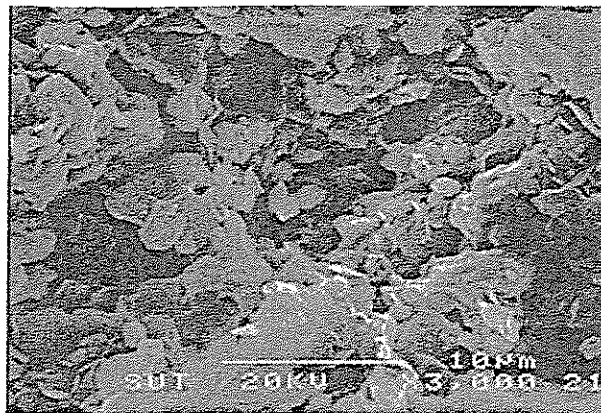


Figure 4.9 Influence of fly ash on the compressive strength of 12% CCR stabilized clay under wetting-drying cycles.

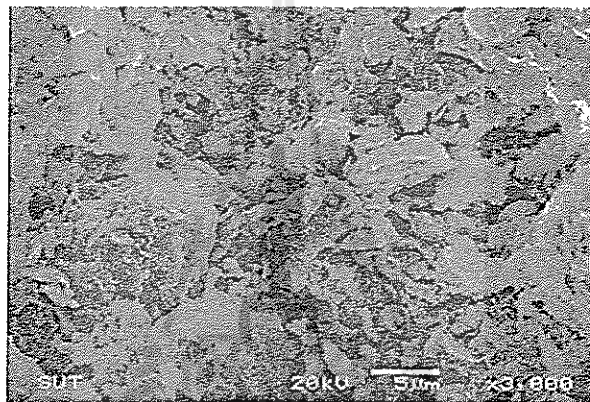
4.4 Analysis and discussion

The unsoaked strength of the CCR-FA stabilized clay is controlled by the densification and chemical reaction. The input FA increases the maximum dry unit weight (densification) of the stabilized clay and CCR particles (*vide* Figure 4.5). The chemical (pozzolanic) reaction develops with curing time as illustrated by the strength development with time. Calcium hydroxide reacts with the silicates and aluminates in the clay and the FA particles to form cementitious materials (Horpibulsuk et al., 2012a; Horpibulsuk et al., 2012b; Kumpala and Horpibulsuk, 2012a; Kumpala and Horpibulsuk, 2012b) consisting of calcium silicates and aluminates hydrates, similar to the reactions of hydrated lime stabilized clay (Diamond and Kinter, 1965; Herrin and Mitchell, 1961; Hunter, 1988). For a particular curing time and mix proportion, attractive forces (cementation bond) depend on the state of water content (*vide* Figure 4.7). The water content at OWC provides the highest cementation bond strength as indicated by the highest unsoaked strength. The lower water content is not enough for pozzolanic reaction while the higher water content causes the high water/stabilizer ratio, W/S. The higher the W/S, the lower the strength (Horpibulsuk et al., 2006; Horpibulsuk et al., 2003; Horpibulsuk et al., 2011b; Horpibulsuk et al., 2011c).

The effect of flocculation and pozzolanic reaction is illustrated by the SEM images. Figure 4.10 shows the SEM images of the compacted clay, the 7% CCR sample and the 7% CCR with 20% FA samples. Both stabilized samples were cured for 7 days. Figure 4.10a shows the morphology of the clay, which is composed of several thin sheets. The CCR stabilized sample develops more aggregates than the clay sample (compared Figure 4.10a to 4.10b).



(a) Compacted sample



(b) 7% CCR sample



(c) 7% CCR with 20% FA

Figure 4.10 SEM photos of compacted clay, CCR stabilized clay and CCR-fly ash stabilized clay.

The aggregation is caused by the pozzolanic products bonding the clay particles. Figure 4.10c shows the morphology of the CCR-FA stabilized sample that the fly ash particles are clearly shown among clay-cement clusters. It is also seen that the morphology of the CCR-FA stabilized sample is denser than that of the CCR stabilized sample (compared Figure 4.10b to 4.10c). The presence of cementing products caused by the pozzolanic reaction between CCR and FA particles is clearly seen in the pores with the well-knitted framework among the soil particles. The cementitious products connect fly ash, clay and CCR particles together. These cementitious products impart the strength and the resistance to deformation to the stabilized clay. The optimal FA content for strength development is found at about 21%. The excess FA content causes the strength reduction. Under this condition, the excess fly ashes surround the CCR grains and hinder the interaction between water and CCR grains (Horpibulsuk et al., 2012a; Horpibulsuk et al., 2011b)

The water absorption induces the repulsive forces among clay particles (Kumpala and Horpibulsuk, 2012a; Kumpala and Horpibulsuk, 2012b) and causes the expansion of the diffuse double layer (Herrin and Mitchell, 1961). The drying process makes the CCR stabilized samples shrinkage due to the reduction of the pore volumes and causes tension and surface cracks on the samples (Tang et al., 2011). The repulsive forces and the tension and surface cracks upon the wetting and drying cycles reduce the strength of the CCR-FA stabilized clay.

The input FA enhances not only the strength development but also the durability against w-d cycles. It is logical to relate the durability to the strength because they are mainly governed by the pozzolanic reaction. Figure 4.11 shows the relationship between the w-d cycle strength, $q_{u(w-d)}$ and the unsoaked strength,

$q_{u(\text{unsoaked})}$ for the 7% and 12% CCR samples with different FA contents. It is of interest to note that the strength at any w-d cycle is directly related to the unsoaked strength, irrespective of the CCR and FA contents. Consequently, it is possible to develop a relationship between generalized strength development and w-d cycle, c (vide Figure 4.12) in the following form:

$$\frac{q_{u(w-d)}}{q_u} = 0.73 - 0.05c \quad \text{for } 1 \leq c \leq 6 \quad (4.2)$$

This relationship is useful as a practical tool for quick approximation of the w-d cycle strength of the stabilized clay to ascertain the serviceability of the pavement material. The development of this relationship is on sound principle. More test data of different stabilized soils (coarse and fine-grained) are required for the future researches to develop a generalized relationship. Based on the equation, the required $q_{u(\text{unsoaked})}$ values are 4006 and 4809 kPa for 7 and 28 days of curing to pass the recommendations by the ACI and US Army Corps of Engineers at the sixth w-d cycle.

Besides the strength improvement against w-d cycles, the input FA improves the resistance to water absorption. Figure 4.13 shows the relationship between the water absorption and the soaking time of 7% CCR samples compacted at the optimum water content (OWC), on the dry side ($0.8OWC$) and the wet side ($1.2OWC$) with different FA contents after 7 days of curing. For all the water contents tested, upon adding the FA, the water absorption decreases. The immersion for 2 hours is considered as appropriate (insignificant change in the water absorption after 2 hours). This is in agreement with the specification by the American Association of State

Highway and Transportation Officials (AASHTO). The optimal FA content to resist the water absorption is about 20%, which also provides the highest strength. The water absorption of CCR-FA stabilized clay is lower than the CCR stabilized clay (without fly ash) in all samples especially at 0.8OWC and 1.2OWC.

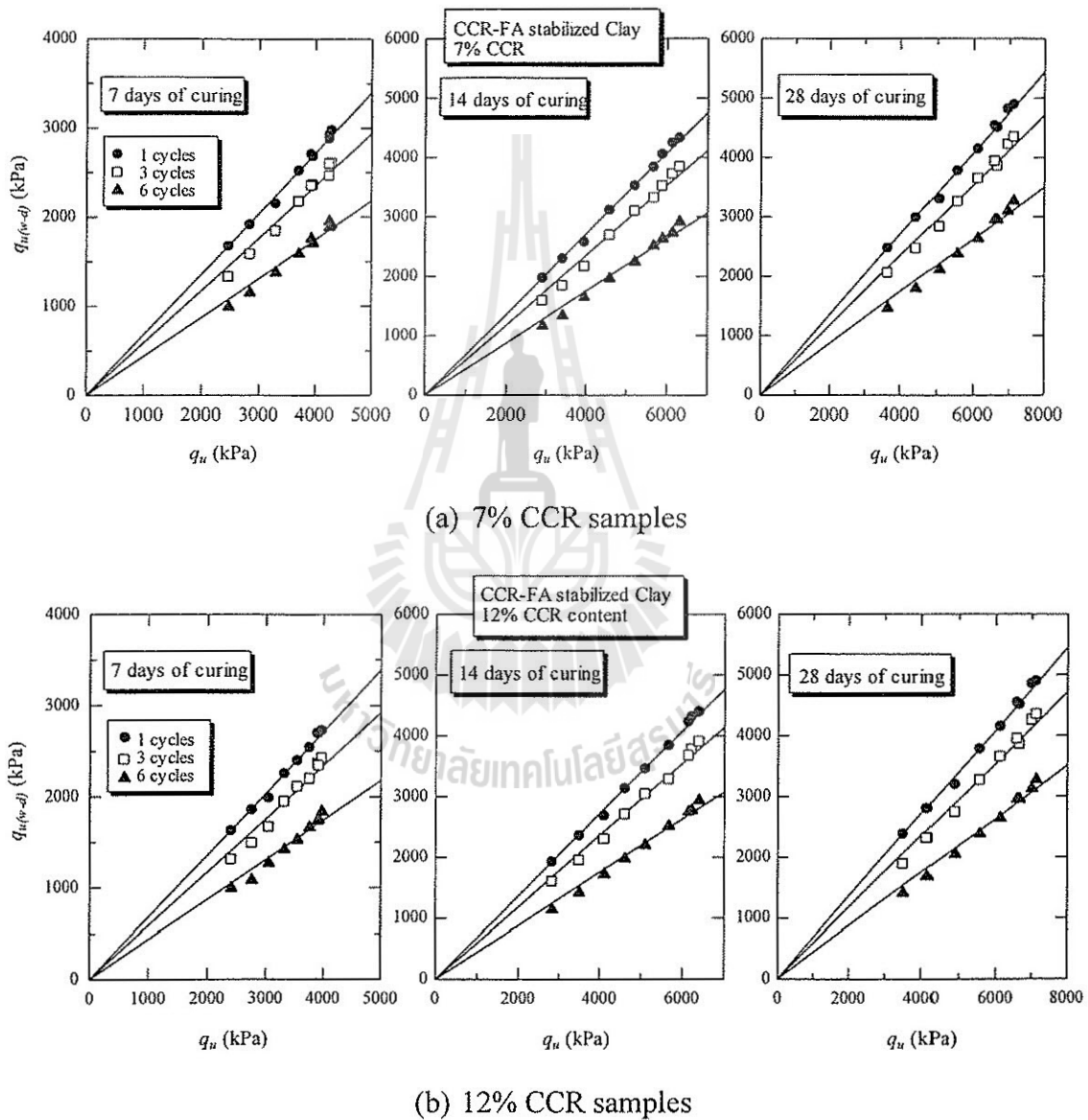


Figure 4.11 Relationship between w-d cycle strengths and unsoaked strengths.

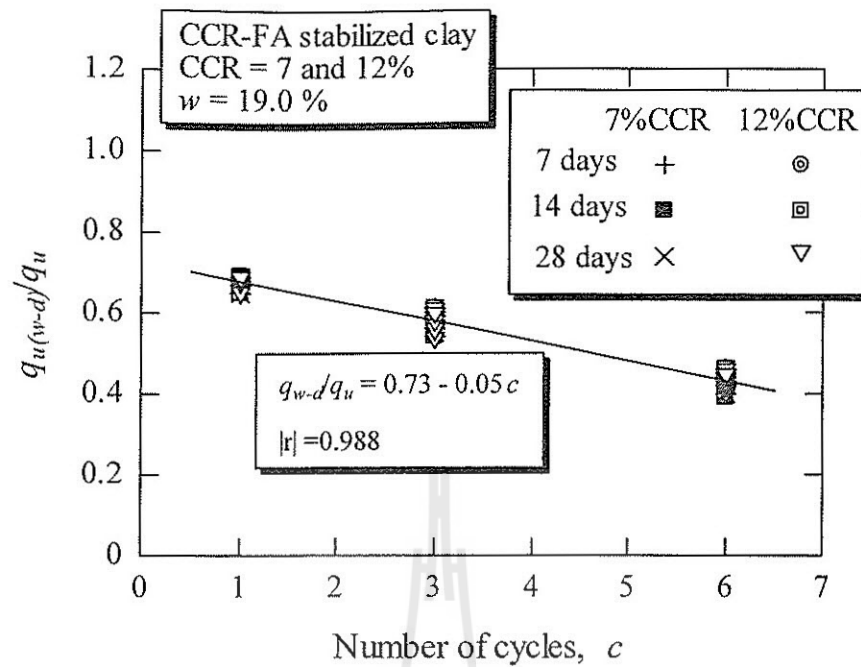


Figure 4.12 Relationship between the normalized strength and number of cycles.

4.5 Suggested Laboratory Mix Design Properties

Based on the laboratory test results, a mix design procedure to arrive at the target strength of the stabilized material subjected to required number of wetting-drying cycles is suggested and presented by the following steps:

1. Determine the CCR fixation point of the stabilized clay by the index test.
More detail on the test can be referred to Horpibulsuk et al. (2012b).
2. Perform the strength tests on the CCR-FA stabilized clay at the CCR fixation point with varied FA contents and cured for 7 days. Then, develop the relationship between unsoaked strength and FA content.
3. Approximate the 7-day unsoaked strength required to resist the target number of wetting and drying cycles using Eq. (4.2). The recommended

values by the ACI and the US Army Corps of Engineers can be used as reference.

4. Determine the FA content to attain the required unsoaked strength from the relationship developed in the second step.
5. Perform the cyclic wetting-drying tests on the CCR-FA stabilized sample to confirm the designed ingredient at the required wetting-drying cycles.

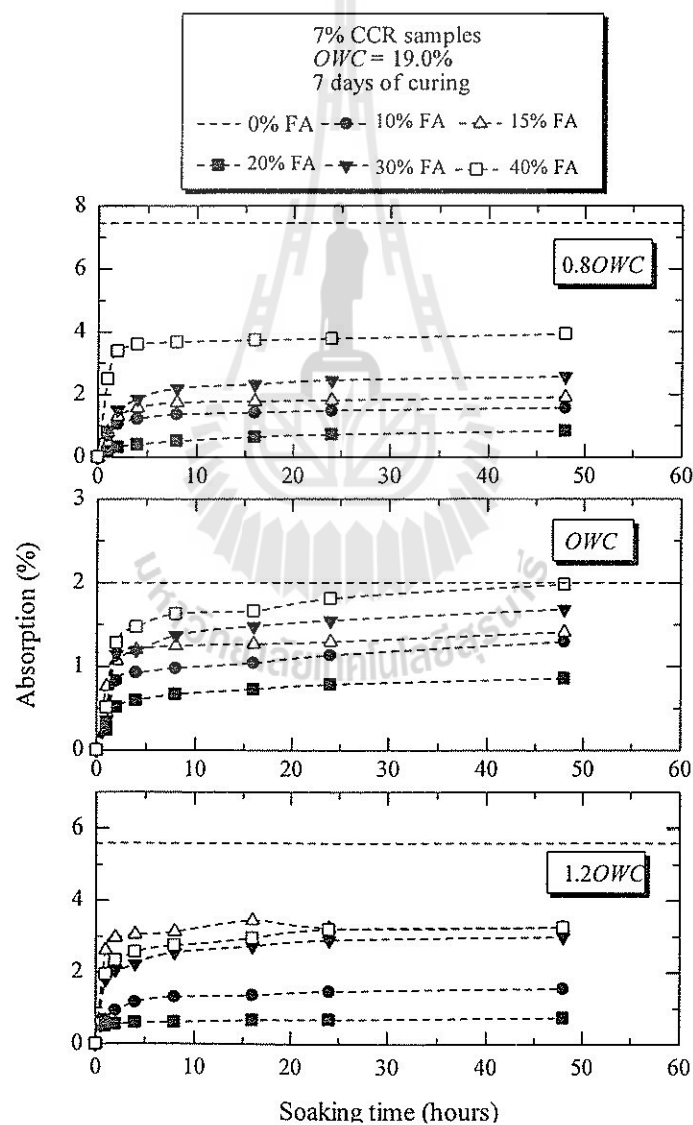


Figure 4.13 Influence of fly ash on the absorption of CCR stabilized clay.

4.6 Conclusions

This article investigates the influence of wetting-drying cycles on the durability of CCR-FA stabilized clay in terms of strength and water absorption to ascertain its serviceability. The following conclusions can be drawn as follows:

1. Even though the CCR stabilization improves significantly the strength, the durability against wetting and drying of the stabilized clay is considered as low according to the recommendations by the ACI (1990) and U.S. Army Corps of Engineers (2004). The wetting induces the repulsive forces among the clay particles and the drying causes the tension and surface cracks on the stabilized material. Consequently, the strength of the CCR stabilized clay reduces significantly with the number of w-d cycles.
2. The input FA improves both the densification and the pozzolanic reaction of the stabilized clay; hence the improvement in strength and durability. The improvement of the densification and the pozzolanic reaction is clearly shown by the SEM images. The optimal CCR and FA contents are found at about 7% and 20% respectively. The excess fly ashes surround the CCR grains and hinder the interaction between water and CCR grains; hence the strength reduction.
3. The suitable molding water content is found at the optimum water content, which provides the highest densification and pozzolanic products. The lower water content is not enough for pozzolanic reaction while the higher water content causes the high water/stabilizer ratio, W/S. The higher the W/S, the lower the strength.

4. The strength test results show that the durability against wetting and drying cycles for all curing times tested is strongly dependent upon the unsoaked strength (prior to the wetting and drying test). It is logical to develop a relationship between the normalized strength and number of cycles. This relationship is useful for a quick determination of strengths under different number of wetting and drying cycles to ascertain the service life of the stabilized material.
5. Besides the strength improvement against w-d cycle, the input FA improves significantly the resistance to water absorption. The optimal FA content to resist the water absorption is at about 20% FA, which also provides the highest strength. The water absorption of CCR-FA stabilized clay is lower than CCR stabilized clay (without fly ash) in all samples especially at 0.8*OWC* and 1.2*OWC*.
6. From the critical analysis, the laboratory mix design procedure to ascertain the serviceability of the stabilized material is suggested. It is useful in terms of engineering and economic viewpoints.

4.7 References

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CHAPTER V

BASIC AND ENGINEERING PROPERTIES OF RECYCLED CEMENTED CLAY

5.1 Introduction

Reuse of waste materials for pavement applications is of great international interest. The urgent need for reuse of waste materials is driven mainly by environmental considerations, due to the increased scarcity of natural resources and the increasing cost of land fill in most countries (Arulrajah et al., 2011a). The previous researches included recycled crushed brick (Arulrajah et al., 2011b; Debieb and Kenai, 2008; Poon and Chan, 2006), recycled crushed glass (Disfani et al., 2011; Grubb et al., 2006; Wartman et al., 2004), recycled concrete aggregate (Courard et al., 2010; Khalaf and DeVenney, 2005; Tam and Tam, 2007), other types of construction, demolition and excavation wastes (Khalaf and DeVenney, 2005; Arulrajah et al., 2011a; Bennert et al., 2000) and waste biosolids (Arulrajah et al., 2011c). Besides, the improvement of engineering properties of waste materials by the cementing agent is an alternative means (Hoyos et al., 2011; Puppala et al., 2011; Taha et al., 2002).

Calcium Carbide Residue (CCR), a by-product of acetylene production process, was proved as one of the waste cementing agents for soil stabilization (Consoli et al., 2001; Horpibulsuk et al., 2012a; Horpibulsuk et al., 2012b). Its production is described in the following equation:



From Eq. (5.1), it is seen that 64 g of calcium carbide (CaC_2) provides 26 g of acetylene gas (C_2H_2) and 74 g of CCR in terms of $\text{Ca}(\text{OH})_2$. Presently, the demand of CaC_2 for producing acetylene gas in Thailand is 18,500 tons/year. This provides 21,500 tons/year of calcium carbide residue and the demand is continuously increasing each year.

Kumpala and Horpibulsuk (2012a, 2012b, and 2012c) investigated the engineering properties of the CCR stabilized clay to ascertain the performance in the fill and pavement applications. They proved that the CCR stabilization is more effective than the lime stabilization in terms of engineering, economic and environmental viewpoints. The strength improvement for a particular curing time is classified into three zones: active, inert and deterioration. In the active zone, the strength increases remarkably with increasing the CCR content. All the input $\text{Ca}(\text{OH})_2$ in the CCR is consumed by the natural pozzolanic materials in the soil for pozzolanic reaction. This active zone can be determined from the CCR fixation point, which is simply obtained from the index test. The CCR fixation point is defined as the CCR content that the plasticity index of the CCR-clay mixture insignificantly changes with the input of CCR. The strength development in the inert zone slows down; the incremental gradient becomes nearly zero and does not make any further significant improvement. This is because the natural pozzolanic materials in the clay are not sufficient to react with excess $\text{Ca}(\text{OH})_2$ from the CCR. The strength reduction is observed for very high CCR contents in the deterioration zone. The excess free lime

causes the unsoundness of the clay-CCR structure. In practice, the CCR fixation point is the maximum CCR content for soil stabilization.

With a large number of cyclic loads by heavy traffic, the large permanent deformation and radial tensile strain cause the pavement damage. Even though there are available researches on the engineering properties of the CCR stabilized soils to ascertain their performance on pavement applications, very few researches have focused on the possibility of reusing this damaged material (over their service life) in pavement application such as pavement recycling technique. This research attempts to investigate basic and engineering properties of the recycled CCR-FA stabilized clay to ascertain its possibility of reuse as backfill and subbase materials. The basic properties involved are specific gravity, grain size distribution, index properties, linear shrinkage and free swell. The engineering properties are compaction behavior, compressibility and strength development. The scanning electron microscope (SEM) was used to illustrate the presence of pozzolanic products attached on the recycled particles. The knowledge obtained from this research can be applied to the other recycled clayey soils stabilized with different cementitious materials produced from $\text{Ca}(\text{OH})_2$ rich materials to explain and analyze the strength development.

5.2 Materials and methods

5.2.1 Materials

A silty clay, collected from the Suranaree University of Technology campus in Nakhon Ratchasima province, Thailand, was mixed with the CCR at different contents. Then, the CCR stabilized samples were crushed and remolded at water content of about 1.5 times liquid limit. The grain size distribution (Figure 5.1)

shows that the clay is composed of 2% sand, 43% silt and 55% clay. The specific gravity is 2.72. The liquid and plastic limits are approximately 55% and 27%, respectively. Based on the Unified Soil Classification System (USCS), the clay is classified as high plasticity (CH). The free swell test proposed by Prakash and Sridharan (2004) shows that the clay is classified as low swelling with a free swell ratio (FSR) of 1.6. The Cation Exchange Capacity, CEC is 27.6 meq/100g. Scanning electron microscope (SEM) image of the clay shown in Figure 5.2 indicates that its particles are irregular in shape. The chemical composition of the clay is shown in Table 5.1. A x-ray diffraction (XRD) pattern of the clay (Horpibulsuk et al., 2012b) shows that main chemical composition is SiO_2 . The sum of SiO_2 , Al_2O_3 and Fe_2O_3 is 60.54%, which is considered as high for pozzolanic reaction.

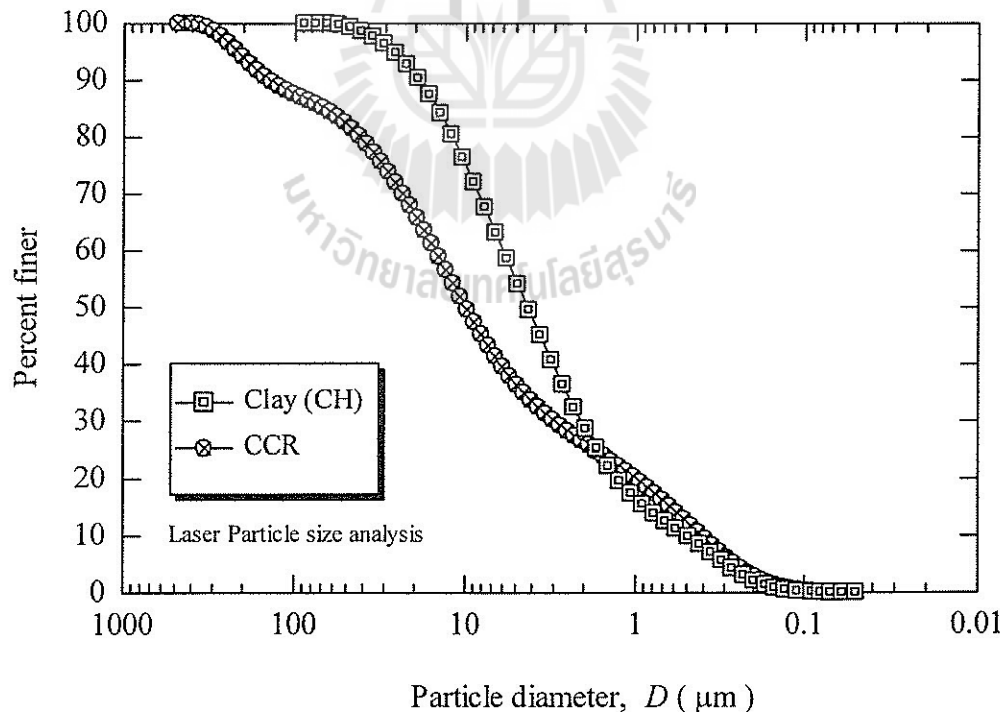


Figure 5.1 Grain size distribution of the clay and the CCR.

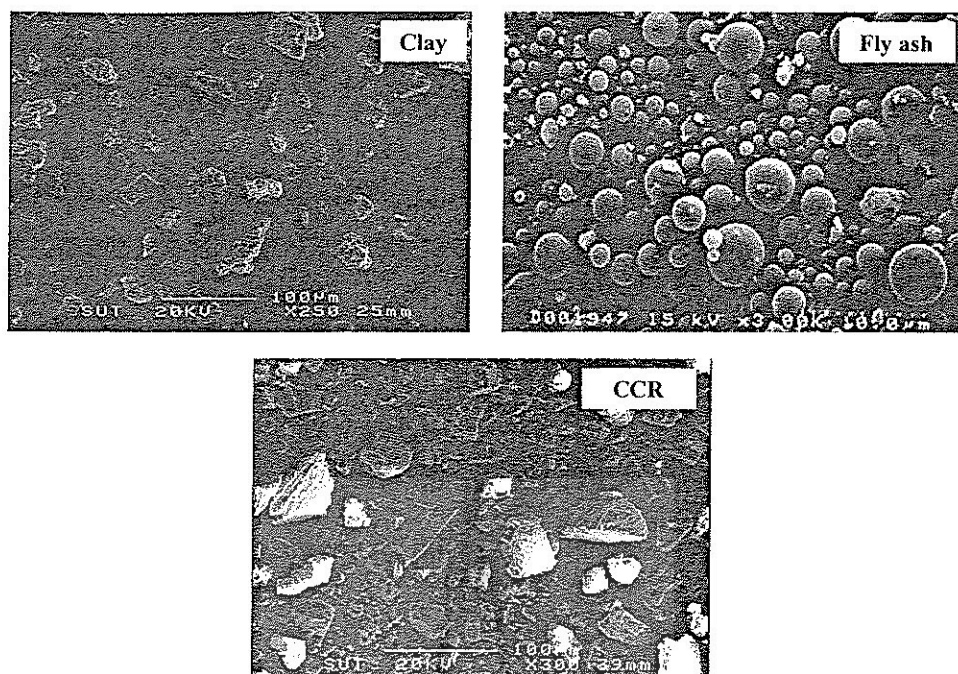


Figure 5.2 SEM photos of the clay, the fly ash and the CCR

Calcium Carbide Residue (CCR) from the Sai 5 Gas Product Co., Ltd was used in this study. The CCR was oven-dried at 100°C for 24 hours and ground in a Los Angeles abrasion machine. The CCR was passed through sieve No. 40 (425 μm). The specific gravity is 2.32. Table 5.1 shows the chemical composition of CCR compared with that of a hydrated lime. The chemical composition (Table 5.1) shows the CaO contents of 90.13% and 70.78% for the hydrated lime and the CCR, respectively. The grain size distribution of the CCR compared with that of the silty clay is shown in Figure 5.1. The D_{50} of the CCR is 0.01 mm. Scanning electron microscopic (SEM) image of the CCR is also shown in Figure 5.2. From the grain size distribution and the SEM images, it is found that the silty clay particles are much smaller than the CCR particles.

5.2.2 Methodology

After 28 days of curing, the CCR stabilized clay was crushed to small particles by a hand hammer and then remolded by a mixer. The CCR contents were varied from 0 to 30% to represent the three improvement zones: active, inert and deterioration. It was proved that the CCR content about 7% is suitable for the stabilization of the tested silty clay (Horpibulsuk et al., 2012b; Kumpala and Horpibulsuk, 2012a). The remolded paste was passed through a 2.75-mm sieve to remove coarser particles and oven-dried at 60°C. All the basic and mechanical tests on the recycled material were carried out according to the American Standard for Testing and Materials (ASTM) and compared with the unstabilized and stabilized materials. To evaluate the basic properties of the recycled material, a series of tests, such as specific gravity, grain size distribution, consistency index, linear shrinkage, and free swelling were performed. The details of testing can be referred to Kumpala and Horpibulsuk (2012a). The compaction of the recycled material was carried out under the modified Proctor energy (ASTM D 1557). The recycled samples at optimum water content, OWC , and the maximum dry unit weight, $\gamma_{d,max}$ were cured for 7 days prior to unconfined compression tests under soaked condition.

The testing program is summarized in Table 5.2. For all tests at each curing time and combination of water content and CCR content, at least five samples were tested under the same condition to check for consistency of the test. In most cases, the results under the same testing condition were reproducible with a low mean standard deviation, SD ($SD/\bar{x} < 10\%$, where \bar{x} is the mean strength value).

The presence of cementitious products attached on the recycled particles was observed by the scanning electron microscope (SEM). The SEM samples were

frozen at -195°C by immersion in liquid nitrogen for 5 minutes and evacuated at a pressure of 0.5 Pa at -40°C for 5 days (Horpibulsuk and Raksachon, 2010; Miura et al., 1999). All samples were coated with gold before SEM (JOEL JSM-6400) analysis.

Table 5.1 Chemical properties of the clay, the CCR, the hydrated lime, and the fly ash

Chemical composition	Clay (%)	CCR (%)	Hydrated lime (%)	Fly ash (%)
SiO ₂	20.10	6.49	1.289	49.04
Al ₂ O ₃	7.55	2.55	0.238	37.91
Fe ₂ O ₃	32.89	3.25	0.489	2.75
CaO	26.15	70.78	90.129	1.03
MgO	0.47	0.70	0.223	0.39
SO ₃	4.92	0.66	0.866	0.18
Na ₂ O	ND	ND	N.D.	0.38
K ₂ O	3.17	7.94	3.303	0.52
LOI	3.44	1.35	1.21	4.70

Table 5.2 Summary of the testing program.

Test	Sample	Stabilized	Curing time (days)	Admixed ratio (%)
Specific gravity	Remolded CCR stabilized clay	CCR	28	0, 3, 7, 10, 20, 30
Particle size	Remolded CCR stabilized clay	CCR	28	0, 3, 7, 10, 20, 30
Index properties	Remolded CCR stabilized clay	CCR	28	0, 3, 7, 10, 20, 30
Linear shrinkage	Remolded CCR stabilized clay	CCR	28	0, 3, 7, 10, 20, 30
Free swell	Remolded CCR stabilized clay	CCR	28	0, 3, 7, 10, 20, 30
Compaction	Remolded CCR stabilized clay	CCR	28	0, 3, 7, 10, 20, 30
Unconfined compression (soaked condition)	Remolded CCR	<i>OWC</i> = 19%	7	-

5.3 Test results

Figure 5.3 shows the unsoaked and soaked strength development of CCR stabilized clay with input 5%, 7%, and 10% CCR (before recycling). Initially as the curing time increases, the strength increases rapidly until 28 days of curing. Beyond

this curing time, the strength development slows down. In this study the stabilized samples were remolded after 28 days of curing. Influences of the CCR content on basic properties: specific gravity, grain size distribution, index properties, linear shrinkage and free swell ratio are presented in Figures 5.4 to 5.8. The understanding of the change in basic properties is vital to analyze the change in engineering properties such as compaction behavior, compressibility and strength development.

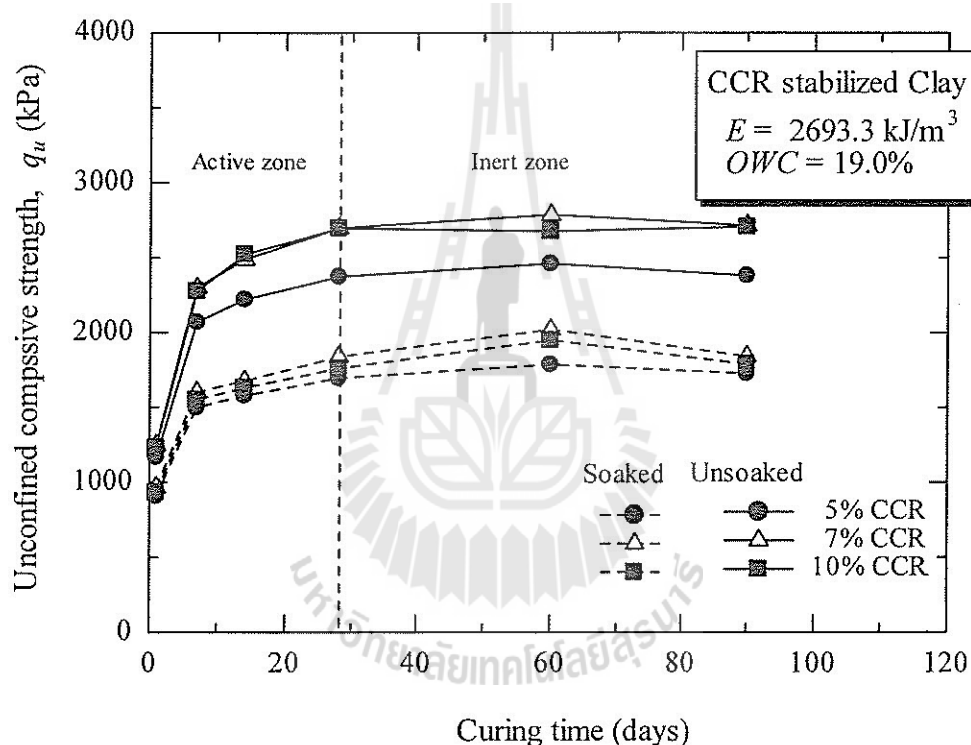


Figure 5.3 Strength development in the CCR stabilized samples.

Figure 5.4 shows the specific gravity of recycled samples. The specific gravity, G_s , values of recycled samples are practically unchanged with the CCR content and are almost the same as the specific gravity value of the unstabilized clay ($G_s = 2.75$).

Figure 5.5 shows the particle size distribution of the recycled samples. The particles of the recycled samples with for different CCR contents are larger than those of the unstabilized clay and CCR as indicated by larger average grain sizes, D_{50} . The recycled particles have higher coarse grains and lower fine grains than the CCR and clay particles. The particle size distributions of all recycled samples are essentially the same for different CCR contents after thoroughly remolding.

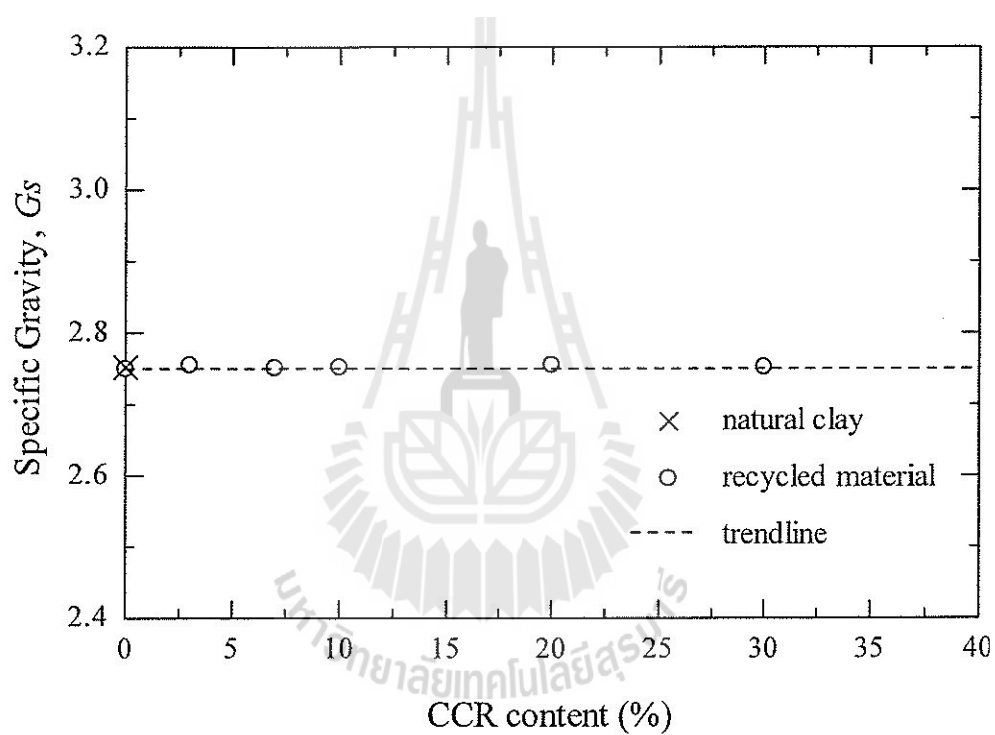


Figure 5.4 Specific gravity of the recycled CCR stabilized clay.

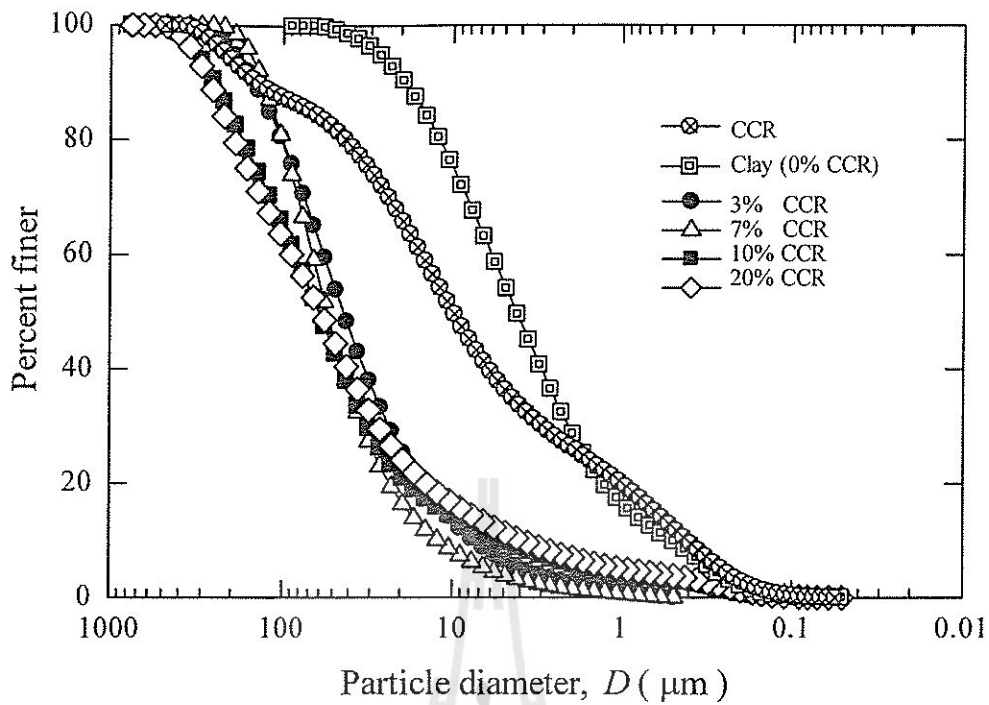


Figure 5.5 Grain size distribution of the recycled CCR stabilized clay.

Figure 5.6 shows the index properties of recycled samples with different CCR contents. The change in the index properties with CCR content is of the same pattern for both CCR stabilized and recycled samples. Initially as the CCR content increases, the plastic limit, PL , values of stabilized and recycled samples significantly increase while their liquid limit, LL values tend to change with small magnitude, resulting in a decrease in the plasticity index, PI . The PI becomes constant when the CCR content is greater 7%. Because the LL and PL values for the recycled and stabilized samples are essentially the same, their CCR fixation point is essentially the same and approximately equal to 7%.

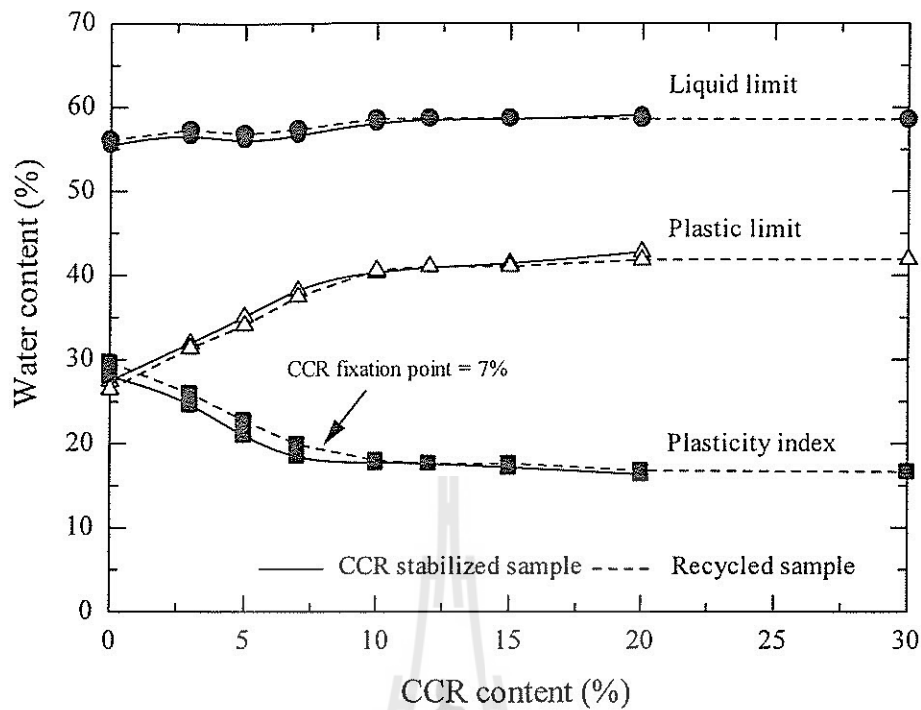


Figure 5.6 Index properties of the recycled CCR stabilized clay.

Figure 5.7 shows the linear shrinkage of recycled samples compared with that of the stabilized samples. The linear shrinkage of both stabilized and recycled samples significantly decrease with the CCR content when the CCR content is lower than 7% (CCR fixation point). Beyond this content, the linear shrinkage is approximately constant. Even with the same pattern and the CCR fixation point, the recycled samples show much lower linear shrinkage than the stabilized samples at the same CCR contents tested. This concludes that the CCR fixation point controls the shrinkage pattern but the shrinkage magnitude is strongly dependent upon the amount of coarse and fine grains. The higher the coarse grains and the lower the fine grains, the lower the linear shrinkage. This is an advantage of the recycled material over the stabilized clay.

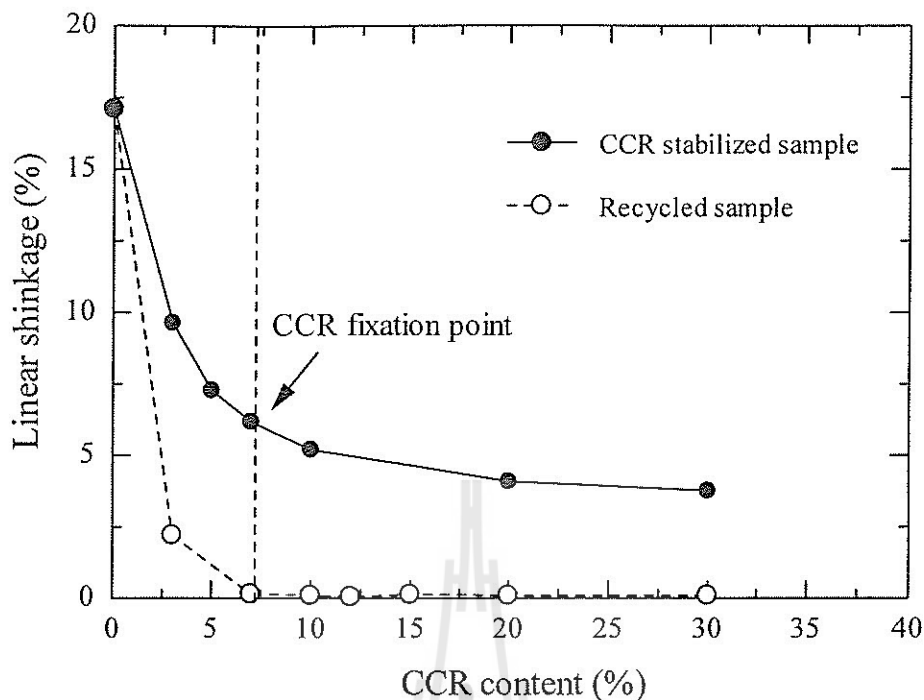


Figure 5.7 The linear shrinkage of recycled CCR stabilized clay.

Figure 5.8 shows the free swell ratio, FSR, of the recycled samples compared with that of the CCR-clay mixtures (prior to hardening) for different CCR contents. The FSR values significantly decrease with the CCR content. The FSR values of the recycled samples for different curing times are almost the same for the CCR contents tested. This might be because the recycled samples for different curing times have approximately the same particle distribution. When the CCR content is greater than 7% (fixation point) for both stabilized and recycled samples, the FSR value tends to increase. The relationships between FSR and CCR content of the recycled samples and the clay-CCR mixture are similar but the FSR values of the recycled samples are slightly lower for all CCR contents.

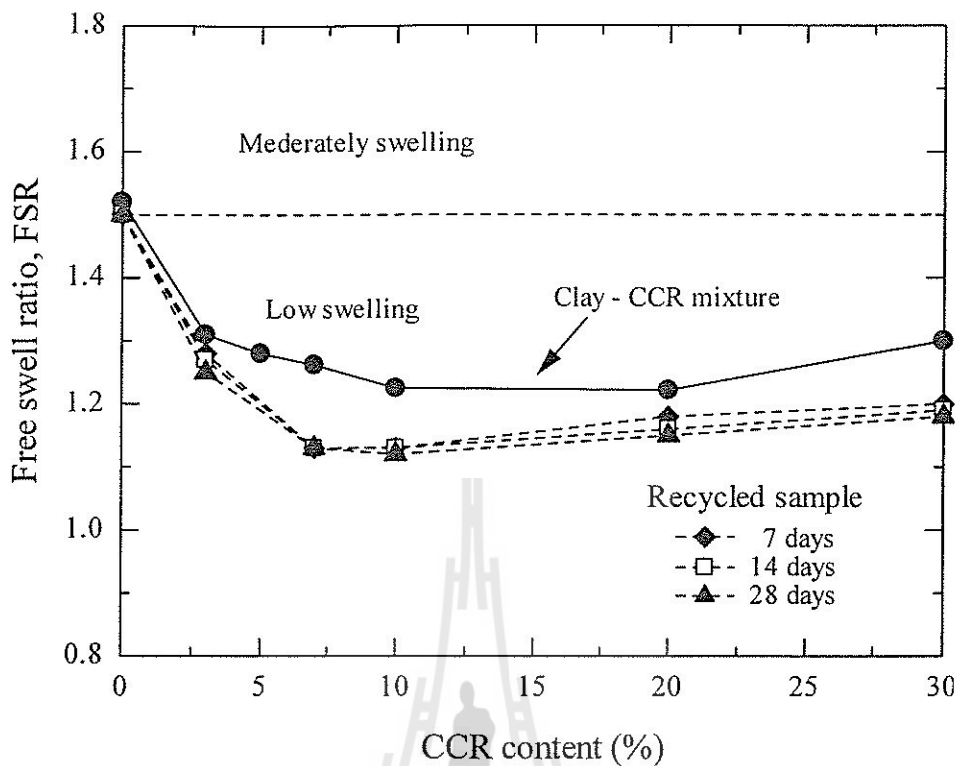


Figure 5.8 Free swell ratio of recycled CCR stabilized clay.

Figure 5.9 shows the compaction curves of the clay, CCR stabilized sample with 7% CCR, and recycled samples with low to high CCR contents. The compacted clay exhibit best compaction characteristics; i.e., highest maximum dry unit weight and lowest optimum water content. The unit weight of the CCR stabilized clay possesses lower dry unit weight than the compacted clay because the specific gravity of the CCR is lower than that of the clay (Kumpala and Horpibulsuk, 2012a). The compaction curves of the recycled samples with CCR contents lower than the CCR fixation point are essentially the same while the dry unit weight tends to decrease with the input CCR content when the CCR contents are greater than the fixation point. The reduction in maximum dry unit weight is associated with the increase in *OWC*. It is also note that as the CCR content increases, the water-sensitivity of the recycled

sample decreases i.e., a large increase in water content can cause only a minor change in dry unit weight. This characteristic is similar to that of the compacted coarse-grained soils.

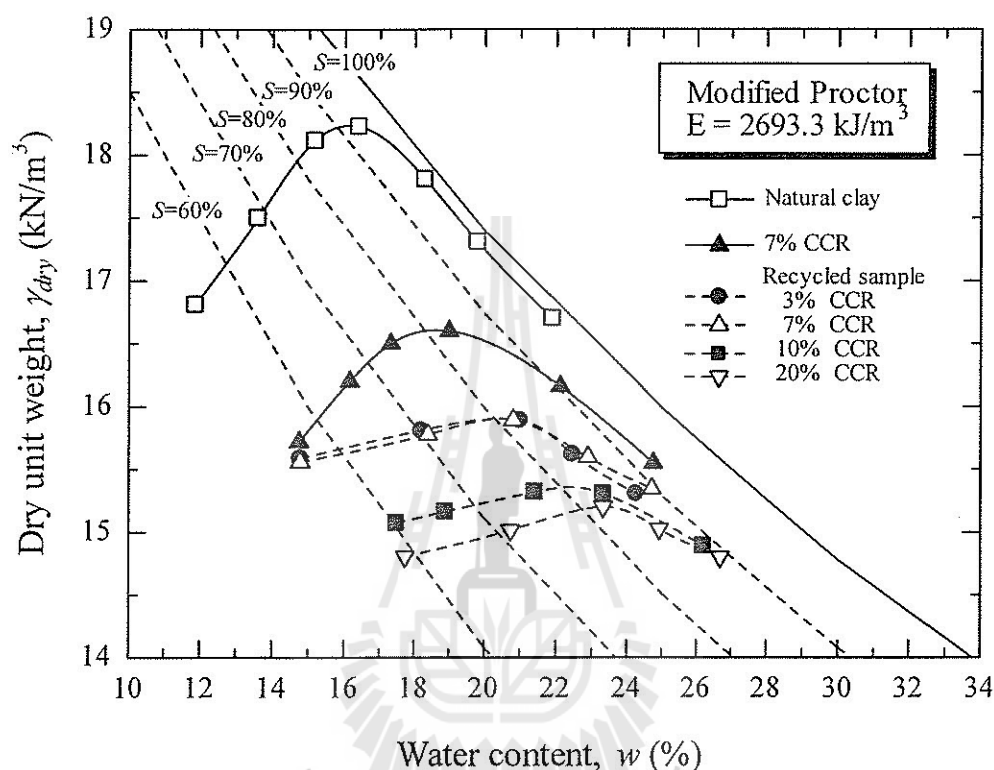


Figure 5.9 Compaction curves of clay, CCR stabilized clay and recycled CCR stabilized clay.

Compression behavior of the recycled samples with different CCR contents at 7 and 28 days of curing compared with that of compacted clay are presented in Figure 5.10. All samples were compacted under the same compaction energy (modified Proctor) at the optimum point. For short curing time (Figure 5.10a), the initial void ratio of the recycled samples decreases with the input CCR and lower than that of the compacted clay (0% CCR) when the CCR contents are lower than the CCR fixation point. The yield stress increases with the input CCR (decreases with the initial void

ratio). The same is not true for the recycled samples with the CCR contents greater than the fixation point. The initial void ratio increases with the CCR content, which is associated with the increase in FSR value. The yield stress decreases as the initial void ratio increases. For longer curing time (28 days), the initial void ratios for all recycled samples decrease and are lower than the void ratio of the compacted clay.

Figure 5.11 shows the 7-day soaked strength versus CCR content plots for recycled samples and CCR stabilized samples. The strengths of the recycled samples are slightly lower than the stabilized samples for the same CCR contents. The strength improvement of the recycled samples can be classified into four zones: inactive, active, inert and deterioration. The soaked strength of recycled samples with CCR contents lower than 3% (inactive zone) is null, which is the same as that of the compacted clay. The samples collapse after few minutes of soaking. The strength increases sharply with the CCR content (active zone) and is approximately constant when the CCR contents are greater than 7% (inert zone). The strength decreases when the CCR content is greater than 15% (deterioration zone).

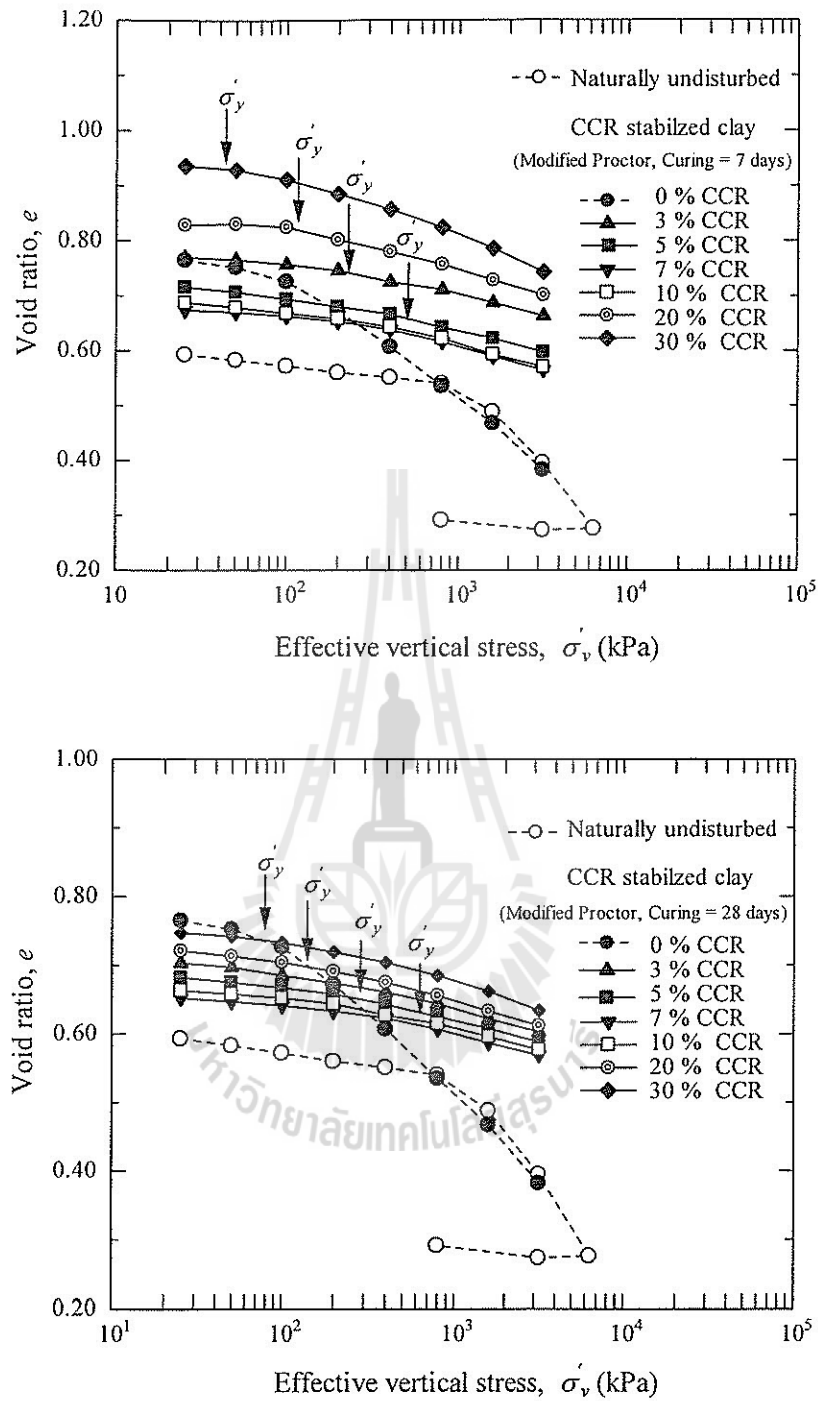


Figure 5.10 Void ratio versus effective stress of naturally undisturbed, compacted silty clay and compacted CCR stabilized clay (Kumpala and Horpibulsuk, 2012b).

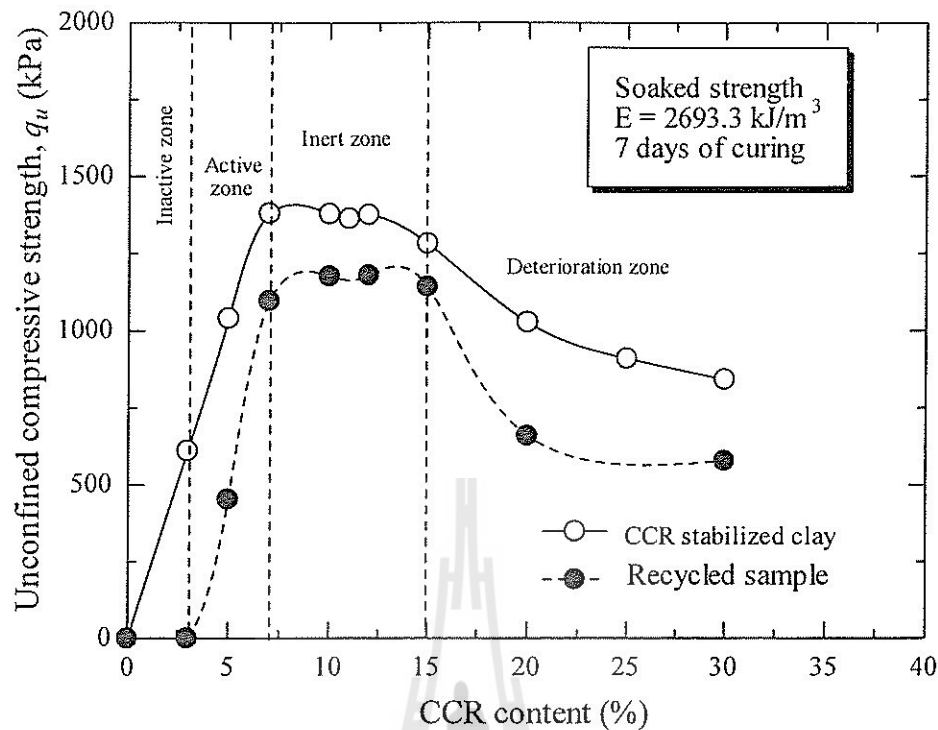
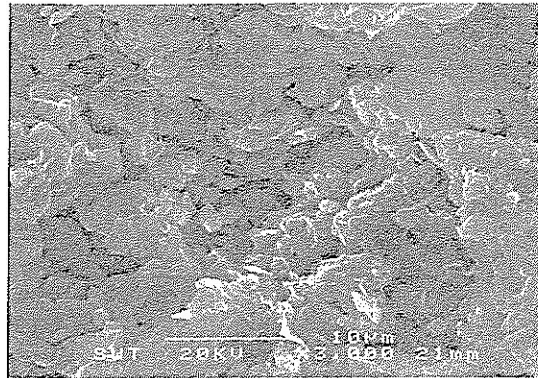


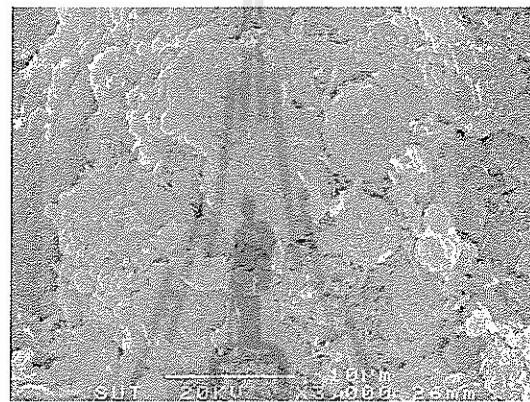
Figure 5.11 Compressive strength of remolded CCR stabilized clay.

5.4 Analysis and discussion

Figure 5.12 shows the SEM images of the compacted clay, CCR stabilized clay and recycled samples. The clay and the CCR stabilized samples were compacted at the same modified Proctor energy. The compaction enhances the interparticle forces; hence the flocculation of the clay particles (*vide* Figure 5.12a). The pozzolanic products (occurrence of glassy-like phases) are clearly shown in the pore space between the clay particles (*vide* Figure 5.12b). It is of interest that even after remolding the CCR stabilized clay, the pozzolanic products are attached on the recycled particles (*vide* Figure 5.12c). As such, the recycled particles are larger than the CCR and clay particles, as seen by the grain size distribution curves (*vide* Figure 5.5).



(a) compacted sample



(b) CCR-FA stabilized sample



(c) Recycled sample

Figure 5.12 SEM photos of the compacted soil, CCR-FA stabilized soil, and recycled CCR-FA stabilized soil after curing 28 day.

The index properties (*LL*, *PL* and *PI*) of the recycled and stabilized samples are essentially the same for the same CCR content, regardless of particle size. The reduction in *PI* shows that the CCR in the recycled material is still active with water and silica and alumina ions in the clay and the index properties are strongly dependent upon the clay-CCR-water interaction rather than the grain size. However, the larger grains reduce the specific surface and the repulsive forces due to diffusion double layer. The FSR values (swelling potential) of the recycled particles are thus lower than those of the clay-CCR mixtures for the same CCR contents. Due to lower swelling potential of the recycled particles, the recycled samples exhibit lower linear shrinkage than the stabilized samples. As the CCR content increases between 1% and 7% (*vide* Figure 5.8), the FSR values of the recycled particles and clay-CCR mixtures decrease due to the cation exchange process causing the clay flocculation (compression of diffusion double layer). But for larger CCR contents (greater than fixation point), the unsoundness due to free lime enhances the repulsive forces between the recycled particles and causes the increase in FSR values.

For the compacted clay, the water softens the clay particles and makes the soil particles slip over each other and move into a densely packed state. But for the recycled material, the compaction behavior at a given water content is strongly dependent upon the hard coarse particles (clay-CCR clusters attached with pozzolanic products) and the swelling potential due to unsoundness of free lime. The swelling potential comes into play at the very high CCR content of greater than fixation point. Kumpala and Horpibulsuk (2012a) illustrated that due to the unsoundness, the dry unit weight of the stabilized clay decreases and the swelling pressure under K_0 -condition increases as the CCR content increases in the inert and deterioration zones.

Consequently, the unit weight of the recycled samples decrease with the CCR content when the CCR contents are greater than 7% (Figure 5.9). The repulsive forces among the clay-CCR clusters prevent the attractive forces due to the compaction. For low CCR content (CCR content < 7%), the hard coarse particles resist the compaction; hence the unit weight of the recycled sample is lower than the stabilized sample at the same 7% CCR (*vide* Figure 5.9). Because the grain size distributions of the recycled samples are approximately the same for different CCR contents, the compaction curves are almost the same.

The reduction in void ratio with time (compared Figure 5.10 and 5.10b) confirms that the pozzolanic reaction still develops in the recycled samples. The reduction in void ratio is associated with the increase in yield stress. When the moist clay is mixed with CCR, the clay and CCR flocculation develops due to the cation exchange process and forms clay-CCR clusters. The pozzolanic reaction mainly happens on the surface of clay-CCR clusters where some of the CCR particles are surrounded with the clay particles. When the stabilized samples are remolded, the surrounded CCR particles are free to react with the clay and develop the pozzolanic products.

Besides the compression test result, the pozzolanic reaction of the recycled material is illustrated by Figure 5.13, which shows the 1-day and 7-day strengths for different CCR contents. Fly ash, FA, which contains high amount of silica and alumina, was mixed with the recycled material to investigate the presence of the surrounded active CCR. The CCR contents were 7%, 10% and 15%, which are in inert and deterioration zones. Kumpala and Horpibulsuk (2012a) illustrated that $\text{Ca}(\text{OH})_2$ in the inert and deterioration zones are abundant and prompt to react with

FA. The strength of the CCR-FA stabilized clay is contributed from packing effect (the small and spherical FA particles can simply fill up the pore between clay and CCR) and pozzolanic reaction. Similarly, the 1-day strengths of the recycled CCR-FA samples are higher than those of the recycled CCR samples for all the CCR contents due to the packing effect. The effect of pozzolanic (long-term) reaction is clearly observed after 28 days of curing. The strengths of recycled CCR-FA samples are significantly higher than those of the recycled CCR samples for the same CCR content. The decrease in strength for CCR content of 15% (in the deterioration zone) is caused by the unsoundness due to free lime.

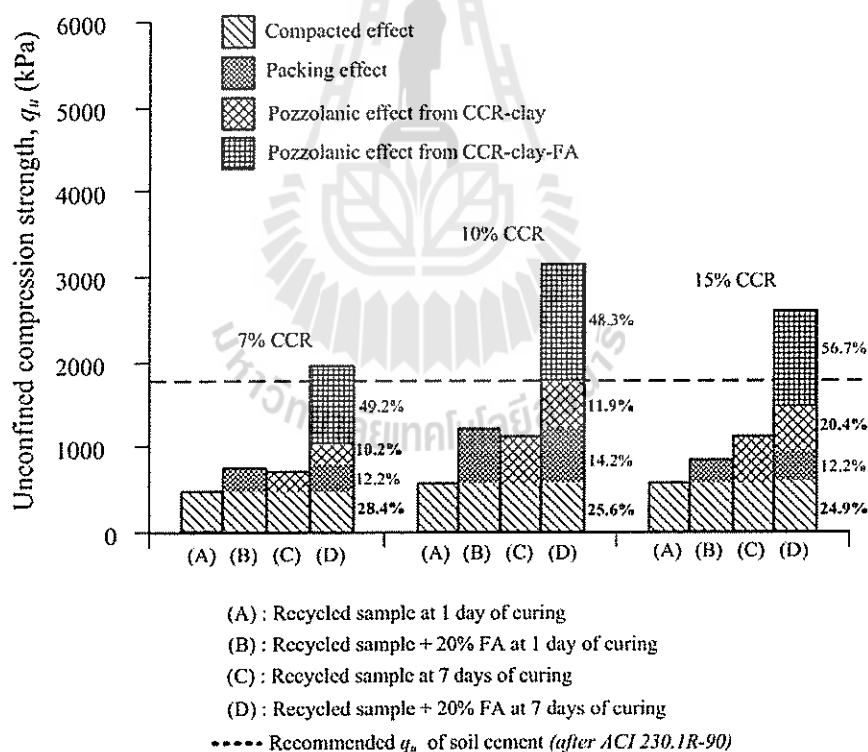


Figure 5.13 Influence of FA on strength development of the remolded CCRstabilized clay compacted.

It is logical to relate the elastic properties with unconfined compressive strength because they are governed by the pozzolanic reaction. Figure 5.14 shows the relationship between modulus of deformation at 50% strength, E_{50} and unconfined compressive strength, q_u , for recycled and stabilized samples. The relationship between E_{50} and q_u is unique for both recycled and stabilized samples and represented by a square root function. This implies that the pozzolanic reaction in both recycled and stabilized samples are the same and hence strength development for both recycled and stabilized samples is of the same pattern (*vide* Figure 5.11).

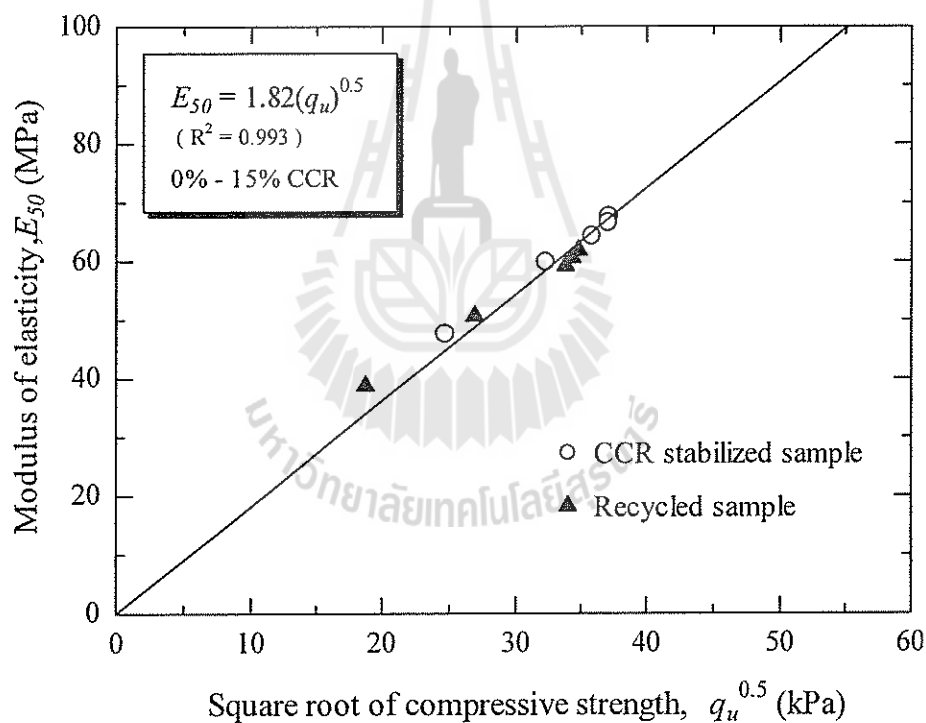


Figure 5.14 Relationship between modulus of elasticity, E_{50} and compressive strength, q_u of CCR stabilized clay and recycled material.

The recycled CCR stabilized clay with CCR contents between 5% and 7% can be used as a fill and pavement materials. For the low CCR content of 3%, the amount

of surrounded CCR particles is small and insufficient to react with available silica and alumina in the clay when remolded. As such, the soaked strength development of the recycled material with this content is null.

5.5 Conclusions

This paper presents basic and engineering properties of the recycled CCR stabilized clay to ascertain to its applications as fill and pavement materials. The following conclusions can be advanced from this study.

1. Scanning electron microscopic images show the pozzolanic products attached on the recycled particles. Due to the attached pozzolanic products, the recycled particles are larger than the CCR and clay particles as indicated by the grain size distribution and average grain size, D_{50} .
2. The index properties of the recycled samples are essentially the same as those of the clay-CCR samples at the same CCR contents. This indicates that the index properties are strongly dependent upon the clay-CCR-water interaction, regardless of grain size distribution. Consequently, the CCR fixation point of the stabilized and recycled samples is practically the same.
3. The attached pozzolanic products reduce the specific surface and the repulsive forces due to the diffusion double layers of the recycled particles. Consequently, the linear shrinkage and free swell ratio of the recycled particles are lower than those of the clay-CCR samples.
4. The compaction behavior of the recycled material diverts from that of the clay and the stabilized clay. The harder and larger recycled particles resist

the compaction; hence the recycled material exhibit lower unit weight and higher optimum water content. The lower unit weight reduces the overburden on the foundation. The water sensitivity of the recycled material is lower than that of the stabilized clay, which is the typical characteristic of the compacted silt and sand.

5. The compaction curves of recycled samples with the CCR contents lower than fixation point are identical because the recycled samples possess approximately the same grain size distribution. The unsoundness due to free lime comes into play when the CCR contents are greater than the fixation point. The dry unit weight of the recycled material increases with CCR content.
6. The strength and yield stress of the recycled material increase with curing time due to pozzolanic reaction. The recycled material with CCR contents between 5 and 7 can be used as fill and pavement materials. The soaked strengths are null for CCR contents lower than 3%. The unsoundness due to free lime causes the strength reduction in recycled samples with high CCR content.
7. The relationship between modulus and strength is unique for both recycled and stabilized samples and represented by a square root function. This implies that the pozzolanic reaction in both recycled and stabilized samples is the same and hence strength development for both recycled and stabilized samples is of the same pattern.

5.6 References

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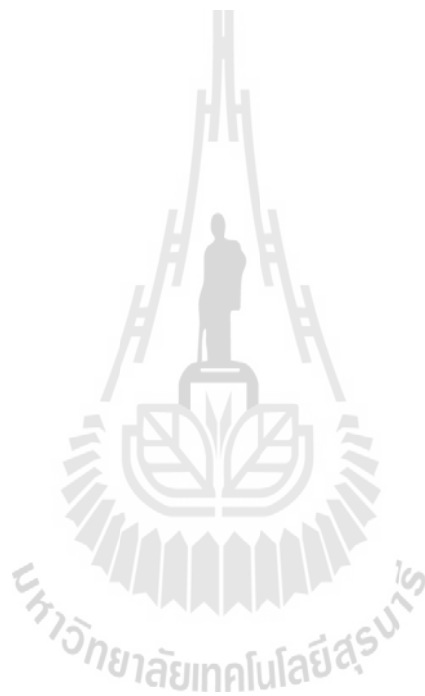
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CHAPTER VI

CONCLUSIONS

6.1 Summary and conclusions

The soil in northeast Thailand is a problematic soil, which is sensitive to change in water content. Its collapse behavior due to wetting is illustrated by Kohgo et al. (1997) and Kohgo and Horpibulsuk (1999). The soil in the dry state has high bearing capacity and low compressibility. The bearing capacity significantly decreases as the water content increases, which causes several problems for foundation and pavement structures (Horpibulsuk et al., 2008). This thesis presents a technique to improve the engineering properties of a problematic soil by Calcium Carbide Residue (CCR); CCR which is a by-product of acetylene (C_2H_2) production process through the hydrolysis of calcium carbide (CaC_2). The CCR is generated as an aqueous slurry and is composed essentially of calcium hydroxide ($Ca(OH)_2$) with minor parts of calcium carbonate ($CaCO_3$), and unreacted carbon and silicates (Hologado et al., 1992; Ramasamy and Periathamby, 2002).

This thesis is made to meet three main objectives. The first is to study the effect of influential factors such as water content, CCR content and curing time on the physical and engineering properties of a problematic silty clay stabilized with the CCR. The second is to study the durability characteristic against the wetting – drying cycles (w-d cycles), which is a main aspect for pavement design. The third is to investigate basic and engineering properties of the recycled CCR-FA stabilized clay to ascertain its possibility of reuse as backfill and sub-base materials.

6.1.1 Engineering properties of CCR stabilized clay

The engineering properties involved are soaked and unsoaked strengths, swelling and collapse behavior, water absorption, volume change due to water immersion and unsoaked and soaked bearing capacities. This study is to ascertain the performance in fill and pavement base and subbase applications. The calcium carbide residue can be regarded as an alternative cementing agent.

The chemical composition and the XRD-pattern of the CCR are similar to those of the hydrate lime. The CCR contains very high amount of Ca(OH)_2 (about 76.7%). Because the silty clay contains the large amount of natural pozzolanic materials (SiO_2 , Al_2O_3 and Fe_2O_3) of 60.5%, the Ca(OH)_2 from the CCR can react with natural pozzolanic materials, resulting in the improvement of the engineering properties of the CCR stabilized silty clay. This stabilized clay can be considered as a pavement geomaterial.

The soil improvement by the CCR can be classified into three zones: active, inert and deterioration. The CCR fixation point, which is simply obtained from the laboratory test can be used to identify the active zone. The CCR fixation point indicates the capacity of the clay to absorb Ca^{2+} ions and react with Ca(OH)_2 . The applicability of the fixation point is verified by the available test results of the clayey soils stabilized with the Ca(OH)_2 rich material.

Because the compaction curves and compaction characteristics of the clayey soils are governed by the plasticity index, they are changed with the CCR content. The *OWC* of the stabilized clay increases as the CCR content and insignificantly alters when the CCR content is higher than the CCR fixation point where the plasticity index becomes essentially constant. The CCR enhances the chemical bonding among the clay particles. The *OWC* is the suitable mixing state,

providing the best engineering properties (highest strength and bearing capacity and lowest water absorption, swelling and collapse). The lower water content is not enough for pozzolanic reaction while the higher water content causes the high water/stabilizer ratio, W/S.

The water absorption induces the swell pressure. The samples compacted on the dry side of optimum shows higher water absorption potential than those on the wet-side due to lower degree of saturation. Consequently, the samples compacted on the dry side exhibits poorest engineering properties. The soil stabilization by the CCR yields higher strength than that by the hydrated lime. This is possibly because the CCR contains more pozzolanic materials (SiO_2 , Al_2O_3 and Fe_2O_3) of about 12.3%. Thus, this stabilization is useful in terms of engineering, environmental and economic viewpoints.

6.1.2 Influence of wet-dry cycles on compressive strength

The w-d cycles cause tension crack and surface crack, which damage the stabilized pavement structure. The investigation of the life time of the stabilized materials via wetting and drying test is thus significant. Even though the CCR stabilization improves significantly the strength, the durability against wetting and drying of the stabilized clay is considered as low according to the recommendations by the ACI (1990) and U.S. Army Corps of Engineers (2004). The wetting induces the repulsive forces among the clay particles and the drying causes the tension and surface cracks on the stabilized material. Consequently, the strength of the CCR stabilized clay reduces significantly with the number of w-d cycles. The input FA improves both the densification and the pozzolanic reaction of the stabilized clay; hence the improvement in strength and durability. The improvement of the

densification and the pozzolanic reaction is clearly shown by the SEM images. The optimal CCR and FA contents are found at about 7% and 20% respectively. The excess fly ashes surround the CCR grains and hinder the interaction between water and CCR grains; hence the strength reduction.

The strength test results show that the durability against wetting and drying cycles for all curing times tested is strongly dependent upon the unsoaked strength (prior to the wetting and drying test). It is logical to develop a relationship between the normalized strength and number of cycles. This relationship is useful for a quick determination of strengths under different number of wetting and drying cycles to ascertain the service life of the stabilized material.

Besides the strength improvement against w-d cycle, the input FA improves significantly the resistance to water absorption. The optimal FA content to resist the water absorption is at about 20% FA, which also provides the highest strength. The water absorption of CCR-FA stabilized clay is lower than CCR stabilized clay (without fly ash) in all samples especially at 0.8OWC and 1.2OWC.

6.1.3 Basic and engineering properties of recycled cemented clay

The index properties of the recycled samples are essentially the same as those of the clay-CCR samples at the same CCR contents. This indicates that the index properties are strongly dependent upon the clay-CCR-water interaction, regardless of grain size distribution. Consequently, the CCR fixation point of the stabilized and recycled samples is practically the same. The attached pozzolanic products increase specific surface and reduce the repulsive forces due to the diffusion double layers of the recycled particles. Consequently, the linear shrinkage and free swell ratio of the recycled particles are lower than those of the clay-CCR samples.

The compaction behavior of the recycled material diverts from that of the clay and the stabilize clay. The harder and larger recycled particles resist the compaction; hence the recycled material exhibit lower unit weight and higher optimum water content. The lower unit weight reduces the overburden on the foundation. The water sensitivity of the recycled material is lower than that of the stabilized clay, which is the typical characteristic of the compacted coarse-grained soils. For the CCR content lower than fixation point, the compaction curves of recycled samples with different CCR contents are essentially the same because the recycled samples possess approximately the same grain size distribution. The swelling comes into play when the CCR contents are greater than the fixation point. Consequently, the dry unit weight of the recycled material increases with CCR content. For low CCR content, even with practically the same compaction curve, the initial void ratios are different for different CCR contents due to the effect of pozzolanic reaction. For high CCR content, the void ratio increases with CCR content due to high swelling potential.

The strength and yield stress of the recycled material increase with curing time due to pozzolanic reaction. The recycled material with CCR contents between 5% and 7% can be used as fill and pavement materials. The soaked strengths are null for CCR contents lower than 3%. The unsoundness due to free lime causes the strength reduction in recycled samples with high CCR content. The relationship between modulus and strength is unique for both recycled and stabilized samples and represented by a square root function. This implies that the pozzolanic reaction in both recycled and stabilized samples is the same and hence strength development for both recycled and stabilized samples is of the same pattern.

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BIOGRAPHY

Mr. Apichit Kumpala. I was born 1 October 1978; I obtained my bachelor degree in Civil Engineering from the Department of Civil Engineering, Khon Kaen University since 2003. I then worked as a civil engineer at KC construction Co., Ltd. from 2003 to 2004. In May 2003, I pursued my master studies at the School of Civil Engineering, Suranaree University of Technology and obtained my master's degree in civil engineering in December 2005. During 2005 to 2008, I worked as a lecturer at the Department of Civil Engineering, Faculty of Engineering and Architecture, Rajamangala University of Technology Isan, giving courses on Soil mechanics, Foundation Engineering, and Geology. I started my Ph.D. study in January 2009 under a financial support from the commission on Higher Education. In September 2012, I visited the Hokkaido University to present a paper in the International Conference on Transportation geotechnics II. During my Ph.D. Study, I published 3-international, 1-international conference paper and 5-national conference papers.