THE DEFORMATION CHARACTERISTICS OF DEEP MIXED COLUMNS IN SOFT CLAYEY SOILS: A MODEL STUDY

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ABSTRACT

THE DEFORMATION CHARACTERISTICS OF DEEP MIXED COLUMNS
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Deep Mixing involves the introduction of cementitious or specially formulated solutions directly into the ground through the use of purpose built blending injection augers. The system is mainly designed to increase strength and reduce compressibility of treated soil.

In the first stage of the research effective mixture ratios and mixture types of stabilizing agents were investigated for soft clays (CL form Eymir lake and kaolinite) by means of unconfined compression (UC) tests on stabilized soils. The unconfined compressive strength (UCS) values were obtained for 7,28,90 and 365 days of curing time. The ratio of elastic modulus at 50% failure load (E_{50}) to (UCS) of the stabilizing agents were also investigated.

In the second part of the research programme, deep mixed model columns with the three column materials and four different column spacings are formed within the large scale consolidation tanks, and the consolidation characteristics of deep mixed improved clay were investigated.

Based on the results of large scale consolidation tests on deep mixed columnar improved soft clay, compressibility characteristics of improved soft clay were determined in relation to spacing of columns namely, effective replacement ratio and binder content. The cement content (also UCS) of the column material was found to be the most important parameter for the improvement effects of DMM applications. Validity of the relations for the estimation of bulk compression modulus of soilcrete were discussed. The use of constrained modulus of the soil and the column material were found to be effective in predicting the compression modulus of the soilcrete. Settlement reduction factor versus replacement ratio and cement content relations were determined which may be used for preliminary design works. The stresses on the soil and the columns were backcalculated from the settlement values. The stress ratios were obtained.

Keywords: Deep mixing, laboratory model, mixture ratio, cement content, replacement ratio, unconfined compression strength-UCS, E_{50}/UCS ratio, compression modulus, settlement reduction factor, stress ratio

ÖZ

YUMUŞAK KİL ZEMİN İÇİNDE DERİN KARIŞTIRMA KOLONLARININ DEFORMASYON KARAKTERİSTİKLERİ, BİR MODEL DENEY ÇALIŞMASI

Şengör, Mahmut Yavuz
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Derin karıştırma, bu amaçla tasarlanmış karıştırıcı enjeksiyon burguları kullanılarak zemine doğrudan çimentolu ve özel formüllü solüsyonlar uygulanması işlemini ifade eder. Bu sistem esasen zemin içerisinde geçirimliliğin azaltılmasını ve/veya dayanımın arttırılmasını sağlamak amacıyla tasarlanmıştır.

Bu araştırmanın ilk aşamasında, iyileştirilmiş zeminler üzerinde tek eksenli basınç deneyleri yapılarak yumuşak kil zeminler için (Eymir gölü kili ve kaolen) etkili karışım oranları ve karışım tipleri araştırılmıştır. 7, 28, 90 ve 365 günlük kür süreleri sonunda tek eksenli basınç dayanım değerleri elde edilmiştir. Bunun beraber iyileştirilmiş zeminlerin elastik modül (E_{50}) /basınç dayanımı (UCS) oranları da ayrıca araştırılmıştır.

Araştırma programının ikinci aşamasında, büyük ölçekli konsolidasyon tankları içerisinde dört farklı yerleşimde ve üç tip karışım oranında model derin karıştırma kolonları oluşturulmuş ve DMM ile iyileştirilmiş kil zeminin konsolidasyon özellikleri araştırılmıştır.

Derin karıştırma kolonları ile iyileştirilmiş yumuşak kil zemin üzerinde yapılan büyük ölçekli konsolidasyon deneylerinin sonuçlarına göre, iyileştirilmiş yumuşak kil zeminin sıkışma özellikleri kolon parametrelerine, özellikle etkili alan oranı ve karışım malzemesi miktarına bağlı olarak belirlenmiştir. İyileştirme açısından kolon çimento miktarının (buna bağlı olarak tek eksenli basınç dayanımı, UCS) en etkili parametre olduğu tespit edilmiştir. İyileştirilmiş zeminin sıkışma modülünü belirlemek için kullanılan bağıntıların geçerliği irdelenmiştir. İyileştirilmiş zeminin sıkışma modülünün tespitinde zeminin ve kolon malzemesinin sıkışma modüllerinin kullanılmasının daha iyi sonuç verdiği görülmüştür. Ön tasarım işlerinde kullanılabilecek şekilde, oturma azaltım faktörü ile alan oranı ve çimento miktarı arasındaki ilişkiler tespit edilmiştir. Oturma değerlerinden zemin ve kolon üzerindeki gerilmeler geri-hesaplanmıştır. Gerilme oranları tespit edilmiştir.

Anahtar Kelimeler: Derin karıştırma, laboratuar modeli, karışım oranı, çimento miktarı, alan oranı, tek eksenli basınç dayanımı-UCS, E_{50}/UCS oranı, sıkışma modülü, oturma azaltım faktörü, gerilme oranı

To my family and friends Pinar and Yagmur

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a_s	Replacement ratio; Ratio of area of the treated soil to the area of the unit cell
a_w	Cement content; dry weight of cement / dry weight of soil to be stabilized
$A_{columns}$	Total cross sectional area of the columns
$A_{stabilized\ soil}$	Tributary area of stabilized soil
β	Settlement reduction factor, ratio of settlement of untreated soil to settlement of treated soil, $s_{untreated}/s_{treated}$
C	Cement
c_u	Undrained shear strength of the soil
DMC	Deep mixing columns
DMM	Deep Mixing Method
E	Elastic modulus
E_{50}	Secant modulus evaluated at stress levels related to 50% of the failure load
E_{col}	Young's modulus of the column
FA	Fly-ash
FEM	Finite element method
M	1D, Oedometer compression modulus
M_{col}	Oedometer compression modulus of the columns
M_{soil}	Oedometer compression modulus of the untreated soil
MD	Marble dust
m_v	Coefficient of volume compressibility
n	Stiffness ratio between the treated and untreated soils,modular ratio, ratio of oedometer compression modulus of column to that of soil (M_{col}/M_{soil})
UCS	Unconfined compressive strength
UK	Untreated kaolinite

CHAPTER 1

INTRODUCTION

1.1 OVERVIEW AND PROBLEM STATEMENT

Deep mixing method (DMM) is a columnar or mass type of ground improvement technique used to strengthen the soil skeleton by directly injecting cementitious or specially-formulated materials using special-purpose blending injection augers. A typical scheme for DMM application is given in Figure 1.1. In general, the purposes of these methods are to reduce permeability and compressibility and/or to increase the strength within the soil mass. Since 1980s, various DMMs such as lime columns, cement mixing, and jet mixing have been heavily used to improve the soft ground, especially highly compressible clayey soils. For example, in Japan, thousands of kilometers of mixed columns are performed every year. Although DMMs are frequently used in practice, there are many unknowns at the design stage when a DMM is needed in a geotechnical project.

The design of DMM is made based on mixed parameters calculated using empirical relations. These relations mostly use the basic parameters of the natural and improved soil, i.e. soilcrete. Although they have been widely used in geotechnical design, these empirical relations may not always reflect the real behavior of the soilcrete. They were developed using the laboratory modeling works, which may suffer from several issues such as the effects of scaling on model dimensions, application (mixing) method, and boundary effects, etc.

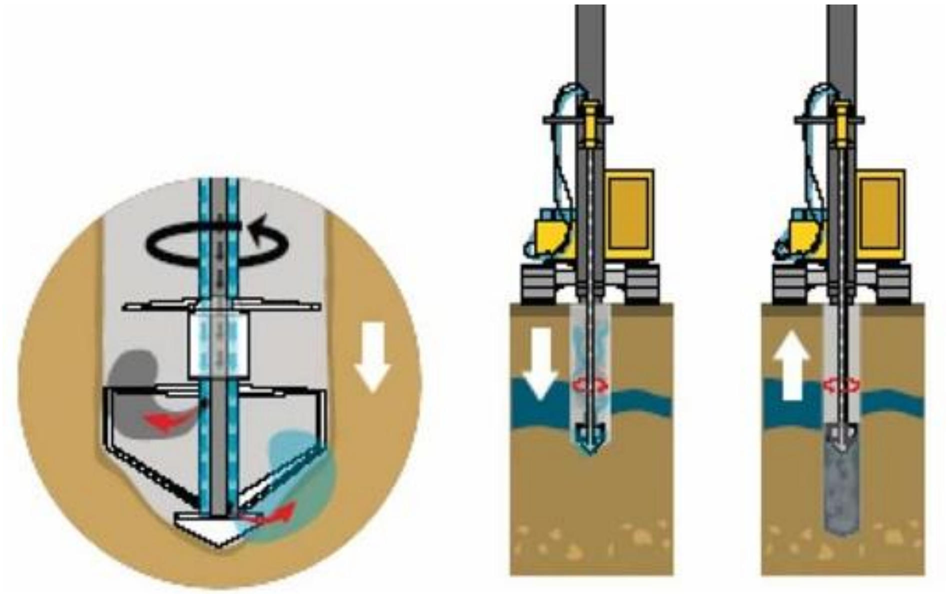


Figure 1.1 Typical scheme for DMM application

DMM applications in the field are generally very complex in terms of geomechanical behavior. The parameters such as loading levels, preconsolidation effects, and efficiency of mixing procedures etc. make the behavior of soil complicated to understand. Therefore either highly instrumented field loading tests or large scale laboratory model tests are attempted to understand the effect of various factors. However the large scale laboratory model studies investigating DMM column improvement, which is much more common compared to mass type of improvement, are limited in the literature.

Therefore, a large scale modeling work is crucial to better understand the behavior of DMM column improved soils.

1.2 RESEARCH METHODOLOGY AND OBJECTIVES

The behavior of DMM column improved soil has been investigated at a two-staged laboratory work. In the first phase, laboratory mixed samples were prepared with different binders (cement, fly ash and marble dust). Unconfined compression tests were then applied on cured samples (curing times: 7, 28, 90, and 365 days). Using the results of these tests, the most efficient binder mixes were determined as the column material. These mixes were then used as the improvement material at the next stage of the laboratory work.

In the second phase of this study, a setup for the large scale laboratory model test was prepared. Several difficulties such as the methodology of column production or the continuity of consolidation pressures for large scale model tests were taken into account. To overcome such challenges, the mixed column material was placed in the prebored pile holes with a special injection system. Pile material hardened and interacted with the neighbouring soil and binder material was diffused to the periphery. Then large diameter consolidation tests on soft clays reinforced by end-bearing DMM columns were performed.

Through successful completion of this research, the following objectives are achieved:

- Compression characteristics of kaolinite clay reinforced by soil-cement (soilcrete) mixes are explained through large scale 1D laboratory model tests.
- Settlement reduction factors for different binder mixes and replacement ratios are determined.

- The load sharing between the soil and deep mixed columns are enlightened.
- The analytical expression of the compression modulus for the stabilized system is obtained. Its validity is also examined.
- The effects of replacement ratio, stress level and type/amount of stabilizing agent are studied.

1.3 THESIS OUTLINE

This thesis is organized as follows: Chapter 2 gives the background work for DMMs. The experimental setup and testing procedure of the large scale laboratory model tests are described in Chapter 3. Results of these tests and the discussion are given in Chapter 4. Finally, the conclusions are provided in Chapter 5.

CHAPTER 2

REVIEW OF LITERATURE

The aim of DMM improvement is to enhance the strength and to reduce compressibility by means of cementation occurring between binders and soil. In this chapter the material found in the literature about the binders and also the properties of stabilized soils by deep mixed columns (DMC) is presented. Typical proportions of areas of applications for DMM is given in Figure 2.1.

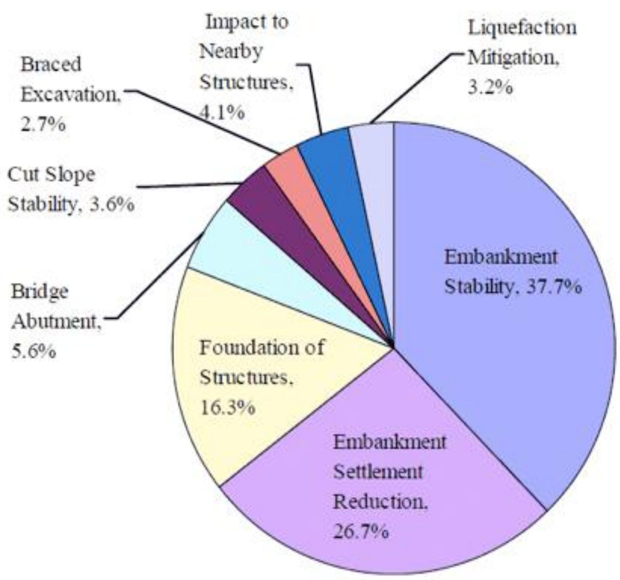


Figure 2.1 Typical areas of application for DMM (Terashi, 2009)

Al-Tabbaa (2005) described the general composition for a stabilized soil as shown in Figure 2.1.

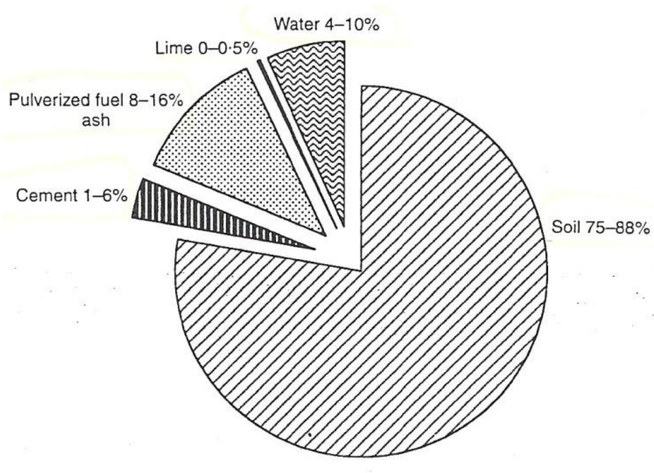


Figure 2.2 Typical proportions of constituents for stabilized samples (Al-Tabbaa, 2005)

2.1 BINDER TYPES AND AMOUNT

The choice of binder is a significant factor affecting the performance of improvement. There exist many research studies in the literature to find the most suitable type of binders, their volumetric content and possible combinations with other additives for different soil types. These studies investigate different types of agents that are used to meet different economical and/or environmental constraints. The most common conclusion of these studies is that, in soil stabilization works, an increase in the quantity of the stabilizing agent increases the compressive strength at different rates, depending on the properties of the soil and binder. They also mention the cement (C) is the most powerful binder for soft soil stabilization (Ahnberg et al., 1995; EuroSoilStab, 2001). Among the others, the most pronounced ones are lime, blast furnace slag cement (BFSC), pulverized fly-ash (PFA), gypsum, marble dust (MD), etc. By adding one of these additives to the cement, the amount of cement is reduced and the long term strength gain, i.e., durability, is achieved (Al-Tabbaa and Boes, 2002; Ahnberg and Johansson, 2005). Table 2.1 provides the list of appropriate binders for different soil types to provide guidelines for stabilization works (EuroSoilStab, 2001).

Table 2.1 Suitability of binders for different soils (EuroSoilStab, 2001)

Binder type	Silt	Clay	Organic Soils	Peat
Cement	G	M	M	G
Cement+gypsum	M	M	G	G
Cement+furnace slag	G	G	G	VG
Lime+cement	G	G	M	U
Lime+gypsum	G	G	G	U
Lime+slag	M	M	M	U
Lime+gypsum+slag	G	G	G	U
Lime+gypsum+cement	G	G	G	U
Lime	U	G	U	U

VG:very good in many cases; G: good in many cases; M: good in some cases; U: not suitable

Effective soil stabilization with different binders is generally achieved through the following reactions (Janz and Johansson, 2002):

i. The reaction of cement with water, and formation of calcium-silicate-hydrate (CSH) gel

ii. Pozzolanic reactions between $Ca(OH)_2$ and pozzolanic minerals in the soil

iii. Ion exchange between Ca+ ions from binders and ions in the soil

Some binders in this respect can be classified as cement; (i)hydration of tricalcium silicate ($3CaO.SiO_2$, C_3S) and dicalcium silicate ($2CaO.SiO_2$, C_2S) forming calcium silicate hydrate (CSH) gels, (ii) hydration of tricalcium aluminate ($3CaO.Al_2O_3$, C_3A) and ferrit ($4CaO.Al_2O_3.Fe_2O_3$, C_4AF) forming Calcium Aluminate Hydrates (CAH) gels, (iii) lime; formation of $CaOH_2$ and reaction with pozzolan and water forming CSH gel, (iv) blast furnace slag; after activation by $CaOH_2$ with the hydration of lime and cement; forms CSH gel, (v) Fly-ash (FA); forming CSH and CAH gels with $CaOH_2$, and (vi) silica fume; same reaction chain with FA.

2.1.1 Fly-ash (FA)

PFA (mostly called FA) is a synthetic pozzolan created by the combustion of coal. It can be described as a siliceous and aluminous material, which has a very little (C class) or no (F class) cementitious component. FA consists of inorganic matter present in the coal that has been formed during combustion. This material is solidified while suspended in the exhaust gases and is collected by electrostatic precipitators, an example of which is shown in Figure 2.2. FA particles are usually of silt size (0.074 - 0.005 mm). A typical particle size distribution is given in Figure 2.3.

Figure 2.3 A photo from electrostatic precipitators of Soma Thermal Plant

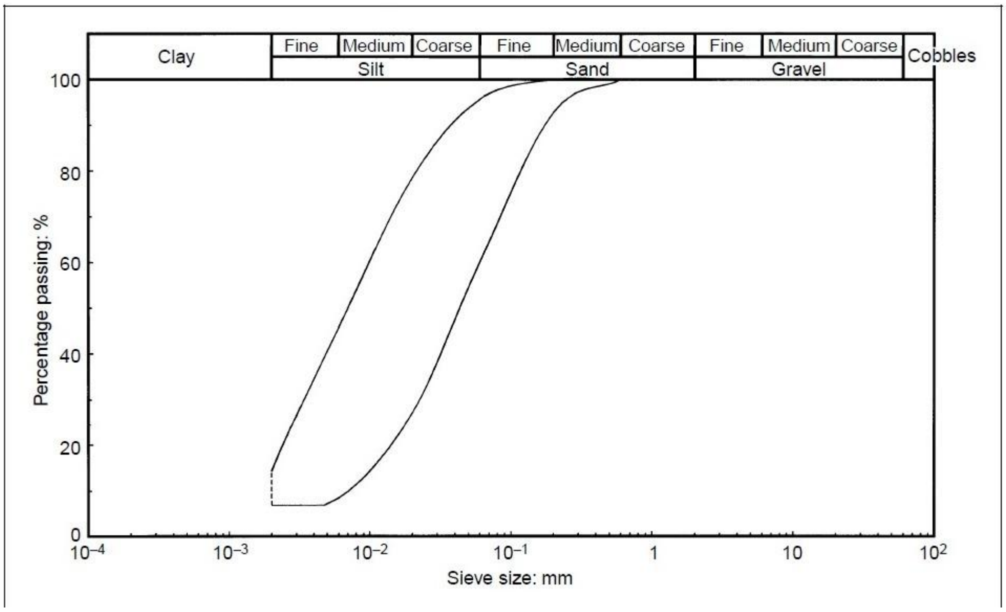


Figure 2.4 Typical range of particle size distributions of FA

The chemical composition and characteristic of FA changes with the type, origin and character of the coal. The classification of FA can be done based on its chemical ingredients. The basic classification is composed of two classes, class F and class C. The chemical requirements to classify any FA are given in Table 2.2. (ASTM C-618).

Class-C FA is produced from lignite and sub-bituminous coals and usually contains significant amount of Calcium Hydroxide (CaO) or lime. This class of FA, in addition to having pozzolanic properties, has some cementitious properties (ASTM C 618-99). The FA produced in Soma thermal plant is of C type generally.

Table 2.2 Chemical requirements for FA classification (ASTM C-618)

Properties	FA Class	
	Class F	Class C
Silicon dioxide (SiO_2) plus aluminum oxide (Al_2O_3) plus iron oxide (Fe_2O_3), min, %	70.0	50.0
Sulfur trioxide (SO_3), max, %	5.0	5.0
Moisture Content, max, %	3.0	3.0
Loss on ignition, max, %	6.0	6.0

Class-F FA is produced from burning anthracite and bituminous coals. This FA has siliceous or siliceous and aluminous material, which possesses little or no cementitious value. However, in a finely divided form and in the presence of moisture, it chemically reacts with calcium hydroxide at ordinary temperature to form cementitious compounds.

The strength enhancement in FA admixture is caused by the hydration reaction. Hydration is the formation of cementitious material by the reaction of free lime (CaO) with the pozzolans (AlO_3, SiO_2 and Fe_2O_3) in the presence of water. The hydrated calcium silicate (CSH) gel or calcium aluminate (CAH) gel (cementitious material) can bind inert material together. For Class-C FA, the calcium oxide (lime) of the FA can react with the siliceous and aluminous materials (pozzolans) of the FA itself. Since the lime content of Class-F FA is relatively low, addition of lime is necessary for hydration reaction with the pozzolans of the FA. For lime stabilization of soils, pozzolanic reactions depend on the siliceous and aluminous materials provided by the soil. The chains of pozzolanic reactions are given in Equations 2.1 to 2.4:

$$CaO+H_2O=Ca(OH)_2 \quad (2.1)$$

$$Ca(OH)_2 \Rightarrow Ca^{++} + 2[OH]^- \quad (2.2)$$

$$Ca^{++} + 2[OH]^- + SiO_2 \Rightarrow CSH \text{ (silica gel)} \quad (2.3)$$

$$Ca^{++} + 2[OH]^- + Al_2O_3 \Rightarrow CAH \text{ (alumina gel)} \quad (2.4)$$

Hydration of tricalcium aluminate in the ash provides one of the primary cementitious products. The rapid rate, at which hydration of the tricalcium aluminate occurs, results in the rapid set of these materials. It is the reason why delays in compaction result in lower strengths of the stabilized materials. The hydration chemistry of FA is very complex in nature. Therefore the stabilization application must be based on the physical properties of the FA treated stabilized soil and cannot be predicted based on the chemical composition of the FA.

To achieve maximum compressive strength in clayey soils, the lime content should be 5 to 9 % and the FA content 10 to 25 %. The ratios of lime and FA are

3 to 6 and 10 to 25 for granular soils, respectively. For self-cementing FAs, addition of 10% and 15% material is sufficient for sandy soils and clays, respectively (Vazquez, 1991).

Indraratna et al. (1995) investigated the effect of FA on the strength and deformation characteristics of Bangkok clay. They concluded that with the addition of a small quantity of cement or lime (5%) in addition to FA, a significant improvement in strength and compressibility properties of the treated soil can be achieved. Also noted that excessive amounts of FA (in the order of 25%) cause a reduction in overall undrained shear strength and also reducing the enhancement of compressive strength in the long term.

Tomohisa et al. (2000) found that several kinds of FAs are effective hardening additives on the muddy soil treatment. They stated that CaO and SO_3 content of the FA are effective in the stabilization. The main reaction products which contribute to strength are ettringite and calcium silicate hydrate (CSH). 9% cement stabilizer and 0, 5, 10% hardening additives were mixed with the soil. Compressive strength values generally increases as additive percentage increases.

Çokça (2001) has investigated the improvement of Soma (High calcium 19%) and Tuncbilek (Low calcium 2%) Class-C FAs mixed with the swelling soils. This study reported that addition of 20% FA decreased the swelling potential to nearly the one obtained with the addition of 8% lime. It was observed that it is better than 8% cement addition. There is a slight decrease in swelling potential by increasing FA from 20 to 25%. Consoli et al. (2001) reported the most efficient binder mixture as 4% lime and 25% FA.

The results of the studies by Mohamed and Hossein (2004) showed that 5% lime and 10% FA is needed to form ettringite (aluminum is added to facilitate and enhance the formation of ettringite). Application of aluminum added fly-ash (ALFA) process to high sulphate content soil has resulted in forming a solid monolith capable of producing more than 1000 kPa of unconfined compressive strength (UCS).

Yaprak et al. (2004) investigated the effects of Çayırhan FA (Ç-FA) and Kardemir blast furnace slag (BFS) on the properties of the concrete. The highest compressive strengths were obtained with 10% FA (382.5 kg PC42.5+4.25 kg Ç-FA) and 20 % BFS (340 kg PC42.5+85 kg BFS) admixed concretes.

Table 2.3 The results presented by Yaprak et al. (2004)

UCS strength	7 days	28 days	90 days
Control (425 kg PC 42.5)	56,7	57,7	59,5
Ç-FA10	51,3	55,3	63,0
Ç-FA20	48,7	54,5	58,2
Ç-FA30	43,4	48,0	49,2
BFS10	54,0	58,5	59,8
BFS20	54,5	62,3	63,8
BFS30	51,1	59,0	60,8

Aydilek (2004) stated that, due to the absence of self-cementing potential, Class-F FAs may be used with the addition of some amount of lime and/or cement for improvement works. In this study, to investigate the effect of cohesion on engineering properties of stabilized soil, kaolinite is also added to some mixtures.Lin et al. (2007) reported that the bearing capacities of soft clay (UCS=33 kPa) were increased by 3 times with an addition of 16% FA only. Jaroslaw (2007) stabilized the clayey mud, marl (calcareous clay) and peat with cement and FA addition. The summary of this research is 75% cement-25% FA is an effective mixture and it is given in Figure 2.5.

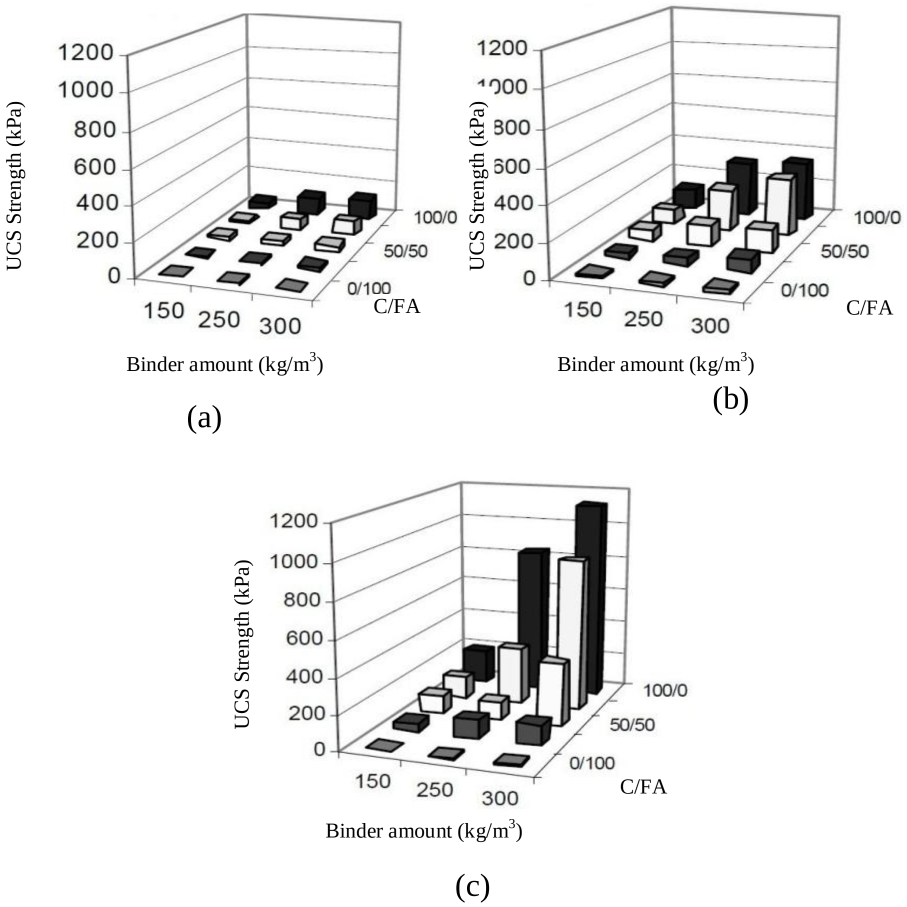


Figure 2.5 The unconfined compressive strength (UCS) for stabilized a) peat, b) clayey mud, and c) marl (Jaroslaw, 2007)

2.1.2 Cement (C)

Cement is the most effective material that can be used in soil improvement. The unconfined compression tests performed on cement stabilized soils show that increasing cement content (a_w, dry weight of cement/dry weight of soil) and curing time directly increases UCS (Bergado et al., 2005; Lade and Overton, 1989; Lorenzo and Bergado, 2004).

Lorenzo and Bergado (2004) explained this phenomenon by the hydration of cement. This reaction, as a result of calcium ions reacting with soil silica and soil alumina, produces pozzolanic products bounding the clay minerals leading to enhanced strength. It is assumed that there is enough water for chemical reactions of hydration. As cement hydration and pozzolanic activity continues, the strength of the treated soil continues to develop.

Bergado et al. (2005a) stated that the UC strength of cement stabilized soils decrease with increasing total water content for specific cement content. This can be explained as the water content increases the volumetric increase leading to the increase in the distance between clay minerals and so the bonding between the minerals and resulting strength decrease.

Miura et al. (2001) gives the 28 days strength of the stabilized soil with the following relation;

$$UCS=A/B^{wc/c} \tag{2.5}$$

where, wc/c is the ratio of water content of stabilized soil to the cement content, UCS is 28 days UCS, and A and B are constant dependent on the soil type and binder.

Hayashi et al. (2003) performed detailed investigation on 17 years old deep mixed (cement-mixed) columns and surrounding soils. They concluded that the strength enhancement continues for long time for the central part of the columns. The leaching of Ca ions to the surrounding soil causes some small deterioration causing minor strength reduction. Kitazume et al. (2003) found the same trend by performing laboratory tests on 1-year old cement-stabilized kaolinite clay for different environmental conditions. The leaching of Ca ions, the by-product of hyration of CaO present in the binder, from the Lime-C columns to the surrounding clay has been detected. The maximum distance at which these ions transferred was determined as 50 mm. No valuable correlation between Ca ion transfer and strength were drawn.

The ion transfer was also investigated by Larsson and Kosche (2005) through laboratory testing of the transition zone surrounding seven different lime cement columns installed in laboratory prepared kaolinite clay. The methods to mix the cement were (i) dry mixing, (ii) wet mixing and (iii) casted columns. A large number of laboratory and field tests; fall cone tests, natural water content, Atterberg limits, undrained shear strength tests on cored samples from the boundary of columns, were performed in the soil surrounding the columns for 7, 14, 30 and 90 days of curing times. The natural water content and the plastic limit were unchanged in the boundary layer. The conclusion was that the migration of calcium ions increases the undrained shear strength in a transition zone, surrounding the columns about 30 mm from the column boundary.

Horpibulsuk et al. (2004) performed series of consolidated undrained triaxial compression tests (CU-TX) on cement admixed (a_w=6, 9, 12 and 18%) clay samples. They investigated the cementation effects on strength for confining pressures lower than the effective yield stress (p_y). As Horpibulsuk (2003) stated that the cementation effect is insignificant after 28 days of curing, the tests were performed on 28 days cured samples. According to the results of this laboratory study, the deviator stress vs. strain plots for cement admixed clay show a peak value which is the same for all confining pressures. This is related to the effect of contribution of cementation on strength for $q_c<p_y$ (for stresses below the yield stress). For post yield state ($q_c>p_y$), the contribution of soil fabric effects on strength comes into picture. Therefore the deviator peak strength increases with the increase in confining pressure at this state. To conclude, for confining pressures lower than the yield stress, strength is only dominated by cementation effect. However, as confining pressure increases and passes the yield stress, the strength is a combination of cementation and the effects of soil fabric.

Kwan et al. (2005) performed a research program for the suitable ground improvement technique which may be applied on the selected soft clay in Australia. In addition to physical and index properties of the clays, UC tests and oedometer tests were performed on stabilized soils to evaluate the strength increase and the reduction in compressibility. In this study, highly compressible Coode Island Silt (CIS) were improved by C, C+FA, C+BFS mixing. The binder amounts were 10%, 15%, 20% and 30%. For 15%C mixes, FA and BFS were added as 25%, 50%, and 75% of C to the binder. Water/cement ratio by mass are 0, 1% and 2%. Strength of treated soil increased along with the number of curing days. C+FA was more effective in the improvement than C+BFS mixes. The effective ratio of C and FA is 25% and 75%, respectively. Stabilization with these binders changes the material behavior into a more brittle state. Maximum stress was attained at a lower strain (1.25%) for cement treated samples. The consolidation yield stress was observed in the oedometer test results. Increasing the amount of cement shifted the compression curve towards the higher stress region (yield stresses increased). Therefore it was concluded that a significant improvement on the soil properties can be obtained by 15%C mixing.

Kwan et al. (2005) also strengthened Queensland soft compressible clay by cement mixing. The results of UC tests and oedometer tests were presented for this clay. The UCS of specimens were increased from 27 kPa to 482 kPa with only 5%C addition. For 15%C addition 28 days UCS was reported to be 631 kPa. The oedometer test results were on the order of 700-800 kPa for compression yield stresses obtained with 10%C and 15%C addition, which also shows the considerable compressibility improvement for these binder amounts.

Ahnberg and Johansson (2005) also studies the variation in strength increase when using different types of binder for three group of stabilized soils up to two years after mixing in the laboratory. Various combinations of binders such as C, lime, slag and FA were used in this study. The study was performed on both soft clays and organic soil. Cement, lime, cement/lime, cement/slag, cement/FA, slag/lime were binder mixtures used. The increase in strength of the samples was investigated from UC tests performed on 7, 28, 91 and 364 days cured samples. All mixes showed a considerable long-term increase in strength.

According to Jacobson et al. (2005), drying and re-wetting soils prior to mixing can decrease mixture strength and it confirmed that lime can reduce mixture strength for some soils. For cement-soil mixtures without lime, strength decreases with increasing w/c ratio. In this study, the wet and dry methods for DMM improvement were considered to support the new embankments to be constructed on very soft and highly compressible organic silts and clays. Replacement of C with lime by 25% to 75%C results in a relatively low increase in mixture strength. UC strength plots for different w/c ratios were prepared and used for estimating the amount of cement required to reach a desired laboratory value of the UC strength.

Hernandez-Martinez and Al-Tabbaa (2005) presented UC test results on stabilized peat specimens. Six different cementitious materials (C, cement-bfs, cement-pfa, cement-pfa-lime, cement-pfa-MgO and lime-gypsum-bfs) were used as additive to the peat to increase the UCS and stiffness. In addition, the results of scanning electron micrograph analyses for the stabilized material were presented. From UC tests on the stabilized specimens, the UCS and the initial tangent elastic modulus were presented. The results showed an increase in strength by curing time. It was concluded that cement alone binders are the most effective binder material for strength enhancement of peat soils.

In a laboratory study by Hayashi and Nishimoto (2005), it was reported that in some cases ordinary Portland C may not be sufficient to obtain the desired strength enhancement. If this is case, blast furnace cements must be used. Similarly, Butcher (2005) intended to show the long term performance of DMC improved organic soil using a slag cement binder 4 years after the improvement. This study concluded DMC improvement of organic soils can provide a durable long term solution in case of a correct binder design.

A study by Löfroth (2005) showed the long term performance of the 10 year-old lime-cement columns at two different sites in Sweden. Long-term strength increase in soilcrete and the influence on the surrounding soil was studied. Determination of the calcium content in the clay indicates a slow migration of calcium from the columns to the untreated clay closest to the column. The shear strength of the lime-cement columns was determined by traditional column tests in the field and also by unconfined compression tests on the coring samples in the laboratory. At these two sites old column test results (50 days-old for one and 1 year-old for the other) were also available. The comparison of the results showed that the strength increase continued for the lime-cement columns for long time. The elastic modulus at 50% failure load (E_{50}) values of the cored samples was 220 times the UCS.

All of the above studies reported that the use of binders was effective to a certain extent such that they improve the performance when the appropriate amount of binder is used for long-enough curing times.

2.2 ENGINEERING PARAMETERS OF STABILIZED SOIL

In this section, the studies about the characteristics of the improved soils are provided. The addition of different binder to the natural soil changes the strength and deformation parameters by processes such as ion exchange, cement reactions and pozzolanic reactions.

The addition of cement (C) to natural soils changes their engineering properties through chemical reactions, namely the formation of CSH and CAH (Swedish Deep Stabilization Research Centre Report No. 9, 2001). Uddin et al. (1997), Kamruzzaman et al. (2000), Kasama and Zen (2000) and Miura et al. (2001) reported that addition of C to clay soils for improvement changes the plasticity behavior of the stabilized soil, which also results in a more brittle soil.

Uddin et al. (1997) and Miura et al. (2001) showed that the change in liquid limit (LL) due to cement addition is insignificant. However, the plastic limit (PL) significantly increases with the addition of C. Both studies concluded that the plasticity index (PI) decreases with increasing C content.

Lorenzo and Bergado (2004) conducted UCS and oedometer tests on cement (cement content is 5%, 10%, 15%, and 20%) stabilized soft Bangkok clay with different remolding water contents (100%, 130% and 160%). They stated that the unit weight of the treated soil increases as the cement content increases. This is because the formation of cementing products increases the amount of solids in a unit volume. In the same year, Horpibulsuk et al. (2004) stated that the friction angle considerably increases with addition of small amount of C (6%) to the soil. But further increase of C will not improve the performance significantly.

Massarsch (2005) reported the results of extensive static and dynamic field tests (static loading test, various seismic tests and bender element test) and static laboratory tests (triaxial and direct shear tests) on soft plastic clay improved by dry mixing. The amount of binder used in lime cement columns100 kg/m^3 and 150 kg/m^3. Shear wave velocity, Vs and primary wave velocity, Vp were determined by seismic tests. The deformation properties was assessed from shear wave velocity values by

- $Mmax=\rho.V_p^2$
- $Gmax=\rho.V_s^2$.

Semi empirical relationships were proposed to estimate the shear modulus (G) and E_{50} of fine grained soils as follows;

- G/UCS = 200 (for plastic clays),
- G/UCS = 2000 (for silty clays),
- E_{50}/UCS = 160 (from a scattered range of 240-475).

It was proposed that the modulus values from laboratory tests are about 2 to 3 times higher than those determined by in-situ tests.

Van Impe et al. (2005) performed laboratory tests on the mechanical behavior of dredged sediments stabilized with ordinary Portland cement (C) and blast furnace slag cement (BFC). The UCS was between 4 to 8 kPa. Laboratory UC tests showed that in 2 years, UCS may increase to values between 1500 to 2000 kPa with the addition of 275 kg/m^3 binder (~15%). The results suggested that BFCs have higher potential for the stabilization of soil compared to other binders. In addition, cores taken from the field was also tested in the laboratory. The comparison between UCS of samples obtained from the field to ones in the laboratory yielded the ratio of 2 to 5.

2.3 DEFORMATION CHARACTERISTICS OF STABILIZED SOIL

Terashi and Tanaka (1993) carried out series of model tests in large scale oedometer cell (diameter: 300mm, height: 100 mm) to clarify the consolidation behavior of composite ground. The first series of oedometer tests were performed on soil-cement and soil-lime mixes, where Japanese marine clay was stabilized. They defined a new concept called the consolidation yield pressure or pseudo pre-consolidation pressure (p_y), which is directly proportional to UCS the columns with a ratio of 1.3. The treated soils` coefficient of volume compressibility (m_v) is approaching to that of untreated one because the loading on the composite ground is larger than value of p_y. In addition, large scale oedometer tests were performed on clayey soil containing 15% cement treated core in the middle. The results showed that the compressibility of the treated soil is the same as the untreated one for consolidation stresses higher than p_y.

Kamruzzaman et al. (2000) performed laboratory UC and oedometer tests on cement stabilized Singapore marine clay. It was found that the consolidation properties of the soil are improved greatly by increasing C content. The addition of C increases the pre-consolidation pressure of the stabilized soil. The values of p_y were reported to be 60 kPa, 400 kPa and 1500 kPa for untreated clay, 10% cement treated clay and 30% cement treated clay, respectively. The same phenomenon was also observed in other research studies (Bergado et al., 2005; Lorenzo and Bergado, 2004). The yield compression stress was affected only by the C content. However, Lorenzo and Bergado (2004) stated that the compression index at the post yield state, where compression stress are greater than the yield stress, is effected by the C content. In other words, an increase in a_w results in an increase in post yield compression index. This is mainly because of the excessive yielding of the soil at high stress levels and sudden break of the cementation bonds. Lorenzo and Bergado (2004) also showed that for the same

C content, the post yield compression line is the same for different water contents.

The treated soil will behave as the untreated one in consolidation view of point for consolidation stresses beyond the pre-consolidation pressure (normally consolidated region-post yield compression). The same finding was also reported by Terashi and Tanaka (1993). The same concept is also emphasized in the research for strength characteristics of cement treated soils conducted by Kasama and Zen (2000). They performed unconfined compression and oedometer tests on cement treated clayey and sandy soils (cement content, a_w=5, 7, and 10%, water contents changing from 1.5 to 2.5 w_L for each series). They concluded that the consolidation yield stress and related to it the overconsolidation ratio can be two major factors in predicting the strength of cement treated soil. As determined form oedometer tests on cement stabilized soils the consolidation yield pressure increases by increasing cement content. It is also emphasized in the research that the strength in the overconsolidated zone (the stresses below the yield stress) depends on the stress level, overconsolidation ratio. Although it is not stated by the authors, from the undrained shear strength/consolidation pressure, c_u/p'_c vs. OCR graph presented in the paper, the consolidation yield pressure, p_y/undrained shear strength, c_u ratio is calculated as 2.4-3.0 for different water contents. This means that the p_y / UCS ratio is between 1.2-1.5. This is in good agreement with the value of 1.3 stated by Terashi and Tanaka. The ratio of p_y / UCS is stated as 2.2 by Horpibulsuk (2001) and as 1.5 by Liu et al. (2006).

The uncertainty in the calculation methods for settlement of cement stabilized mass is emphasized in the research by Baker et al. (1997). They performed in situ field load test for measuring the modulus of deformation of a short lime cement column (60 cm diameter, 5 m length) up to failure. As stated in the paper because of full scale experimental difficulties it is hard to obtain the deformation modulus of field cement treated soil mass. By performing a parametric study with 2D FE Plaxis analysis, they concluded that a drastic reduction of settlement can be obtained by good quality (high modulus of deformation) lime-cement columns. They concluded that the deformations in the stabilized clay depends on stress carried by the columns and hence on the quality (modulus) of the columns.

As stated by Horpibulsuk et al. (2004), the cement admixed clay with high cement content, a_w, shows high yield stress and low compressibility with the increasing confining pressure.

Miura et al. (2001) and Balasubramaniam et al. (1999) also stated that the cementation is responsible for the resistance against compression for vertical stresses less than the yield stress, p_y. The change in soil fabric is dominant for the compression behavior for the stresses greater than the yield stress where the cementation bond is broken. It is similar to the case for strength enhancement.

Bergado et al. (2005) stated that the cement content (a_w) specifies the position of slope of the compression line at post-yield state, whereas the yield stress at specific a_w is influenced by the after-curing void ratio (e_{0t}). This phenomenon is already stated by Miura et al. (2001) by the wc/c (water content/cement content ratio) value. They determined that the lower the wc/c, the greater the yield stress.

Hayashi et al. (2005) reported that consolidation characteristics of the cement-treated soils are affected by the delay of consolidation loading. Consolidation tests were performed on cement stabilized low liquid-limit silt. Water content is 170% and cement content is10% (water-cement ratio of 100%). As consolidation loading was delayed, the settlement strain became smaller but the consolidation yield stress became greater. This phenomenon is because of the cementation.

Bai et al. (2001) performed a loading test on trial cement mixed columns at a foundation site in China. The 0.4 m diameter, 8m long soil-cement columns (a_w=16%) were loaded vertically at the column center (incremental loading, 120 min between load steps) and column settlements were recorded. Axisymmetric finite element model were prepared and analysis were performed using ABAQUS computer program. The variables in the analyses were column dimensions (diameter=0.3, 0.4, and 0.5 m; length=5, 8, and 11 m), replacement ratio (a_s=0.0816, 0.145, and 0.227), column/soil modular ratio (M_{col}/M_{soil}=5, 10, 20, and 50), and load intensity (p=50, 100, 150, 200, 30, and 400 kPa). The field results are in good agreement with the analysis for column/soil modular ratio of 20. The load distribution and settlement behavior of soil-cement columns were discussed. The settlement of the columns decreases with increasing replacement ratio and modular ratio. Similarly the load on the column will get larger with increasing modular ratio and replacement ratio. As the columns get stiffer and closer the system have greater load resistance and transfer less stress to the surrounding soil resulting in less settlement.

Indraratna et al. (1995) performed oedometer tests on cement-FA stabilized soft Bangkok clay. The deformation properties of the soil are not changed substantially with 5% cement treatment. But the addition of small amount of cement (5%) and FA (greater than 10%) improved the compression behavior of the soil (substantial reduction in compression index, increase in the yield stress). The yield stress of the natural soil is increased from 80 kPa to 300 kPa by the addition of 5% cement and 25% FA. The compression index also decreases with the increasing FA content. The coefficient of consolidation is increased 15-20 times by high (18% to 25%) FA content with 5% cement. These improvements are related to the pozzolanic activities of the binders.

Bergado et al. (1993) gave the bearing capacity, settlement and stability evaluations on DMM improved foundations of highway embankments in Thailand. The measured surface settlements of DMM improved soft soils was agreed well with the values predicted using the conventional method (conventional settlement calculation of untreated ground times the settlement reduction factor which is equal to the ratio of E_{soil} to E_{system}) and also FEM analyses.

Miki and Furumoto (2000) conducted large scale laboratory model tests about the settlement of DMC supported embankment loading to evaluate the stress concentration ratio (the ratio of vertical load acting on improved part to the vertical load acting on the unimproved part). The improvement ratio and settlement values with respect to stress concentration ratio were obtained. According to the test results the stress concentration ratios are obtained as 5 to 20. As the height of embankment (vertical stress on the improved system) and improvement ratio was increased, the stress concentration ratio was increased. As a result, the researchers concluded that the DMM with low improvement ratio can be used as an economical way of improvement depending on site conditions.

Alen et al. (2005) performed settlement measurements on field trial lime/cement column stabilized soft clay at four different sites in Sweden. The settlement calculations according to the traditional method (calculations using the bulk modulus of the stabilized system found using the modulus of column, modulus of soil, and the replacement ratio) overestimated the real measured values. This is based on the underestimation of the modulus values of the whole system.

Bergado et al. (2005b) monitored a full scale DMM improved soft clay ground and investigated the compression mechanism of the system under bridge approach embankment in Thailand. Full scale embankment loading on soil-cement columns constructed by jet-mixing method of diameter 0.5 m, length of 9 m, and spacing of 1.5 m was monitored up to one year. According to the results the settlement of the soft clay under embankment loading was reduced by at least 70%.

The stress concentration ratio phenomenon is also investigated by Yin and Fang (2010) by large scale laboratory model tests. From the instrumentation of plane strain physical model created for the investigation of the bearing capacity and failure mode of soft soil improved by end bearing group of deep mixed columns (DMC), the researchers obtained the stress concentration ratio with respect to vertical displacement. According to the test results, the average stress on the columns (and stress concentration ratio) increases to a peak and then gradually decreases to a residual value with displacement. The peak and residual values of the stress concentration ratio for soft clay improved with DMC with replacement ratio of 12.6% was obtained as 11 and 7, respectively.

CHAPTER 3

EXPERIMENTAL SETUP AND PROCEDURE

In this chapter, the details of the laboratory experiments were described. First, the selection of soil materials to be used in this research is provided. Then, the procedures applied to prepare soil specimens are explained step by step. The characteristics of different binders are also given in this chapter. Next, the particulars of unconfined compression (UC) tests and large scale consolidation tests are given; the steps to prepare soil samples and performing the experiments are enlightened. Finally, the summary of the laboratory work is provided at the end.

3.1 MATERIAL SELECTION AND SAMPLE PREPARATION

3.1.1 Natural Soft Soil

The selection of weak soil to be improved was the first step of this study. Based on the results of a previous work (Özkeskin, 2004), the soil samples were taken from Eymir Lake-Ankara, where the soil type was reported to be low plasticity clay (CL). The site nearby the lake area was excavated by hand; then the soil samples were carefully taken from the depth of 2 m and transferred to the Soil Mechanics Laboratory.

The processing of natural soft soils was as follows: The extracted soil samples were dried in the oven at 110^{o}C for 24 hours. After drying, the bulk mass was broken into pieces by tampering to increase its workability. Then cobble and boulder size particles were removed from the soil mass. Standard classification tests (Specific Gravity Tests, Sieve Analysis, Hydrometer Test, Atterberg Limit Tests) were performed on the cleansed soil for identification purposes. Using the above laboratory tests G_s was found to be 2.66. The liquid limit (LL) was 31, and the plastic limit (PL) was 18%. Therefore the plasticity index (PI) of the soil was calculated to be 13. These results verified that the soil is low-plasticity silty-clay (CL), as mentioned in the previous study (Özkeskin, 2004). This soil obtained after physical processes were used at the initial stages of laboratory experiments, specifically in UC tests. The samples used in UC tests were sieved using No. 4sieve (5mm sieve opening).

3.1.2 Kaolinite Clay

For the second stage of this research, to fill the large consolidation tanks, large amount of soil was needed. One tank is generally filled by about 70 kg of soil slurry with water to solid ratio (W/S) of 0.7. Since there were several tanks to be used in the experiments, total of 1500 kg of soil would be needed during the research. As it was difficult /impractical to find and transport such soil masses from the natural deposits, commercially available industrial soils were preferred for practical purposes. Among those, the most suitable one to simulate clay behavior was kaolinite clay due to the less expansive character of the kaolinite mineral.

The mineralogical and chemical properties of the kaolinite used in this study are given in Table 3.1.The standard physical tests were also repeated for kaolinite.

Accordingly, 90% of kaolinite is of clay size. There were no remaining particles detected on No.200 sieve; therefore, the remaining 10% was considered to be silt. The PL of kaolinite was 33 and LL was 49 (PI of kaolinite was calculated to be 16).

Table 3.1 Mineralogical and chemical composition of kaolinite used

Mineralogical Structure	**Volumetric Content**	**%**	**Chemical Analysis**	**%**
kaolinite	Clay Mineral	90.5	(loss on ignition)	12.73
Quartz	Free Quartz	2.71	SiO_2	47.89
Illite	Sodium Feldspar	0.08	Al_2O_3	36.75
	Potassium Feldspar	4.45	TiO_2	0.61
			Fe_2O_3	0.40
			CaO	0.39
			MgO	0.09
			Na_2O	0.01
			K_2O	0.75
			SO_4	0.37

3.1.3 Binder Materials

The selection of binder materials was an important stage of this research. Both selection of the binder material and determination of the exact amount to be used as an additive were critical as they directly affect the structural performances of the ground improvement method. Considering these, several of those improvement materials were collected. First, the ordinary Portland cement (C) was chosen since it improves the strength of natural soil dramatically. This material was used throughout the testing program. Typical mineralogical composition of ordinary Portland cement used in this study is given in Table 3.2.

Table 3.2 Mineralogical composition of ordinary Portland cement used

Oxides	Amount, %
Calcium Oxide (CaO)	65
Magnesium oxide (MgO)	3
Aluminum oxide (Al_2O_3)	6
Ferric oxide (Fe_2O_3)	3
Silicon dioxide (SiO_2)	20
Sulfur trioxide (SO_3)	2.5

The second alternative to be used as an additive into soil mix was chosen considering environmental effects and the overall cost of the proposed solutions. The previous research (Aydilek, 2004; Zorluer and Usta, 2003; Yaprak et.al., 2004) showed that industrial by-products are effective way of increasing the strength of soil. In Turkey, fly-ash (FA) material is generally used as an effective way of increasing strength; and it is plenty in the local market. Using FA decreases the total cost of deep-mixed columns as it reduces the amount of cement used. Another alternative considered as an additive was to use marble-dust (MD). MD has also been used in improvement studies although its use is not as frequent as FA. Both FA and MD materials have calcium-oxide (CaO) content, which results in pozzolanic reaction when interacted with soil.

The FA used in the experiments was taken from a coal-fueled power plant located in Manisa-Soma. The mineralogical analysis was performed in the laboratories of General Directorate of Mineral Research and Exploration (Maden Tetkik ve Arama Genel Müdürlüğü – MTA). The results of these analyses are given in Table 3.3. Based on these results, the classification of FA was determined to be Type-C since the total proportion of Al_2O_3, SiO_2 and Fe_2O_3 exceeded 70%. The FA material was not preprocessed at all before using as an additive material.

Table 3.3 Mineralogical composition of the FA used

Oxides	Amount, %
Silicon dioxide (SiO_2)	48.2
Aluminum oxide (Al_2O_3)	22.3
Calcium Oxide (CaO)	15.8
Ferric oxide (Fe_2O_3)	5.3
Magnesium oxide (MgO)	1.2
K_2O	1.2
TiO_2	0.8
Sodium Oxide (Na_2O)	0.5
P_2O_5	0.2
BaO	0.09
SrO	0.06
ZrO_2	0.04
MnO	<0.1

Similar to FA, the marble dust (MD) used in this study was obtained from a local marble processing unit in Ankara. The mineralogical analysis was performed in the laboratories of General Directorate of Mineral Research and Exploration (Maden Tetkik ve Arama Genel Müdürlüğü – MTA). The results of this analysis is given in Table 3.4. MD material was first dried, and then it was grounded by hammering. MD was only used in UC tests and in the powder form.

Table 3.4 Mineralogical composition of the MD used

Oxides	Amount, %
Silicon dioxide (SiO_2)	0.2
Calcium Oxide (CaO)	56.2
Magnesium oxide (MgO)	0.2
Al_2O_3, Fe_2O_3	0.1
K_2O, Na_2O, TiO_2, Pb_2O_5	<0.1

3.2 PREPARATION FOR UC TESTS

The objective of the UC tests on improved CL is to find the proper binder type and to determine its volume when mixes with the soil to supply the desired strength. The UC tests were performed on CL-type clay mixed with (i) cement (C), (ii) cement and fly-ash (C+FA), and (iii) cement and marble dust (C+MD). The cement content (a_w), i.e., the ratio of dry weight of cement to dry weight of soil, was chosen to be 5%, 10%, and 15% throughout the experiments. The prescribed amount of clay, cement, and FA/MD is mixed in dry powder form without compaction. When mixed with 5%C, the amount of FA added, i.e., the ratio of dry weight of additive to that of soil, was selected to be 8%, 15%, and 20%. These quantities were kept the same when MD was used as an additive in addition to C.

The method to prepare the improved soil specimens in the laboratory was standardized by the Japanese Geotechnical Society (JGS, 2000). This standard describes a procedure of making and curing a cylindrical specimen of treated soil without compaction. Following this standard, the soft soil, initially, was sieved using No. 4 sieve. Then soil mixture was prepared through mixing soil, water and the stabilizing material by means of an electric mixer; the natural soil was mixed with the stabilizing agents in dry powder form and then water was added thoroughly to achieve W/S = 0.7. The duration of mixing was 10 minutes to supply homogeneity in the soil mix as recommended in the literature (JGS, 2000). However, after 10 minutes, the binders were susceptible to hardening.

The prepared mixture was placed in cylindrical PVC molds (Diameter: 50 mm, Height: 100 mm) with a special injection system to fill the mold from bottom to the top without having air bubbles and voids. The inside of the molds were lubricated in advance to make extrusion of the soil easier at the end of initial setting time. The mold was vibrated slightly by hand to remove the entrapped air bubbles. The specimens prepared in this way were then cured in the moisture room where the temperature was kept 20 ± 3^{0}C and relative humidity was 95%.

Initial setting was achieved after 2 to 3 days. Then the mixture was removed from the molds and trimming was done to have smooth boundaries. The mixed samples were then put in special moisture bags in the moisture room and kept closed till the end of the desired curing period. The pre-determined curing times used in this study were 7, 28, 90 and 360 days. The cured specimens of CL and kaolinite were then tested in the UC testing machine under 0.5 mm/min loading rate. The results of these tests to determine the most suitable binder and its volume in a given mix are presented in chapter 4.

3.3 PREPARATION FOR LARGE SCALE CONSOLIDATION TESTS

The second stage of laboratory experiments was large scale consolidation tests on reconstituted soft kaolinite clay improved with deep mixed group of columns (DMC). There are three main stages in large-scale experiments (1) preparation of soft clay (2) preparation of deep mixed columns and (3) performing consolidation tests to determine the deformation characteristics of the stabilized soil.

3.3.1 Preparing kaolinite for large scale consolidation test

Dry kaolinite in the powder form was mixed with water using large scale electric mixer to have a water content of 40%, almost at the LL. As in preparation of soil samples for UC tests, the clayey mass of soil was put in plastic bags and kept in the moisture room for 2-3 days to have homogeneous water and soil mix. Then clay soil was put into large tanks where consolidation test will be performed.

The diameter and the height of the consolidation tanks are 41 cm and 39 cm, respectively. The height of the clay sample that was put into tank was around 30 cm. There are holes at the bottom plate of the consolidation tank to allow drainage. Each consolidation tank was then placed in a plastic bath tub to have continuous water supply. When placing the kaolinite in the tanks, small lumps of clay was placed and spread with hand to avoid air bubbles and cavities. When the placement was done, the total weight of the material in the tank was 63 kg.

To avoid drying of clay paste and also to allow drainage path, the bottom and top of soil mass was covered with filter paper and geomembrane covers. The whole setup was left untouched for about 3 days to allow consolidation under its own weight (An overview of the setup for one of the large scale consolidation tanks and the equipment used to prepare group of columns are shown in Figures 3.1 and 3.2, respectively).

Figure 3.1 Consolidation tank filled with kaolinite

Figure 3.2 An overview of the equipment used in the tests

Next, the 5 mm thick-loading plate was placed on top of the initially consolidated specimen. Then an air piston with 100 mm diameter was placed on the loading plate and 50 kPa consolidation pressure was applied to the system using an air compressor. The air pressure was susceptible to changes due to several reasons and therefore regulated through a regulator. The regulator was connected to compressor and its performance during the consolidation was observed using a pressure dial gauge (Figure 3.3) placed on the top of the loading plate (Figure 3.4). Using this setup, the consolidation was completed in about 25-30 days for each specimen (Figure 3.5). The preparation of kaolinite was the same for different mixes. Therefore, it was assumed that the compression modulus of the soil for different mixes would be the same.

However, the modulus of the deep mixed columns and the improved soil would vary based on different the column materials and binder type.

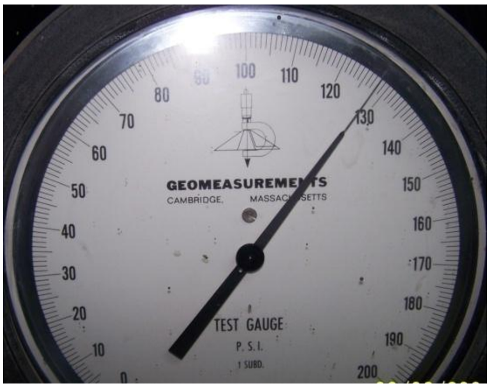


Figure 3.3 The air pressure regulator (from the compressor to the air pistons)

Figure 3.4 The dial gauge checked consolidation under 50 kPa loading

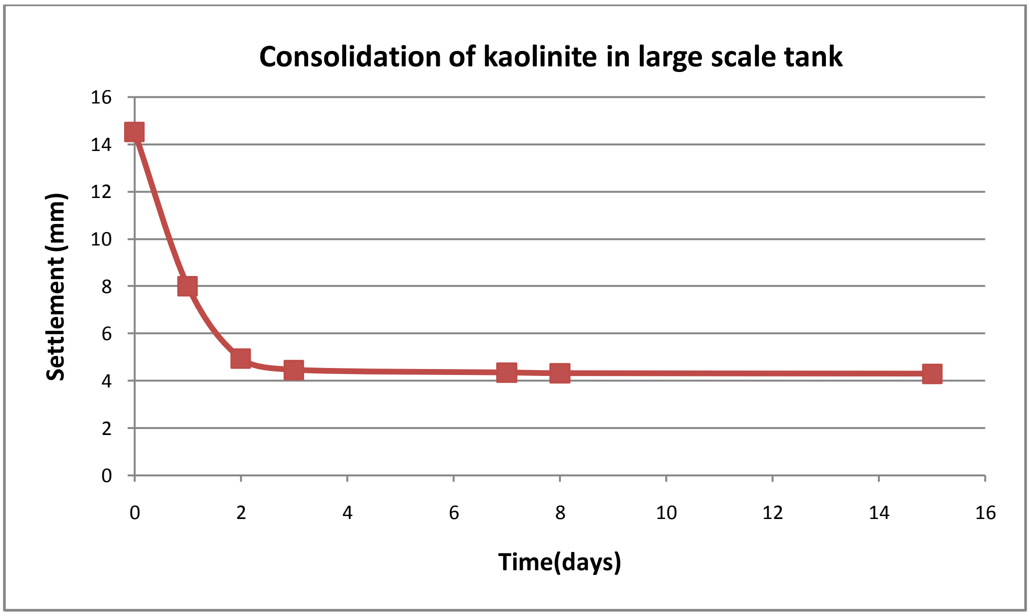


Figure 3.5 Typical consolidation curve of kaolinite in the large scale consolidation tank

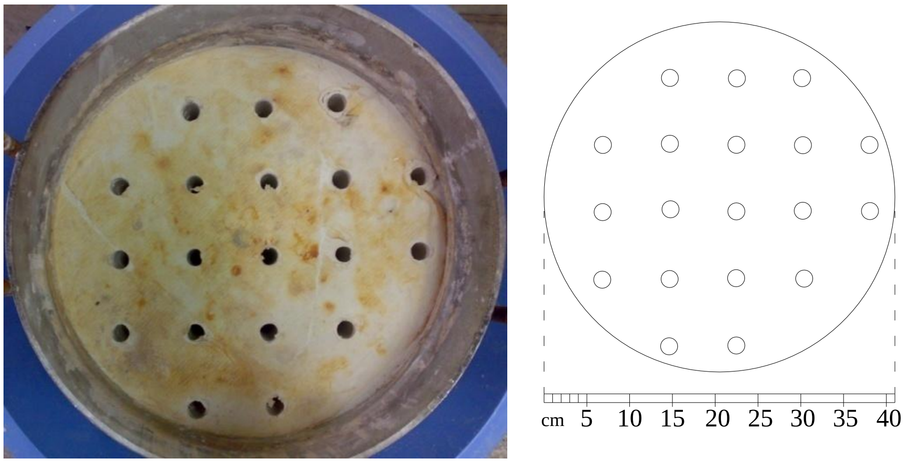
After the consolidation was completed, the loading mechanism was taken off. The surface of clay soil was flattened through a trimmer and some of the soil was removed to bring its height 25 cm (Figure 3.6). The soil was ready for constructing DMC inside that will be explained in the next section.

Figure 3.6 The leveling and height adjustment of clay in the tank

3.3.2 DMC construction

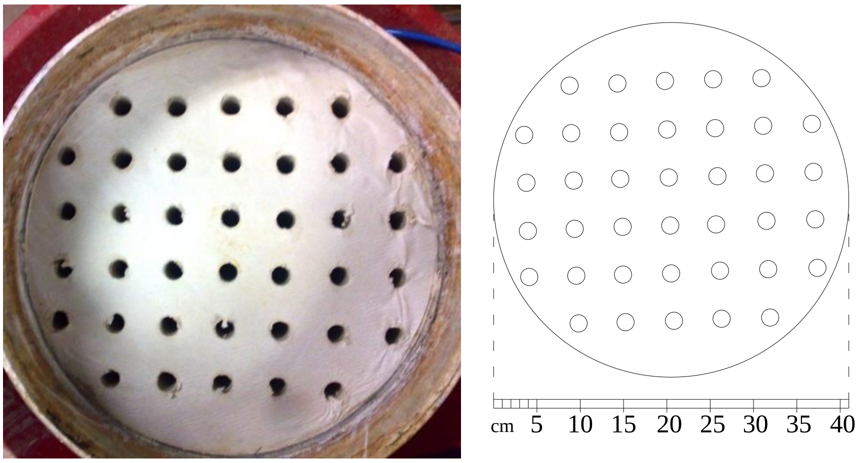
In this research, the performances of several pile configurations were investigated. They were created using different number of columns, specifically 19, 38, 55, and 85 piles. The replacement ratios corresponding to these column configurations were 0.045, 0.09, 0.13, and 0.20, respectively (these configurations are shown in Figures 3.7 to 3.10). There exist 3 types of guide plates (pre-bored steel plates) for drilling operations with 38, 55, and 85 punched holes on them.

The DMCs were prepared using a technique similar to the ones used to construct bored piles. Before drilling, the guide plates that were used to assure the accuracy of geometry were placed carefully on top of the consolidated soil. The holes opened to build DMC in the soft clay were prepared using a standard electric-hand drill. The verticality of the operation is maintained by using the guide plates. Two identical steel guide plates with 20 cm distance in between is placed on the top of the surface and the drill bit is passed through these plates and the vertical boring is performed. The auger of the drill is continuous (length: 40 cm, diameter: 2cm). The drill bit for wood has sharp sides to minimize the sample disturbance during drilling. To not to leave any soil material at the end of the hole during the boring operation, the conical end of the drill bit is cut perpendicular to the axis of the drill bit. The picture of drilling operation is shown in Figure 3.11.



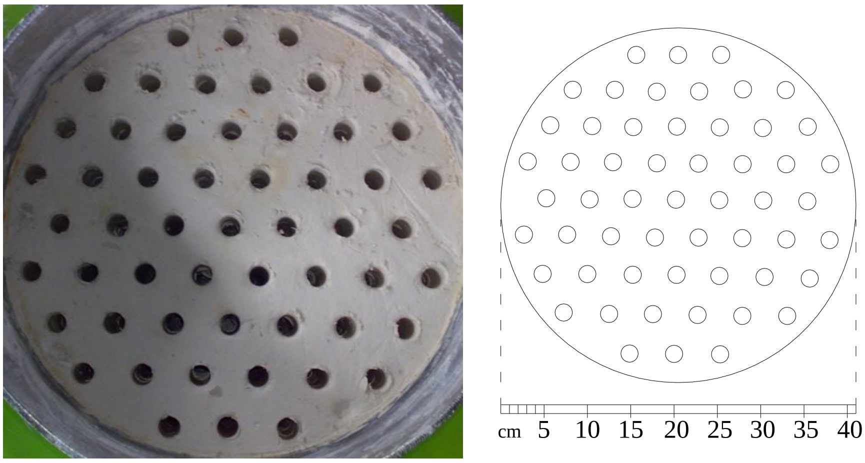
(a) Real Medium (b) Scaled Drawing

Figure 3.7 The plan view of 19 column system (replacement ratio, as = 0.045)



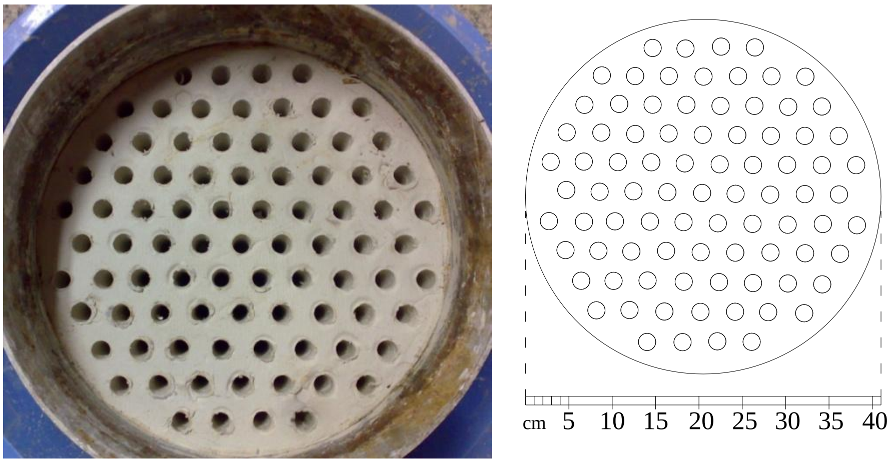
(a) Real Medium (b) Scaled Drawing

Figure 3.8 The plan view of 38 column system (as = 0.09)



(a) Real Medium (b) Scaled Drawing

Figure 3.9 The plan view of 55 column system (as = 0.13)



(a) Real Medium (b) Scaled Drawing

Figure 3.10 The plan view of 85 column system (as = 0.20)

Figure 3.11 Drilling operation

The pile material was then injected into pre-bored holes using a special injection system. The mix material was filled in the shaft (bucket) of the system and the holes were filled up through inserting the 30 cm long hose with 1.5 cm outer diameter to the bottom of each hole. The operation for filling from bottom to top (Figure 3.12) was very similar to the Tremie pipe method that is frequently used in the field applications. Next, the columns were left to rest, i.e., without loading, for 7 days for the initial setting of the binders (Figure 3.13). Finally, 50 kPa loading was kept constant through the air jacked system for an additional 21 days. This was done to guarantee 28 days for the setting time of cement mixes.

Figure 3.12 Filling operation

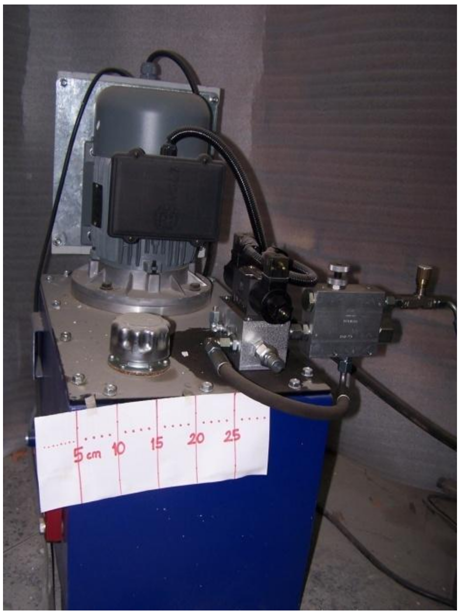
Figure 3.13 The top view after the formation of the piles

3.3.3 Performing the consolidation test

The consolidation test was performed using a loading mechanism that was specifically designed for this research. It is composed of an oil pressure supplier (Figure 3.14a), a pressure transducer, a loading piston (diameter 80 mm) and a process controller (Figure 3.14b). The process controller maintains the pre-set oil pressure until a next stage of loading is desired. This mechanism is capable of applying 150 bar oil pressure, i.e. 6 kg/cm^2 on 41 cm^2 area.

In order to perform the consolidation test, the previously applied consolidation pressure on the deep mixed soil mass was first removed. The top loading plate was then changed with a more rigid one which is 20 mm thick and stiffened with vertical steel strengtheners, to resist the higher amount of loads without bending. The loading mechanism designed for large scale consolidation tests was put on the top plate. In addition to the loading mechanism, three dial gauges were placed on the loading plate with a radial angle of 120 degrees among them. Similarly, three LVDTs were placed next to the dial gauges. The whole experimental setup to apply loading is shown in Figure 3.15.

Before starting the test, the laboratory testing software CODA was started to control the hardware, to initialize the readings of LVDT, and to record the findings of the software (Figure 3.16). The application of load was done through the load mechanism that was mainly controlled by the process controller. The controller was adjusted using preset values of oil pressure for loading increment of 0.5, 1, 1.5, 2, 2.5, 3, and 4 kg/cm^2 (The oil pressure is calculated using a simple load conversion between the loaded area and area of loading piston). The pressure transducer sends signal to the pressure supplier to start loading. When the desired oil pressure, i.e., loading pressure, is reached, the process controller sends another signal to the pressure supplier and it stops. Using this mechanism, the load was maintained on the system for the desired duration until consolidation for that loading step is completed (about 24 hours).



(a) Oil pressure supplier

(b) Process controller to maintain the desired load throughout each load step (Step loading)

Figure 3.14 The components of consolidation loading mechanism

Figure 3.15 The assembled system of test

During the test, the settlement readings were taken from LVDTs using the CODA software. “Settlement vs. time” plots were prepared in order to check completion of consolidation (time to complete the consolidation was determined to be 24 hours for each loading step). As a result, each test was completed in about one week.

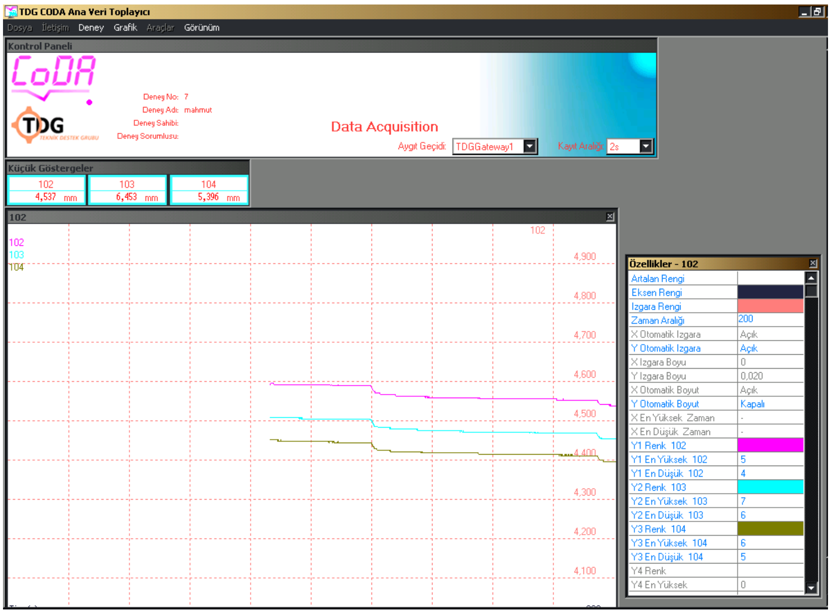


Figure 3.16 CODA interface

The deformation (settlement) values of the mixed soil system were obtained for different consolidation stresses. These values were used to obtain the stress-strain relations. Using these, the bulk compression moduli for each test were calculated, the relation between the bulk compression modulus and the type of columns, the effect of replacement ratio (number of piles, a_s), and also the compression behavior of DMC stabilized soil mass was investigated.

3.4 SUMMARY

In this chapter, the specifics of material selection, the preparation of soil materials, DMCs and binders were discussed. The steps of all the tests were given in detail to highlight the details as they were difficult to perform and repeat. The results of both UC and large scale consolidation tests are discussed in the next Chapter.

CHAPTER 4

EXPERIMENTAL RESULTS AND DISCUSSION

In this chapter, the results of the unconfined compression (UC) tests and large scale consolidation test are provided together with the discussion about the effects of improvement. In section 4.1, UC tests performed on CL-type clay and kaolinitekaoliniteite clay are given. The development of unconfined compressive strength (UCS) with time is investigated. Then, the relation between the elastic modulus at 50% of the failure load (E_{50}) and UCS is highlighted. Finally, the determination of effective binder mixes, i.e., the type and the amount of binder, is discussed. In the next section, the results of large consolidation tests are presented. Deformation properties of the soil improved by end-bearing group of columns are studied through investigation of relations among the physical variables. Finally, the conclusive remarks for the performance of soil systems improved by using Deep Mixing Method (DMM) are given at the end of the chapter.

4.1 UC TESTS FOR DETERMINING EFFICIENT BINDER TYPE

4.1.1 UC Tests on improved CL

In order to understand the performance of different improvement materials, first, the development of UCS and E_{50} with time is investigated. For this purpose, the results of UC tests performed on soil systems improved with these materials are provided in Table 4.1. The results include the values of UCS and E_{50} that were recorded for different curing times. The data shows that UCS and E_{50} reached their maximum values (376 kPa and 53.7 MPa, respectively) after 90 days of curing time for soils mixed only with cement. When the curing is continued, UCS decreased about 20% after a year. Similarly, the decrease in E_{50} was about 30% as compared to its maximum for the same mix after a year. This may be because of calcium (Ca^{+}) ions leaching towards the outer boundary of the treated samples.

Table 4.1 Results of UC tests on CL improved with different binders

Binder Type	Curing Time (Days)	UCS (kPa)	E_{50} (MPa)	E_{50}/UCS
5%C	7	198	22.5	114
	28	269	41.9	156
	90	376	53.7	143
	365	300	37.5	125
5%C+8%FA	7	115	15	130
	28	255	41.6	163
	90	340	32.7	96
5%C+15% FA	7	290	32.2	111
	28	366	38.1	104
	90	381	38.2	100
	365	645	93	144
5%C+20%FA	7	263	31.3	119
	28	372	51.7	139
	90	514	64.3	125
	365	630	126	200

Table 4.1 Results of UC tests on CL improved with different binders (ctd.)

Binder Type	Curing Time (Days)	UCS (kPa)	E_{50} (MPa)	E_{50}/UCS
5%C+8%MD	7	200	32.25	161
	28	260	27.1	104
	90	320	40	125
5%C+15%MD	7	217	36	166
	28	385	47.1	122
	90	269	27	100
	365	385	38	99
5%C+20%MD	7	245	49.2	201
	28	329	66	201
	90	238	40	168
	365	480	91.7	191
10%C	7	493	117.6	239
	28	684	155.5	227
	90	1033	215	208
15%C	7	620	193.8	313
	28	1051	328.1	312
	90	1450	500	345
CL consolidated under 50 kPa	-	25	0.46	18.4

Figure 4.1 shows the unconfined compression strength of binder mixed CL with respect to curing time. In general, because of the pozzolanic character of the binders (CaO present in cement, fly-ash, marble dust) the UCS increases with the curing time. Cement shows substantial increase in UCS by time. The UCS at 28 days is 1.36, 1.39, and 1.7 times the UCS at 7 days for 5%C, 10%C, and 15%C, respectively. The ratio of $UCS_{,90\ days}/UCS_{,7\ days}$ is 1.9, 2.1, and 2.34 for 5%C, 10%C, and 15%C, respectively. From the Figure 4.1 and also from these ratios it is clearly seen that as cement content (a_w) increases the UCS increases

as expected. Generally, 5%C admixture gives about 300-400 kPa compressive strength. This value is changing between 500-1000 kPa, and 600-1400 kPa for 10%C and 15%C addition, respectively.

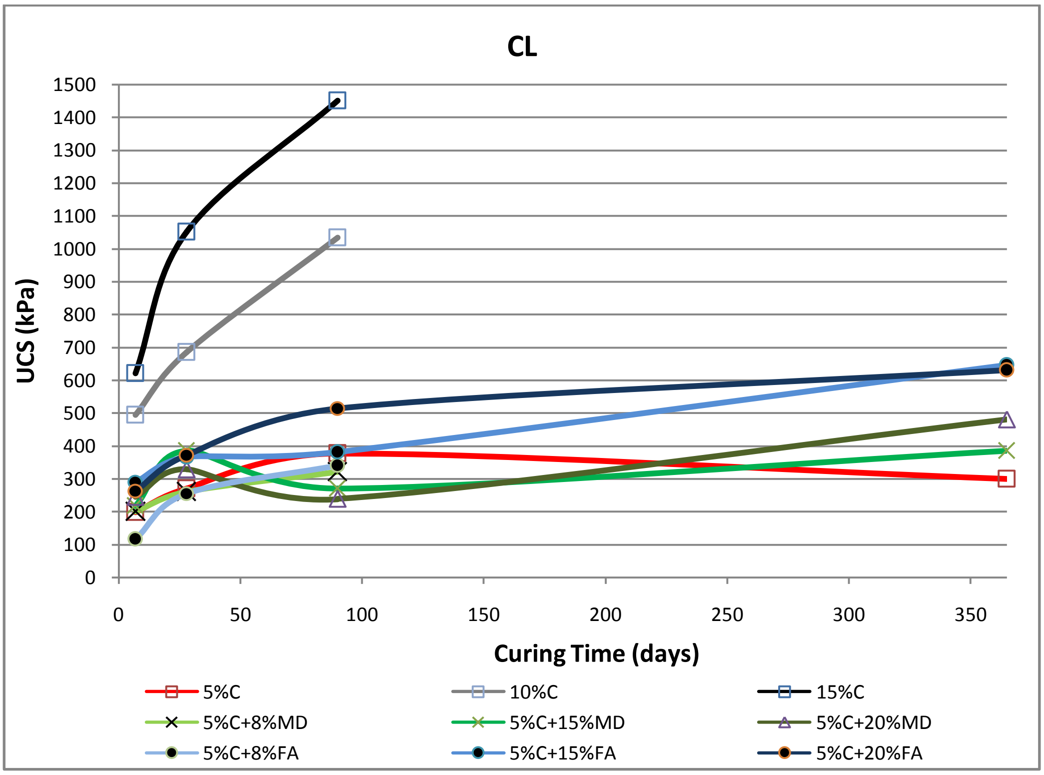
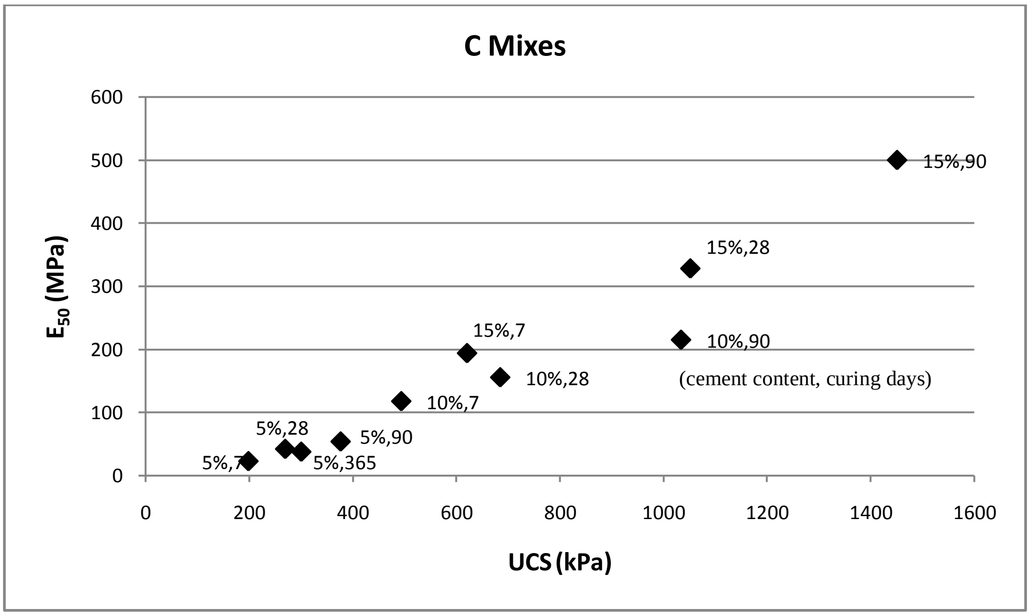


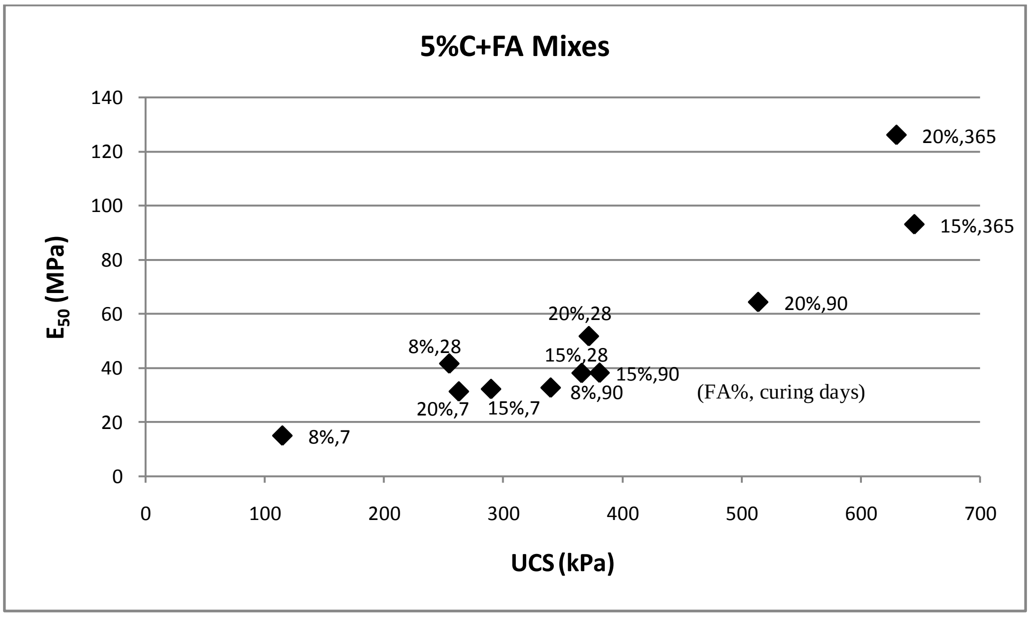
Figure 4.1 UCS vs. curing time for different mixes (CL)

The strength enhancement can also be achieved by adding 5% cement and some percent of (8% to 20%) fly-ash and marble dust (Vazquez, 1991; Indraratna et al., 1995). The binders with the marble dust show a decrease in 90 days time, but in 1 year period it is also effective in increasing the strength. Generally, the compressive strength values for fly-ash and marble dust added mixtures are changing between 200-600 kPa. The curing time does not cause substantial increase in UCS for fly-ash and marble dust added mixtures except the 15%FA and 20%FA additions. For 5%C+20%FA mixes the UCS at 365, 90, 28 days curing time is 2.4, 1.95, 1.41 time $UCS_{,7\ days}$, respectively. The same factors are 2.22, 1.31, and 1.26 for 5%C+15%FA mixes. As it is understood from these values especially fly-ash is an effective additive in strength enhancement of cement treated soils. 5% cement with the addition of 15% and 20% fly ash gives relatively high strength in 28, 90, and 365 days. These findings are in good agreement with the statements of Vazquez (1991), Mohamed and Hossein (2004), Consoli et al. (2001), and Indraratna et al. (1995). Briefly, fly-ash is an effective binder also for long term strength enhancement.

Another important physical property of binder mixed soils is the modulus of elasticity. In this respect the ratios of E_{50} to UCS for different mixes are presented in Figure 4.2.a and 4.2.b. As can be seen, although there is some scatter, the E_{50} can be correlated to the UCS. E_{50} of the original soil was 18.4 times the UCS. The cement mixes give higher ratio (E_{50} is equal to 284UCS for C mixes and 141UCS for C+FA mixes). These values are comparable with the values given in the literature (220UCS, Löfroth, 2005; 160UCS, Massarch, 2005; 110UCS, VanImpe, 2005; 50-200UCS, Bruce et al., 1999; 30-300UCS, Fang et al., 2001; 350-1000$UCS_{,lab}$, 150-500$UCS_{,field}$, Bruce, 2001; 350-1000$UCS_{,lab}$, FHWA, 1999).



(a)



(b)

Figure 4.2 E_{50} vs. UCS for a) cement mixes b) cement+fly-ash mixes

In Figure 4.3 the E_{50}/UCS ratio against curing time is shown. No consistent trend for cement mixes can be seen but the cement+fly-ash mixes show similar behavior especially for 15% and 20% additions.

Generally, 15%C mixes give a ratio of E_{50}/UCS as 300-350. This ratio is about 200-250 for 10%C mixes. For the other mix types (5%C, 5%C+FA, 5%C+MD), E_{50} is about 100-150 times the UCS. For 5%C+20%FA mixes in one year time the E_{50}/UCS ratio reaches the value of 200.

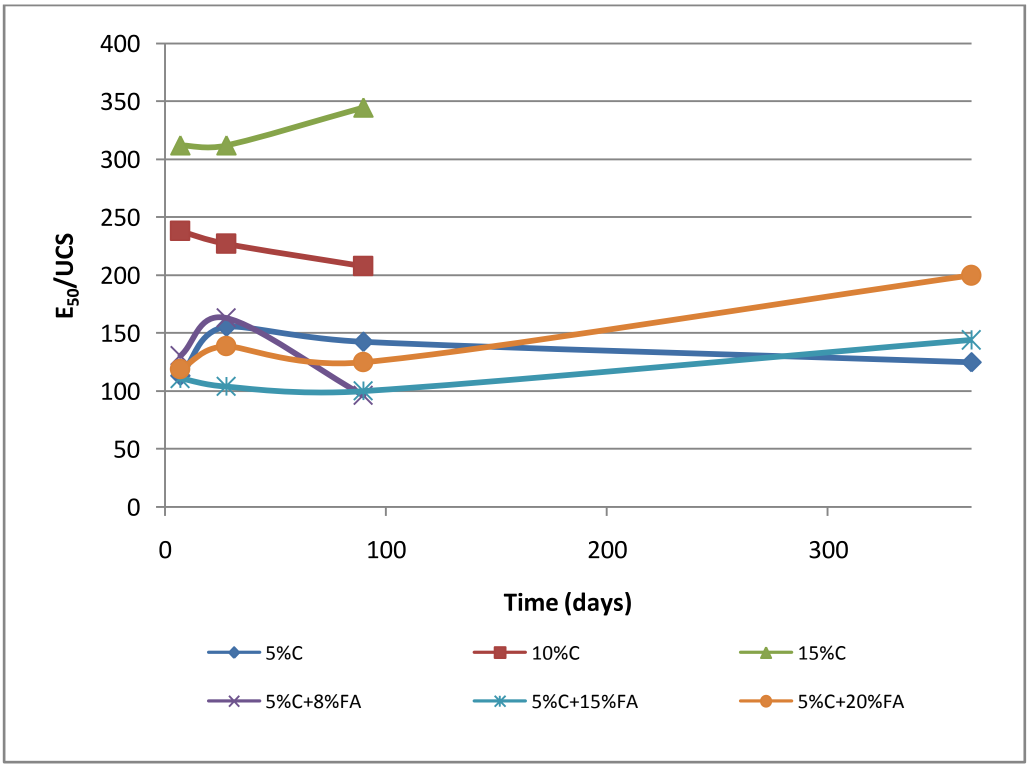


Figure 4.3 E_{50}/UCS vs. time for C and C+FA mixes

4.1.2 UC Tests on improved kaolinite clay

The efficient binder types for the clayey soils considered in the experimental program is determined as 5%C, 5%C+20%FA, 15%C, and 30%C on the basis of the first series tests on CL. The 7-28-90 days unconfined compressive strength values (UCS), and elastic moduli at 50% failure load (E_{50}) for different kaolinite-binder mixes and also for 28 days consolidated (consolidation pressure of 50 kPa) kaolinite clay (taken from the consolidation tank) are given in Table 4.2.

Table 4.2 Results of UC tests on kaolinite improved with different binders

Binder Type	Curing Time (Days)	UCS (kPa)	E_{50} (MPa)	E_{50}/UCS
5%C	7	100	31.2	312
	28	165	105	636
	90	220	129	586
	365	115	25	217
5%C+20%FA	7	108	34	315
	28	330	154	467
	90	560	254	454
	365	710	130	183
15%C	7	225	66	293
	28	400	190	475
	90	680	280	412
	365	755	151.5	201
30%C	7	395	87	220
	28	960	218	227
	90	1440	350	243
kaolinite consolidated in the tank under 50 kPa	28	35	0.3	8.6

Figure 4.4 provides the results of the UC tests on cement and fly-ash improved kaolinite. As the cement content increases the 7-28-90 day UCS increases as expected. Some researchers (Terashi et al., 1980; Kwan et al., 2005; Chew et al., 2004) stated that a minimum amount of 5% cement will be required to improve the strength and deformation properties of soft soils. Figure 4.4 also shows that. 5% cement addition will not improve the UCS substantially. When the cement content increased or another type of pozzolanic binder is used with the cement, the desired amount of strength improvement can be obtained. The most efficient binder in terms of strength enhancement is 30% cement. The compressive strength value for 30% cement mixed kaolinite reaches to about 1 MPa for 28 days curing.

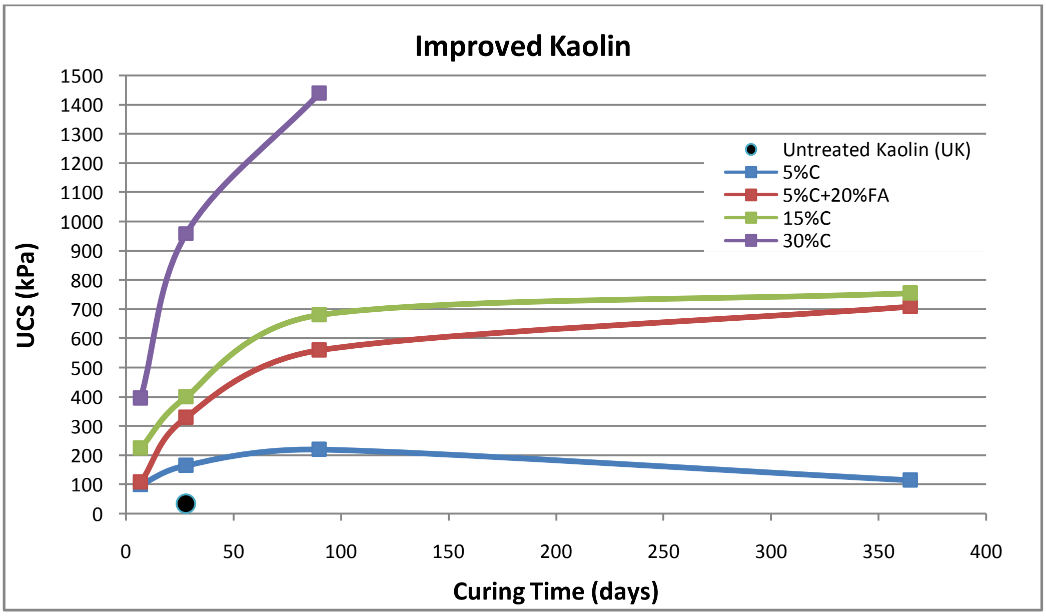


Figure 4.4 UCS vs. curing time for different mixes (kaolinite)

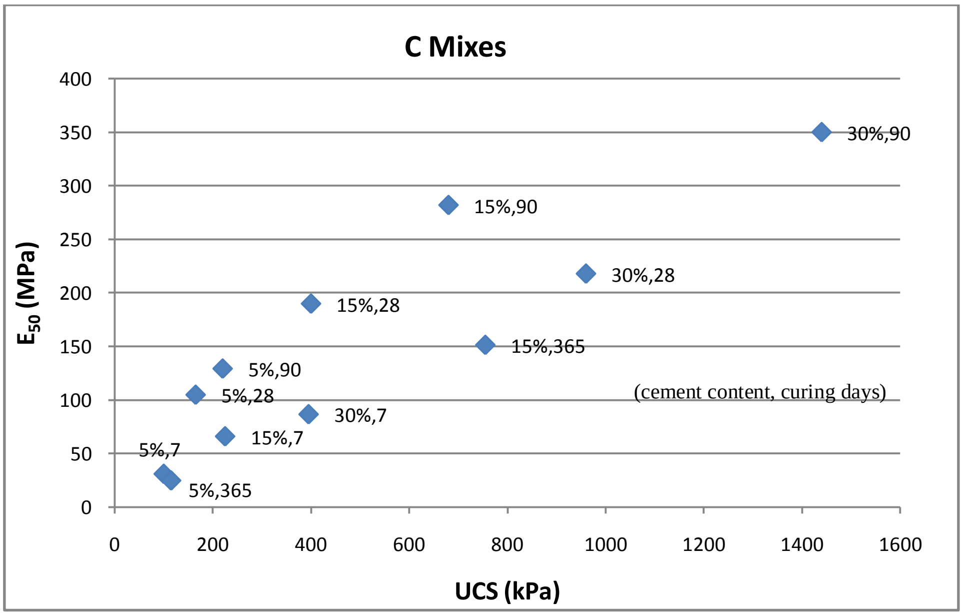
20%FA addition to 5%C is an effective way of improvement as this increased the UCS of improved kaolinite to the level for 15%C addition.

The ratio of $UCS_{,28\ days}/UCS_{,7\ days}$ is 1.65, 1.78, and 2.43 for 5%C, 15%C, and 30%C mixes, respectively. This value is 3.06 for 5%C+20%FA admixture. The UCS at 90 days time is 2.2, 3.02, and 5.19 times the $UCS_{,7\ days}$ for 5%C, 15%C, and 5%C+20%FA mixed soils. This shows the efficiency of fly-ash in long term strength enhancement. The relative increase in compression strength for the improved kaolinite is shown in Figure 4.7.

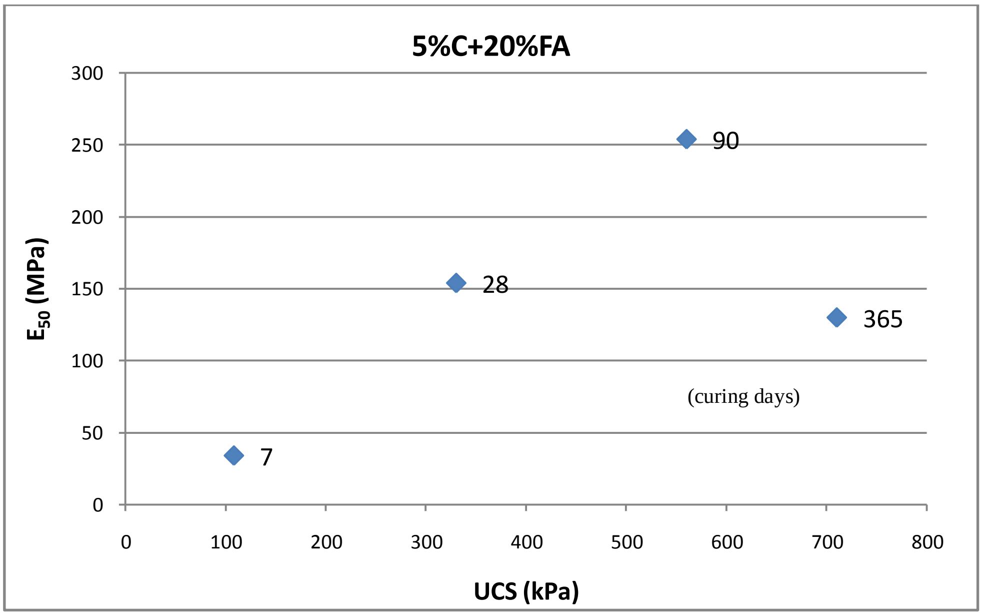
The E values for the improved soils are increased by binder addition. In this respect the E_{50} of the treated soil may be given as a multiple of UCS. Although there is a great scatter in the data for cement mixes, the relation may be given as in Figure 4.5a. The same relation for cement+fly-ash mixes are given in Figure 4.5b.

The ratio of E_{50}/UCS is higher for cement + fly-ash mixes than that of cement mixes.

As it is seen in Figure 4.5b, the 5% cement with the addition of 20% fly-ash mixed soil gives an elastic modulus (254 MPa at 90 days) as much as that of 15% (280 MPa at 90 days) and 30% (218 MPa for 28 days) cement mixed samples. This shows the efficiency of fly-ash addition not only for the improvement of strength but also for the modulus of soft soils.



(a)



(b)

Figure 4.5 E_{50} vs. UCS for a) cement mixes b) cement+fly-ash mixes

E_{50}/UCS ratio vs. time graph is given in Figure 4.6. For all mixes, as this ratio increases up to 28 days, then there is a sharp decrease after this time. This ratio is all about 200 for 5%C, 5%C+20%FA, and 15%C mixes. 5%C+20%FA mixes give the same ratios as that of 15%C mixes. This ratio is around 200 for 30%C mixture. The E_{50}/UCS ratios are lower for higher cement contents. For long term this ratio converges to 200 as in the case for mixed CL.

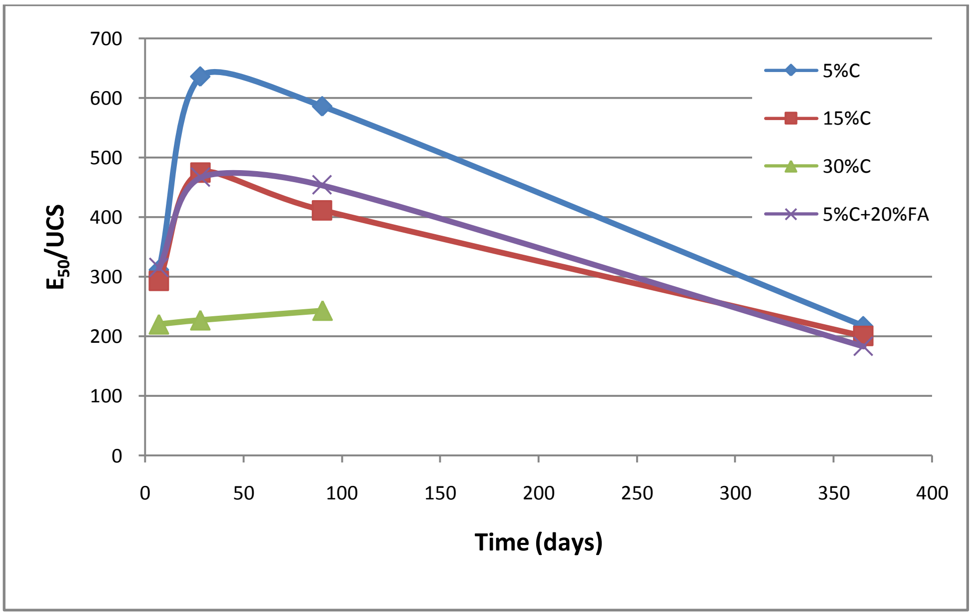


Figure 4.6 E_{50}/UCS vs. time for C and C+FA mixes

As it is seen in Figure 4.7, as cement content increases, the ratio of UCS of treated soil to the one of untreated soil increases linearly. The trend may be given by a linear regression. This means that the UC strength of the cement treated soil is $\boldsymbol{0.885a_w}$ times the strength of the untreated soil (Equation 4.1).

$$\frac{UCS_t}{UCS_u} = 0.885.a_w \qquad (4.1)$$

where, UCS_t and UCS_u is the 28 days UCS of the treated and untreated soil, respectively. a_w is the cement content in %.

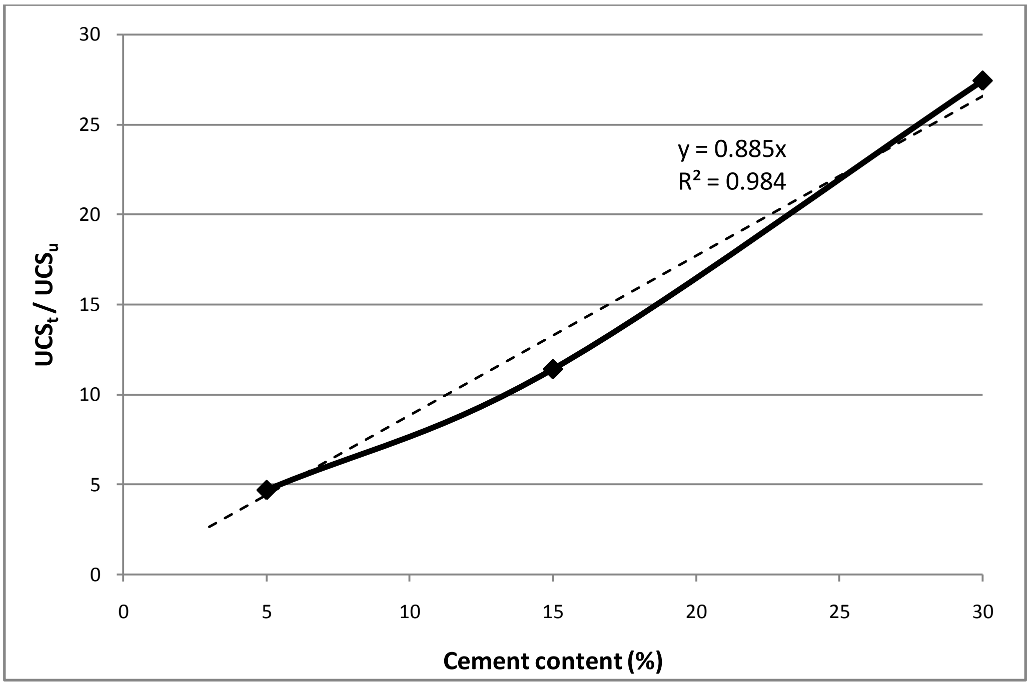


Figure 4.7 UCS treated,$_{28days}$ / UCS untreated,$_{28days}$ for different cement contents

The fly-ash also gives some strength increase. This is due to type C fly-ash as in this study that performs hydration (CaO+H_2O) process. The 28 day strength of 5%C+20%FA treated soil is 9.43 times the untreated soil. From the above relation, it can be back-calculated that 5%C+20%FA gives the same strength enhancement as 10% (9.43/0.885 = 10.65%) cement addition does.

4.1.3 Comparison of results of tests on improved CL and kaolinite

From Figure 4.8 the relation between axial strain at failure and UC strength obtained from UC test results can be seen. Although at lower strength region there is a great scatter in this relation, at higher UC strengths, the strain at failure value is at a narrow range of 0.75-1.25 %. The strains at failure show a rapid decrease at the strength of 400 kPa. This trend is related to the brittle behavior of the stabilized soils. As UCS of the mixed soil increases, the soil shows a more brittle behavior. This can also be seen in figures of stress and strain for CL and kaolinite, respectively (Figures 4.9 and 4.10). These results are in good agreement with the results presented by Ahnberg et al. (2003).

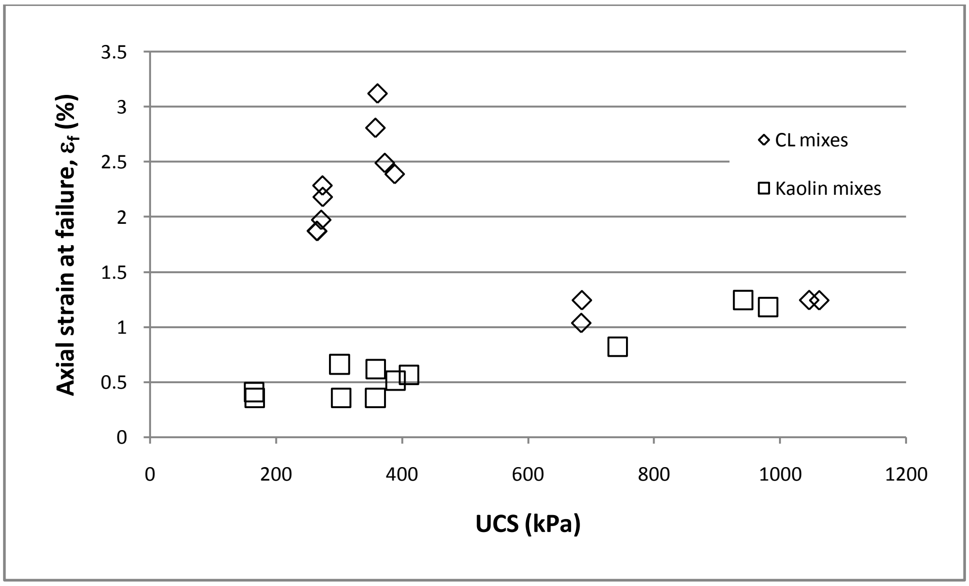


Figure 4.8 Axial strain at failure load vs. UCS for C and C+FA mixed CL and kaolinite soils

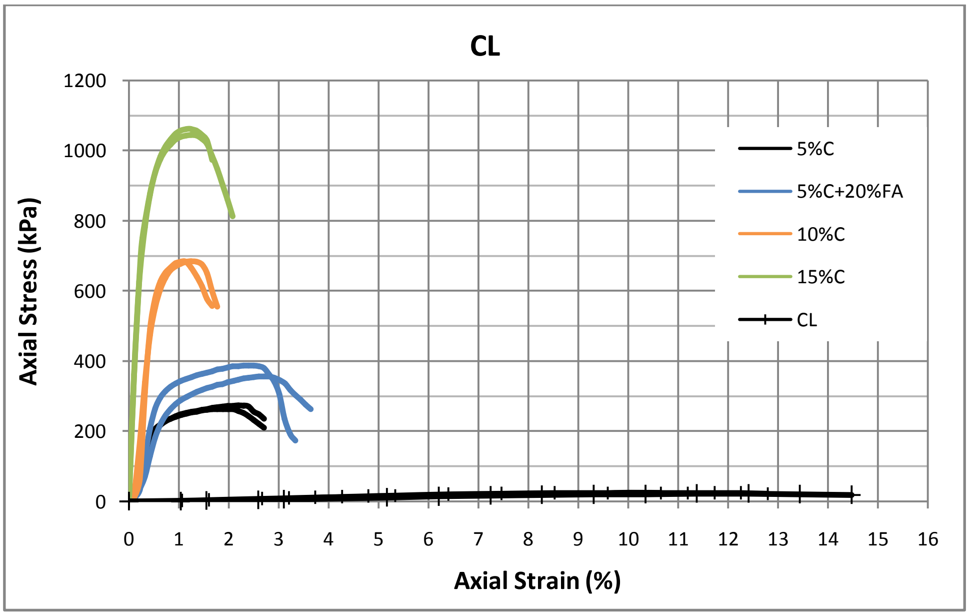


Figure 4.9 Stress-strain for mixed CL

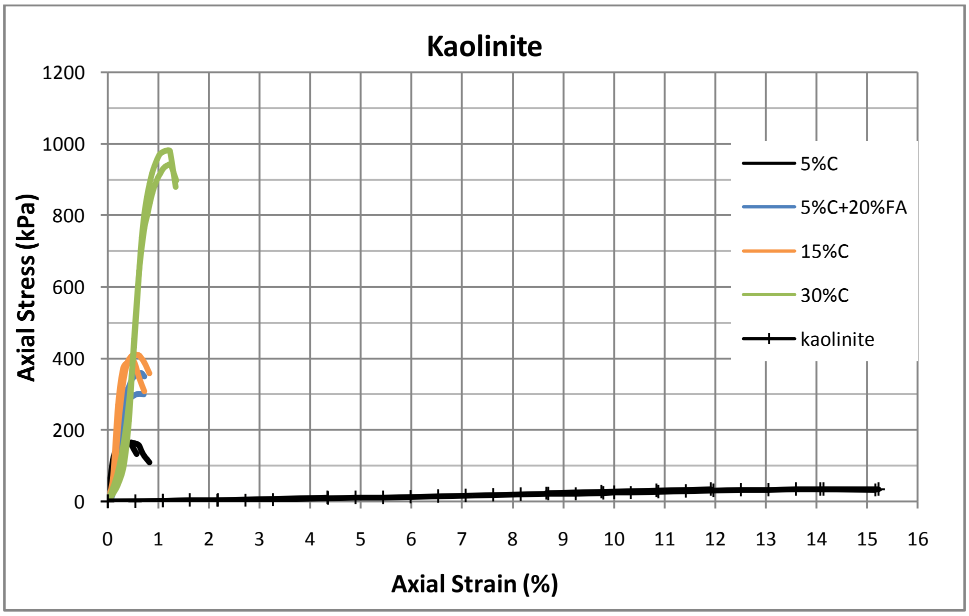


Figure 4.10 Stress-strain for mixed kaolinite

The E_{50}/UCS ratio vs. cement content relation for different curing times can be seen in Figure 4.11. This ratio converges to a smaller range from 5% to 15% cement content values (100-600 for 5%C, 300-500 for 15%C). For the cement content of 30%, this ratio is about 200.

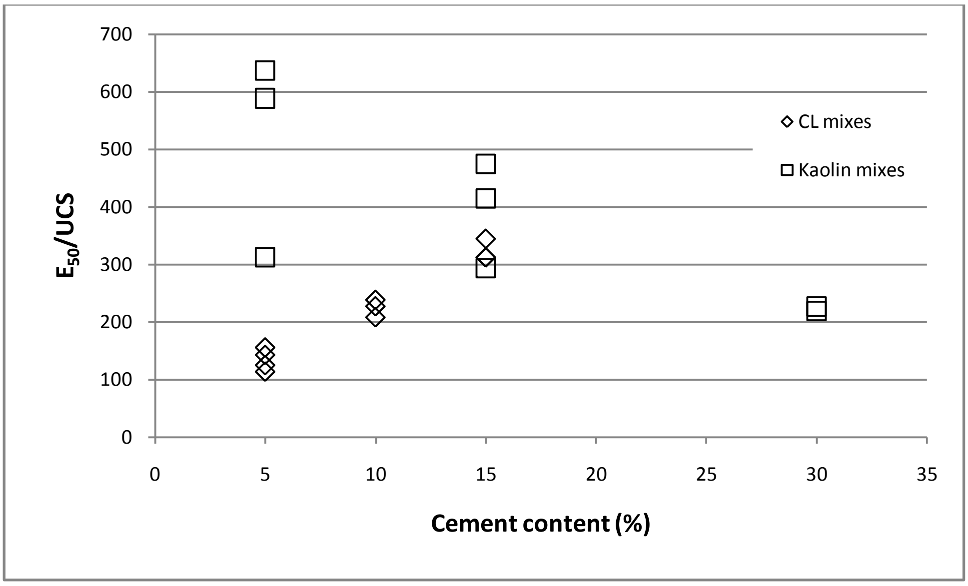


Figure 4.11 E_{50}/UCS vs. cement content for C mixed CL and kaolinite soils

4.2 LARGE SCALE CONSOLIDATION TESTS

Results of large scale consolidation tests on deep mixed group column improved kaolinite soil mass are presented in Figures 4.12 to 4.16.

The stress-strain graphs for kaolinite improved with cement/cement+fly-ash columns are presented in Figure 4.12. It is obvious that the deformation behavior of kaolinite is changed by the C/C+FA treatment. The effect of change of deformation behavior depends on the binder type and also the number of columns (replacement ratio, a_s). The replacement ratios (ratio of total area of DMC to that of stabilized soil) for 19, 38, 55, and 85 column groups are 0.045, 0.09, 0.13, and 0.20, respectively.

In Figure 4.12 5%C treated group of columns indicate more strains than the other type of columns. 5%C+20%FA and 15%C group of columns show the strains more than 5%C and less than 30%C group of columns. The most efficient binder mix seems to be the high amount of cement (30%). The improvement ratios are discussed in detail in the following sections for each binder type.

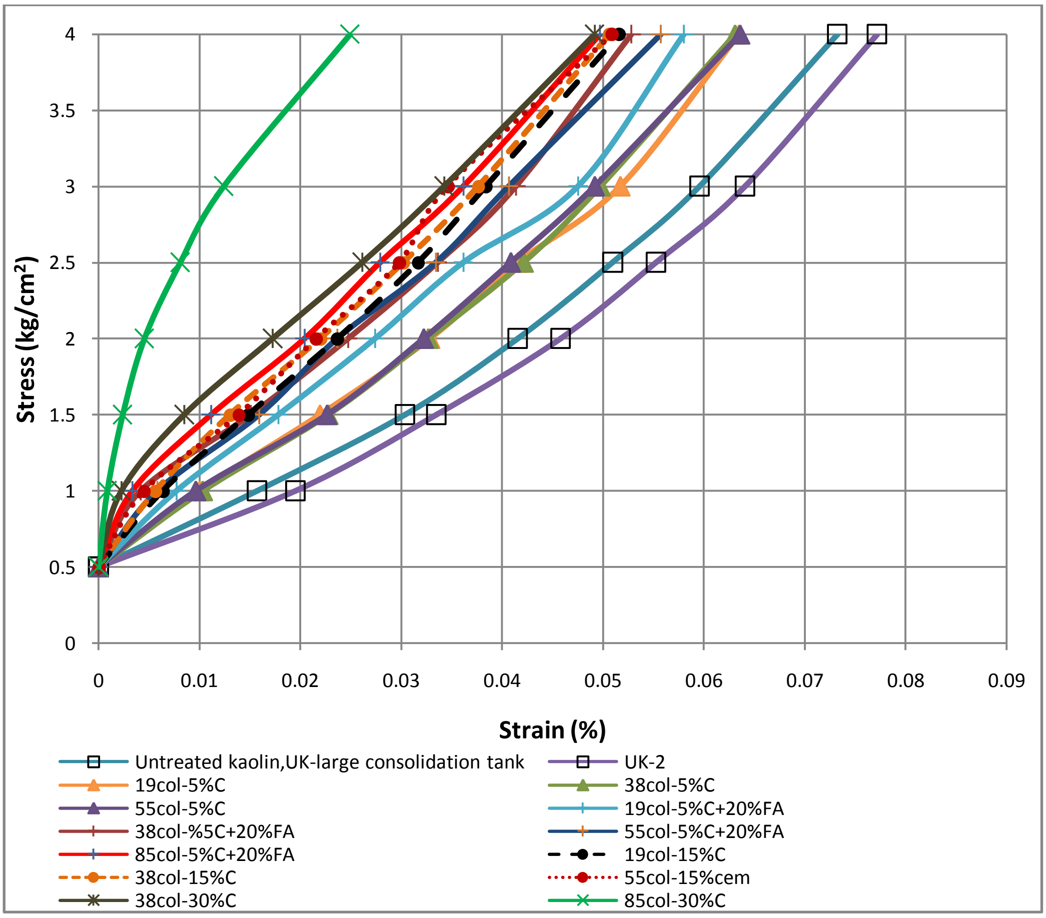


Figure 4.12 Stress-Strain diagrams for all tests

As can be seen in Figure 4.13 the average strain of the soft clay can be reduced by 13% to 36% with the improvement by 5% cement DMM column groups. These values are calculated at 1 and 4 kg/cm^2.The replacement ratio or number of columns does not cause much difference in the improvement for 5% cement binder stabilized kaolinite.

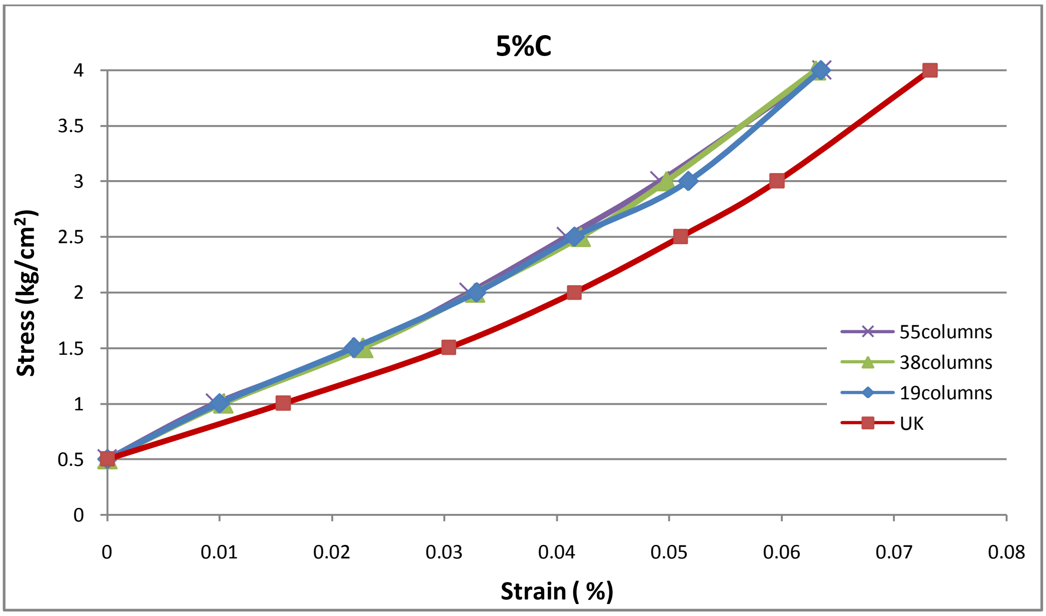


Figure 4.13 Stress-strain diagram for 5%C column improved tests

Figure 4.14 shows that the average strain of the soft clay can be reduced by 24% to 63% with the improvement by 5%C+ 20%FA DMM column groups for 19-55 columns. The number of columns makes some difference in the deformation behavior of the stabilized soil. The 19, 38, and 55 column groups give the similar results, whereas the 85 column group causes a better improvement. The average strain can be reduced by 28% to 79% for 85 columns.

As the number of columns (replacement ratio) increases the final strain at the end of consolidation decreases.

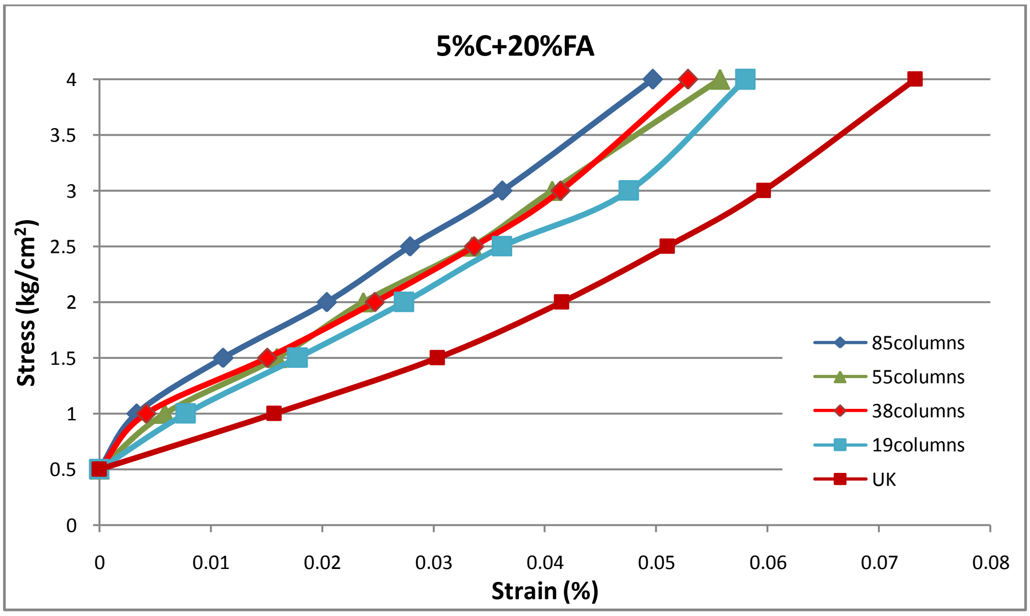


Figure 4.14 Stress-strain diagram for 5%C+20%FA column improved tests

As can be seen in Figure 4.15 the average strain of the soft clay can be reduced by 30% to 71% with the improvement by 15% cement DMM column groups. The number of columns does not change the stress-strain behavior significantly.

As it is concluded from Figures 4.13 to 4.15, increase of replacement ratio or number of columns does not cause significant difference in the improvement with 5%C, 5%C+20%FA and 15%C columnar improved clay.

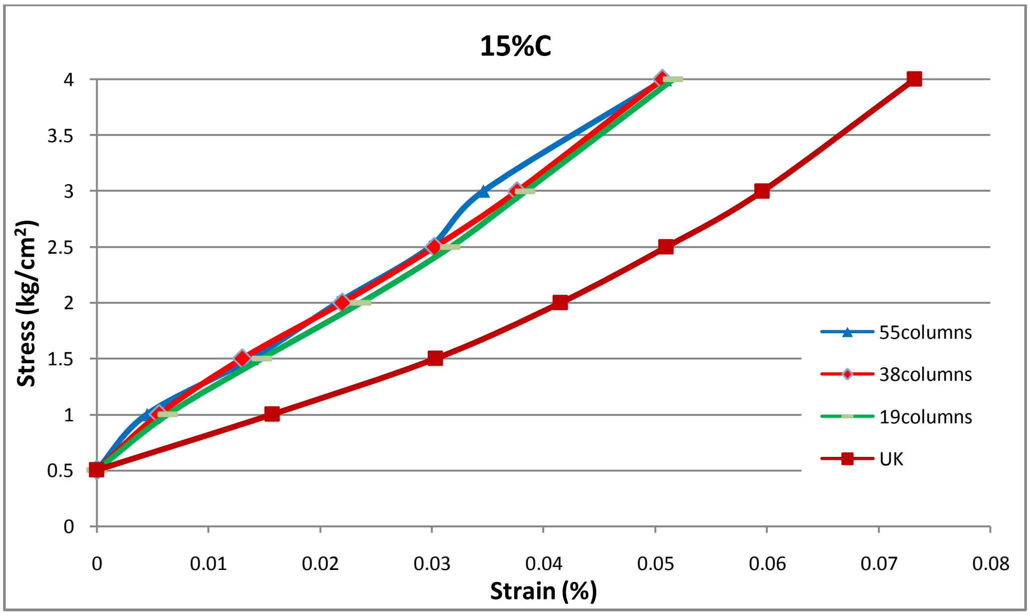


Figure 4.15 Stress-strain diagram for 15%C column improved tests

The average strain of the soft clay can be reduced by 33% to 86% with the improvement by 38 columns of 30%C (Figure 4.16). These values are 66% to 94% for 85 columns of 30%C. The number of columns makes a substantial change in the deformation behavior of the treated soil.

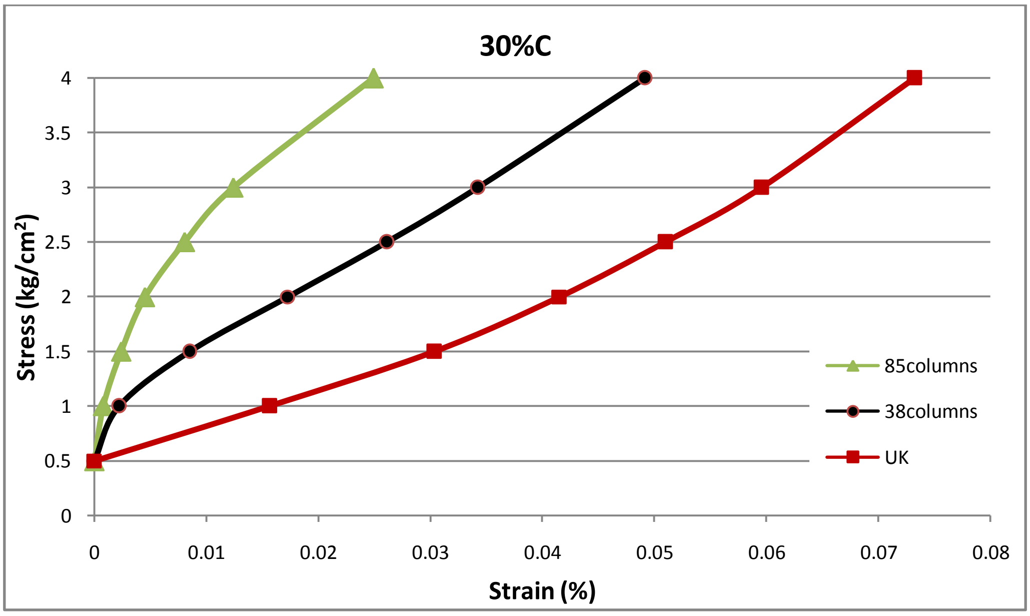


Figure 4.16 Stress-strain diagram for 30%C column improved tests

In Figure 4.17 the average settlement reduction factors (β values obtained for different replacement ratios are averaged for each type of columns) are shown for different consolidation stress levels for different binder mixes. Accordingly, the most effective binder mix is 30%C. 30%C mixed group of columns will improve the system (will reduce the settlement of the system) 2.2 to 13 times for consolidation stresses of 4 and 1 kg/cm^2, respectively.

15%C and 15%C + 20%FA mixed group of columns will show a settlement reduction factor above 2 for 1-1.5 kg/cm^2 stress range. These values drop under 2 for stress ranges above 2 kg/cm^2.

The less effective binder mix is 5%C. The settlement reduction factors are under 2 for all stress levels. These are shown in detail in Figures 4.19 and 4.20.

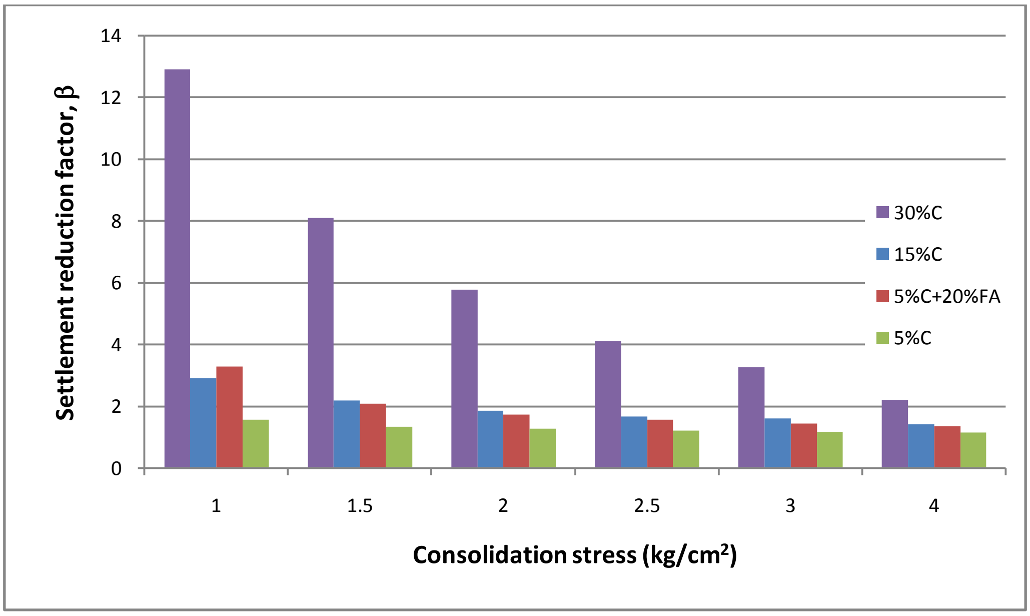


Figure 4.17 Average settlement reduction factors ($s_{untreated}/s_{treated}$) at different consolidation stress levels

In Figure 4.18 settlement reduction factors for different consolidation stresses are shown for each individual test. 30%C columns give the upper bound and 5%C columns give the lower bound.

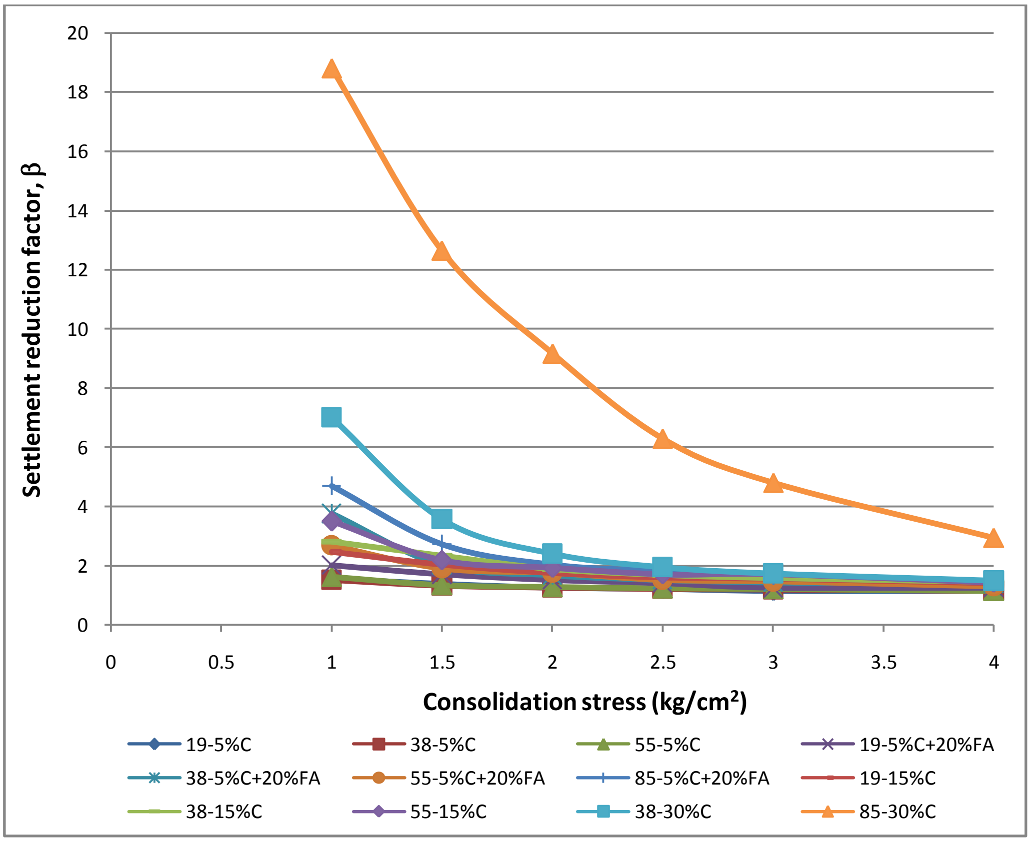


Figure 4.18 Settlement reduction factor for different stress ranges for all tests

In Figure 4.19 and 4.20 the settlement reduction factors for 2 and 2.5 kg/cm^2 consolidation stresses, which is the typical range of loading for DMM improved projects, for different binder mixes and different replacement ratios can be seen. There is a limited increase of improvement ratio with increasing replacement ratio for cement treated clays except 30%C columns. The β values are in the range of 1.31 to 1.48, and 1.77 to 2.12 for 5%C, and 15%C, respectively. The β values are in the range of 1.73 to 2.28 for 5%C+20%FA column improved system.

In general, while β values for 5%C, 5%C+20%FA, and 15%C columns are below 2, the 30% cement mixed group of columns makes a substantial improvement in terms of settlement reduction (2-2.5 for 38 columns, 6-9 for 85 columns).

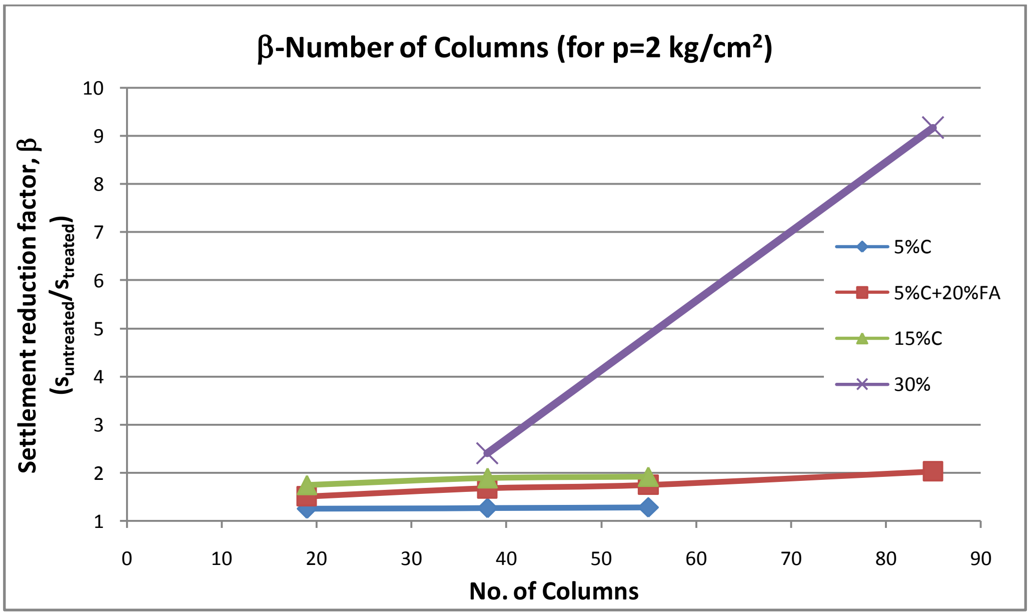


Figure 4.19 Settlement reduction factor (β) vs. number of columns for consolidation pressure of 2 kg/cm^2

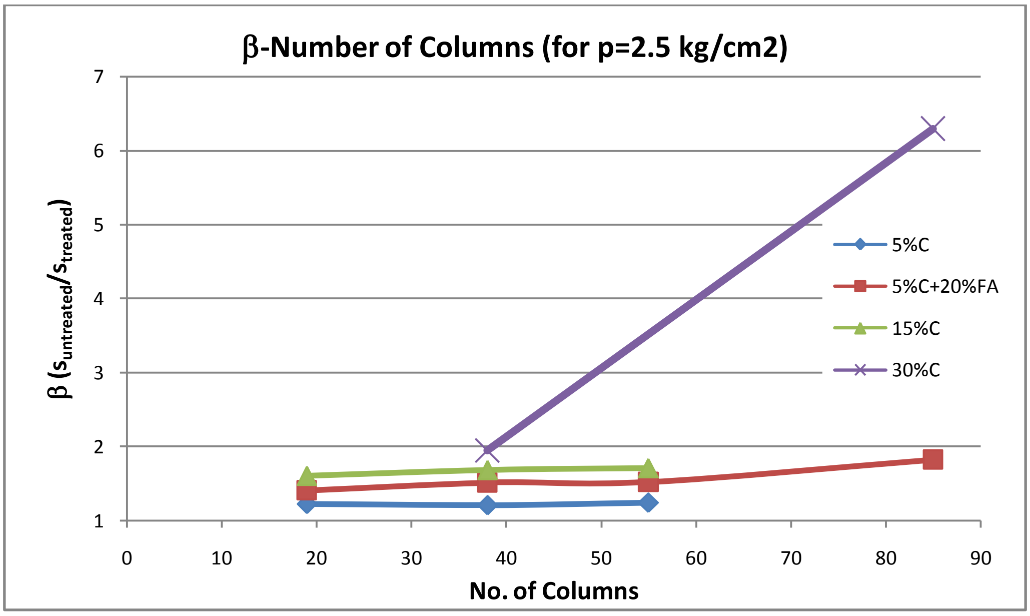


Figure 4.20 Settlement reduction factor vs. number of columns for consolidation pressure of 2.5 kg/cm^2

Change of settlement reduction factor with stress level at different replacement ratios for different binder materials is shown in Figure 4.21. A similar plot is presented in Figure 4.22 where β is related to a_s at different stress levels for different binder materials. It is observed that β decreases consistently with increasing stress level at all a_s, but effect of a_s is not much pronounced. On the other hand high cement content results in very high improvement (β) at higher a_s (a_s=0.2).

For all binder materials and for all replacement ratios settlement reduction ratio (a_s) decreases at higher stress levels (3-4 kg/cm^2) (Figures 4.21 and 4.22). This seems to be related to p_y values of composite groups and small oedometer tests of stabilized soils.

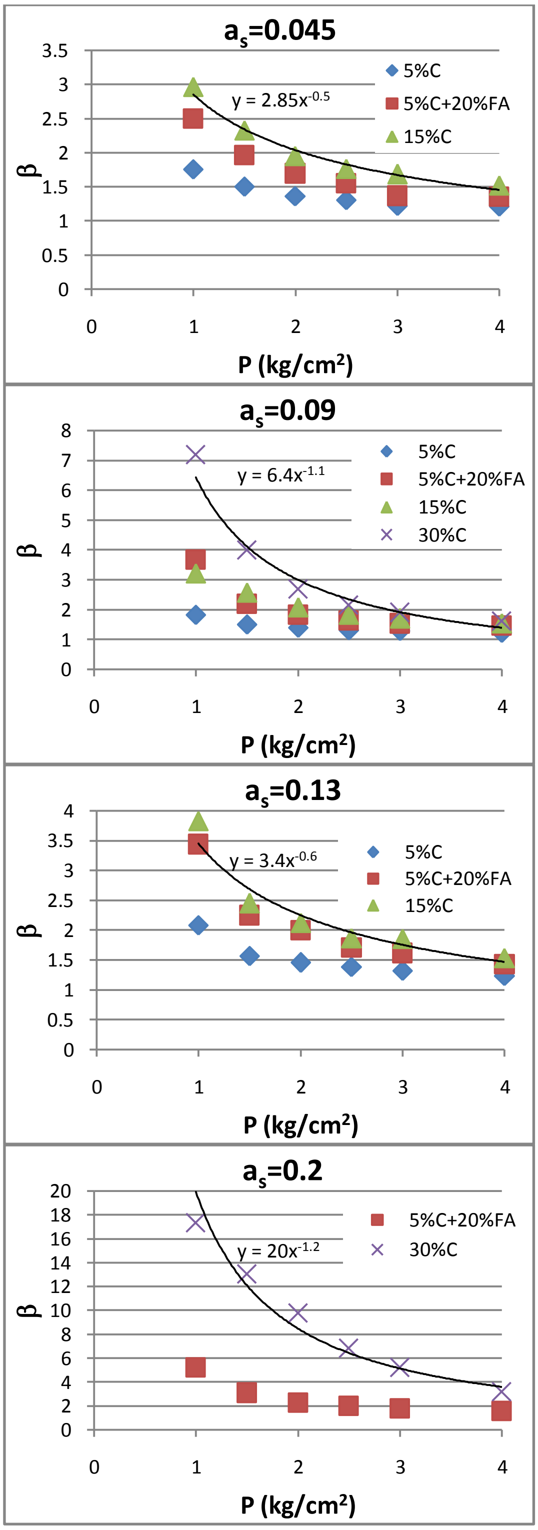


Figure 4.21 β vs P for a_s from 0.045 to 0.2

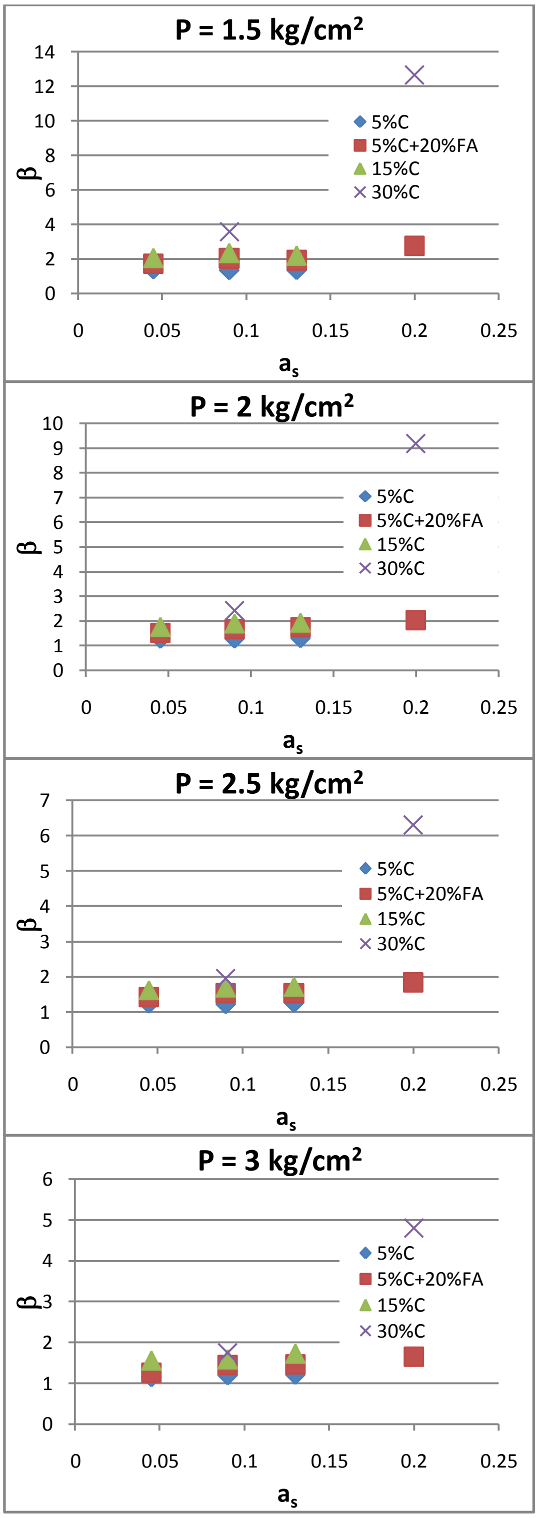


Figure 4.22 β vs a_s for P from 1.5 to 3 kg/cm^2

The total displacement vs. consolidation stress (in Log scale) plots are presented in Figures 4.23 to 4.26. As these curves are similar to that of e-logP curves of oedometer tests, the break points on the curves for improved soils are similar to the recompression-compression passage. This phenomenon is called the consolidation yield pressure or pseudo pre-consolidation pressure, p_y (especially for single binder mixed samples in the literature). The compressibility of the treated soil is almost the same as that of the untreated soil when the stress in the treated soil exceeds this value.

As stated by some researchers (Terashi and Tanaka, 1993; Kasama and Zen, 2000; Kamruzzaman et al., 2001) beyond the pseudo pre-consolidation pressure, the behavior of treated sample was almost parallel to the virgin consolidation line (VCL) of the untreated clay which implied that the soft clay matrix may still control the behavior of treated clay if it is stressed beyond pre-consolidation pressure. This phenomenon is observed on stabilized soil samples in oedometer by these researchers. But this study shows that this is also relevant to the improvement of soft soils by DMM column groups. This can be seen in Figures 4.23 to 4.26.

The value of p_y is studied in the literature by a number of researchers. Terashi and Tanaka (1993)(30 cm diameter, 10 cm height consolidation tests on composite ground- 15%C, a_s of 30%) stated the value of p_y as 1.3*UCS. Horpibulsuk (2001) and Liu et al. (2006) performing similar tests determined this value as 2.2 and 1.5, respectively.

From Figures 4.23 to 4.26, the p_y values are obtained as 1.2, 1.3, and 1.4 kg/cm^2 for 5%C, 5%C+20%FA, and 15%C mixes, respectively. The number of columns does not change this value for these mixes. p_y/UCS values are 0.72, 0.39, and 0.35 for 5%C, 5%C+20%FA, and 15%C mixes, respectively. For the 30%C mixes, increase in the number of columns increase the value of p_y. The p_y values for 30%C mixes are 1.5 and 2.7 kg/cm^2 for 38 and 85 columns, respectively. The values of p_y/UCS are between 0.16 and 0.28 and these are not similar to p_y/UCS values reported in the literature. Research work by Terashi and Tanaka (1993), Horpibulsuk (2001), and Liu et al. (2006) was performed on single column treated soil systems. Their contribution for relating the yield pressure to the UC strength of the system is valid for single column improved systems. The group column improved systems however show a different behavior in terms of yield pressure phenomenon.

Cement content (a_w) of DMC highly affect the p_y value of the stabilized system. High cement content (30%C) yields higher p_y values hence better improvement at higher stress levels. Higher replacement ratios (a_s) end up with higher p_y values and much better improvement.

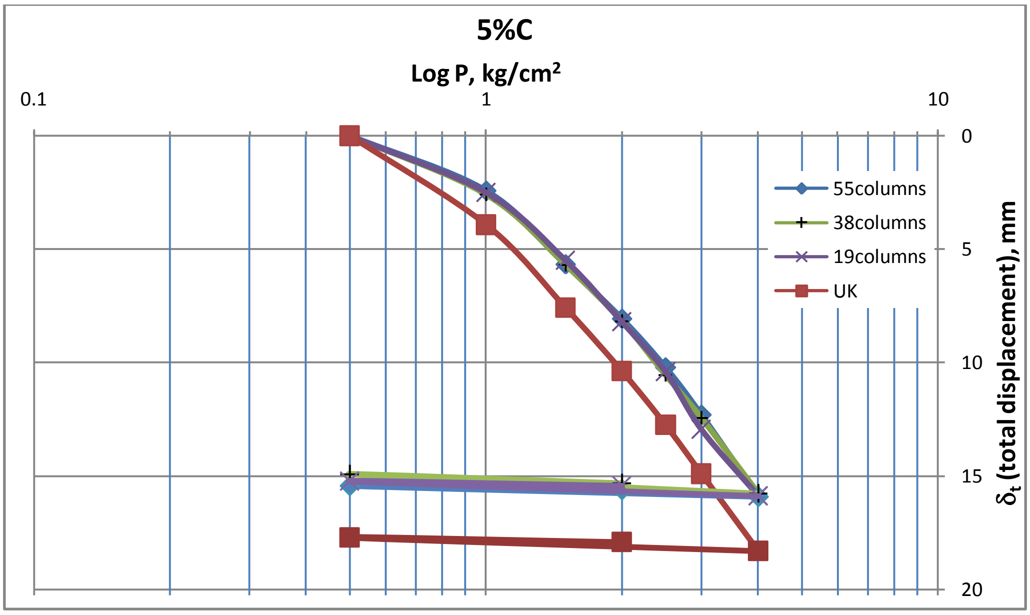


Figure 4.23 δt – LogP curve for 5%C column tests

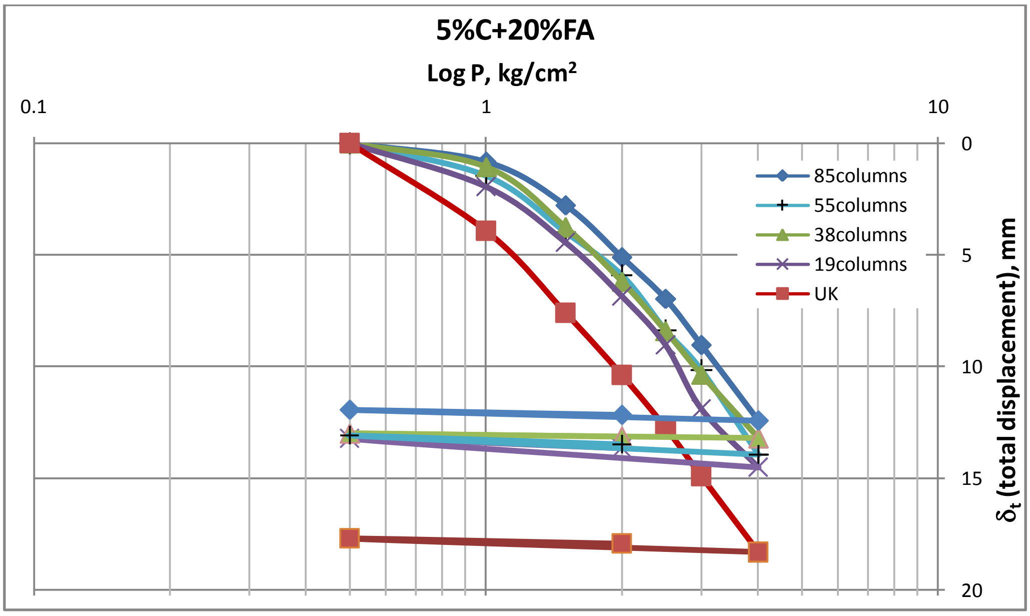


Figure 4.24 δt – LogP curve for 5%C+20%FA column tests

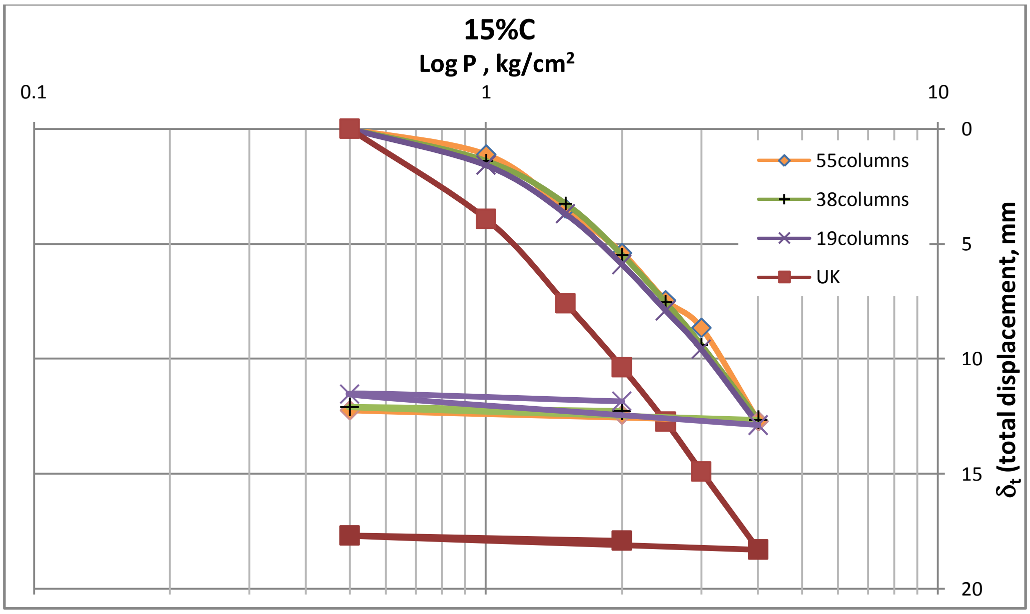


Figure 4.25 δt – LogP curve for 15%C column tests

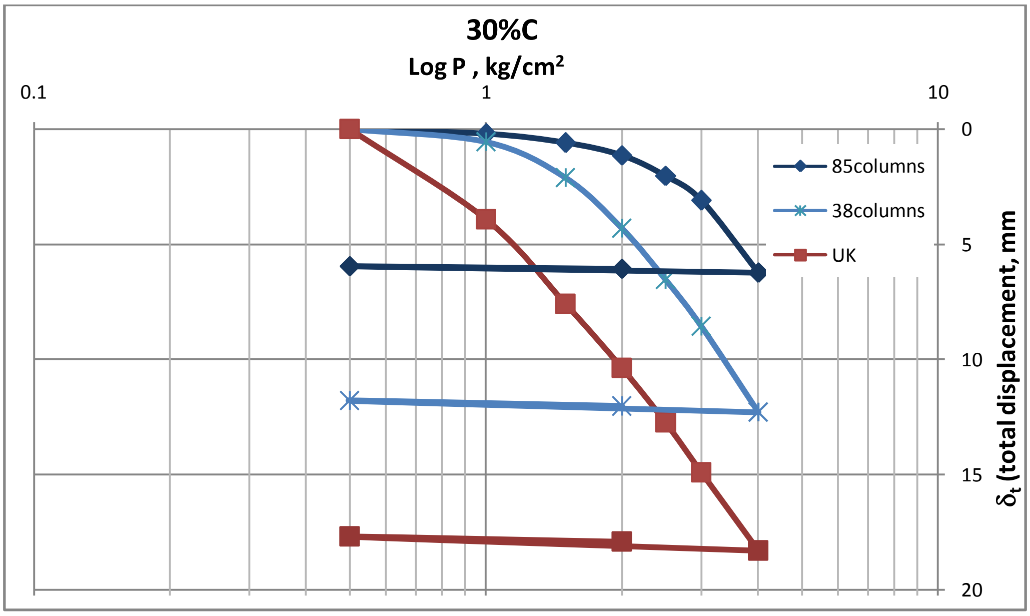


Figure 4.26 δt – LogP curve for 30% C column tests

Figures 4.27 to 4.30 show the comparison of constrained modulus (M) of soils improved with different number of DMCs and binder mixes with the untreated kaolinite (UK) for different consolidation stress ranges. Figure 4.27 provides the comparisons for soils treated with 5%C DMC and shows that M values of improved soils are higher compared to that of UK for consolidation stresses up to 2 kg/cm^2. However, this trend is not observed for higher consolidation stresses. In addition, increasing the number of columns did not help improving the soil when 5%C columns are used.

Figure 4.28 shows M values of soils improved with group of columns made by mixing 5%C and 20%FA as compared to UK. For consolidation stresses between 0.5 and 2 kg/cm^2, using DMC and increasing the number of columns will significantly increase the moduli. In this consolidation stress range, unlike the addition of 5%C, when FA is used as an additive together with C, increasing the number of columns will substantially affect the performance of improvement. The maximum increase in M value is about 3.75 MPa (100% increase) which was obtained when 85 columns were used.

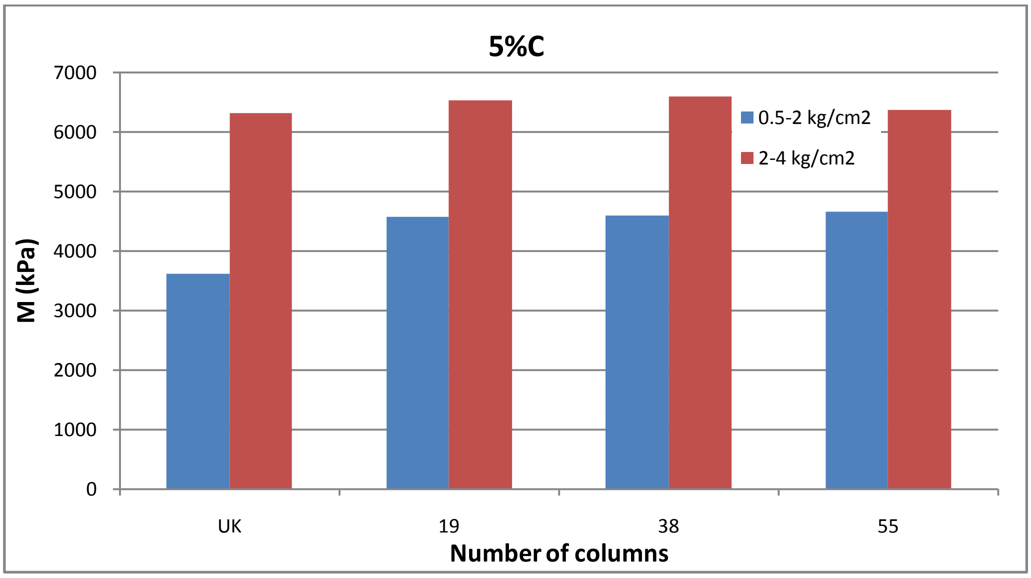


Figure 4.27 Comparison of M of soils improved with DMC of 5%C

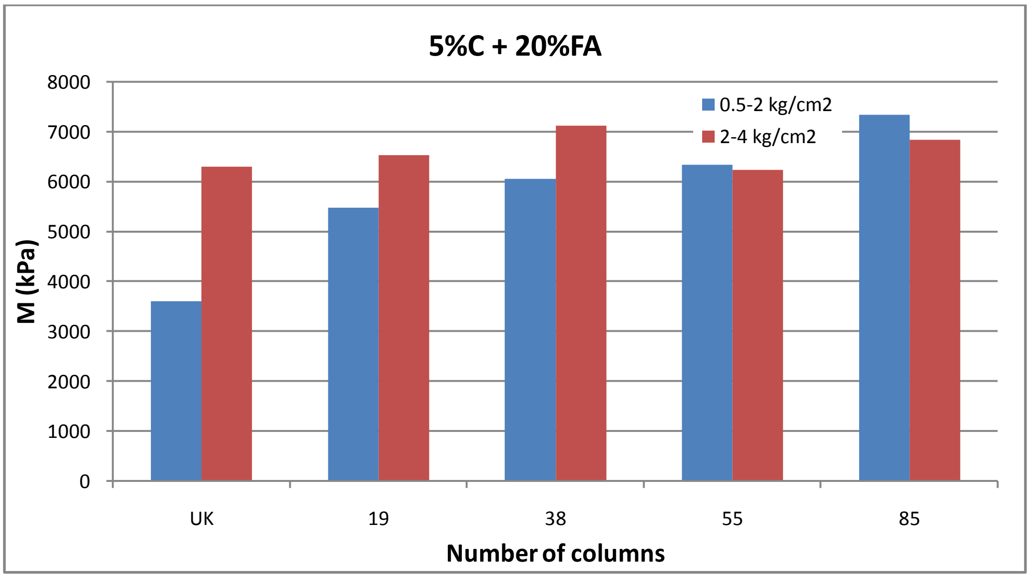


Figure 4.28 Comparison of M of soils improved with DMC of 5%C+20%FA

Figure 4.29 shows M values of soils improved with group of columns made by mixing 15%C. For consolidation stresses between 0.5 and 2 kg/cm^2, using DMC and increasing the number of columns will greatly help obtaining higher moduli. Increasing the number of columns will substantially affect the performance of improvement. The maximum increase in M value is about 3.3 MPa which was obtained when 55 columns were used. The improvement level for 38 and 55 columns are at the same order.

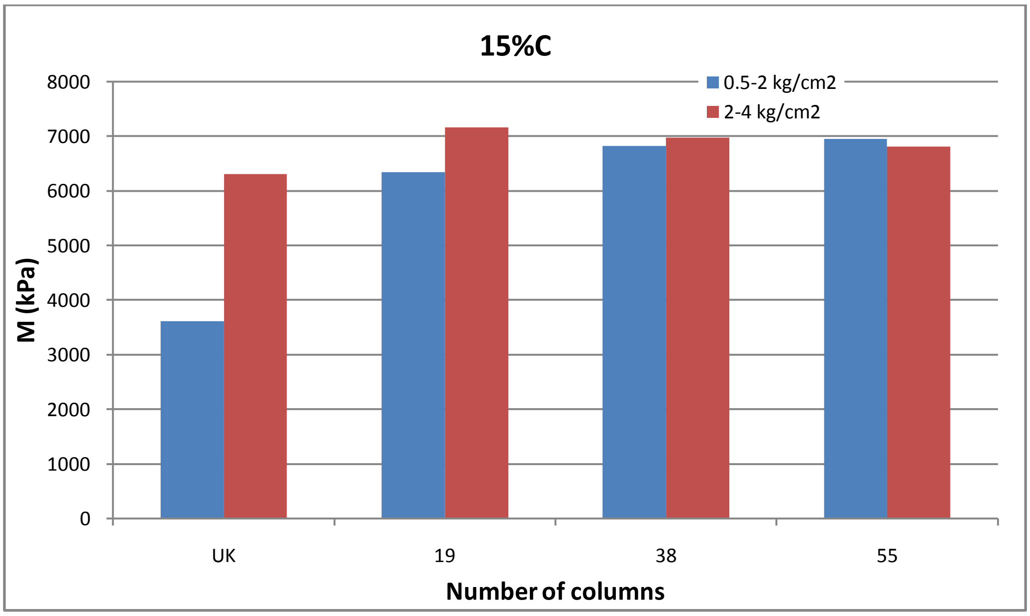


Figure 4.29 Comparison of M of soils improved with DMC of 15%C

Figure 4.30 provides the comparison of M values for 38 and 85 columns of 30%C with those of UK. For 0.5-2 kg/cm^2 stress range 38 columns of 30%C increases the M value of UK from 3.6 to 8.6 MPa. 85 columns of 30%C increase the M value of UK from 3.6 to 33 MPa for the stress range of 0.5-2 kg/cm^2.

Figure 4.31 provides the summary of modulus (M) values for all groups.

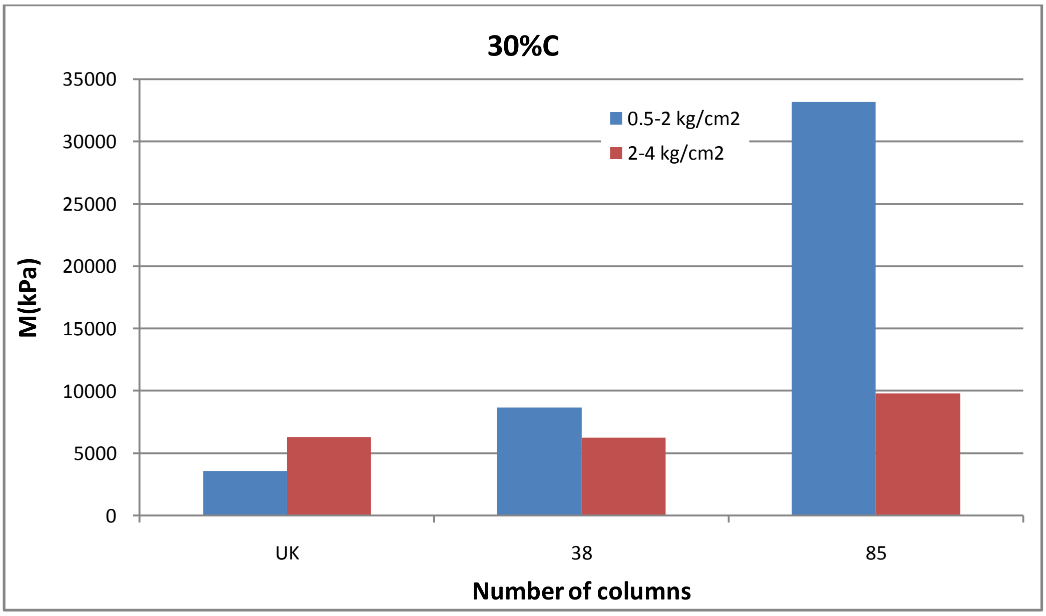


Figure 4.30 Comparison of M of soils improved with DMC of 30%C

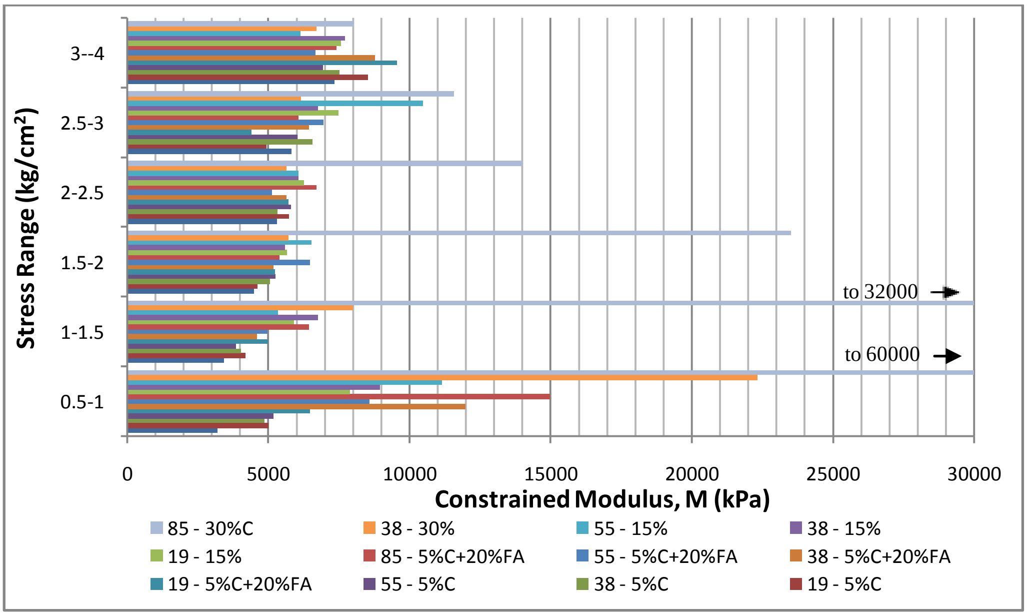


Figure 4.31 Comparison of M for all consolidation stress ranges

The data in Figures 4.27-4.31 can also be presented as in Figure 4.32. The M values for the two stress ranges for all tests are drawn.

From Figure 4.32, it is concluded that group columns of 5%C (19, 38, 55 columns) increases M from 3.6 to about 5 MPa. The M values for 5%C+20%FA column improved system are between 5.5 and 7.3 MPa. 15%C columns increase M value of the system to about 6.8 MPa. The modulus values of 38 and 85 columns of 30%C treated soil is 8.7, and 33 MPa, respectively. These data are presented for 0.5-2 kg/cm^2 stress range. No substantial differences in M values were observed for 2-4 kg/cm^2 stress range.

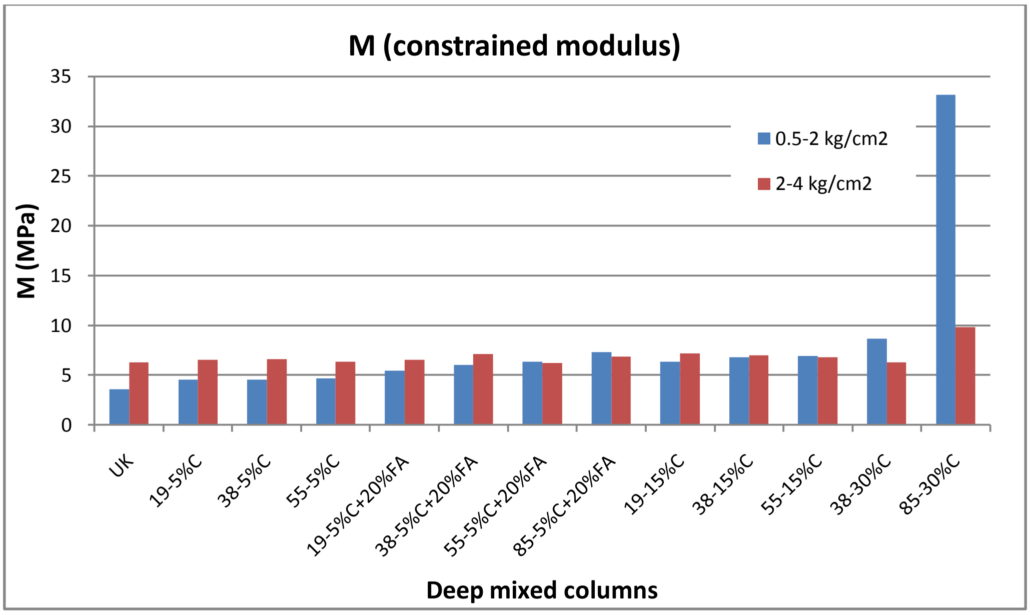


Figure 4.32 Change in M for different stress ranges

The increase in M for 0.5-2 kg/cm^2 stress range is presented in Figure 4.33. As cement percentage increases, the increase in M (or level of improvement) for different a_s values are noted.

Using cement alone binders the optimum a_s value for improvement may not be higher than 0.1 for low cement contents (up to 15%), since the increase in M value converges to 100% asymptotically. The 30%C admixed soil shows great increase in M (from 140% to 810%).

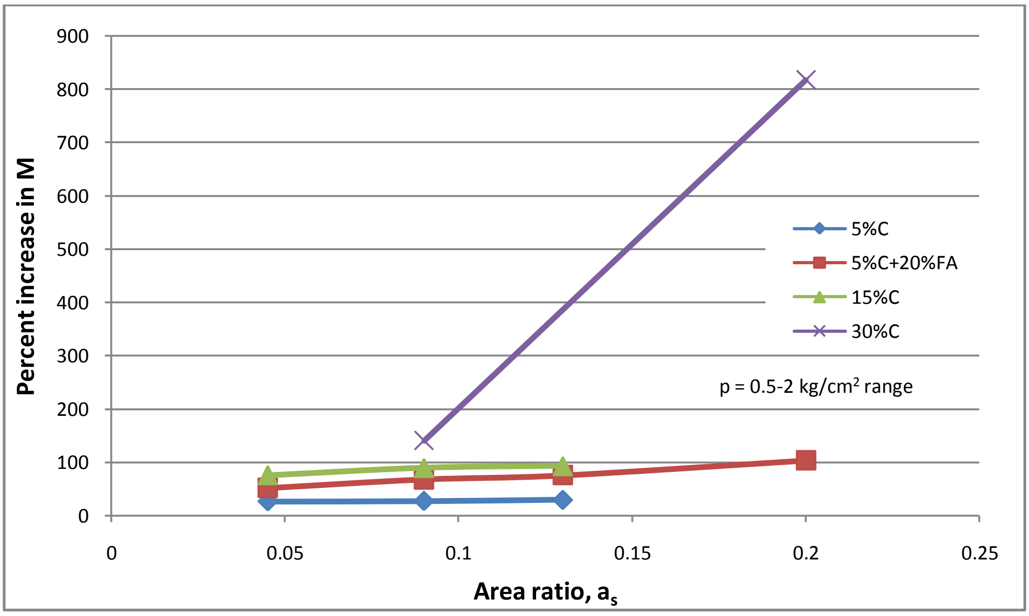


Figure 4.33 Change of % increase in M for different replacement ratios for the stress range of 0.5-2 kg/cm^2

For most of binders (except 30%C) studied the percent increase in M is between 30-100%.

In general design methodology, for computing the deformations of the stabilized composite mass, the bulk compression modulus of the system is obtained by the formulation which is given in Equation 4.2.

$$M_{system} = M_{column}.a_s + M_{soil}.(1-a_s) \quad (4.2)$$

where; a_s: replacement ratio, M_{column}: 1D (constrained) compression modulus of DMC from oedometer tests, M_{soil}: 1D (constrained) compression modulus of soil from oedometer tests. When computing for design (settlement calculations) 1D compression (constrained) modulus (M) of the column material must be used.

If cement columns due to stress concentration deforms in the lateral direction (i.e. deviation from 1-D compression) E_{col} may be used. In this case M_{system} is written as in Equation 4.3.

$$M\text{-}E_{system} = E_{column}.a_s + M_{soil}.(1-a_s) \quad (4.3)$$

where; a_s: replacement ratio, E_{column}: elastic modulus of DMC from UC tests, M_{soil}: 1D (constrained) compression modulus of soil from oedometer tests.

There is also common use of E moduli in professional practice. Equation 4.4 with E (elastic modulus) values for both column and soil are employed (Equation 4.5).

$$E_{system} = E_{column}.a_s + E_{soil}.(1-a_s) \quad (4.4)$$

where; a_s: replacement ratio, E_{column}: elastic (shear) modulus of DMC from UC tests, E_{soil}: elastic (shear) modulus of soil from UC tests. The calculated E_{system} value is then used instead of M_{system} for settlement calculations.

The experimental M_{system} values determined from the large scale test data were compared with the M_{system} and E_{system} values calculated from Equations 4.2, 4.3 and 4.4. The ratio between these values are given in Tables 4.3-4.5 and Figures 4.34-4.37.

The calculated settlements of the composite system are not similar to the measured values at all replacement ratios. Best approach seems to conduct oedometer tests on stabilized soil and on untreated soil in the laboratory to calculate compressibility of the composite mass.

From the values in Table 4.3 it is understood that the calculation always give higher values for M. As this classical approach overestimates the constrained modulus (M) of the system for 1-1.5, 1.5-2 and 2-2.5 kg/cm^2 stress ranges

Table 4.3 M_{system}/M_{exp} ratios for P=1-1.5 kg/cm^2 stress range

Binder Type	a_s	M_{calc}/M_{exp}	E_{calc}/M_{exp}	$M\text{-}E_{calc}/M_{exp}$
5%C	0.045	0.74	1.20	1.83
	0.09	0.80	2.44	3.06
	0.13	0.85	3.64	4.26
5%C+20%FA	0.045	0.66	1.46	1.98
	0.09	0.78	3.09	2.15
	0.13	0.77	4.13	2.00
	0.20	0.66	4.88	1.54
15%C	0.045	0.57	1.50	1.95
	0.09	0.54	2.59	2.98
	0.13	0.74	4.72	5.17
30%C	0.09	0.53	2.5	2.82
	0.20	0.18	1.39	1.46

Table 4.4 M_{system} /M_{exp} ratios for P=1.5-2 kg/cm^2 stress range

Binder Type	a_s	M_{calc}/M_{exp}	E_{calc}/M_{exp}	M-E_{calc}/M_{exp}
5%C	0.045	0.97	1.09	1.96
	0.09	0.87	1.94	2.69
	0.13	0.83	2.67	3.37
5%C+20%FA	0.045	0.91	1.39	2.16
	0.09	0.97	2.74	2.17
	0.13	0.81	3.16	1.74
	0.20	1.05	5.84	2.09
15%C	0.045	0.84	1.57	2.27
	0.09	0.89	3.12	3.84
	0.13	0.80	3.85	4.41
30%C	0.09	0.92	3.5	4.17
	0.20	0.26	1.89	2.03

Table 4.5 M_{system} /M_{exp} ratios for P=2-2.5 kg/cm^2 stress range

Binder Type	a_s	M_{calc}/M_{exp}	E_{calc}/M_{exp}	$M\text{-}E_{calc}/M_{exp}$
5%C	0.045	0.93	0.88	1.72
	0.09	0.99	1.84	2.71
	0.13	0.91	2.42	3.18
5%C+20%FA	0.045	0.97	1.27	2.11
	0.09	1.01	2.52	2.14
	0.13	1.15	3.99	2.36
	0.20	0.92	4.70	1.80
15%C	0.045	0.89	1.42	2.19
	0.09	0.95	2.88	3.67
	0.13	0.98	4.15	4.87
30%C	0.09	1.38	3.55	4.36
	0.20	0.77	3.17	3.45

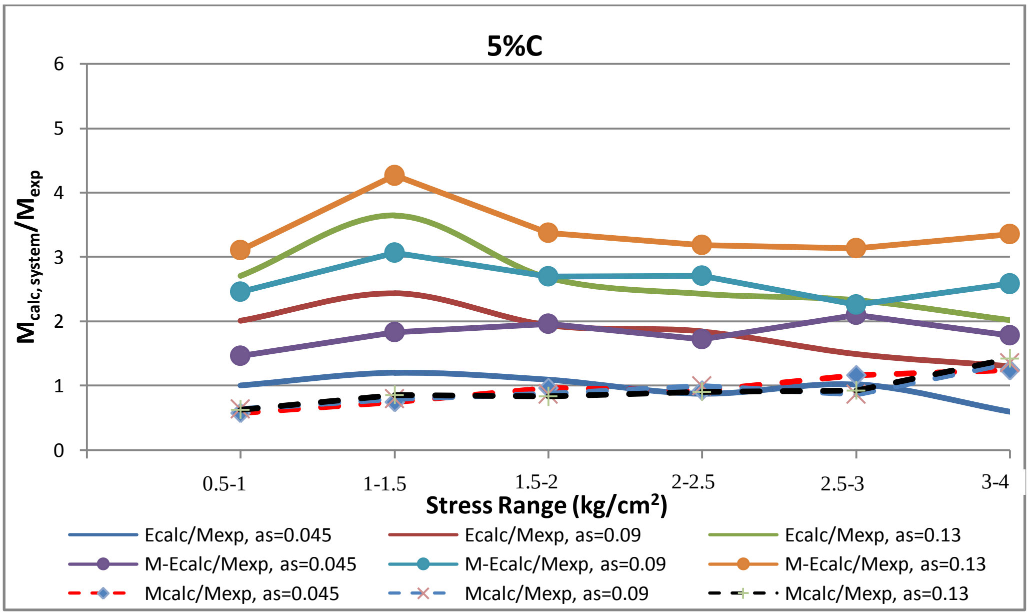


Figure 4.34 Comparison of M values calculated for 5%C stabilized soils

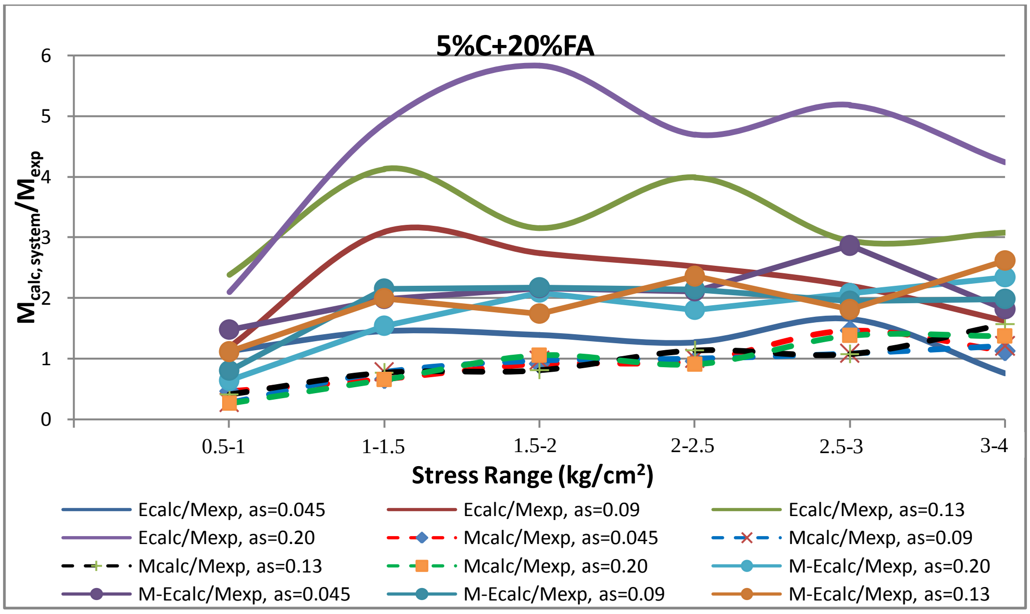


Figure 4.35 Comparison of M values calculated for 5%C+20%FA stabilized soils

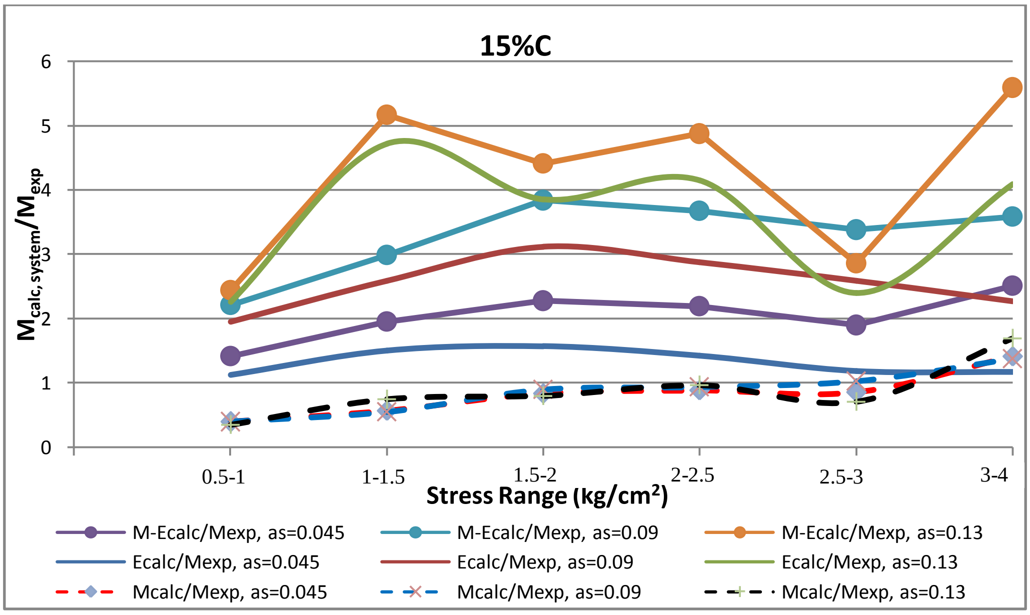


Figure 4.36 Comparison of M values calculated for 15%C stabilized soils

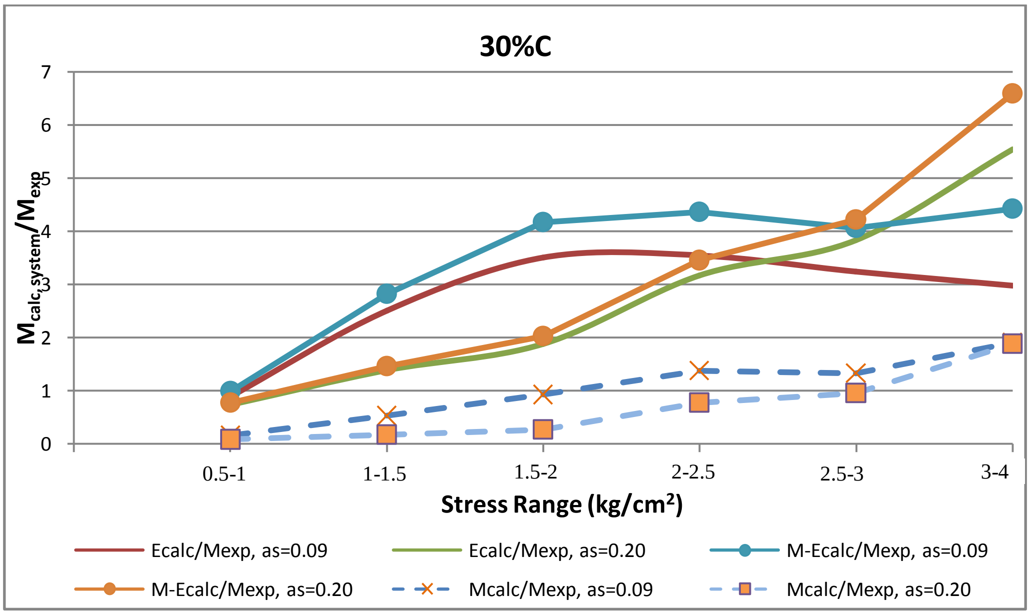


Figure 4.37 Comparison of M values calculated for 30%C stabilized soils

As in Figure 4.38 the M values found from standard consolidation (oedometer) tests are in good agreement with the values obtained from the large scale consolidation tests (Maintained Step Loading Test) in the tank. The oedometer sample was taken from clay consolidated in the tank by means of pushing oedometer ring. Then it was transferred to the standard consolidation testing apparatus.

The results from the two testing system is well suited for the range of p=1.5 to 3 kg/cm^2 which may be considered as the normal loading stress range for deep mixed systems.

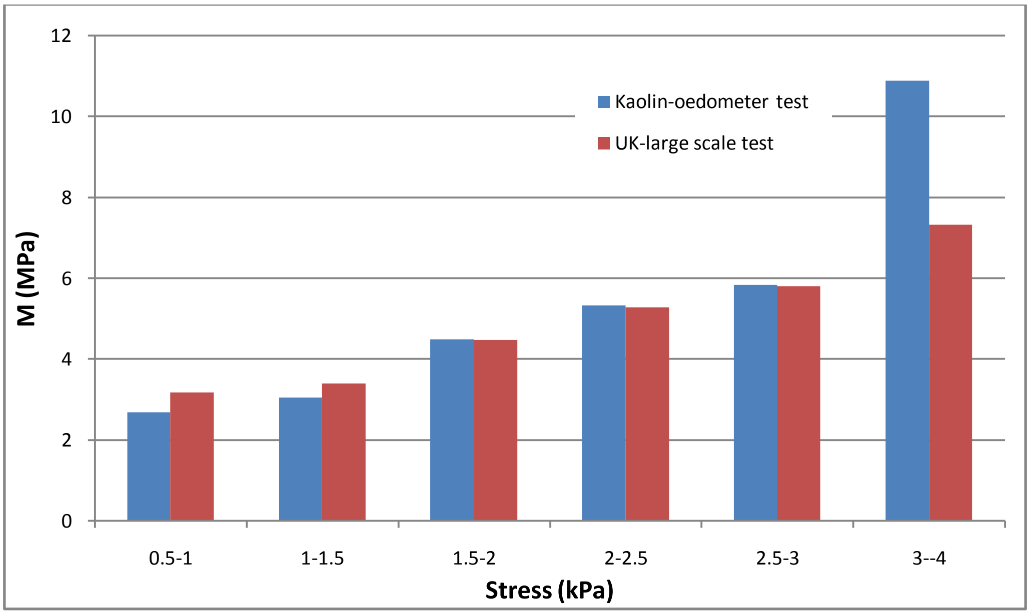


Figure 4.38 Comparison of constrained modulus (M) values for kaolinite in the oedometer and large scale consolidation test

Stresses on deep mix columns and on soil may be back-calculated for each individual test. The compression modulus (M) values of kaolinite for each stress range is known from the large scale consolidation test results on UK. The settlement values for each stress range are already measured for all groups of column improved systems. The stress on the soil is calculated form the conventional equation (Equation 4.5).

$$\sigma_{soil} = \frac{(\delta_{soil}).M_{soil}}{H} \tag{4.5}$$

Total stress in the stabilized system is given conventionally as in Equation 4.6.

$$\sigma_{system} = \sigma_{col}.a_s + \sigma_{soil}.(1-a_s) \tag{4.6}$$

Then the stress in the column can be calculated from Equation 4.7.

$$\sigma_{col} = \frac{\sigma_{syst} - \sigma_{soil}.(1-a_s)}{a_s} \tag{4.7}$$

Summary of the calculations is presented in form of ratio of stress on column to that on soil in Figure 4.39 against replacement ratio at different stress levels. The ratios are higher at lower stress levels, and they show a decreasing trend with increasing replacement ratio. Stress ratios in case of 30%C stabilized columns are very high namely 43, 22, 9 at replacement ratio of 0.19. Stiff columns take more loads. For other binder materials studied $\sigma_{col}/\sigma_{soil}$ ratios vary between 1-17 depending on stress level and a_s.

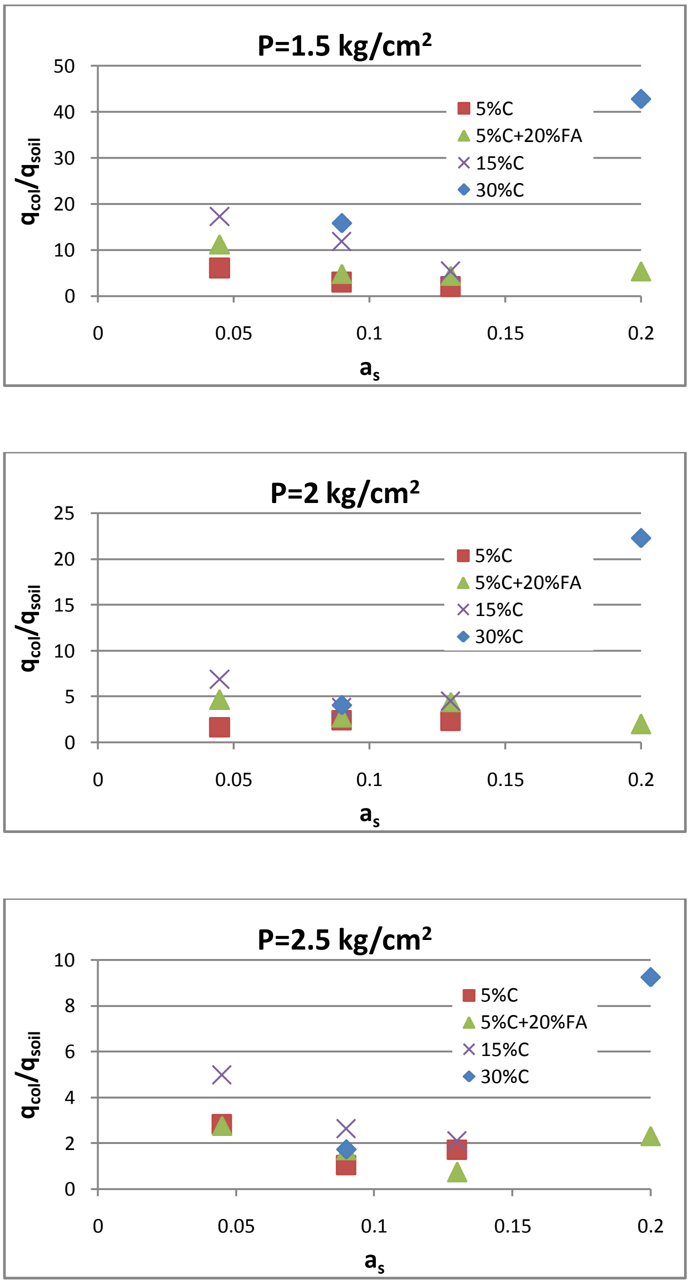


Figure 4.39 q_{col}/q_{soil} vs. a_s for P from 1.5 to 2.5 kg/cm^2

4.3 SUMMARY

In this chapter, the results of UC tests on the binder mixed soils and large scale consolidation tests on DMC columnar improved soft clay were presented. The findings for consolidation behavior of DMC improved systems were discussed. The effect of parameters of DMC (binder content, replacement ratio) on the improvement was determined. The conclusions are given in the next Chapter.

CHAPTER 5

CONCLUSION

5.1 GENERAL

Compression behavior of a soft clay improved by groups of deep mixed columns has been studied by performing large scale laboratory model tests. The effects of replacement ratio, binder type, stress level, amount of binder have been investigated in these tests. Before the large scale testing program unconfined compression strength of all mixes has been determined.

5.2 COMPRESSIVE STRENGTH OF CEMENT/CEMENT+FLY-ASH STABILIZED SOFT CLAY

1. The 5% cement addition does not improve the UC strength substantially. But as the cement content is increased or another type of pozzolanic binder is used with the cement, better strength improvement can be obtained. Especially 30%C admixture results in higher strength.
2. As cement percentage in the stabilizer increases, the ratio of UC strength of the treated soil to the one of untreated soil increases linearly (from 5 to 25 times). 20% FA addition with 5%C results in 1o times increase in UCS.
3. The C and C + FA addition increases the elastic modulus (E_{50}) of the treated soil. The 5% cement with the addition of 20% fly-ash mixed soil

gives an elastic modulus as much as that of 15% and 30% cement mixed samples. This shows the efficiency of FA addition for the improvement of strength and also modulus of soft soils. The E_{50}/UCS ratios for 5% and 15% cement content values are 100-600 and 300-500, respectively. For the cement content of 30% this ratio is about 200. The ratio of E_{50}/UCS is 450 for C + FA mixes.

5.3 CONSOLIDATION BEHAVIOR OF DMM GROUP COLUMN IMPROVED SOFT CLAY

1. The deformation characteristics of C/C+FA stabilized soft clay by columns has shown a change compared to the untreated soil. The effect of change of deformation behavior depends on the binder type and also the number of columns (replacement ratio, a_s).
2. Stiffer columns provide much better improvement. It should be stressed that there is a threshold cement content beyond which significant improvement occurs and it is practically very important to determine it.
3. Depending on the required level of improvement, cement content and a_s may be designed based on Figures 4.21 and 4.22 (β-p relationships).
4. For consolidation stresses between 0.5 and 2 kg/cm^2, using DMC and increasing the number of columns will significantly increase the moduli.
5. For most of binders studied (5%C, 5%C+20%FA, 15%C), percent increase in constrained modulus (M) is between 30-100%. This increase reaches to 140-800% for 30%C group of columns.
6. Large scale model tests have shown that the best approach to calculate the compressibility of deep mix stabilized soils is to conduct laboratory oedometer tests on stabilized and untreated soil samples and measure M moduli to be used in Equation 4.2. Equations 4.3 and 4.4 overestimate

the composite moduli roughly 2 to 5 times considering various stress levels and a_s values.

7. Stresses on the columns (σ_{col}) and on the soil (σ_{soil}) during composite loading is the basis of calculations on columnar improvement works. $\sigma_{col}/\sigma_{soil}$ ratios decrease with increasing replacement replacement ratio (a_s) at all pressure levels (Figure 4.39). Overall range is from 17 to 1 depending on the replacement ratio, stress level and binder type. If cement content is higher (e.g. 30%) $\sigma_{col}/\sigma_{soil}$ value is 22 at p=2.0 kg/cm^2 and 9 at 2.5 kg/cm^2 for a_s=0.20. Stiff columns take higher loads. At practical replacement ratios and common stress levels values vary roughly between 3-6.

5.4 RECOMMENDATIONS FOR FUTURE RESEARCH

1. As stiffer columns carry more load, it is recommended that compression characteristics of soils reinforced by stiffer binders should be studied (upper limit is mortar columns and piles).
2. Behavior of footings on DMC is expected to be different than that of 1D loaded areas. A laboratory model study on footings supported by DMC would be valuable.
3. Floating DMC improved system will show a different behavior than the site improved by end bearing DMC. Floating DMC improved system may be studied with the similar setup.
4. Numerical modelling of DMC improved systems may give valuable results for a comparison with the experimental findings.
5. The long term strength gain for stabilized soft clays may be investigated by chemical and mineralogical analysis.

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