KSCE Journal of Civil Engineering
Vol. 6, No. 4 / December 2002
pp. 389~398

Mechanical Properties of Weakly Bonded Cement Stabilized Kaolin

By Kyu Hwan Lee* and Song Lee**

Abstract

Soil mixing with cement is one of the fastest growing specialties in many coastal and offshore areas and is becoming means of improving poor ground conditions. The engineering properties of clay soils can be enhanced by addition of cement, thereby producing an improved construction material. While this ground improvement technique is gaining popularity, the strength-deformation behaviour over the long term and the soil structure of the clay-cement mixture are not well understood. This paper aims to investigate the mechanical behaviour of cement-stabilized Kaolin with Portland cement. An extensive laboratory program was carried out to determine the mechanical characteristics of kaolin-cement, with some brief examination of the effects of curing environment. Results show that while cement increases strength, it also reduces axial strain required to achieve failure under drained conditions. Furthermore, when the cement content is 5 percent or less, kaolin-cement may not improve with time after 28 to 56 days; this may be partially due to softening during curing. Addition of cement increases the degree of flocculation of the clay particles; this is associated with an increase in pH of the pore wate However, the pH decreases over time. The formation of various new reaction products has been identified using scanning electron microscopy (SEM) technique.

Keywords: *cement, cement-stabilized kaolin, clay, deep mixing, kaolin, portland cement*

1. Introduction

Development on soft clay deposits, typical of marine and coastal environments, is necessary to sustain a healthy economy associated with expanding ports and harbors. Soft clay is highly compressible, therefore, where deposits are thick, construction on soft clay can only take place by installing costly deep foundations or by altering the in situ soil conditions to improve the bearing capacity and reduce the compressibility of the unfavorable foundation material. The use of empirical correlations, based primarily on the unconfined compressive strength of the improved soil as prepared in the laboratory, is the current design practice for deep mixing. The unconfined compressive strength test, however, does not consider the undrained behaviour nor the effect of confining pressure, which have been shown to greatly affect the mechanical behaviour. Furthermore, the unconfined compressive behaviour is typically brittle and tests yield negligible residual strength (Tatsuoka and Kobayashi, 1983) so that design based only on results of unconfined compression tests may be extremely conservative. Furthermore, design is normally based on the strength of samples cured for no more than 28 days, so that the long-term behaviour of the soil is not considered. In the current study, it is found that the method of curing may have a large influence on the behaviour of the stabilized soil; this too must be considered carefully when establishing the soil properties in the laboratory. Little work prior to 1995 has been done to research the behaviour of clay-cement beyond the unconfined compressive strength. The scope of the study is to collect high quality laboratory results from a series of undrained and drained consolidated triaxial tests, oedometer, isotropic consolidation tests and unconfined compressive strength tests.

2. Description of Experimental Program

The laboratory tests were conducted on samples with moisture contents of 70 and 100%, cement contents of 2, 5 and 10% and curing periods of 7, 28, 56 and 112 days. Confining pressures for triaxial tests were 50, 100 and 400 kPa.

A lot of effort was spent in establishing a procedure for preparation of high-quality samples. The goal was to produce homogeneous samples with minimum air voids, and

*Member, Full time Lecturer, Konyang University (E-mail: khlee@konyang.ac.kr)
**Member, Professor, University of Seoul (E-mail: songlee@uos.ac.kr)

The manuscript for this paper was submitted for review on Decemer 12, 2001.

to produce a group of samples for each mix type that had the same physical properties. Images captured using a SEM have been analyzed to increase the understanding of how the resulting structure contributed to the strength/deformation characteristics of the clay-cement mixture. The tests included both drained and undrained consolidated triaxial tests, isotropic consolidation tests and oedometer tests, unconfined compressive strength tests with humid and water curing samples. pH tests also included.

2.1 Properties of Soil and Cement

The soil used in the clay-cement samples for the laboratory tests was kaolin from Indonesia. The atterberg limits, grain size distribution, specific gravity and pH of the Kaolin were determined in the laboratory. The specific gravity of the kaolin was determined with a pycnometer; at least 50 g of dry material was mixed with distilled water. The soil pH was measured by mixing 10.0 g of dry soil with 40 ml of distilled water. The physical and chemical properties of Belitung Utama Kaolin and the mineralogical composition of soils are given in Tables 1 and 2.

Emerald brand Portland cement (Type I) from the Green Island Cement Company in Hong Kong was used as the stabilizing agent in all laboratory samples; this is equivalent to ordinary Portland cement. The chemical composition of the cement, as provided by the manufacturer, is summarized in Table 3.

2.2 Preparation of Mixing Sample

The strength of soil-cement samples achieved in the laboratory is normally far greater than that achieved in the field for the same mix. It is nearly impossible to mimic the exact field conditions in the laboratory, which makes it difficult to design efficient deep mixing programs. Therefore, it is very important, when preparing laboratory samples, to match as best as possible the field conditions. Most importantly, this includes the method of mixing and the curing environment. Very little detailed information was in literature regarding sample preparation for laboratory tests cement-stabilized clay. Methods were often described. But significant problems frequently encountered in where were not addressed so that when these methods were attempted for the current project, the results would be unsatisfactory. Weak seams and trapped air pockets between lifts were the most common problems encountered, particularly for the samples prepared with only 70% moisture content.

According to the reviewed literature, all samples used for strength tests were cylindrical and had a length to diameter ratio of 2; this complies with both ASTM standards and British Standards.

In most cases, the triaxial and unconfined compressive strength test samples had a diameter of 50 mm and a length of 100 mm.

Table 1. Physical Properties of the Kaolin

Properties	Symbol	Value
Plasticity index	PI (%)	37.0
Liquid Limit	LL (%)	77
Specific Gravity	Gs	2.65
Clay fraction pH Soil Type	%<2 μm	47 4.58 MH

Table 2. Chemical Analysis of the Kaolin

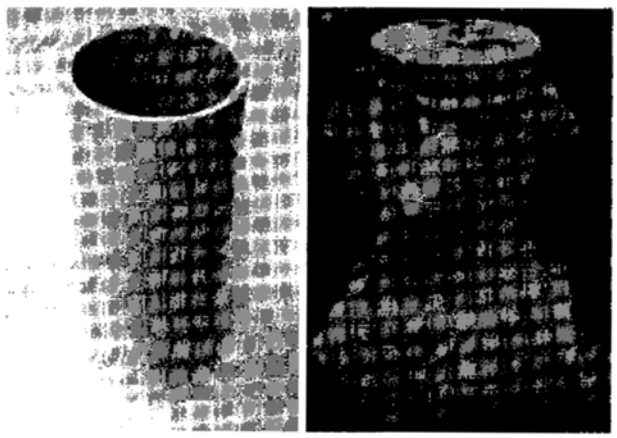
Constituent	%	Constituent	%
SiO_2	49.7	MgO	0.02
Al_2O_3	34.5	K_2O	0.60
Fe_2O_3	0.55	Na_2O	0.04
TiO_2	0.21	Loss of ignition	14.2
CaO	0.03		

Table 3. Chemical Composition of Cement

Constituent	%	Constituent	%
SiO_2	20.2	MgO	0.6
Al_2O_3	6.3	SO_3	2.4
Fe_2O_3	3.3	Na_2O	0.35
CaO	65.7	Loss of ignition	1.2

2.3 Mixing and Casting of Sample

Sample preparation for the current study was conducted twice a week; preparation of each series of samples took place over a two-day period. On the first day, oven-dried kaolin was mixed with the appropriate mass of distilled water to obtain the desired moisture content. Both the kaolin and water were at room temperature when mixed. Mixing was done using a large electric mixer fitted with a paddle type-mixing blade until a homogeneous consistency was achieved throughout. The wet clay was sealed in plastic and allowed to soak at room temperature for approximately 24 hours. This allowed moisture to penetrate any small clumps of dry clay minerals and for any physical changes in the clay caused by the addition of water such as swelling to occur prior to casting the samples. On the second day, following the soaking period, the appropriate mass of dry cement required to achieve the desired cement content was measured and added to the wet clay. Immediately, the material was mixed thoroughly using the same electric mixer until a homogeneous mixture was achieved. This second



(a) (b)

Fig. 1. Sample Mixing Apparatus, Mould for Sample Casting and Mould with Fabric on Ends

mixing stage took less than ten minutes and was monitored closely to ensure a uniform mix within a minimum time period. The exact mixing time was not considered to be a critical factor contributing to the quality of the samples. However it was kept to a minimum.

The material was then immediately cast in the moulds. The moulds used for preparation of samples consisted of high-density plastic tubes, with a 50 mm internal diameter and a length of approximately 150 mm (Fig. 1(a)). The tubes were lined with PVC to reduce friction between the sample and the mould during casting and extrusion.

2.4 Sample Curing

Once the material was compacted in the mould, the ends were covered with heavy plastic and held tightly in place with thick rubber bands (Fig. 1(b)). The moulds were placed vertically in a constant temperature (20°C) and constant humidity (65%) room for 24 hours. After this period, the plastic was replaced with porous fabric and the samples were submerged vertically in a distilled water bath at 20°C for the remainder of the curing period. The samples were in the constant temperature and constant humidity room for the duration of their curing period. Due to the high moisture content of the samples prepared for this study, extruding the samples prior to curing was not possible as the material was too soft. Preparation of the oedometer samples was done in a similar manner. Material was cast in small, PVC-lined steel cylindrical moulds, slightly larger than the diameter of the consolidation ring. After casting, the top of the mould was covered with plastic for the initial 24-hour curing period. After this time, the steel base-plate was removed, and both ends were tightly covered with fabric and the sample was submerged upright in the bath of distilled water for the remainder of the curing period.

Three oedometer tests were performed without cement, at 40, 50 and 70 percent moisture content. These samples were compacted directly in the consolidation ring using the wooden rammer. At the start of the sample preparation program, the intent was to seal the ends of the sample tubes in wax, and allow them to cure upright in the moisture and humidity controlled room. For samples cured in this way, some shrinkage and desiccation (horizontal hairline cracking) was observed following extrusion after 7 days of curing, and therefore, the wax curing method was abandoned for future samples and the 7-day wax-sealed samples were discarded. However, out of interest, the 112-day samples with a moisture content of 100 percent were cured in wax for the full curing time.

When extruded, the moisture content of these samples was 3 to 6 percent below that of the same samples, cured for 112 days in water, following the method described previously. All laboratory tests were carried out on the 112-day samples to compare curing methods. Besides the shrinkage and desiccation observed, the failure behaviour of the material was found to be much more brittle than of those cured in distilled water. It was concluded that the wax sealed curing method is not suitable for the series of laboratory tests carried out herein due primarily to the shrinkage and cracking. However, if the humidity was higher or temperature lower in the curing room, the wax sealed curing method may have been more successful and appropriate.

3. Results of Tests and Discussion

3.1 The pH Value

The dependence of pH value on the cement content is showed in Fig. 2. For the tests, the required cement quantity was admixed to the air dry pulverized kaolin, then 40 mls distilled water was added. After the last shake-up of the suspension settled down and the glass container of the pH elec-

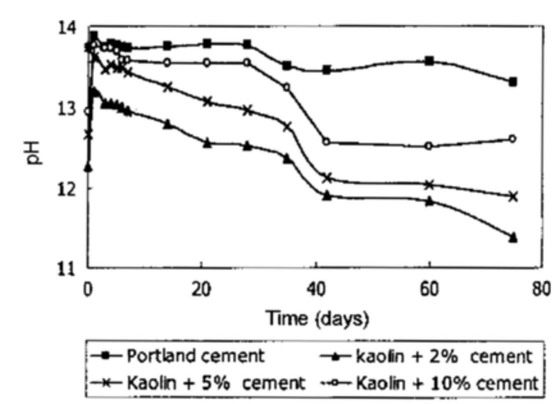


Fig. 2. pH Value of the Mixture with Kaolin and Cement as a Function of Time

trometer was then filled up with pure water by means of a pipette. The high pH value of samples containing hydrating cement may be attributed to the dissociation of OH^- ions from the $Ca(OH)^2$ produced during hydration. According to the hypothesis of clay cement interaction, a primary and secondary process must be distinguished in the consolidation of the clay cement mixture.

The primary process includes hydrolysis and hydration of cement, in the course of which the usual hydration products appear, and the pH value of the water increases.

The calcium hydroxide produced in this period and consumed during the course of secondary processes is partly replaced by the lime produced by cement hydration. As similar to the general case of concrete, the pH value of the pure Portland cement, and pure clay did not much change with time in the given case but that of the soil cement decreased, implying a process consuming OH^- ions during hardening.

As discussed above, the pH value of mixing sample increased initially due to the process of hydrolysis and the hydration of cement and then decreased with time due to the pozzolanic action of soil cement. The drop rate of pH is larger at small cement ratio than high cement ratio.

3.2 Plasticity and Volume Change

Liquid and plastic limits were obtained for both the untreated soils and cement-treated kaolin according to BS specifications. For the cement-treated soils the limits were run immediately after adding the cement pastes to the dry soil and mixing in the water. The kaolin was added to water and then cured in humid room during 24 hours before mixed with cement. Table 4 shows that cement agent caused a drop in the plasticity index of the kaolin. As to the cement ratio increased from 2% to 10%, liquid limit of mixing soil increased a little bit. The change in viscosity by adding cement and cement paste soaking up the water results in a change of liquid limit so that more water needed to liquefy the mixing soil.

It is interesting to note that in the given soil the reduction of plasticity index value was due mainly to the increased plastic limit. According to the Lee (1999) when a cement ratio below 5% is added to clay slurry, it becomes very viscous and has a higher liquid limit than clay itself. On addition of more cement, the liquid limit then drops. However this study shows that liquid limit increases to the 10% cement ratio.

Table 4. Variation of Atterberg Limit with Cement Ratio

Cement ratio (%)	Liquid limit (%)	Plastic limit (%)	Plastic Index(%)
None	77.0	40.0	37.0
2%	77.5	46.0	31.5
5%	78.0	46.2	31.8
10%	79.0	46.5	32.5

3.3 Unconfined Compressive Strength (UCS)

The unconfined compressive strength of improved soil is most frequently used as an index of the strength of stabilized ground. An examination was made on the relationship between UCS and curing conditions in respect of different conditions including water content, cement ratio and curing times. For the desired strength of cement stabilized soil, proper curing is paramount for obtaining optimum performance from a given set of mixture proportions. Because curing method contributes a great deal to the subsequent strength properties of soil cement. Also, for the curing process of soil cement a well-defined amount of water is needed to allow the cement to react with soil. It is very difficult to adjust the curing condition and well-defined water content in deep mixing work so that we have to followed the field conditions. For these, a series of unconfined compression tests under water and humid curing have been performed.

Fig. 3 shows the relationship between unconfined compressive strength of water and humid water curing samples at 5% cement ratio and 70% water content for various curing days. The results indicated that the axial strain of peak strength of humid curing samples is larger than that of the water curing samples. Other test results with 2% and 10% cement ratio also showed similar behaviour to the 5% cases.

For all mixes, the strength increased with age, at decreasing rate. The peak strength following humid curing was far greater than that following water curing. At 5 percent cement, the strength following humid curing was up to 3 times that following water curing.

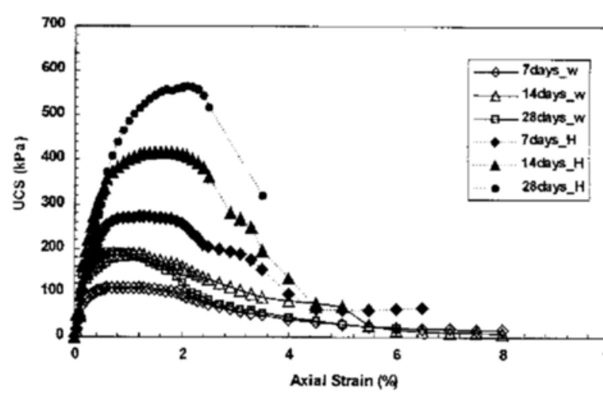


Fig. 3. Unconfined Compressive Strength Curves of Water and Humid Curing Sample with 5% Cement Ratio

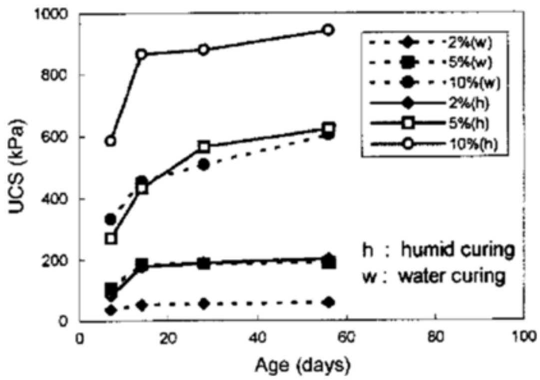


Fig. 4. Unconfined Compressive Strength with Cement Ratio, Curing Time and Conditions

The failure behaviour of water curing samples showed more brittle and very week bonding effect compare to the humid condition, so the braking of bonding occurred quickly in water cured samples. Because placed in direct contact with a free supply of water the clay will swell before the cement has time to harden and it will develop of disruption of the fabric as the curing is taking place. Even the specific surface of cement is larger than clay; small cement ratio is not enough to cover the clay surface so that clay particle expanded by water soaking with time.

Fig. 4 is the result of unconfined compressive strength of humid and water-cured samples with curing time. The UCS increased quickly and lineally up to the 14 days in both of water and humid cured samples. But after that the increasing ratio of strength decreased with time or almost same. The strength of water curing samples with 2 and 5 percent samples were almost the same after 14 days curing.

The moisture content following curing was determined for each sample. Results show that the moisture content following humid curing is significantly less than that following water curing. Furthermore, the humid cured samples continued to lose moisture with time, whereas the moisture content of water cured samples dropped initially and then increased after 28 days. Considering the data from samples with no cement, it can be seen that there is a larger difference in moisture content between water cured and humid cured samples. This indicates that either cement reduces the capacity of kaolin to absorb moisture during water curing, or reduces the capacity of the kaolin to lose moisture during humid curing.

Under unconfined conditions, failure is brittle and by crushing, particularly at greater curing times (Porbaha *et al.*, 2000). The top of the sample normally remains intact and nearly vertical cracks were formed in the bottom of the sample as crushing proceeds. Occasionally, the opposite is true so that the bottom of the sample remains intact. Spalling is also seen from time to time. Also, oblique fractures sometimes occurred, separating the sample into two nearly equal pieces, as was the case for most of the triaxial tests.

3.4 Results of Consolidation Tests

Some of the significant concepts observed from the oedometer and isotropic consolidation tests on pure kaolin and kaolin-cement are summarized below. Note that conclusions are based on somewhat limited test results and are not intended to characterize condition not considered in the current study.

3.4.1 Apparent Consolidation Yield Stress

When the cement content is sufficiently high, the consolidation behaviour of kaolin-cement resembles that of overconsolidated clay in that an apparent consolidation yield stress can be determined from the consolidation curve. However, in no way does this apparent consolidation yield stress provide information about the stress history of the kaolin-cement. The apparent consolidation yield stress is roughly equivalent to the pressure at which the cement bonds begin to fail during consolidation.

A plot of the apparent consolidation yield stress against curing time is provided in Fig. 5. As seen the oedometer tests sometimes yield a greater value due to the larger applied loads. The apparent consolidation yield stress increases with cement to moisture content ratio and curing time, up to at least 56 days. However, when the cement content is sufficiently low, a reduction in the apparent consolidation yield stress was observed following 56 days of curing. This indicates deterioration of the kaolin-cement with time; the same observation was made of the strength test results. When exposed to water during curing, the apparent consolidation yield stress of weakly cemented materials may decrease due to softening.

3.4.2 Compression and Swelling Indices

Compression index increases with moisture content but is

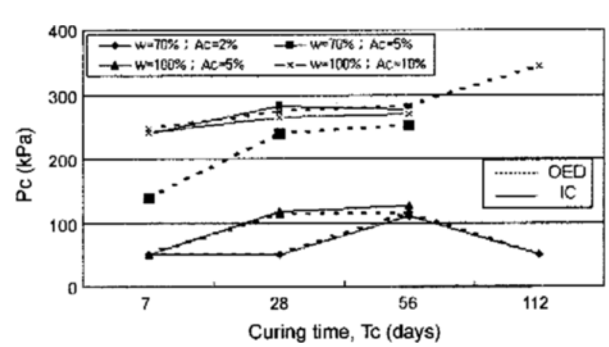


Fig. 5. Apparent Consolidation Yield Stress vs. Curing Time

relatively independent of cement content. This is because the cement bonds have failed at the consolidation pressures at which the compression index was calculated. The results of the oedometer and isotropic consolidation results were typically different; usually, this was because the oedometer tests were consolidated further beyond the apparent pre-consolidation pressure. However, backpressure applied during isotropic consolidation reduces compression; this was indicated by a decrease in the compression index and the coefficient of volume compressibility of the isotropic consolidation test results as compared with the oedometer test results.

The pozzolanic reactions that occur when cement is added to wet clay cause changes in both the chemical and physical properties of the soil. Cement increases the pH of the pore water so that it causing flocculation of clay particles. As curing proceeds, however, the pH decreases and the degree of flocculation may be reduced, causing a very small increase in the void ratio up to at least 56 days (Fig. 6). But the change was very small and may not be meaningful.

The trends in the swelling index are very similar to those in the compression index, except the swelling index is a function of cement content.

This is because cemented particles interlock during consolidation, causing permanent deformation, which is a function of the degree of cementation.

3.4.3 Coefficient of Consolidation

Based on limited data, it was observed that the coefficient of consolidation generally increased with curing time and cement content up to a pressure roughly corresponding to the apparent consolidation yield stress. When the cement to moisture content is low, there is very little change in the coefficient of consolidation with consolidation pressure.

While it is difficult to see this trend, up to a given consolidation pressure, which depends on the sample mixture, the coefficient of consolidation generally increased with curing time (Fig. 8). This is the desired effect and the same observation was made by Uddin (1995) at all pressures. However, at high consolidation pressures for the current study, the coefficient of consolidation actually decreased with curing time. It is suggested that this reversal occurs at approximately the apparent consolidation yield stress and is due to failure of cement bonds and subsequent collapse of structure during consolidation. Patterns observed in the apparent consolidation yield and compression index also support this theory.

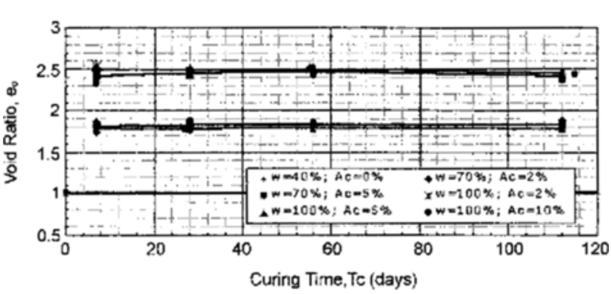


Fig. 6. Average Void Ratio vs. Curing Time

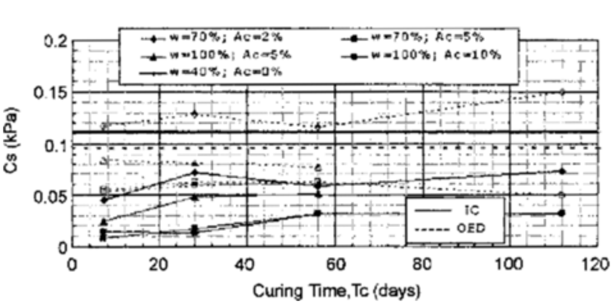


Fig. 7. Swelling Index vs. Curing Time

3.4.4 Coefficient of Volume Compressibility

An increase in curing time, up to at least 56 days, generally leads to a decrease in the coefficient of volume compressibility during both loading and unloading (Fig. 9), except when the cement is 2% or occasionally during the initial loading stage for the oedometer tests. This indicates

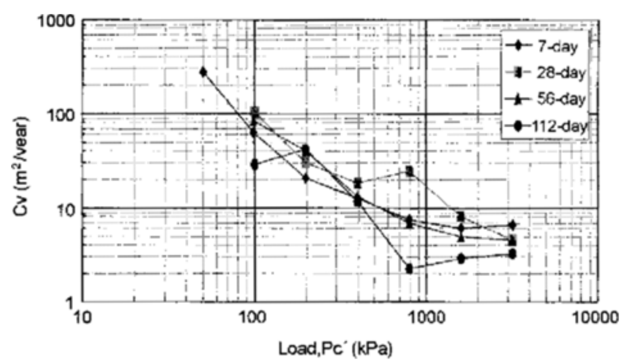


Fig. 8. Coefficient of Consolidation vs. Load for w=100% & A_c=10% (oedometer tests, loading only)

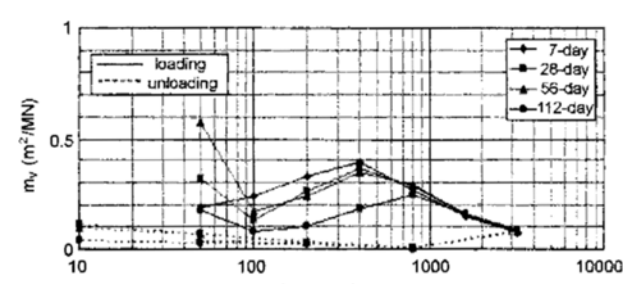


Fig. 9. Coefficient of Volume Compressibility vs. Load for A_c=10% & w=100% (oedometer tests)

that as curing proceeds, the material becomes more stiff and consolidation takes more time.

At high loads, the change in the coefficient of volume compressibility with curing time was small to negligible. At 2% cement and in some cases, beyond approximately 56 days of curing, softening may occurring with curing time due to insufficient cementation.

3.5 Triaxial Compression Test Results

Both drained and undrained triaxial compression tests were conducted at confining pressures between 50 and 400 kPa. While general observed trends often lead to similar conclusions, the drained test results were typically far easier to interpret than the undrained test results. This is partially due to the significant excess pore pressures generated during undrained tests, which can greatly influence the resulting shear behaviour, particularly for bonded soils.

3.5.1 Peak Strength

The peak strength, as determined from drained tests, generally increased with cement to moisture content ratio and curing time, up to at least 28 days (10(a) and (b)); However, the material often did not continue to improve beyond 28 or 56 days of curing, and when the moisture content was 70%, the peak deviator stress sometimes decreased significantly following an initial improvement.

Some factors contributing to the reduced improvement or deterioration include insufficient cement and/or too much moisture, curing in water leading to softening and/or leaching of cement over time, and lack of coarse-grained particles in the stabilized material. Furthermore, due to the different compaction methods, a greater degree of softening beyond 28 days of curing may be associated with the samples with 70% moisture content.

More research is required to understand exactly what phenomena lead to the observed reduced improvement or deterioration with curing time.

3.5.2 Fully-Softened Strength

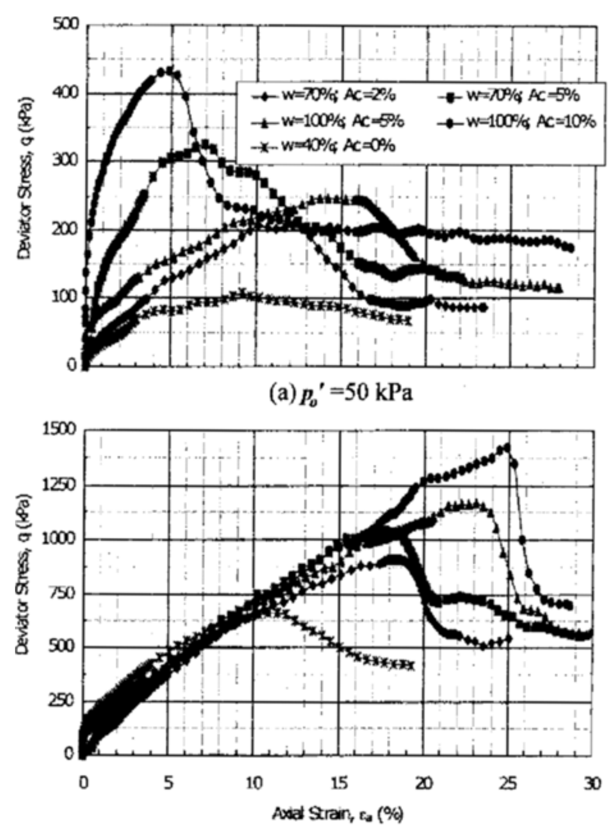
Cement content and confining pressure generally govern fully softened strength; curing time and moisture content have no little influence over fully softened values.

For drained tests, the fully-softened stress increases with cement content and confining pressure; for undrained tests, only confining pressure influences the fully-softened strength.

At high confining pressures and/or low cement contents, when the cement bonds have failed during consolidation, the kaolin-cement behaves more like an un-bonded material, and the fully softened strength is a function of confining pressure only.

3.5.3 Material Stiffness

The material stiffness is evaluated by considering the axial strain associated with peak stress conditions. Under undrained conditions, behaviour was stiffer than under drained conditions. Furthermore, material stiffness was largely influenced by confining pressure, which also governed the volume change during consolidation, and hence the material stiffness during shear. At 50 kPa confining pressure, the axial strain at peak conditions decreases with an increase in the cement to moisture content ratio (Fig. 10(a)); this is an indication of an increase in the materials stiffness. As confining pressure increases to 400 kPa, the opposite trend occurs; the axial strain is directly related to the materials strength and the materials stiffness are governed by confining pressure (Fig. 10(b)). When pure kaolin is considered, cementation effects are irrelevant and the behaviour is only a function of the moisture content. At 400 kPa confining pressure, the pure kaolin has the same stiffness as the cemented material but it fails at a much lower axial strain due to its reduced strength (Fig. 10(b)).



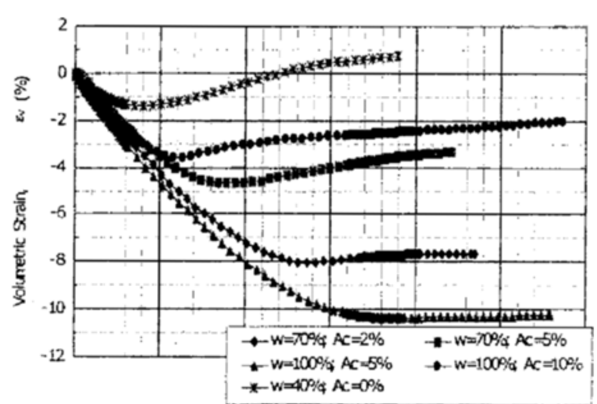
(a) p_o' =50 kPa

(b) p_o'=400 kPa

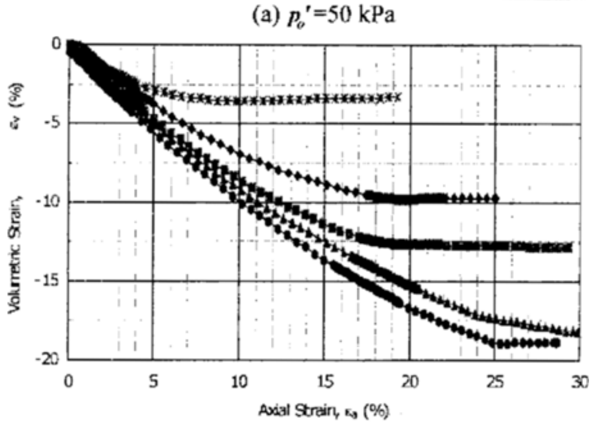
Fig. 10. q vs. ε_a for T_c=56 days (CID)

3.5.4 Volumetric Strain

When interpreting the volume change characteristics during drained shear, it was found that trends are largely influenced by confining pressure, and hence the volume change that occurred during consolidation, prior to shear. At sufficiently high confining pressures or low cement contents, cement bonds failed during consolidation causing significant volume change, leading to very little volume change during shear. For this reason, at low confining pressures (i.e. 50 kPa), volumetric strain tended to decrease with an increase in strength associated with an increase in cement to moisture content ratio and curing time (Fig. 11(a)); the opposite trend was observed at high confining pressures (i.e. 400 kPa) (Fig. 11(b)) when the structure collapsed during consolidation. It is commonly reported that cement-stabilized clay behaves as an over-consolidated material (i.e. Kohata *et al.*, 1997); based on the strain-softening behaviour observed in the stress-strain plots, this theory is reasonable. However, volume change during drained shear indicates the behaviour more closely resembles that of normally consolidated to slightly over-consolidated material. For all drained tests on kaolin-cement, volumetric strain remained negative throughout shear. Most samples contracted up to peak stress conditions, and then proceeded to dilate slightly or maintain a constant volume after failure; low confining pressures increased the tendency for the sample to dilate.

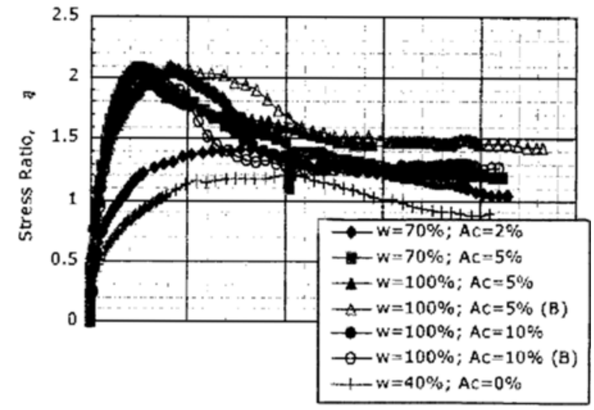


(a) p_o'=50 kPa

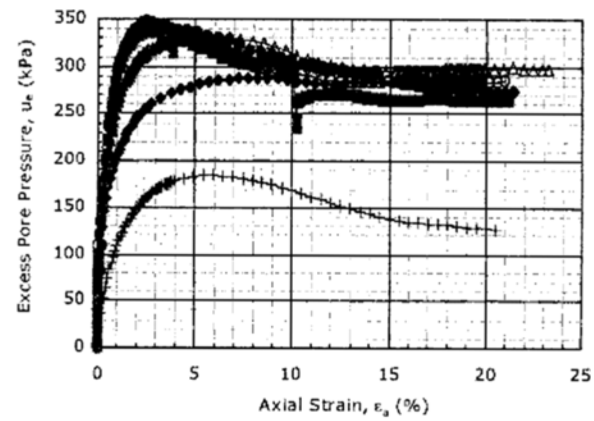


(b) p_o'=400 kPa

Fig. 11. ε_v vs. ε_a for T_c=56 days (CID)



(a) η vs. ε_a



(b) u_e vs. ε_a

Fig. 12. p_o'=400 kPa & T_c=28 days (CIU)

However, pure kaolin at a moisture content of 40% behaved as an over-consolidated material at a confining pressure of only 50 kPa (Fig. 11(a)); this is indicated by the positive volumetric strain at the end of shear. It is suggested that this observed contraction of kaolin-cement up to peak stress conditions, followed by very little to no dilation during strain-softening, is a fundamental concept of cement-stabilized soils, that differs from un-bonded clay. Progressive failure of cement bonds during shears results in strain-softening behaviour, but in no other way does cement-stabilized clay resemble that of an over-consolidated material. This fundamental concept may be more evident in samples cured in water and with relatively low cement contents.

3.5.5 Excess Pore Pressure

Similar to drained test results, behaviour based on the excess pore water pressure generated during undrained shear tests resembled that of a normally consolidated to slightly over-consolidated material.

For all kaolin-cement samples, the excess pore water pressure remained positive throughout shear; all samples contracted up to approximately peak stress conditions, and then proceeded to dilate after peak. The degree of dilation following peak conditions increased with cement to moisture content ratio (Fig. 12(b)) and decreased with confining pressure. Samples of pure kaolin exhibited over-consolidated behaviour when the confining pressure was only 100 kPa as indicated by the negative excess pore pressure at fully softened stress conditions. The peak excess pore pressure generally increased with curing time and cement to moisture content ratio, although the strong influence of confining pressure often minimized the effects of these variables.

3.5.6 Failure Behaviour

The failure behaviour for the drained tests was brittle, particularly at high cement to moisture content ratios and curing times and low confining pressure. A clear shear plane develops upon failure for all of the drained triaxial tests due to plastic shearing; occasionally, two planes develop simultaneously. More sample disturbance near the failure plane is associated with lower cement to moisture content ratios and lower curing times. Sample barrelling occurred during shear. The undrained triaxial tests exhibited strain-softening failure behaviour. Unlike the drained tests, failure was not brittle. In most cases, a clear shear plane developed during shear due to plastic shearing. In some cases, sample failure was by crushing. As the cement to moisture content ratio and curing time increased, sample disturbance near the shear plane decreased. Some sample barrelling occurred during shear, but to a lesser extent than for the drained tests.

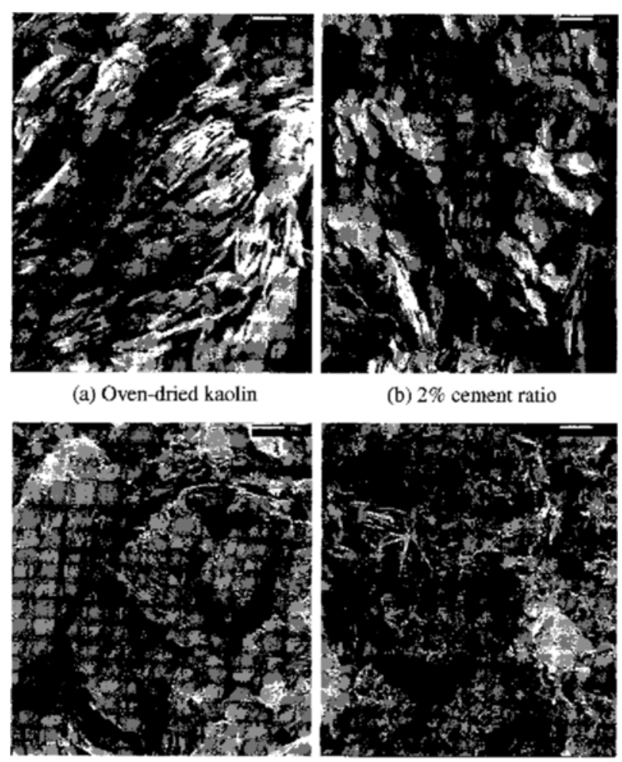
4. Micro Structure of Kaolin-Cement

The mechanical behaviour of soil is strongly influenced by the shape, size and surface characteristics of the soil particles (Mitchell, 1993). Furthermore, the soil structure, which is also a function of the individual particle properties, strongly influences the soils behaviour. Sufficient detail of the particle properties can only be observed with the aid of a Scanning Electron Microscope (SEM). Prior to sample preparation, the pore water must be removed, replaced or frozen. This is very difficult to do without disturbing the structure of the sample and there is some debate as to what method is most suitable for different types of soils.

The edges of kaolin particles have a negative charge when in a high pH environment and a positive charge when in an acidic environment (Mitchell, 1993). When only kaolin is present, the pH was measured to be roughly 3.6 to 4.0, so that the edges are positively charged. However, when at least 2 percent cement is added to kaolin, the pH of the pore water increases to above 13 after one day of curing, and is very near the pH of 100 percent cement, which approaches 14. This increase in pH is due to an increase in the electrolytic concentration of the pore water that results from dissociation of calcium hydroxide (Probaha *et al.*, 2000). Therefore, Ca cations are attracted to the negatively charged clay particles and the clay particles become flocculated.

The micrographs obtained for cemented treated soil set up with kaolin and 2%, 5%, 10% cement ratio are shown in Figs 13(a), 13(b), 13(c) and 13(d).

Kaolin generally has an aggregated and edge-to-face and edge-to-edge flocculated structure. The addition of cement causes the kaolin to be less aggregated but more flocculated, as compared with the mixture with no cement. At only 2 percent cement, the cement could not be seen in the SEM photomicrographs, yet the particle structure was still affected by the cement. At 5 percent cement, the cement was seen only sometimes indicating that not all kaolin particles are cemented. At 10 percent cement, the presence of cement was clear throughout the material. These observations agree with those made of the laboratory test results; with 5 percent cement or less, the mechanical properties of the stabilized material are difficult to detect or do not



(a) Oven-dried kaolin (b) 2% cement ratio

(c) 5% cement ratio (d) 10% cement ratio

Fig. 13. Scanning Electron Micrographs of Soil Systems

change significantly with curing time. Flocculation decreases with an increase in curing time, as indicated by a slight increase in void ratio, and is due to a reduction in pH with curing time. This observation corresponds to a slight reduction in friction angle with curing time, which is directly related to the degree of flocculation. Other authors (i.e. Uddin, 1995) observe an increase in the friction angle with curing time, but this may be due to an increase in flocculation that is associated with a loss of moisture during humid curing.

Shrinkage occurs during drying of the samples prior to SEM analysis. Therefore, the void ratio cannot be interpreted accurately from the SEM images.

It is not known if shrinkage during drying was significant enough to damage the microstructure of the kaolin and kaolin-cement; it is believed that any damage was relative so that the comparisons are still relevant. More research is required with respect to SEM images of kaolin-cement.

5. Conclusions

Based on the results of this study on the mechanical characteristics of kaolin-cement, some general overall conclusions can be made, as follows. Note that conclusions are based on somewhat limited test results and are not intended to characterize conditions not considered in the current study.

Curing environment has a significant influence over the properties of soil-cement. Particularly at low cement contents, water cured samples have a much lower strength than humid cured samples, but they will be less stiff and possess more ductile failure behaviour. Furthermore, humid cured samples often lost significant amounts of moisture during curing so that some samples were damaged with cracks. Therefore, when developing a laboratory program for deep mixing projects, the curing environment must be considered carefully so that it simulates the field conditions as best as possible. Sites where DM is used typically have thick deposits of soft, wet clay. Therefore, water curing of laboratory samples is often more appropriate than humid curing when designing an effective and efficient DM program.

Cement treatment of clay causes the clay particles to be more flocculated but less aggregated. This increase in the degree of flocculation is likely due to the rise in pH and hence the electrolytic concentration of the soil pore water due to the addition of cement. As the cement content increases, so does the pH and the degree of flocculation. This flocculation causes an increase in the peak friction angle of cement treated clay. Furthermore, the pH of the pore water decreases as curing proceeds. It is suggested that the degree of flocculation and hence the friction angle may also decrease with curing time.

While an increase in cement content does increase the peak strength of the treated soil, it also increases the stiffness thereby reducing the strain at which failure occurs. Therefore, cement causes failure to be more brittle and catastrophic under drained conditions.

In the past, cement treated clay has been described by some authors (e.g. Kohata *et al.*, 1997) as an over-consolidated material. Consolidation curves obtained from cement-treated clay do resemble that of an over-consolidated material, however, it is proposed that the apparent consolidation yield stress represents the pressure at which the cement bonds begin to fail and the virgin compression curve is formed when the structure collapses. Therefore, unlike over-consolidated material, the consolidation curve for soil-cement does not reveal information regarding the soils stress history.

Many questions remain unanswered with regards to the properties of soil-cement, and how they change with curing time and cement content. More research is recommended as follow-up to the current study.

References

Bergado, D.T., Anderson, L.R., Miura, N., and Balasubramaniam, A.S. (1996). *Soft ground improvement in lowland and other environments.* ASCE Press, New York.

Kedzi, A. (1979). *Stabilized earth roads, developments in geotechnical engineering 19,* Elsevier Scientific Publishing Company, New York.

Lee Yeong (1999). *Annual report on the cement mixing soil,* BBR Ground Engineering Singapore.

Mitchell, J.K. (1981). *Soil improvement methods and their applications in civil engineering,* Department of Civil Engineering, North Carolina State University, Raleigh.

Mitchell, J.K. (1993). *Fundamentals of soil behavior,* 2^{nd} Ed., John Wiley & Sons, Inc., New York.

Porbaha, A., Shibuya, S., and Kishida, T. (2000). "State of the art in deep mixing technology." Part III: Geomaterial Characterization. *Ground Improvement.* Vol. 3, 91-110.

Sivapullaiah, P.V., Sridharan, A., and Bhaskar Raju, K.V. (2000). "Role of amount and type of clay in the lime stabilization of soils." *Ground Improvement.* Vol. 4, 37-45.

Tatsuoka, F. and Kobayashi, A. (1983). "Triaxial strength characteristics of cement-treated soft clay". *Improvement of Ground, Proceedings of the 8^{th} ECSMFE,* Helsinki, 23-26 May 1983. Vol. 1. pp. 421-426.

Uddin, M.K. (1995). *Strength and deformation characteristics of cement treated Bangkok clay,* D. Eng. Dissertation No. GT-94-1, Asian Institute of Technology, Bangkok, Thailand.

Wissa, A.E.Z. and Ladd, C.C. (1964). *Effective stress-strength behavior of compacted stabilized soils,* Soil Mechanics Division, Department of Civil Engineering, Massachusetts Institute of Technology. Research reports R 64 -32.