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Mechanical Characteristics of Kaolin-cement Mixture

카올린-시멘트 혼합재료의 공학적 특성

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요 지

연약 지반개량을 위한 심층혼합처리 공법의 사용은 점차로 증가 추세에 있으며 특히, 일본과 해안가에 인접한 동남아 국가는 물론 스칸디나비아지역에서도 보편화된 개량공법으로 각광을 받고 있다. 시멘트는 지반의 강도를 증가시키고 압축성을 감소시키는 역할을 한다. 따라서 심도가 깊은 퇴적지반이나 해안지역에서 지반의 지지력 증가나 압밀침하를 감소시키기 위해 시멘트를 개량재료로 사용할 수가 있다. 연약지반 처리를 위한 고화제로서 시멘트의 사용이 증가하고 있음에도 불구하고 이에 대한 응력-변형특성이나 혼합처리 흙의 구조적인 특성 등의 역학적인 거동특성을 명확하게 파악하지 못하고 있다. 본 연구에서는 시멘트 고화처리 흙의 역학적 성질을 파악하기 위해 카올린을 이용하여 최대 10%의 시멘트를 첨가하여 7일에서 112일까지 양생기간을 변화시켜 삼축압축시험, 일축압축시험, 등방압밀 및 표준압밀시험등을 수행하였으며 이에 따른 시멘트 고화처리 흙의 역학적인 특성을 파악하고자 하였다. 또한 혼합토의 시료 제조 및 양생방법등 일련의 시험과정에 대한 절차 및 방법에 대하여 기술하였다.

Abstract

Ground improvement technique of cement stabilization via Deep Soil Mixing with dry cement is gaining popularity, particularly in Japan and other parts of Southeast Asia and in Scandinavia. Cement can be mixed with deep soft clay deposits, typical of marine environments, to improve the bearing capacity and/or reduce the compressibility of the material so that an otherwise poor site can be developed. However, the strength/deformation behaviour and resulting soil structure of the clay-cement mixture is presently not well understood with respect to both dry and wet mix methods. An extensive laboratory test was carried out to determine the mechanical characteristics of kaolin-cement, with some brief examination of the effects of curing environment. Laboratory tests include triaxial tests, unconfined compression tests, isotropic consolidation tests and oedometer tests. Cement contents up to 10 percent were considered and water curing was employed. Samples were cured for 7 to 112 days while submerged in distilled water. Conventional laboratory tests were also performed. In this paper, the laboratory testing program is described and various sample preparation techniques are discussed. Preliminary triaxial compression test results and trends at varying moisture contents, cement contents, confining pressures and curing times will be presented.

Keywords : Cement, Deep Mixing, Deformation, Ground Improvement, Soft clay, Strength

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1. Introduction

Ground improvement of soft clay deposits through Deep Soil Mixing methods has become an extremely popular industry in Southeast Asia, as well as areas of Scandinavia, and more recently, in the United States. In the past decades, research and development in the area of DSM has been rampant, particularly in Japan, as design standards and formula are being established and updated and new construction methods and machinery are being designed and tested with great success. Currently, empirical formulas relating in situ moisture content to desired bearing capacity, settlement tolerances and cement requirements are being used in applications of DSM methods. However, these relationships are generally conservative and do not accurately predict the long term behaviour of the clay-cement mixture. Furthermore, often the unconfined compressive strength is the strength value used for design purposes as it can easily be obtained in the laboratory and it allows for quick confirmation of the in situ strength of the improved soil (Kohata *et al.*, 1997). Various reduction factors are applied in design so that the quality of stabilized material in the field as compared with that of ideal laboratory samples is considered.

The scope of this study is to collect high-quality laboratory results from a series of undrained and drained consolidated triaxial compression tests, unconfined compression tests and oedometer and isotropic consolidation tests. The laboratory tests were conducted on samples with moisture contents of 70 and 100 percent, cement contents of 2, 5 and 10 percent 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 to produce a group of samples for each mix type that had the same physical properties.

2. Deep Soil Mixing Methods

Deep Soil Mixing is a soil modification technique used to improve deep deposits of soft soil. In this context, the term soft soil refers to soil that is cohesive with a high moisture content, or soil that is fine, granular, saturated and in a loose state(Porbaha, 1998). Masses of stabilized soil, in the shape of columns, walls, grids or blocks are formed by DSM(Fig. 1). The method mixes a reagent, which can be either cementitious chemical or biological, in the form of a wet slurry or dry powder, with the soft in situ material. Mixing is done using hollow, rotating shafts with cutting tools, mixing paddles and/or augers attached to penetrate to varying distances beyond the tip of the shaft(Fig. 2).

The soil is penetrated to the desired depth and mixing

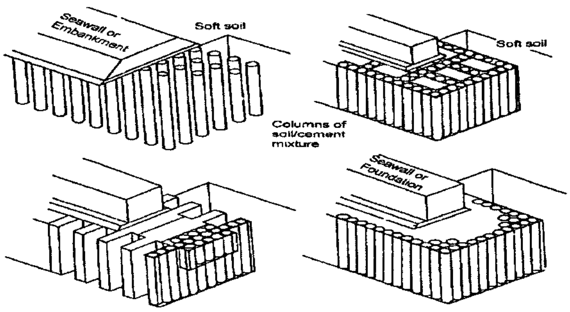
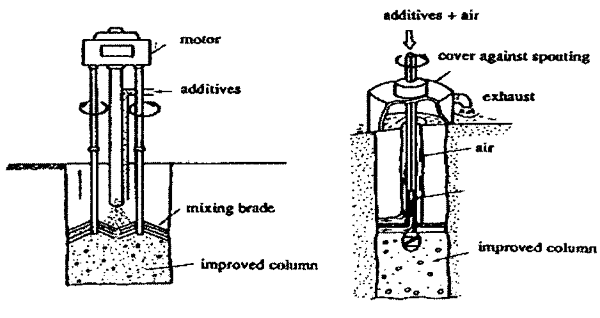


Fig. 1. Soil improvement patterns using DSM(Taki and Yang, 1991)

Fig. 2. Installation of cement piles using DSM method (Bergado *et al.*, 1996)



(a) Slurry state (b) Dry powder state

Fig. 3. Mechanical methods for DSM (Kamon, 1997)

is normally done mechanically during withdrawal of the tool(Fig. 3).

The types of improvements achieved by DSM can be classified into three general categories.

(1) *Mechanical properties:* These include increased bearing capacity, prevention of deformation, reduction of earth pressure and improvement of slope stability.

(2) *Hydrological properties:* Prevention of liquefaction and erosion caused by drainage and running water fall into this category.

(3) *Environmental properties:* Included here is environmental preservation and waste management.

3. Sample Preparation

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, making it difficult to design efficient DSM programs. Therefore, it is very important, when preparing laboratory samples, to match as best as possible the field conditions.

Very little information of sufficient detail was found in literature regarding sample preparation for triaxial or unconfined compression tests on cement stabilized clay samples. Methods were often described but significant problems that are frequently encountered were not addressed so that when these methods were attempted for the current project, the results were 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. It was found that this method of static compaction produces samples which are not uniform in density. Wall friction during compaction leads to a lower dry density at the centre of the sample. However, the distribution of density is symmetrical across the middle of the sample, provided that the compaction load at either end is applied at the same rate. All samples used for compression tests that were found in reviewed literature 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 compression test samples had a diameter of 50 mm and a length of 100 mm. Uddin (1995) conducted unconfined compression tests and triaxial tests on samples that were 35.5 mm in diameter and 71 mm in height.

Samples were extruded and then humid cured in a sealed glass desiccator, with water in its base, for the desired curing time. Stabilized samples were immersed in water for at least 24 hours prior to the start of saturation in the triaxial cell.

3.1 Properties of Soil and Cement

The soil used in the clay-cement samples for the laboratory tests was kaolin from Indonesia. The liquid limit and plastic limit of the kaolin was 77 and 40 percent, respectively, and the plasticity index was 37 percent. The specific gravity of kaolin was found to be 2.57. The soil pH was measured by mixing 10.0 g of dry soil with 40 ml of distilled water, the soil pH was 4.58.

Emerald brand Portland cement from Green Island Cement Company in Hong Kong was used as the stabilizing agent in all laboratory samples; this is equivalent to ordinary Portland cement.

3.2 Sample Mixing

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 (Fig. 4) 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,

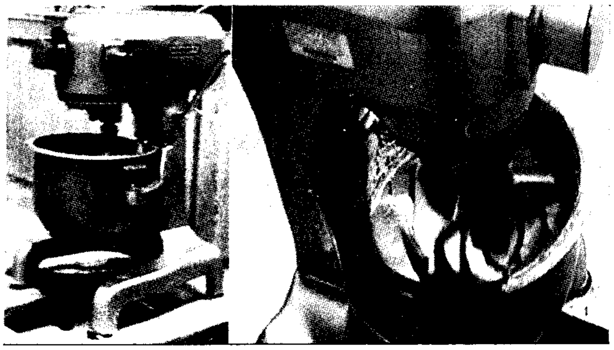


Fig. 4. Sample mixing apparatus

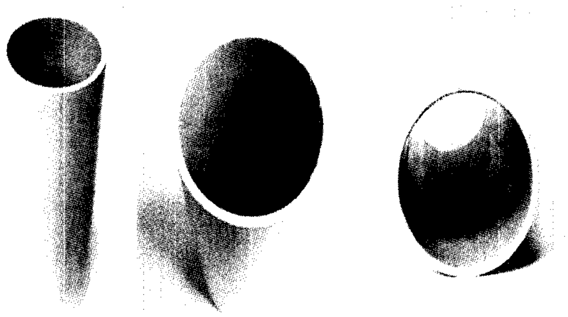


Fig. 5. PVC moulds for casting samples

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 mixing stage took less than ten minutes and was monitored closely, with some hand-mixing required, 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.

3.3 Sample Mixing

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. 5.). The tubes were lined with PVC to reduce friction between the sample and the mould during casting and extrusion.

3.3.1 Sample Casting at 100% Moisture Content

When the moisture content of the kaolin was 100%, the consistency of the material was extremely soft. When vibrated at a high frequency on a vibrating table used for casting concrete samples, the material liquefied. Therefore, to prepare high quality samples with a moisture content of 100 %, the moulds were placed vertically on a vibrating table and held firmly against a flat rigid plastic plate. With the vibrating table switched on, the material was quickly scooped into the mould using a large metal spatula. As the mould vibrated with the table, the soft material easily slid down the inside of the mould, creating a high quality sample with minimum voids and constant bulk density throughout.

Occasionally, problems were encountered when large air voids formed at the bottom of the mould, working their way upwards causing a bubbling effect on the surface of the sample. It is suspected that this air entered the sample at the base of the mould between the plastic plate and the mould, and occurred more frequently towards the end of the batch (when the mixture became slightly stiffer and the technician holding the mould firmly against the plate more tired). This problem was alleviated by working quickly and relieving fatigued technicians during the process.

Following this method, it took less than fifteen minutes to prepare a batch of six to nine samples with a moisture content of 100 %. This meant the total time between the addition of cement and casting of each sample was 25 minutes or less.

3.3.2 Sample Casting at 70% Moisture Content

Material with a moisture content of only 70% was much stiffer than the material with a moisture content of 100%. Therefore, sample preparation using material with a moisture content of 70% was more laborious and therefore, it was more difficult to prepare high quality samples. Material was usually mixed in small batches to minimize the curing of the cement prior to completion of casting of the samples in the mould. Several different methods were attempted before arriving at a final decision on how these samples should be prepared; each method is described below.

Method A: An attempt was made to mimic the method used by Wissa and Ladd (1964) to prepare cement-treated samples. The material stabilized by Wissa and Ladd had a moisture content below 20 %; they used a hydraulic press to achieve two-end static compaction. For the current study, an aluminium piston attached to a heavy duty C-clamp was used to compact the soil-cement. The piston fit exactly the inside diameter of the mould and was pushed into the mould manually by screwing the C-clamp, until refusal was reached. The mould was filled with approximately 10 to 12 equal lifts of material. It was originally thought that this method would produce a sample with a constant density throughout in little time and with little effort. However, there was no means for the air to escape and therefore, upon extruding initial trial samples, voids were found around the perimeter of the sample between lifts.

Method B: Small amounts of material were first kneaded into a ball and then firmly tossed into the bottom of the mould so that the material adhered to the underlying material prior to tamping. Tamping was done using a cylindrical wooden rammer. The use of other rammers such as a steel rod and a steel rod covered with PVC was attempted, but the material was very sticky and only the wooden rammer provided satisfactory results. Tamping was done in such as way as to result in a more-or-less consistent density throughout the sample and between samples of the same mix.

Method C: Kneading a large amount of sample at once and attempting to squeeze it into the sample was also attempted; this method was also unsatisfactory due to the resulting large trapped air voids.

Method B was the only sample preparation method which provided satisfactory results; this method was employed to prepare all samples at 70% moisture content. Some samples were also prepared without cement at 40 and 70% moisture content following this method. Preparation of these samples was the same as for the cement-stabilized samples, except curing was not necessary and therefore, laboratory tests were conducted immediately following sample casting.

3.4 Sample Curing

Immediately after casting, the samples were sealed in plastic and allowed to cure upright for 24 hours in a temperature and humidity controlled environment (20°C, 65%). Following this period, the plastic was removed, the ends of the sample were covered with permeable fabric (Fig. 6), and the samples were submerged upright in distilled water at 20°C for the remainder of the curing time.

A wax curing method was also attempted whereby the sample ends were sealed with beeswax. However, horizontal micro-cracking and shrinkage was observed in these samples upon extrusion and the moisture content was approximately 3 to 6% below that of the equivalent samples cured in water. It was concluded that wax sealing was not a suitable curing method. A humid room, similar to that used for curing concrete, may be more appropriate for this curing method.

4. Laboratory Program

To characterize the mechanical properties of the clay-cement, undrained and drained triaxial tests, unconfined compressive strength tests, isotropic consolidation tests and oedometer tests were performed. Confining pressures of

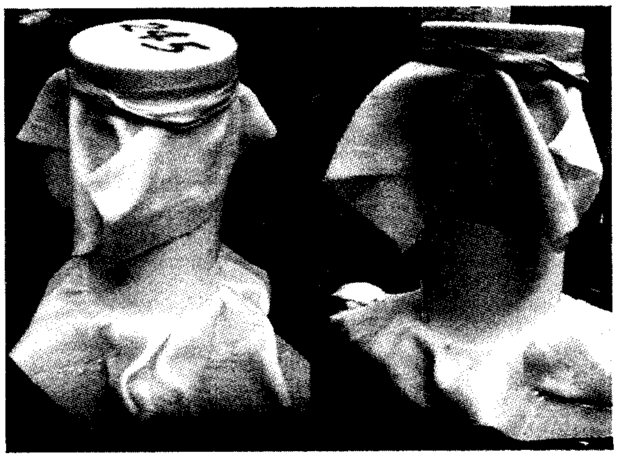


Fig. 6. Samples with fabric on ends for water curing

50, 100 and 400 kPa were chosen for the triaxial tests. Curing times ranged from 7 to 112 days so that the long-term behaviour of the material could be evaluated.

4.1 Sample Saturation

Saturation of the sample took place over a period of approximately 36 hours. A minimum final back pressure of 300 kPa was applied to all tests. A B-test was conducted initially with the back pressure valve shut and the cell pressure set at 50 kPa. The initial saturation stage was implemented once the pore pressure equalized; during this stage the cell pressure was maintained at 50 kPa and the back pressure at 40 kPa. Each subsequent B-test and saturation stage was done by increasing the cell pressure 40 kPa (for the B-test) and then increasing the cell pressure an additional 10 kPa and increasing the back pressure 50 kPa for the saturation stage. The cell and back pressure were increased in this manner until a B-value of at least 0.8 was obtained. After which, for the tests where the confining pressure was at least 100 kPa, the stress increments were increased from 50 kPa to 100 kPa, as recommended by Head(1998). For the tests with a confining stress of only 50 kPa, the stress increments were limited to 50 kPa. Pressure increments proceeded until the cell pressure reached 700 kPa; the back pressure was such that the desired confining pressure was achieved. A final B-test was conducted to confirm adequate saturation.

Throughout saturation, the B-value and degree of saturation (based on the volume change during the saturation stages and the initial moisture content and specific gravity) were calculated. Prior to the final back pressure increment and final B-value test, the B-value and degree of saturation were checked to ensure that they were above 0.95 and 99%, respectively. Occasionally, particularly for samples with 10% cement content, this criteria was not met and the B-value was sometimes below 0.9. Wissa and Ladd (1964) found that the B-value decreased as consolidation pressure and curing time increased and suggest that a B-value less than 1.0 for cement stabilized soil is due to the high rigidity of the soil skeleton and not the presence of air within the system.

4.2 Sample Consolidation

Isotropic consolidation was commenced by opening the back pressure valve and took place over a roughly 12 to 14 hour period, after which time primary consolidation was completed. Plots were generated of change in volume vs. time. The coefficient of consolidation(c_{vi}) and coefficient of volume compressibility (m_{vi}) were calculated based on the isotropic consolidation triaxial data using the following equations(Head, 1998).

It is important to note here that Head warns against calculating c_{vi} in this way when making consolidation or permeability calculations, as the value can be misleading when side drains are used. The primary function of deriving c_{vi} in this way is to estimate the rate of strain for triaxial tests. For more accurate evaluation of c_{vi} side drains should be omitted. The coefficient of consolidation was also calculated based on the isotropic consolidation and oedometer test results. Therefore, any error in the calculation of c_{vi} from the triaxial data is acceptable.

4.3 Shearing of Sample

The strain rate chosen to shear the triaxial compression tests was 0.005 mm/min(0.3%/h). Kohata et al.(1997) indicate that the normal practice in Japan is to apply a strain rate of 0.04 to 0.5%/min for triaxial tests. This allowed for adequate dissipation of pore water pressure during the undrained tests. Shearing was allowed to continue until the deviator stress became approximately constant or until lateral deformation was such that the stress could no longer be accurately calculated. The axial strain at which the deviator stress remained constant will be termed the failure condition for the purposes of this paper. It was necessary to make some corrections to the initial axial strain data to account for seating error.

Particularly at high confining pressures, volume change during consolidation was significant and the sample often was not perfectly vertical prior to shear. Therefore, the piston was not centred directly over the loading cap at the start of shear and the initial axial strain was actually due to the piston seating itself in the loading cap, and not straining of the sample. Whenever possible, the cell was manually raised prior to shearing so that the piston and loading cap were almost touching. After manual adjustments in sample elevation were made, the valves were left open to allow for equalization of all pressures prior to the start of shear.

4.4 Conventional Laboratory Tests

Conventional laboratory tests including UCS, IC and oedometer tests were also conducted. At least two UCS tests were done on all sample mixes, at all curing times considered. A strain rate of 1 mm/min was applied using a modified CBR apparatus.

Loading proceeded until either the stress became approximately constant or the load reached zero. Isotropic consolidation tests were conducted on samples 50 mm diameter and 50 mm high. Saturation was done in a manner similar to that of the triaxial tests. Isotropic consolidation pressures were applied in the following sequence: 50, 100, 200, 400, 200, 100, 50 and 10 kPa. Each stage continued for 10 to 14 hours.

Oedometer tests were conducted on samples 75 mm diameter and 20 mm high. The normal load was increased in the following increments: 50, 100, 200, 400, 800, 1600, 3200, 800, 200, 50 and 10 kPa. Each load was applied for approximately 24 hours to allow for completion of primary consolidation.

For the isotropic consolidation and oedometer tests, the coefficient of consolidation (c_v) and the coefficient of volume compressibility (m_v) were determined following both the Taylor and Casagrande methods. Void ratio vs. log pressure plots were constructed for each test.

5. Aging of Soil-cement

When cement is mixed with soil, three main reactions normally occur (Porbaha *et al.*, 1998), and are summarized as follows:

(1) *Hydration.* The cement absorbs water from the soil, producing calcium hydroxide, $Ca(OH)_2$.

(2) *Ion Exchange and Flocculation.* The calcium hydroxide dissociates in water so that the electrolytic concentration and pH of the pore water increase. This leads to an attraction between the Ca cations and the negatively charged clay particles, causing flocculation of the clay particles.

(3) *Pozzolanic Reaction.* Calcium hydroxide in the pore water reacts with the silicates and aluminates (pozzolans) in the clay, forming insoluble cementing materials.

These reactions are responsible for the increase in strength of soil-cement with time. As the last statement indicates, it is the pozzolanic reaction that causes the properties of soil-cement to change with time. However, some authors have reported that under some conditions, soil-cement does not necessarily gain strength with age. Babasaki *et al.* (1996) suggest that when stabilized material is exposed to water, the strength of cement stabilized soils may actually deteriorate with time. Azman *et al.* (1995) found the strength of black soil from Malaysia stabilized with only 2% cement and cured in water to decrease between 28 and 60 days of curing; it was suggested that in this instance, cement had a separating effect when the content was low, instead of a bonding effect.

Uddin (1995) observed that when Bangkok clay is stabilized with 5% cement or less and cured in a humid environment, the mechanical properties generally do not improve with time. Kezdi (1979) reports that the unconfined compressive strength of samples cured in water to be 20 to 30% less than that of samples cured in a humid environment. The study of the effects of curing environment on the strength of kaolin-cement results showed that at low cement contents (i.e. 2%), the unconfined compressive strength of water cured samples may be as much as 70% less than that of humid cured samples. The above observations only confirm the large influence that curing environment, soil type and cement content have on effective, long-term cement stabilization.

It is believed that the effect of age is the most complex and least understood influence on the properties of soil-cement; it is clear that more research is required in this area.

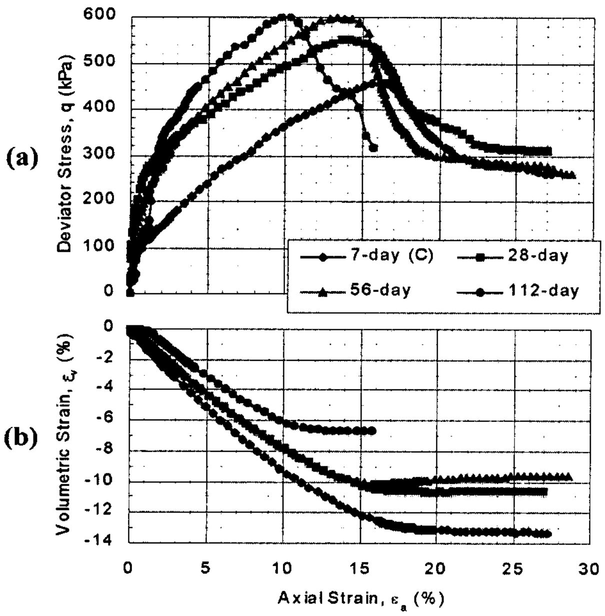
6. Laboratory Test Results

The discussion below considers both the isotropic consolidated triaxial drained and undrained (CID and CIU) compression test results as well as the unconfined compression (UC) test results. To examine the effects of excess pore water pressure, the stress ratio, η (q/p′) is considered for the case of the triaxial tests, which is particularly useful with respect to the undrained triaxial test results, where the excess pore water pressure is significant.

6.1 Deviator Stress (q) and Stress Ratio (q/p′)

The trends in deviator stress with curing time are best interpreted at high cement contents and low confining pressures. Trends were far clearer for the drained tests than for the undrained tests; this is partially because confining pressure dominates the undrained properties to such a large extent. Furthermore, when studying the undrained triaxial test results, the excess pore water pressure is significant and therefore, it is best to consider the stress ratio instead of merely the deviator stress. In general, as expected, the peak deviator stress increases with curing time, at a decreasing rate (Fig. 7a and 9a).

However, a reduction in peak strength after 28 or 56 days of curing is occasionally seen when the cement content is low(Fig. 10). In most cases, the failure strength does not appear to change significantly with curing time and was found to be primarily a function of the confining pressure. Occasionally, lateral deformation of the sample at large strains was significant, and in some cases, tensile fractures occurred in the sample.



(a) stress & axial strain (b) volumetric strain vs. axial strain

Fig. 7. w=100%, Ac=5% and σ'_3 =100 kPa(CID tests).

6.2 Axial Strain at Peak Conditions

When the conditions are undrained or when the conditions are drained and the confining pressure is low (i.e. 50 or 100 kPa), the axial strain corresponding to peak conditions decreases with curing time (Figs. 7a, 9a and 10). The trend is the opposite, however, when conditions are drained at 400 kPa (Fig. 11). This behaviour is due to the increase in rigidity of the stabilized material with age, confining pressure, and undrained conditions.

6.3 Volumetric Strain

In general, as curing time increases, the volumetric stain during drained shear decreases, at a decreasing rate, so that the difference in volumetric strain between 7 and 28 days of curing is the greater than that between 28 and 56 days of curing (Fig. 7b).

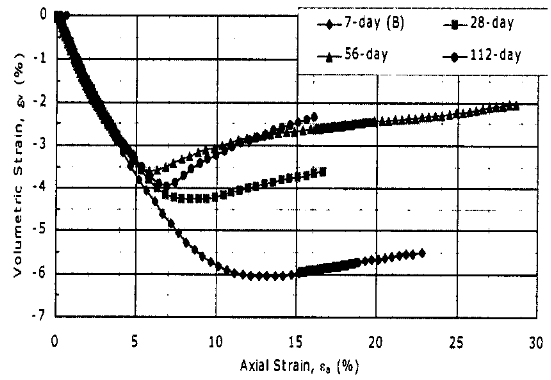


Fig. 8. Volumetric & axial strain for Ac=10%, w=100% vs. σ_3'=50 kPa(CID tests)

This trend is most clear at 50 kPa confining pressure and at 100 % moisture content. When the moisture content is only 70 %, occasionally the trend is somewhat reversed and the volume change is greatest after 56 days of curing. This phenomenon may be due to weakening in the bond strength after a given curing time, and generally agrees with the observed trends in peak strength. Most samples dilated slightly following peak conditions; samples cured for greater curing times tended to dilate more. Occasionally, however, when the curing time is only 7 days, some samples continued to contract following peak but at a slower rate than prior to peak, while samples of the same mixture but greater curing times dilated once peak had occurred(Fig. 8).

6.4 Excess Pore Water Pressure

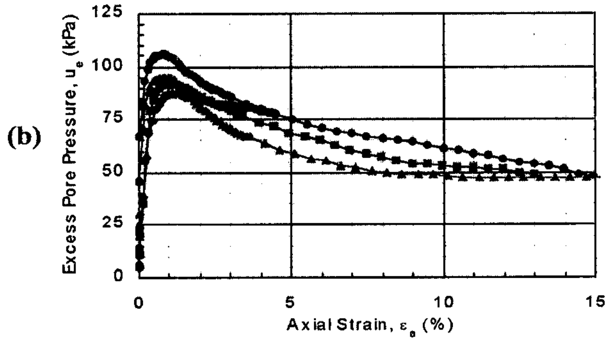
For all of the undrained triaxial tests on kaolin-cement, the excess pore water pressure remained positive throughout the test. For these cases, the excess pore water pressure always increased up to an axial strain roughly coincident with peak conditions (Fig. 9); this is an indication of sample contraction.

Following peak conditions, the excess pore water pressure always decreased, while remaining positive, indicating sample dilation. However, when pure kaolin at 40 % moisture content was tested, the excess pore water pressure was slightly negative at the end of the test when the confining pressure was only 100 kPa. Furthermore, for the cases with no cement, the sample began to dilate before peak stress conditions were achieved.

6.5 Cohesion and Angle of Friction

The effective cohesion and friction angle were determined from the drained triaxial test results and give a good indication of the overall trend in the material strength with age, regardless of confining pressure. Both are expected to increase with curing time (Bergado *et al.*, 1996).

For all cases, the peak cohesion increases between 7 and 28 days of curing (Fig. 12); when the cement to moisture content ratio is high (i.e. w=100% and Ac= 10%; w=70% and Ac=5%), the increase continues up to at least 56 days. However, when the cement to moisture content ratio is low (i.e. w=100% and Ac=5 %; w=70% and Ac=2%), there is a reduction in cohesion following



(a) stress ratio vs. axial strain;
(b) excess pore water pressure vs. axial strain

Fig. 9. w=100%, Ac=10% and σ'_3=100 kPa (CIU test)

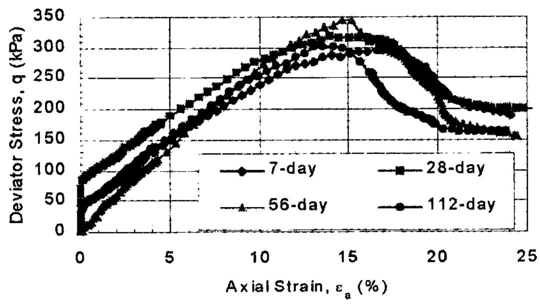


Fig. 10. Stress-strain curve for w=70%, Ac=2% and σ_3=100 kPa (CID test)

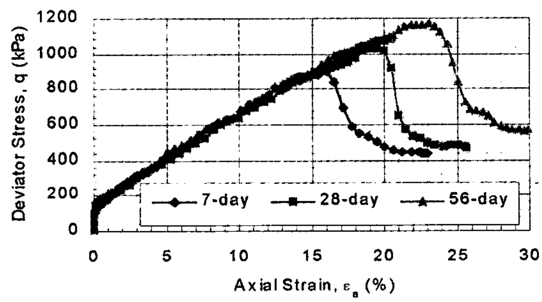


Fig. 11. Stress-strain curve for w=100%, Ac=5% and σ_3=400 kPa (CID test)

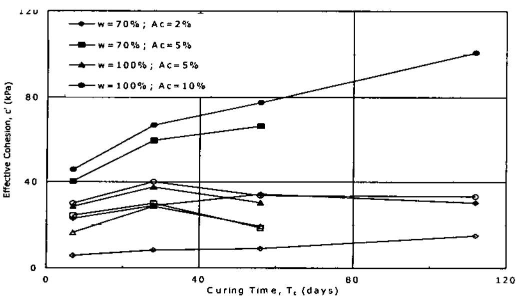


Fig. 12. Effective cohesion vs. curing time

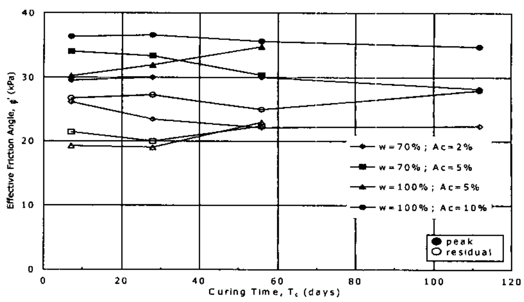


Fig. 13. Effective friction angle vs. curing time

28 or 56 days of curing and the overall change in cohesion with curing time is not great (<12 kPa). These results may indicate that, when the cement to moisture content ratio is low, either there is a reduction in strength with curing time, or there may be no significant change in cohesion with curing time, particularly beyond 28 days. The change in peak friction angle with curing time is more difficult to describe (Fig. 13). The friction angle did not change by more than 5° between 7 and 112 days of curing for any mixture and no statement can describe the observed trends. When the cement content was 2 and 10%, the peak friction angle decreased very slightly with curing time. When the cement content was 5%, however, it either increased (w=100%) or decreased (w=70%) by nearly 5° between 7 and 56 days. Based on these observations, it is suggested that the change in friction angle, for the cases considered, does not change significantly with curing time. Wissa and Ladd (1964) made a similar conclusion based on strength tests on fine-grained glacial till by reporting that friction angle is independent of curing time when the cement content is less than 5%.

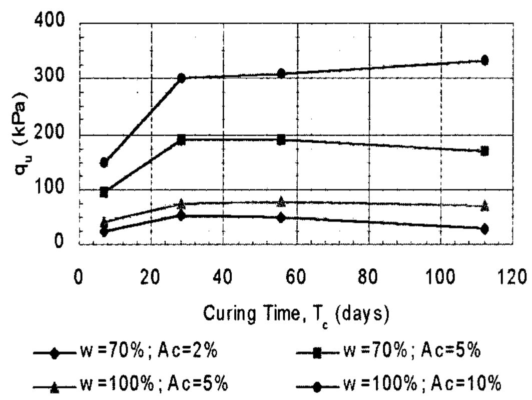


Fig. 14. UCS vs. curing time

6.6 Unconfined Compressive Strength

Preliminary results of unconfined compressive strength (UCS) tests (Fig. 14) generally agree with the trends observed in peak cohesion with curing time. The strength for all mixtures increases between 7 and 28 days of curing. When the cement content is 10%, the increase continues up to at least 112 days such that the increase is roughly linear between 28 and 112 days. However, when the cement content is only 5%, the additional increase in strength between 28 and 56 days is very small and is followed by a reduction in strength after 56 days. With 2% cement, there is no further increase in strength after 28 days of curing and the unconfined compressive strength after 112 days of curing is reduced to a value very close to that after 7 days of curing. Similar to the peak cohesion results, when the cement to moisture content ratio is low, the change in strength with time is very small and may not be meaningful.

7. Failure Behaviour of Samples

7.1 Drained Triaxial Tests

In general, the failure behaviour was brittle and strain-softening, particularly at low confining pressures. An increase in the cement to moisture content ratio and curing time caused the failure to be more brittle. Porbaha *et al.* (1998) notes that as confining pressure increases from zero, failure becomes less brittle. Based on the volumetric strain data, most samples changed from contracting to dilating at peak stress conditions, which is interpreted as failure. However, in some cases, usually when curing time was only 7 days, samples continued to contract after failure, which is often an indication of crushing.

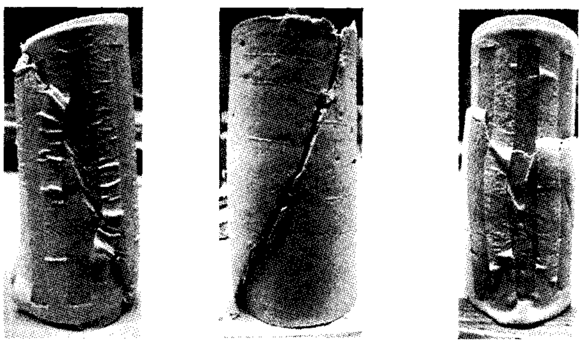
Fig. 15 shows three CID samples after failure; Fig. 15a and 15b are more typical. Normally, some sample barrelling occurred and a clear failure plane was formed during shear; the failure plane was normally about 30° from the vertical axis of the sample. Porbaha *et al.* (1998) comment that a failure plane form even at low confining pressures and is due to the plastic shearing at failure that occurs under triaxial conditions. Typically, this failure plane ran from the top to the bottom of the sample, dividing the sample into two, nearly equal, portions. As the cement to moisture content ratio and curing time increased, less sample disturbance near the failure plane occurred (Fig. 15(b)). In many cases, small shallow tension cracks occurred around the outside of the sample as deformation following failure proceeded. In some cases (i.e. Fig. 15(c)), more than one failure plane was formed simultaneously so that the top of the sample remained intact, with the exception of barrelling, and the two shear planes formed an approximately symmetric V-shape at the bottom of the sample.

7.2 Undrained Triaxial Tests

In general, the failure behaviour was strain softening. Although the material under undrained conditions was more stiff, as indicated by the relatively low axial strain at peak, failure was not brittle, as it was for the drained triaxial tests. Fig. 15 shows a variety of CIU samples after failure with the sample shown in Figure 16a being the most typical.

A clear failure plane, roughly 30° from the vertical axis of the sample, developed during shear; this was also the case for most CID tests. Failure often occurred more through the lower portion of the sample so that following shear, the top piece was larger than the bottom piece.

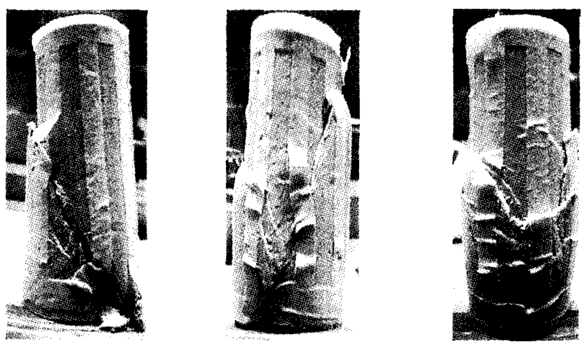
As the cement to moisture content ratio and curing time increased, less sample disturbance near the failure plane occurred. In one test, two clear failure planes developed simultaneously (Fig. 16(b)). Similar to the CID tests, more sample disturbance near the shear plane occurred at lower cement to moisture content ratios. As for the CID tests, barrelling occurred, but to a slightly greater degree. Where a shear plane did not form, failure is believed to be by crushing. This mode of failure caused the top portion of the sample to punch through the bottom portion of the sample, so that the bottom portion of the sample was greatly disturbed and its diameter was far greater than the diameter of the top portion. This type of failure rarely occurred, and was exhibited in the undrained triaxial tests only (Fig. 15(c)).



(a) (b) (c)

(a) Ac=2%, w=70%, p_0'=50 kPa & Tc=7 days;
(b) Ac=10%, w=100%, p_0'=400 kPa & Tc=28 days;
(c) Ac=2%, w=70%, p_0'=400 kPa & Tc=7 days

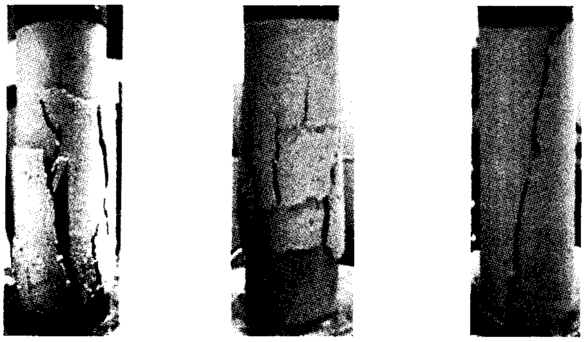
Fig. 15. CID triaxial samples after failure



(a) (b) (c)

(a) Ac=10%, w=100%, p_0'=400 kPa & Tc=56 days;
(b) Ac=2%, w=70%, p_0'=400 kPa & Tc=28 days;
(c) Ac=5%, w=100%, p_0'=100 kPa & Tc=28 days

Fig. 16. CIU triaxial samples after failure



(a) (b) (c)

(a) Ac=2%; w=70% & Tc=28 days;
(b) Ac=5%; w=70% & Tc=56 days;
(c) Ac=10%; w=100% & Tc=7 days

Fig 17. UC samples after failure

7.3 Unconfined Compressive Strength

Under unconfined conditions, failure is brittle and by crushing (Porbaha *et al.*, 1998), particularly at greater curing times. The top of the sample normally remains intact and nearly vertical tension cracks form in the bottom of the sample as crushing proceeds (Fig. 17a). Occasionally, the opposite is true so that the bottom of the sample remains intact (Fig. 17b). Spalling is also seen from time to time (Fig. 17b). Oblique fractures sometimes occurred, separating the sample into two nearly equal pieces (Fig. 17c), as was the case for most of the triaxial tests.

7.4 Discussions

Some general observations can be made with respect to curing time when considering all strength test results: When the cement content is low (i.e. 5% or less), the stabilized material may begin to weaken after a certain curing time beyond 28 days. This reduction in strength begins earlier for lower cement contents. The most significant improvement in strength resulted when the cement to moisture content ratio was high; the majority of this improvement occurred within the first 28 days of curing. When the cement to moisture content ratio is low, the change in peak cohesion and unconfined compressive strength with curing time are small; the observed trends with curing time for these cases may not be meaningful, particularly after 28 days. Only at a cement content of 10% was there consistent improvement up to 112 days of curing in all strength tests. There is no significant change in peak friction angle with curing time. The rigidity of cement stabilized clay increases with curing time, confining pressure and undrained conditions. An increase in curing time leads to a decrease in volumetric strain and an increase in maximum excess pore water pressure during drained and undrained shear, respectively.

8. Conclusions

Based on the results of this study on the mechanical characteristics of kaolin-cement, some general overall conclusions can be made as follows.

When establishing a laboratory program for the design of soil cement, great attention should be paid towards sample preparation and curing environment as the procedures followed may have a great influence on the test results. Sample preparation must be done such that there are no voids or weak seams. The material consistency may govern which sample preparation method is the most appropriate. Curing environment has a significant influence over the properties of soil-cement. Particularly at law cement contents, water cured samples have much lower strength than humid cured samples, but they will be less stiff and more ductile failure behaviour. Futhermore, humid cured samples often lost significant amounts of moisture during curing so that after at least 28days, some samples were damaged with cracks. Therefore, when establishing a laboratory program for effective DSM design, the curing environment must be considered carefully and chosen to best simulate the field conditions. Using too little cement to stabilize soft clay sometimes causes the strength to deteriorate with curing time. This is partly because of the relatively large amount of cement required for effective stabilization of clay particles due to their high specific surface.

At relatively low confining pressures, the failure envelope of kaolin-cement appears linear. However, trends in both peak cohesion and friction angle with curing time suggest that the two properties influence one another: when one increases, the other decreases, and vice versa. This observation leads to the suggestion that the failure envelope is actually curved, like that of soft rock. However, over a narrow range of confining pressures, the failure envelope appears linear.

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.

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