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Basic parameters governing the behaviour of cement-treated clays

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Abstract

Although extensive research has been conducted on the mechanical behaviour of Portland cement-treated soft clays, there has been less emphasis on the correlation of the observed behaviour with clay mineralogy. In this study, experimental results from the authors have been combined with the data found in the literature to investigate the effect of parameters such as curing time, cement content, moisture content, liquidity index, and mineralogy on the mechanical properties of cement-treated clays. The findings show that undrained shear strength and sensitivity of cemented clays still continue to increase after relatively long curing times; expressions are proposed to predict the strength and sensitivity with time. This parametric study also indicates the relative importance of the activity of the soil, as well as the water–cement ratio, to the mechanical properties of cementitious admixtures. Two new empirical parameters are introduced herein. Based on the results of unconfined compression, undrained triaxial, and oedometer tests on cement-enhanced clays, expressions that use these parameters to predict undrained shear strength, yield stress, and the slope of the compression line are proposed. The observed variations in the mechanical behaviour with respect to mineralogy and the important effect of curing time are explained in terms of the pozzolanic reactions. The possible limitations of applying Abrams' law to cement–admixed clays are also discussed.

Keywords: Artificially cemented clay; Undrained shear strength; Sensitivity; Curing time; Activity; Cement–water ratio; Pozzolanic reactions (IGC: D10)

1. Introduction

Soft clays cover large areas of our built environment, including many important coastal and low-land regions, where major urban and industrial areas are located, and are often encountered in land reclamation projects. These clays can have high in situ water contents and are considered to be potentially problematical soils, because of their low strength and susceptibility to large settlements. Although accelerated consolidation of these clays using various drainage techniques (e.g. wick drains) is common, it is not preferable in some cases, due to time constraints and expense. An alternative approach for increasing the stiffness and strength of soft ground is to create cementitious bonds within the soil material by adding cementing agents (e.g. Nagaraj and Miura, 2001).

Depending on the specific needs of different projects, various cement stabilization techniques, such as shallow soil mixing, deep mixing, and jet grouting, have been developed and are now routinely applied (e.g. Bergado et al., 1996; Nagaraj and Miura, 2001). Many of these treatments are used to provide soft soil underlying roads and railways with higher stiffness and bearing capacity. Deep mixed cement columns are also utilized as an

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alternative to piles to stabilize slopes, trenches and deep excavations in soft ground (Bergado et al., 1996). These chemical stabilization methods differ in their approaches for creating soil bonding, but they all utilize at least one form of bonding agent (usually Portland cement, lime, fly ash, or slag). Due to common availability and effectiveness, Portland cement is the most widely used cementing agent for ground improvement projects.

Although the changes in mechanical behaviour of many different clays stabilized by cement has been studied before by various researchers, limited attempts have been made to correlate the data from different soils and to introduce unified approaches describing the effect of clay mineralogy. Such a framework would assist engineers in preliminary design studies and minimize the number of trials needed to determine the required cement content and curing period. The significant volumes of data that have been reported recently by several researchers on the mechanical properties of cement-treated clays at high water contents have made these aims more feasible. The primary objective of this study was to combine data available in the literature, with further results from the authors, to identify pertinent parameters for predicting the strength, stiffness, sensitivity, and other important geotechnical characteristics of soft clays cemented with Portland cement.

2. Literature review

Many researchers (e.g. Uddin et al., 1997; Yin and Lai, 1998; Tremblay et al., 2001; Tan et al., 2002; Rotta et al., 2003; Horpibulsuk et al., 2003, 2004a; Lorenzo and Bergado, 2004; Chew et al., 2004; Lee et al., 2005; Xiao and Lee, 2008; Kamruzzaman et al., 2009; Jongpradist et al., 2011) have investigated the effect of artificial cementation by Portland cement on the mechanical behaviour of clay and have reported increases in strength, stiffness, and brittleness of the soil. Increases in the peak strength and stiffness of the material occur due to the formation of a cementitious structure within the soil skeleton. In soils with high water contents, this cemented structure may be responsible for a significant proportion of the mechanical behaviour of the soil.

Adding Portland cement to a soil body results in a primary hydration reaction in the cement, followed by a secondary pozzolanic reaction. The former happens in any mixture of cement and water, but the latter only occurs in the vicinity of soil particles between calcium hydroxide supplied by the cement, and silica and alumina from the soil (Herzog and Mitchell, 1963; Croft, 1967a; Bergado et al., 1996; Bhattacharja et al., 2003). The primary hydration reactions are (Bergado et al., 1996; Bhattacharja et al., 2003)

$$2C_3S + 6H \rightarrow C_3S_2H_3 + 3Ca(OH)_2 \tag{1}$$

$$2C_2S + 4H \rightarrow C_3S_2H_3 + Ca(OH)_2 \tag{2}$$

where the following symbols represent short forms of the compounds: $H{=}H_2O$, $C{=}CaO$, and $S{=}SiO_2$. The subsequent secondary reactions occur as soon as calcium hydroxide is produced in the mixture (Bergado et al., 1996; Bhattacharja et al., 2003):

$$Ca(OH)_2 + SiO_2 \rightarrow CSH \tag{3}$$

$$Ca(OH)_2 + Al_2O_3 \rightarrow CAH \tag{4}$$

where $A{=}Al_2O_3$. Due to the purity and fineness of the calcium hydroxide produced during the hydration reactions, it reacts more strongly than ordinary lime with the soil minerals (Herzog and Mitchell, 1963; Bhattacharja et al., 2003). Both hydration and pozzolanic reactions lead to the creation of gelatinous and amorphous materials, which later crystallize to form inter-aggregate and inter-particle bonds (Croft, 1967a). The production of cementitious bonds between soil mineral substances creates a matrix that encloses the unbonded particles and aggregates and results in an apparent cohesion in the soil material, making its engineering behaviour more complex (Kasama et al., 2000).

Bonding, composition, and fabric are often identified as important elements that contribute to the overall structure of a clay (Mitchell and Soga, 2005). For artificially cemented soils, these factors depend on cement and water chemistry, soil type, grain size distribution, plasticity of clay, and the process of mixing and curing. Although soil properties can greatly influence the cementation process, few researchers have investigated the effect of soil mineralogy on artificial cementation. Early works on the hardening of soil–cement mixtures considered the soil material to be relatively inert (Croft, 1967b). Croft (1967a) investigated the effect of mineralogical composition of clay on cement stabilization by analysing the behaviour of seven types of clay mixed with Portland cement. He suggested that although most clay minerals eventually consume the lime produced in the hydration process, more expansive clay minerals, such as montmorillonite, are much quicker to react with lime than less active minerals, such as kaolinite and illite. Noble and Plaster (1970) investigated the chemical reactions in Portland cement–clay mixtures and reported that the rate of cement hydration in most clays mixed with cement was slower than normal rate of cement hydration in concrete. Their results also showed that the reaction of calcium hydroxide with the soil minerals (secondary reactions) was related to the magnitude of the clay size fraction and that soil mineralogy and size distribution controlled the strength development. Woo (1971) suggested that an increase in clay content or plasticity index would make artificial cementation by Portland cement less effective. Tremblay et al. (2001) investigated the effect of organic content on artificial cementation of clays from eastern Canada and concluded that the presence of organic matter can negatively affect the efficiency of artificial cementation, especially if the organic content is higher than 3–4%.

New parameters to characterize the behaviour of artificially cemented clays have been introduced during the past decade. Miura et al. (2001) suggested that Abrams' law (Abrams, 1918), which is commonly used in concrete technology, can also be applied to cemented soils. They proposed *w/c* (clay–water/cement ratio) as a parameter for studying the engineering behaviour of cemented clays with high liquidity indices. For any given clay, Miura et al. (2001) showed that lower values of *w/c* result in higher strength and that two mixtures with different cement and water contents will have a similar strength level if their *w/c* ratios are the same. However, they also noted that at low *w/c* ratios (for example

$w/c=7.5$), soil fabric has a bigger influence on the behaviour of cemented clays and using w/c ratio alone is not sufficient to predict the soil behaviour. Horpibulsuk et al. (2003) provided empirical equations correlating the *w/c* ratio with unconfined compressive strength for Ariake and Bangkok clays. They proposed different coefficients in their equation for each clay type and curing period. They also provided a logarithmic equation for predicting strength development with curing time based on the results of experiments performed on different types of clay, up to 180 days after curing. Horpibulsuk et al. (2005), Lee et al. (2005), and Lorenzo and Bergado (2004) also confirmed that shear strength can be expressed as a function of w/c ratio for Ariake, Singapore, and Bangkok clays.

3. Description of laboratory studies and data analysis

3.1. Experimental work

To investigate the effect of mineralogy, laboratory studies were conducted by the authors on cement-treated specimens of three different clays, i.e. EPK kaolin, Nanticoke clay, and Ottawa clay. Portland cement (type I according to ASTM C150-11) was chosen for this study, since it is one of the most commonly added cementing agents (Bergado et al., 1996; Nagaraj and Miura, 2001; Bhattacharja et al., 2003). Most ground improvement projects involving artificial cementation occur in zones with a high water table; hence, the specimens used in this study were covered with water, so that their moisture content did not drop significantly due to cement hydration. The effect of curing time on the strength and sensitivity of the cemented material was investigated by performing laboratory shear vane tests on the cement-treated samples at various time intervals after curing. In addition, the results of oedometer and triaxial tests were used to obtain relationships between soil/cement parameters and the mechanical properties of the artificially cemented clays. The results of these laboratory studies were added to those found in the literature and described in Section 3.3 below.

3.2. Specimen preparation and testing procedures

Each of the clay soils was prepared from a dry, powdered form to create slurries. EPK Kaolin is already available in a powdered form. To make the powdered Nanticoke and Ottawa clays, the soil was cut into small pieces, dried at room temperature, and then finely pulverized into a powder (100% passing Sieve no. 40), using a rubber hammer to avoid crushing the soil particles. Clay powders were mixed with distilled, deionised water to form the slurry with a water content close to the desired value. The slurry was mixed until a uniform paste was achieved. Next, the required amount of cement mixed with water was added to the mixture to increase the water content to the desired level. The slurry was mixed again for a maximum of 15 min, so that the mixing process did not disturb the produced bonds. The mixture was then poured into plastic cups 70 mm in diameter and 120 mm in height. Trapped air bubbles were removed from the samples by gently tapping on the walls of each cup. Some water was added on top of the slurry, 1 h after the material was placed in the cups, to provide it with moisture throughout the curing period. Due to early formation of some structure, the added free water did not significantly affect the porosity of the material. The cups were then covered by plastic wraps and placed in a temperature controlled room to be cured at a constant temperature of 25 °C. Cemented samples were prepared with different moisture contents, w (%), and cement contents, c (%), where cement content is defined as the ratio of mass of cement to the mass of dry soil in terms of percentage. Moisture content (w) was determined based on the total amount of water in the slurry (at the time of mixing), and the Atterberg limits of untreated soil was used in calculations. A few oedometer tests were also performed on undisturbed and reconstituted specimens of Ottawa clay. The reconstituted specimens were prepared at water contents of 1.2 and 1.5 times the liquid limit to obtain the intrinsic compression line (ICL) for the material (Burland, 1990).

Since the undrained shear strength of a large number of cemented specimens were to be measured, the laboratory shear vane was chosen as the primary testing method. The vane had a diameter and height of 19 and 28 mm, respectively. The vane tests were carried out at a rate of one complete revolution per minute (Serota and Jangle, 1972; Kogure et al., 1988). Although it is generally recognized that due to the different modes of failure, undrained shear strengths measured with the shear vane apparatus vary from those measured with other test methods (Kogure et al., 1988), the manufacturer of the shear vane calibrated the device using UU triaxial tests on clay specimens (Serota and Jangle, 1972). Accordingly, the calibration factor obtained based on this approach was used herein, allowing for the comparison of the vane results to those obtained from

Table 1
Geotechnical properties of the four types of clay tested by the authors.

Soil Characteristic	Ottawa clay	Nanticoke clay	EPK kaolin	Speswhite kaolin
Liquid limit, LL (%)	52	48	61	64
Plastic limit, PL (%)	24	23	36	34
Plasticity index, PI (%)	28	25	25	30
Specific gravity, G_s	2.82	2.73	2.61	2.62
Clay fraction (< 2 μm, %)	43	48	53	61
Activity, A	0.65	0.52	0.47	0.49

Table 2
Properties of the cemented soil samples from the literature used in the curing time analysis shown in Fig. 3.

Soil	Plastic limit, PL (%)	Liquid limit, LL (%)	Plasticity index, PI (%)	Clay content (%)	Activity, *A*	Water content, *w* (%)	Liquidity index, LI	Cement content, *c* (%)	Reference
Home Rule kaolin	33	64	31	88	0.35	45	0.4	1, 5, 10, 20	Croft (1967a)
Rotoclay kaolin	30	58	28	40	0.70	115	3.0	10, 20	Flores et al. (2010)
Singapore marine clay	35	87	52	68	0.76	120	1.6	5, 10, 20, 30, 40, 50, 60	Kamruzzaman et al. (2009)
	25	90	65	[a]	[a]	100	1.2	10, 15, 20	Xiao and Lee (2008)
						133	1.7	50	
						150	1.9	77, 100	
	32	72	40			90, 120, 150	1.5, 2.2, 3	10, 20, 30	Tan et al. (2002)
Bangkok clay	43	103	60	69	0.87	86, 106, 136, 166	0.7, 1, 1.5, 2	10	Lorenzo and Bergado (2004)
						80	1.3	5, 7.5, 10, 12.5, 15, 20, 25, 30, 35, 40	Uddin et al. (1997); Uddin (1994); Bergado et al. (1999)
Ariake clay	60	125	65	55	1.18	106	0.7	10, 20	Horpibulsuk et al. (2003)
						130	2.0	10, 15, 20	
						160	2.5	15, 20	
Illite	[a]	[a]	[a]	[a]	[a]	118	[a]	10, 20	Nagaraj et al. (1996)
Brown Indian clay	23	60	37	46	0.80	60, 90, 120	1, 1.8, 2.6	3–24[b]	Narendra et al. (2006)
Brown Indian clay	[a]	[a]	[a]	[a]	[a]	62	[a]	10, 20	Nagaraj et al. (1996)
Black cotton Indian clay	35	97	62	61	1.02	97, 145, 194	1, 1.8, 2.6	5–19.5[b]	Narendra et al. (2006)
Black cotton Indian clay	[a]	[a]	[a]	[a]	[a]	72	[a]	10, 20	Nagaraj et al. (1996)
Black cotton Indian clay	[a]	[a]	[a]	[a]	[a]	86	[a]	20	Nagaraj et al. (1996)
Red earth Indian clay	15	38	23	32	0.72	38, 57, 76	1, 1.8, 2.7	4–15[b]	Narendra et al. (2006)

[a]No data available.
[b]Data ranging between the two values.

unconfined compression or quick triaxial tests shown later in the paper. A similar apparatus was used by Kirwan (2003) and Kainourgiaki (2004) to measure the shear strengths of cemented Speswhite kaolin specimens. To support the shear vane test results, a number of oedometer and CIU triaxial tests were also performed to obtain further parameters. The oedometer and CIU triaxial tests were conducted according to ASTM D2435-04 and ASTM D4767-04, respectively. For testing, the samples were taken out of the plastic cups and were cut with a thin wire trimmer to the required size.

3.3. Additional database compilation

In addition to the experimental studies, a detailed literature survey and analysis of the data was conducted by the authors. To ensure comprehensive data collection, the results of laboratory shear vane, unconfined compression, undrained triaxial, and oedometer tests performed by different researchers on different types of clay treated with Portland cement were included in the analysis. The database covers 24 studies published from 1967 to 2011. For the parametric studies, information on the following cement-treated clays were gathered from the literature: *Speswhite Kaolin* (Ritchie, 2004; Kainourgiaki, 2004; Kirwan, 2003); *Ariake clay* (Miura et al., 2001; Horpibulsuk et al., 2003, 2004a, 2004b, 2005); *Bangkok clay* (Uddin, 1994; Uddin et al., 1997; Bergado et al., 1999; Lorenzo and Bergado, 2004; Horpibulsuk et al., 2004b, 2011); *Singapore marine clay* (Tan et al., 2002; Chew et al., 2004; Lee et al., 2005; Xiao and Lee, 2008; Kamruzzaman et al., 2009); *Hong Kong clay* (Yin and Lai, 1998); *Brown, Black cotton, and Red earth Indian clays* (Nagaraj et al., 1996; Narendra et al., 2006); *Illite* (Nagaraj et al., 1996); *Home Rule Kaolin* (Croft, 1967a); *Louiseville clay* (Tremblay et al., 2001); *Bothkennar clay* (Kirwan, 2003; Kainourgiaki, 2004); *Rotoclay Kaolin* (Flores et al., 2010).

3.4. Material properties

Analysis was performed on laboratory test results conducted by the authors for artificially cemented specimens of two commercially produced kaolin clays: EPK and Speswhite, and two naturally occurring Canadian clays: Nanticoke and Ottawa clays. EPK is a pulverized kaolin clay from Georgia, U.S., and Speswhite china clay is produced from deposits in the southwest of England. Air dried clay powders were also obtained from block samples of a stiff fissured clay with a glacio-lacustrine origin taken from 3 m depth in a test pit in Nanticoke, Ontario, and from samples of a sensitive Leda/Champlain clay (St ~ 20), also taken from 3 m depth in a borehole in Ottawa, Ontario. X-ray diffraction analysis showed that the primary clay minerals of both soils are illite and chlorite, with traces of vermiculite found in the Ottawa clay. The engineering properties of these four clays are summarized in Table 1.

In addition to the aforementioned clay tests, the laboratory results from other researchers on clayey soils listed in Section 3.3 were used in the overall analysis. Some basic geotechnical properties of these soils are given in Tables 2 and 3. The liquid limit (LL), plastic limit (PL), plasticity index (PI), and activity number (A) of these clays range from 38 to 125, 15 to 60, 20 to 65, and 0.47 to 1.18, respectively. It should be noted that except for Bothkennar clay and Rotoclay Kaolin, which are from the U.K, and Home Rule Kaolin, which is from Australia, the remaining clays are from East and Southeast Asia or Canada.

Table 3
Properties of the cemented soil samples from the literature used in the parametric study.

Soil	Activity, A	Water content, w (%)	Cement content, c (%)	Type of test	Reference
Rotoclay kaolin	0.70	115	10, 20	UC	Flores et al. (2010)
Singapore marine clay	0.76	90–150[a]	5–77[a]	UC, CIU, Oed.	Kamruzzaman et al. (2009), Xiao and Lee (2008), Tan et al. (2002), Chew et al. (2004), Lee et al. (2005)
Bangkok clay	0.87	80–209[a]	3–35[a]	UC, CIU, Oed.	Lorenzo and Bergado (2004), Uddin et al. (1997), Uddin (1994), Horpibulsuk et al. (2004b), Horpibulsk et al. (2011)
Ariake clay	1.18	106–250[a]	6–33[a]	UC, CIU, Oed.	Miura et al. (2001), Horpibulsuk et al. (2003), Horpibulsuk et al. (2004a), Horpibulsuk et al. (2004b), Horpibulsuk et al. (2005)
Hong Kong clay	0.91	60–100[a]	5–20[a]	UC, CIU	Yin and Lai (1998)
Brown Indian clay	0.80	60–120[a]	3–24[a]	UC	Narendra et al. (2006)
Black cotton Indian clay	1.02	97–194[a]	5 -19.5[a]	UC	Narendra et al. (2006)
Red earth Indian clay	0.72	38–76[a]	4–15[a]	UC	Narendra et al. (2006)
Louiseville clay	0.56	122	5.3	Oed.	Tremblay et al. (2001)
Bothkennar clay	0.50	60	4.2	CIU, Oed.	Kirwan (2003), Kainourgiaki (2004)

UC: unconfined compression test.
CIU: conventional undrained triaxial test.
Oed: oedometer test.
[a]Data ranging between the two values.

4. Analysis of the data

4.1. Hardening of cemented clays with time

Fig. 1 shows the results of typical laboratory shear vane tests on artificially cemented samples of Nanticoke clay prepared at a moisture content of 98% (liquidity index, LI=3). The tests were conducted on cemented samples cured for up to 40 months. Four cement percentages were used, namely 1%, 2%, 4.2%, and 8.7%. Samples with 1% cement did not produce any measurable strength, even after long curing periods, so the results for these samples are not plotted in the figure. Likewise, the strength of the sample with 8.7% cement after 40 months of curing could not be measured, since it exceeded the maximum capacity of the shear vane device. It should be noted that the reported shear strengths (in Fig. 1) are relatively low for many practical applications. However, this data will be combined with those obtained from the literature later in the paper, to expand the database and provide more comprehensive information. As expected, the shear strength increases with curing time and cement content, since both contribute to the production of more cementing bonds within the soil body, and this confirms the previous findings of many other researchers (e.g. Nagaraj et al., 1996; Uddin et al., 1997; Bergado et al., 1999; Horpibulsuk et al., 2003; Kamruzzaman et al., 2009).

An interesting observation is that the cemented samples kept gaining significant amounts of strength, long after the start of curing, as the curves in Fig. 1 have considerable slope even after 1000 days. It is typically assumed that the hydration rate of ordinary Portland cement drops significantly with time after curing. The observed trend of increasing strength in cemented clay is not usual for cement based construction materials, such as concrete. It may be attributed to the slower secondary reactions that happen between clay minerals and cementation products. Further explanation of this behaviour is provided in Section 5.

The values plotted in Fig. 1 are peak undrained shear strengths measured with the laboratory shear vane device. If these peak strengths are normalized by the peak strength of the same sample after 28 days of curing ($c_{u,\ 28\ \text{days}}$), all three curves in Fig. 1 plot on the same curve (Fig. 2), indicating that the hardening trend is independent of the amount of cement added to the soil. Fig. 2 better illustrates the continuation of the rate of increase of peak shear strength; the peak strength after 400 and 1200 days of curing is 2 and 3 times that for 28 days, respectively. As reported by Tan et al. (2002), the results of unconfined compression tests on cemented specimens of Singapore marine clay suggest the same independence of hardening trend from cement and water contents. Horpibulsuk et al. (2003) have also shown the same independence for Bangkok, Ariake, and Indian clays.

The data plotted in Fig. 2 were obtained for one type of clay, with a single value of moisture content and variations in the cement content. To investigate whether a unique relationship occurs for a number of clays, where the hardening trend is independent from mineralogy and moisture content (as well as from the cement content), unconfined compression test results collected from the literature for the different soils described in Section 3.3 have been plotted, along with the results for Nanticoke clay, in Fig. 3. It can be seen that all data follow a similar trend, yielding the following relationship:

$$\frac{c_u}{c_{u,28\ \text{days}}} = 0.96 \times \left(\frac{t}{t_{28\ \text{days}}}\right)^{0.31} \tag{5}$$

where "c_u" is the undrained shear strength after "t" days of curing. More than 440 data points for 12 different clays, with a

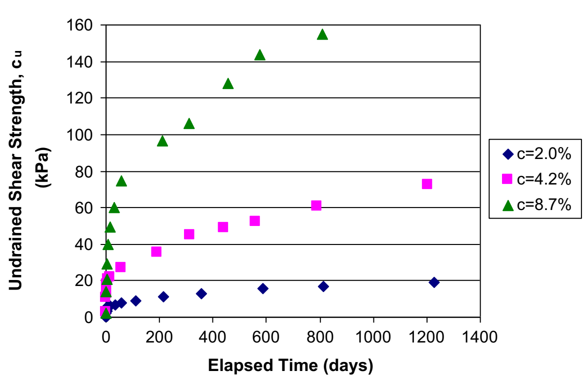

Fig. 1. Shear strength of artificially cemented Nanticoke clay (LI=3, w=98%) with curing time.

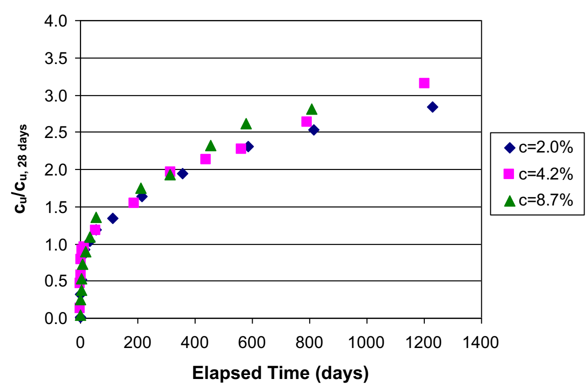

Fig. 2. Normalized undrained shear strength of cemented Nanticoke clay (LI=3, w=98%) versus curing time.

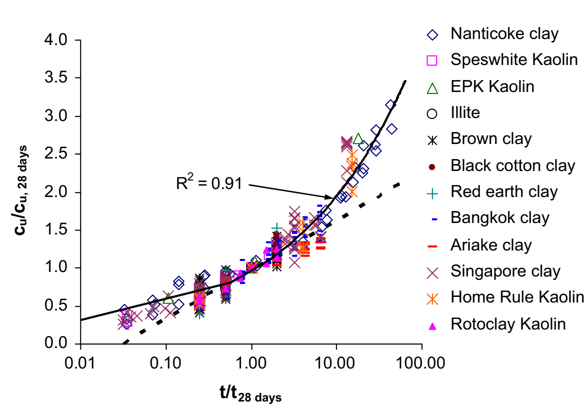

Fig. 3. Normalized undrained shear strength of various clays cemented with Portland cement versus curing time.

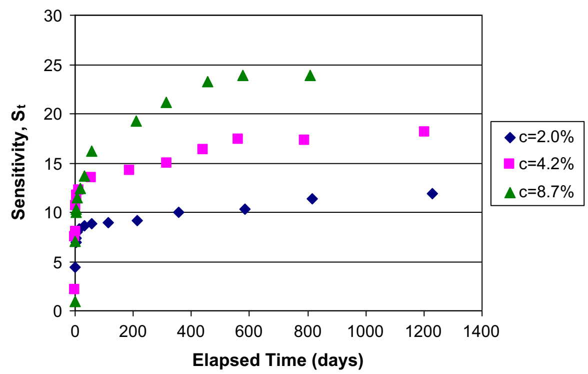

Fig. 4. Sensitivity of artificially cemented Nanticoke clay (LI=3, w=98%) with curing time.

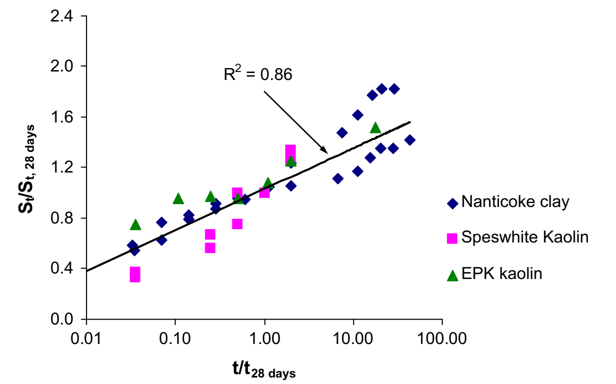

Fig. 5. Normalized sensitivity of three cemented clays versus curing time.

wide range of liquidity indices (LI ~0.4–3.0) and cement contents (c ~1–100%) were used to derive Eq. (5). Table 2 summarizes soil properties of the samples plotted in this figure. The dashed line in Fig. 3 is calculated from the logarithmic equation that was suggested earlier by Horpibulsuk et al. (2003) for soils with a liquidity index of 1.0–2.5. It should be noted that the proposed equation has been obtained for curing times in the range of 1–1250 days and may not provide a good prediction particularly for short curing times less than 1 day.

As well as the peak strengths, residual vane shear strengths of three of the clays were measured by the authors. It was observed that samples with higher cement content and curing time have higher residual strengths. However, the residual strength does not increase at the same rate as the peak strength. As a result, the brittleness and sensitivity of the soil also increase with cement content and curing time. Fig. 4 shows the development of sensitivity in cemented samples of Nanticoke clay with three different cement contents; the sensitivity number of the specimen with 8.7% cement has reached up to 13 and 24, after 28 and 600 days of curing, respectively.

The sensitivity values of Nanticoke clay samples are plotted along with those for samples of EPK and Speswhite kaolin in Fig. 5. Similar to the previous analysis, the value measured after 28 days of curing, $S_{t,\ 28\ \mathrm{days}}$, is used as a normalizing parameter. The sensitivity is seen to increase with curing time in a logarithmic fashion according to

$$\frac{S_t}{S_{t,\ 28\ \mathrm{days}}} = 0.14 \times \ln\left(\frac{t}{t_{28\ \mathrm{days}}}\right) + 1.03 \tag{6}$$

where "S_t" is the sensitivity after "t" days of curing. Comparison of Figs. 3 and 5 shows that both peak strength and sensitivity initially increase linearly with time in semi-logarithmic space. However, after almost 100 days of curing, the peak strength begins to increase with a faster rate, while the sensitivity graph remains essentially linear in semi-logarithmic space.

4.2. Shear strength of artificially cemented clays: state parameters

In addition to curing time and cement content, other parameters, such as water content and liquidity index, plasticity index, clay content, curing temperature, and soil mineralogy,

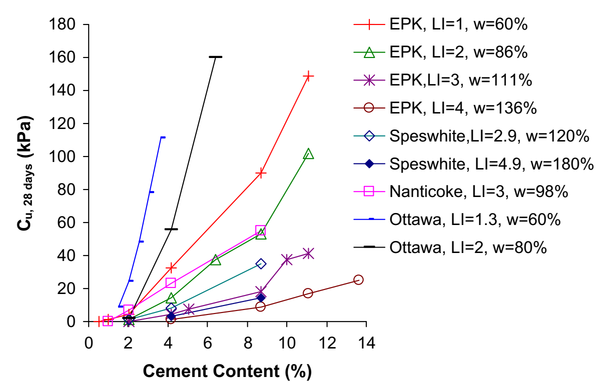

Fig. 6. The relationship between cement content and undrained shear strength for soils with different liquidity indices.

affect the behaviour of artificially cemented clays (Croft, 1967a, 1967b; Woo, 1971; Broms, 1986; Bergado et al., 1996; Uddin et al., 1997; Miura et al., 2001). To further investigate the parameters that have a significant effect on the strength of cemented clays, the results of the laboratory shear vane tests on samples made of different clays with varying moisture and cement contents were studied. Fig. 6 shows the relationship between cement content and undrained shear strength after 28 days, $c_{u,\ 28\ \mathrm{days}}$, for EPK and Speswhite Kaolins and Nanticoke and Ottawa clay. As we can see, Ottawa and Nanticoke clays, which are both predominantly illitic, gain significantly higher strengths than does kaolin clay, when mixed with cement at a similar cement content and a similar liquidity index or water content. This illustrates the importance of soil mineralogical composition in cement stabilization.

Miura et al. (2001) introduced clay–water/cement ratio, w/c, as a key parameter in understanding the behaviour of soft clays admixed with Portland cement. Horpibulsuk et al. (2003) and Horpibulsuk et al. (2005) further discussed the importance of this parameter and proposed an expression for calculating the amount of added cement to stabilize soft clays based on the water content. The results of this study corroborate the previous findings regarding the importance of clay–water/cement ratio. To better illustrate the results, cement–moisture ratio, c/w, which is the ratio of cement content (%) to initial moisture content of the clay (%), is

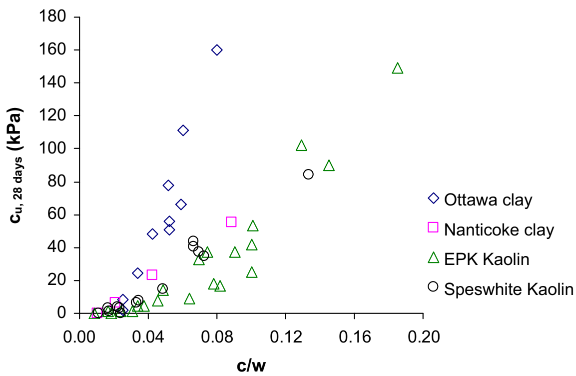

Fig. 7. Variations in undrained shear strength with cement–moisture ratio (*c*/*w*).

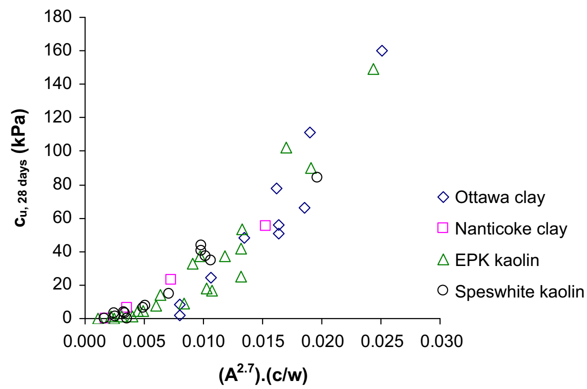

Fig. 9. The effect of cement–moisture ratio (*c*/*w*) and activity number (*A*) on undrained shear strength.

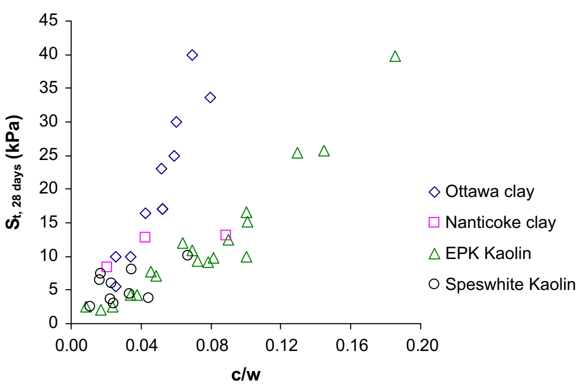

Fig. 8. Variations in sensitivity with cement–moisture ratio (*c*/*w*).

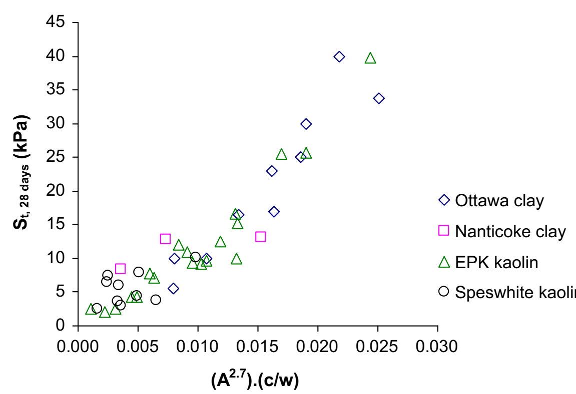

Fig. 10. The effect of cement–moisture ratio (*c*/*w*) and activity number (*A*) on sensitivity.

used instead of water–cement ratio, *w*/*c*. The cement–moisture ratios of the samples tested with laboratory shear vane have been plotted against 28-days undrained shear strength and sensitivity in Figs. 7 and 8, respectively. Similar trends are observed in both graphs as the undrained shear strength and sensitivity are closely related. In both figures, a trend can be detected for each type of clay, i.e. the shear strength and sensitivity increase with an increase in *c*/*w*. Moreover, the increase in the strength with an increase in *c*/*w* ratio is non-linear; the rate of increase in strength and sensitivity with *c*/*w* ratio increases as the *c*/*w* ratio gets larger. However, due to their mineralogical variations, different clay types follow different paths, and the overall data are rather scattered. To create an expression that may be used to predict the behaviour of cemented clays accounting for clay mineralogy, an additional parameter that takes into account the effect of clay type is introduced.

On inspection of Figs. 7 and 8, it can be seen that at similar cement–moisture ratios, the highest and lowest strength and sensitivity belong to samples of Ottawa clay and EPK kaolin, respectively. For Ottawa clay, a cement–moisture ratio of 0.08 results in a shear strength of 160 kPa, while for EPK kaolin, a cement–moisture ratio as high as 0.185 only gives a shear strength of 149 kPa. Further comparison between the activity numbers from Table 1 and the results presented in Figs. 7 and 8 confirms that the higher the activity number (*A*) of the soil, the higher its strength and sensitivity at a given cement–moisture ratio.

Plotting the undrained strength and sensitivity against cement–moisture ratio multiplied by $A^{2.7}$ is found to eliminate the effect of variations in the type of clay and results in a unique curve for all four clays (Figs. 9 and 10). Again, this relationship is non-linear; the strength and sensitivity increase with an increase in $(A^{2.7}) \times (c/w)$, indicating that higher soil activity leads to the cementing agent producing more and/or stronger bonds.

4.3. *Predicting the behaviour of artificially cemented clays*

4.3.1. *Undrained shear strength: unconfined compression*

The data presented in Figs. 9 and 10 only cover a small range of clay types and cement and moisture contents, due to the limitations of using the laboratory shear vane device; cement contents for ground improvement applications are typically higher. Unlike laboratory shear vane, unconfined compression tests can be performed on samples with much higher undrained shear strengths. Several researchers have performed such experiments on various types of artificially cemented clay. Their results, in terms of undrained shear strength, are also plotted versus the *c*/*w* ratio of the specimens in Fig. 11 along with the results presented in the previous figures. The kaolinite data include those for EPK, Speswhite, and Rotoclay kaolins. Although some scatter exists within the data for each clay type, a clear pattern of increase in strength

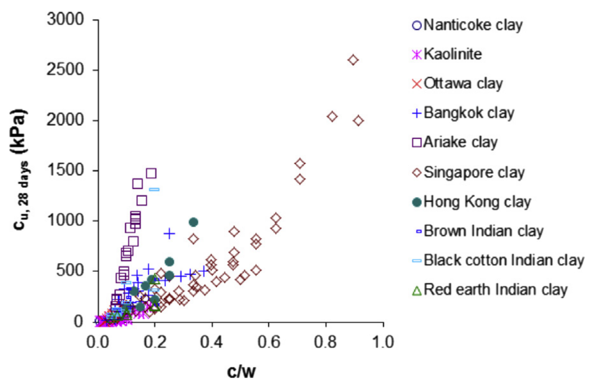

Fig. 11. Undrained shear strength of various cemented clays versus cement–moisture ratio.

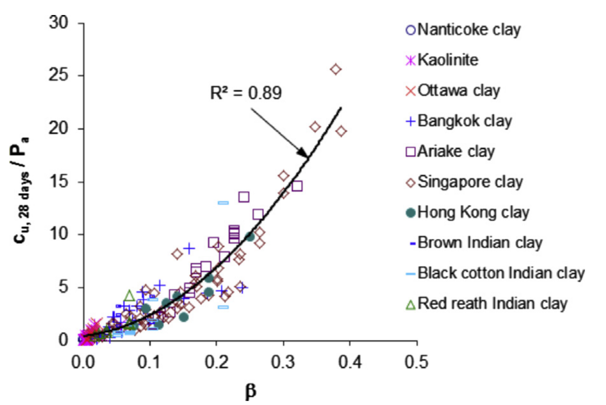

Fig. 12. Undrained shear strength of various clays versus the β parameter.

with c/w ratio is shown. Ariake clay, which has an activity number of 1.18, gains much higher strengths than Hong Kong, Bangkok, or Singapore clays the activity of which is 0.91, 0.87, and 0.76, respectively, confirming the importance of soil activity in the gained strength. Based on this concept, a relationship between activity, c/w ratio and undrained shear strength was defined using the parameter β below:

$$\beta = A^{3.2} \times \frac{c}{w} \tag{7}$$

Fig. 12 shows the values of undrained shear strength normalized by the atmospheric pressure, P_a (=101.3 kPa), plotted against this new β parameter. The relationship can be satisfactorily modelled by the following polynomial function:

$$\frac{c_{u,\ 28\ \text{days}}}{P_a} = 125.24\beta^2 + 7.47\beta + 0.42 \tag{8}$$

This correlation enables an estimate of the shear strength of a cemented soil based on c/w ratio and activity. It should be noted that Eq. (8) has been derived for β values less than 0.4 and may not be applicable to cemented clays with $\beta > 0.4$. During the analysis, it was also noticed that in some cases, w/c ratios lower than 3 (c/w values higher than 0.33) resulted in lower than expected undrained shear strength especially for Bangkok and Singapore clays. This indicates that the addition of high amounts of cement to certain soils might also reduce the efficiency of the soil improvement process.

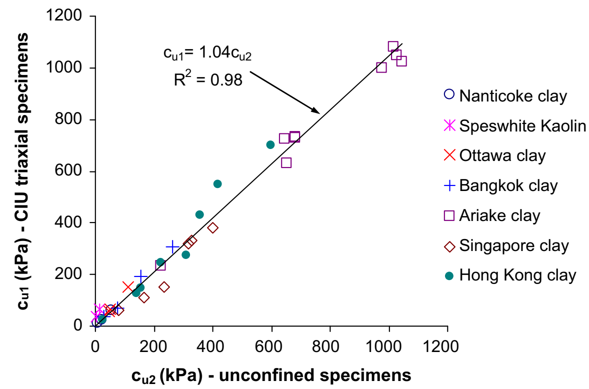

Fig. 13. Comparison between peak undrained shear strengths obtained for consolidated triaxial specimens and those obtained for unconfined specimens.

4.3.2. Undrained shear strength: triaxial

In addition to data from unconfined compression tests, the results of several undrained triaxial test studies on artificially cemented clays were used in the analysis. All of these cemented specimens had been isotropically consolidated to 25, 50, or 100 kPa before undrained shearing, but in all cases, the consolidation pressure had been considerably lower than the yield stress of the specimens, indicating that the cementitious bonds had remained relatively intact. Fig. 13 shows the peak undrained shear strengths measured with triaxial tests for confined specimens, consolidated below the yield stress, versus those measured for unconfined specimens with unconfined compression or shear vane tests. As it shows, the slope of the line fitting the data is very close to unity and indicates only 4% of increase in strength due to confinement. Hence, Eq. (8), which is obtained based on the results of unconfined compression tests, should be able to predict undrained shear strengths obtained from triaxial experiments, as long as the yield consolidation pressure is not exceeded. It can also be concluded that if the isotropic loading phase does not cause the soil to yield, confinement does not have any significant effect on the undrained shear strength of cemented clays. This indicates that cemented clay pre-yield behaviour is predominantly dependent on the cementitious bonds, rather than friction. Fig. 14 shows the peak undrained shear strengths, obtained from triaxial experiments, versus the β parameter. It can be seen that the same trend observed in Fig. 12 also exists for the triaxial specimens. The dashed line, which represents Eq. (8), can also provide a reasonable match for the results of this specific form of undrained triaxial test.

4.3.3. Compressibility and vertical yield stress: oedometer

Since they usually have high moisture contests and void ratios, cemented clays are often metastable and undergo a significant amount of compression after the yield point is passed. Thus, accurate prediction of the yield stress can be very important in the design of cement–clay mixtures for settlement problems. Analysis was conducted on the results of oedometer experiments performed for this study, along with

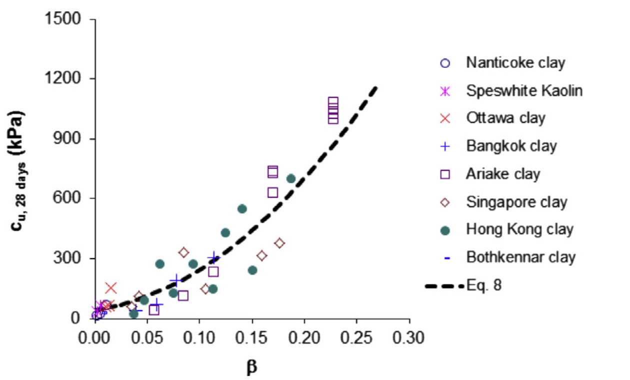

Fig. 14. Undrained shear strength from CIU triaxial tests versus the β parameter.

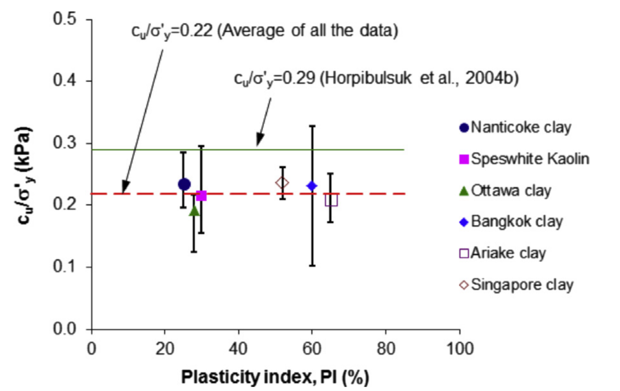

Fig. 16. The relationship between the plasticity index and the c_u/σ'_y ratio.

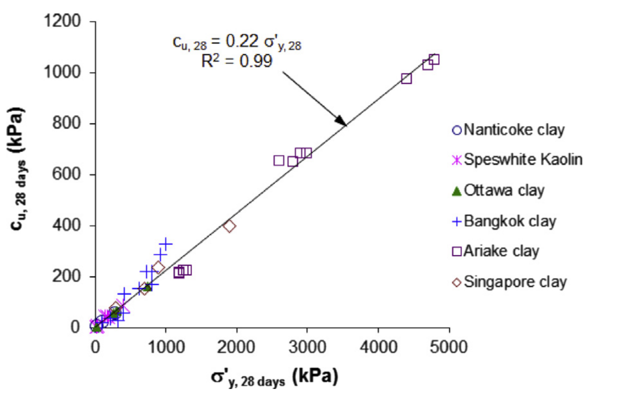

Fig. 15. Vertical yield stress versus undrained shear strength.

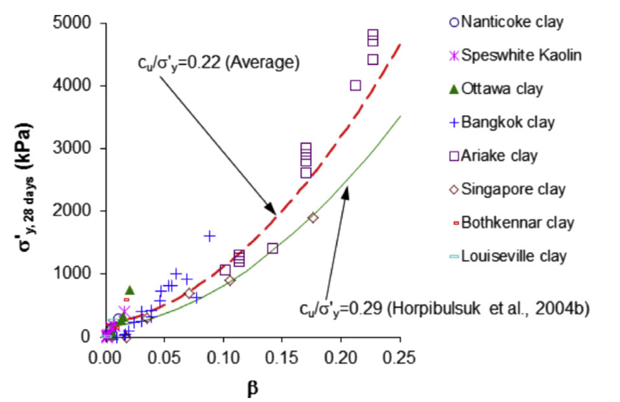

Fig. 17. The relationship between vertical yield stress and β.

those found in the literature. Fig. 15 shows the relationship between σ'_y, which is the vertical yield stress obtained from oedometer tests, and undrained shear strength, c_u, for the cemented specimens. All of the σ'_y values have been calculated based on the Casagrande method (ASTM D2435). The results support the suggestion by Horpibulsuk et al. (2004b) that a linear relationship exists between vertical yield stress and undrained shear strength of cement-admixed clays. Although there is scatter in the data, it can be seen that the c_u/σ'_y ratio is insensitive to the PI. An average value of $c_u/\sigma'_y = 0.22$ is found, which is similar to the values proposed by Mesri (1975) and re-evaluated by other researchers (Trak et al., 1980; Trak and Leroueil, 1983; Jamiolkowski et al., 1985; Mesri, 1989) for normally consolidated and lightly overconsolidated clays. Horpibulsuk et al. (2004b) performed similar analysis on cemented Bangkok, Tokyo, and Ariake clays and suggested a range of 0.23–0.36 for c_u/σ'_y with a slightly higher average value of 0.29. The significant scatter in c_u/σ'_y ratios is better illustrated in Fig. 16. The variation in the data may originate in the effect of additional cementitious fines on the plasticity index, which has not been accounted for in the results.

Since vertical yield stress correlates with undrained shear strength, we may expect it to correlate with the β parameter as well. Using Eq. (8) and assuming a c_u/σ'_y ratio of 0.22, the vertical yield stress of the cemented material can be approximated as follows:

$$\frac{\sigma'_{y,\,28\ \mathrm{days}}}{P_a} = 569.27\beta^2 + 33.95\beta + 1.91 \tag{9}$$

Fig. 17 gives the values of σ'_y versus the β parameter with Eq. (9) plotted as a dashed line. The solid line represents the relationship obtained by using a c_u/σ'_y ratio of 0.29, as suggested by Horpibulsuk et al. (2004b). As the figure shows, using a ratio of 0.22 provides a better approximation of the overall data.

Most naturally structured soft clays experience a relatively abrupt destructuration once the virgin yield stress is exceeded (e.g. Burland, 1990). In comparison, numerous consolidation test results on artificially cemented clays have confirmed that this type of structured material undergoes a more gradual breakage of the bonds and has an approximately linear (in semi-logarithmic space) post-yield compression curve (e.g. Miura et al., 2001; Rotta et al., 2003; Lorenzo and Bergado, 2004; Xiao and Lee, 2008; Kamruzzaman et al., 2009). In common with other structured soils, however, this compression line still converges with the intrinsic compression line (ICL) of the material at high pressures (Burland, 1990; Liu and Carter, 2002; Rotta et al., 2003). Fig. 18 illustrates an example of the different compression behaviours of naturally structured and artificially cemented clays. Naturally structured Ottawa clay undergoes higher amount of pre-yield compression followed by an abrupt post-yield destructuration, while

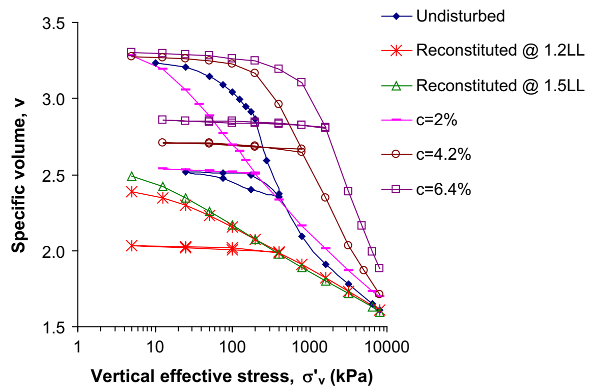

Fig. 18. One dimensional compression curves for undisturbed, artificially cemented, and reconstituted Ottawa clay.

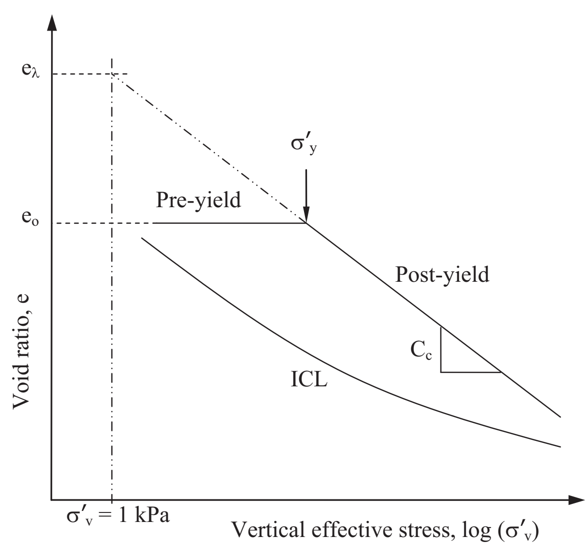

Fig. 19. Idealized compression behaviour of artificially cemented clays (after Horpibulsuk et al., 2004b).

the artificially cemented material displays a stiffer pre-yield response and a more gradual post-yield breakage of the cementitious bonds. Even though artificially cemented clays have been previously treated as "structured soils", they may be better represented in e: σ'_y space using two limiting relationships (Horpibulsuk et al., 2004b); one a neat horizontal pre-yield line followed by a pseudo-normal compression line (Fig. 19). This pseudo-normal compression line (so called since it may represent a family of changing gradient lines with destructuration) can be represented by the following equation:

$$e = e_\lambda - C_c \log(\sigma'_v) \tag{10}$$

where e_λ is the void ratio at $\sigma_y' = 1$ kPa, and C_c is the "average" slope of the compression line. In general, we expect steeper compression lines (i.e. greater values of C_c) to be accompanied by higher interception values (e_λ) for a specific type of clay.

A number of researchers have previously proposed normalized such relationships for predicting the generalized compression behaviour of reconstituted or artificially cemented clays (e.g. Nagaraj and Srinivasa Murthy, 1986; Burland, 1990; Nagaraj et al., 1993, 1994; Horpibulsuk et al., 2004b). These relationships are often written in the following form:

$$\frac{e}{e_n} = a - b \log(\sigma'_v) \tag{11}$$

where a and b are normalized parameters usually obtained from experimental data, and e_n is a normalizing void ratio. The void ratio at the liquid limit (e_L) has been commonly used as the normalizing void ratio for reconstituted clays (e.g. Burland, 1990; Nagaraj et al., 1994). However, e_{100} (the void ratio at σ'_v=100 kPa) was used also by Horpibulsuk et al. (2004b) to normalize the compression behaviour of both cemented and uncemented materials. At a void ratio equal to e_L, reconstituted clays have an undrained shear strength (c_u) of approximately 1.7 kPa (Wood, 1990), corresponding to a vertical effective stress (σ'_v) of 7.7 kPa (based on the relationship suggested by Mesri (1975)). Likewise, e_λ (the void ratio at $\sigma'_v = 1$ kPa) can also be used as another pressure based, normalizing reference volume. Since e_L is a reference state associated with remoulded soil states, it is essentially arbitrary with respect to cemented and structured soils. In such cases, e_λ is preferred since it is a fixed and known reference volume for an effective stress of 1 kPa, without the uncertainty associated with the determination of liquid limit states. Therefore, Eq. (11) can be rewritten as

$$\frac{e}{e_\lambda} = 1 - \frac{b}{a} \log(\sigma'_v) \tag{12}$$

here the ratio b/a is equal to C_c/e_λ (Eq. (10)). An examination of relationships with a similar form to Eq. (11) that are available in the literature reveals that the b/a ratio ($\sim C_c/e_\lambda$ ratio) has a very narrow range (between 0.21 and 0.23) and appears to be independent of the value of the a and b parameters (e.g. Nagaraj and Srinivasa Murthy, 1986; Nagaraj et al., 1993, 1994; Horpibulsuk et al., 2004b). Interestingly, assuming that the vertical yield stress at the liquid limit is approximately 7.7 kPa, the relationship proposed by Burland (1990) between C_c^* (for reconstituted clay) and e_L also gives a C_c^*/e_λ ratio of approximately 0.21.

The average e_λ and C_c parameters were also obtained for the cemented clays studied herein. The results again suggest that a linear relationship exists between the two parameters (Fig. 20):

$$C_c = 0.23 e_\lambda \tag{13}$$

Hence, as was previously suggested by Horpibulsuk et al. (2004b), the same relationship (Eq. (13)) governs both the destructuration/compression of artificially cemented clays and the compression of reconstituted material. This resemblance in behaviour could be related to the more gradual breakage of the bonds within the artificially cemented clay; although the bond breakage is brittle, it appears to be sufficiently disseminated to lead to elasto-plastic frictional behaviour of the cemented material. It should be noted that the data used to find Eq. (13) were obtained by oedometer tests for a vertical effective stress (σ'_v) range of 5–8000 kPa and initial void ratios (e_o) less than 5.5.

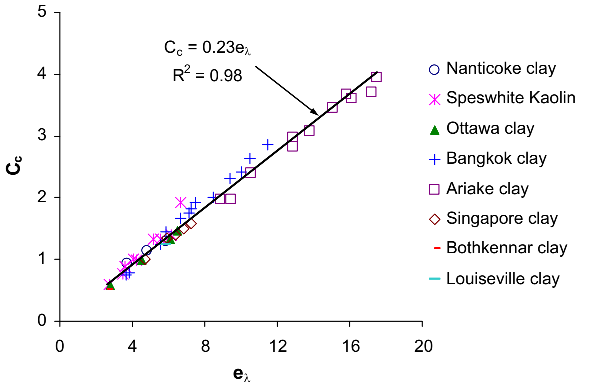

Fig. 20. The relationship between C_c (the slope of the compression line) and e_λ (void ratio at $\sigma'_v = 1$ kPa), for cement-treated clays.

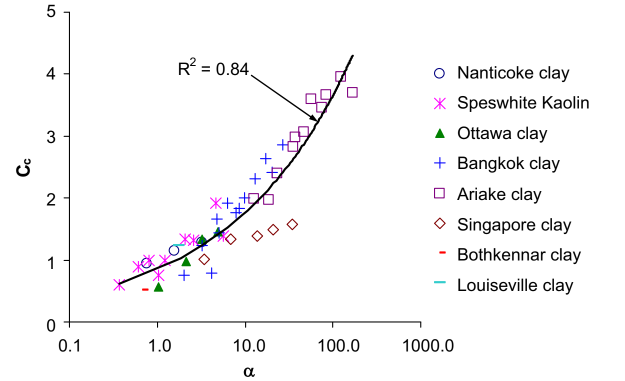

Fig. 21. The relationship between the slope of the compression line, C_c, and α.

Combining Eqs. (12) and (13) gives

$$\frac{e}{e_\lambda} = 1 - 0.23\ \log(\sigma'_v) \tag{14}$$

Thus, Eq. (14) produces a number of discrete lines in the $e - \log(\sigma'_v)$ space, restricting the pseudo-normal compression lines to only moving on certain paths. Therefore, having the value of e_o for a particular cemented clay and estimating σ'_v from Eq. (9), we can utilize Eq. (14) to calculate the values of e_λ and C_c for a specific cemented clay.

A further parametric study was completed to provide a deeper understanding and enable the prediction of e_λ and C_c directly from the basic geotechnical properties of the cemented material. For a given vertical yield stress (σ'_v), a higher post-curing void ratio (e_o) will increase the post-yield destructuration rate of the material and will thus be accompanied by an increase in e_λ and C_c. Similarly, for a given initial void ratio, a higher yield stress results in greater e_λ and C_c values. Based on the information available in the literature it seems reasonable that there are links between form of cementation, post-curing void ratio, yield stress, and clay mineralogy. Horpibulsuk et al. (2004b) suggested that the slope of the compression line (C_c) only depends on the cement content and clay type and is independent of the moisture content. However, the results of this study indicate that the water content can also affect the compressibility of a cemented clay. Since a number of researchers have previously correlated the slope of the compression line of remoulded clays with the plasticity index or liquid limit (Burland, 1990; Wood, 1990), the liquidity index was used in calculations to incorporate the effect of index properties with water content. After analysing the oedometer data, it was found that the post-yield compressibility of the cemented clays is related to the liquidity index (LI), cement content (c, %), and activity number (A). Hence, a new parameter, α, was defined as follows:

$$\alpha = LI\ cA^{3.2} \tag{15}$$

The calculated C_c values are plotted versus the new α parameter in Fig. 21 for different cemented clays. A clear trend can be detected in the results, providing the following relationship:

$$C_c = 0.85\alpha^{0.32} \tag{16}$$

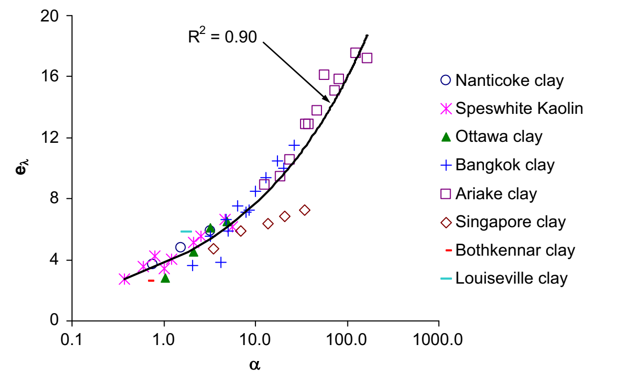

Fig. 22. The relationship between e_λ (void ratio at $\sigma'_v = 1$ kPa) and α.

Similarly, a relationship for estimating the representative e_λ values based on the α parameter can be obtained (Fig. 22):

$$e_\lambda = 3.70\alpha^{0.32} \tag{17}$$

Substitution of Eqs. (16) and (17) into Eq. (10) provides

$$e = 3.7\alpha^{0.32}[1 - 0.23\ \log(\sigma'_v)] \tag{18}$$

It should be noted that the relationships presented in this section are derived based on the available data for a number of clays with a certain range of water content, cement content, and activity number; the properties of these clays, along with the type of experiments performed on each soil, are given in Table 3.

5. Discussion

Two interesting observations emerge from the results presented herein. Firstly, the strength of artificially cemented clays continues to increase with curing time even beyond 3 years. Secondly, the gained strength appears to be a function of the activity number of the soil, as well as the w/c ratio. Both of these phenomena can be further understood by taking into account the importance of the secondary pozzolanic reactions.

The hydration reaction is primarily responsible for the short-term gain in strength, since it produces the primary bonds and reduces the moisture content of the mixture. In contrast, the pozzolanic reactions commence when adequate concentration of hydroxide ions is produced and a certain level of alkalinity is reached in the pore fluid (Herzog and Mitchell, 1963; Xiao and Lee, 2008). The secondary reactions are considerably slower than the hydration reaction and can continue for months, or even years after mixing (Bergado et al., 1996), but only if there is sufficient calcium available in the matrix and the pH remains elevated. According to Stocker (1975), the first layer of secondary reaction products covers the particle surfaces and impedes further reactions. For the reactions to continue, $Ca(OH)_2$ must diffuse through the first layer of reaction products (Stocker, 1975; Croft, 1967b; Bhattacharja et al., 2003). This process of diffused cementation occurs at a much slower rate than the hydration reaction of the cement. Furthermore, clay minerals are much more chemically reactive than concrete aggregates, so the pozzolanic reaction is more pronounced in artificially cemented clays than it would be in concrete or other granular pastes. Thus the continuation of strength gain, which differentiates the hardening response of a cemented clay from that of concrete, could be attributed to the secondary pozzolanic reactions between the clay minerals and cement hydration products.

As stated by Bergado et al. (1996), if cured and mixed under similar conditions, soils with higher pozzolanic reactivity obtain greater strengths, compared to those with lower reactivity. Most physicochemical phenomena that happen in the soil material are affected by the available surface area (Bhattacharja et al., 2003). Croft (1967a) suggested that expansive minerals, which have high surface areas and activity numbers, consume the lime released during the cement hydration more rapidly. Therefore, secondary reactions commence sooner and are of higher intensity in active clays. This could result in higher gained strength in such soils, compared to that of soils with lower activity number. In addition, based on the work of Wissa et al. (1965), the amount of cementitious material produced during the pozzolanic reaction depends on the amount of clay fraction as well as that of amorphous silica and alumina that exist in the soil; having a high surface area and therefore a high activity, poorly crystallized minerals react more readily with calcium hydroxide and produce more cement. Hence, the activity number of the clay is an important parameter in determining the gained strength, as it represents the ability of the material to be involved in the pozzolanic reactions.

Although the results of this work imply that secondary reactions, which are often deemed to be lower in significance compared to the hydration reactions, play an important role in providing the cemented clay with strength, further elucidation from a micro-structural point of view is needed to understand the mechanisms by which these reactions affect the cemented soil behaviour.

Soil aggregates or clusters are the fundamental components controlling soil behaviour. They can act almost as single particles and interact to generate the strength and stiffness in clays (Mitchell and Soga, 2005). Even if the soil–cement mixture is thoroughly blended, clay particles will form aggregates enclosed by the cement slurry (Croft, 1967b; Bergado et al., 1996). During the curing period, hydration reactions form hardened cement bodies, which connect to develop a matrix within the soil mass (also called skeletal cementation by Bhattacharja et al. (2003)). On the other hand, since pozzolanic reactions take place between clay minerals and cement hydration products and hence happen near particle surfaces, they form cementitous material on or near the surface of clay particles. Stocker (1975) suggests that these reactions take place exclusively at particle edges. The produced cement pastes the flocculated particles together at points of contact, developing hardened soil aggregates (Herzog and Mitchell, 1963). The strength of the improved soil will depend on the strength of the hardened cement bodies and soil aggregates (Bergado et al., 1996). However, because the cementitious bonds produced by pozzolanic reactions (CSH and CAH) have a lower strength compared to those created by the hydration reactions [mainly $C_3S_2H_3$] (Bergado et al., 1996), the strength of the whole cemented soil mass should mainly depend on the strength of the hardened aggregates, rather than that of hydrated cement bodies. This is consistent with the findings of Saitoh et al. (1985), who suggested that the lower the pozzolanic reactivity of the soil, the higher its strength dependence on the properties of hardened aggregates. Thus, the strength of the cemented material is greatly dependent on the degree and intensity of the secondary reactions.

Nonetheless, it should be borne in mind that the results presented here are only for soils with low to medium values of activity number (less than 1.2). Very high activity numbers, which are usually encountered in smectitic clays, may have a negative effect on the strength of cement-treated soil (Bergado et al., 1996), as the high affinity of these clays for lime depletes the soil–cement mixture from calcium hydroxide and reduces the pH of the aqueous phase (Croft, 1967a), decreasing the solubility of silicates and aluminates and bringing the secondary reactions to a halt (Herzog and Mitchell, 1963; Bergado et al., 1996). This is further evidenced by the works of Croft (1967a), Noble and Plaster (1970), Ingles and Metcalf (1972), and Osula (1996), who suggested that Portland cement treatment does not develop much strength in highly active clays, such as montmorillonite, and recommended that it be replaced by lime stabilization, so that there would be enough lime available in the mix to keep the pH at a high level.

Other factors could also be contributing to the observed behaviour of the artificially cemented clays. Noble and Plaster (1970), who examined the chemical reactions in mixtures made by the addition of Portland cement to various types of clay, suggested that the hydration reaction *initially* takes place at a slower rate in clays than concrete. This is in accord with the statement made by Bergado et al. (1996) that cemented soil is better improved if type III Portland cement, which provides relatively high early strengths, is used rather than type I. Therefore, another reason why the rate of increase in the strength of cemented clays is lower than that of concrete could be the initially slower hydration reaction due to the presence of clay minerals.

Additionally, to better understand the importance of the activity number compared to that of water–cement ratio, we can further deliberate on Abrams' law. This law states that the strength of concrete only depends on the ratio of “free” water content to the cement content in the mix (Abrams, 1918). To successfully apply Abrams' law, which was originally intended for concrete material, to artificially cemented clays, we should address the main difference between concrete and clay constituents. Unlike concrete, clayey materials tend to absorb significant amounts of water. The extent of this absorption depends on the specific surface area, which can be represented by the activity number of the soil. The amount of adsorbed water increases in more active clays, which usually have higher surface area and charge deficiency (Mitchell and Soga, 2005). Hence, there will be less “free” water available in the paste for cement hydration. This adsorbed water is not taken into account when water–cement ratio (*w/c*) is calculated. Consequently, comparing two different clays that have been artificially cemented and have similar clay water–cement ratios, the one having a higher activity would probably gain higher strength, since the actual *w/c* ratio is lower than the nominal value.

A few researchers have tried to correlate the mechanical behaviour of cemented clays with the index properties (e.g. Woo, 1971). Using the activity number for modelling the behaviour of cemented soil appears to have an advantage over using other soil parameters such as plasticity index, liquid limit, or clay content. As postulated by Skempton (1953), the plasticity of a soil (and its index properties) depends on both the type of clay minerals and the clay content in that soil. Thus using the activity number would eliminate the variability observed in index properties of different samples taken from the same site and would make possible further interpretations.

In addition to soil activity and the *w/c* ratio, the presence of organics in the soil could also significantly affect the cementation process and its outcome. High organic contents hinder the development of strength in soils improved by Portland cement or lime (Tremblay et al., 2001). Miura et al. (1986) suggest that cement rather than lime should be used in the stabilization of organic clays to achieve better results. None of the clays described in this study contains amounts of organic material large enough to affect the cementation process. Thus, caution should also be exercised should the equations provided herein be used for organic clays.

6. Summary and conclusions

To study the effect of soil mineralogy and activity on the properties of cemented clays and to find important parameters governing cement-treated clay mechanical behaviour, the results of laboratory shear vane, unconfined compression, undrained triaxial, and oedometer tests on many different types of clays treated with Portland cement have been examined. The results indicate that

- Although the hydration rate of ordinary Portland cement used to stabilize clay drops significantly with curing time, cemented clays continue to gain significant amount of strength long after the curing has started. This behaviour, which is not typical of cement-based construction materials can be attributed to slower pozzolanic reactions that happen between clay minerals and cementation products.
- The hardening trend of artificially cemented clays (normalized against shear strength after 28 days, $C_{u,\ 28\ \mathrm{days}}$) is the same for all clay types and can be modelled by a power function.
- The residual shear strength of the cemented soil increases with increasing curing time or cement content. However, the residual strength increases with a slower rate than does the peak strength. Consequently, the sensitivity and brittleness of the material increases with curing time and cement content.
- Undrained shear strength of cemented clays is dependent on their mineralogical composition. The strength of the cement-treated material is also closely related to its cement–water ratio, *c/w*. However, this ratio alone is not enough to predict the behaviour of cemented clays. The activity of the soil, which represents its mineralogy and its ability to be involved in chemical reactions, should also be considered. The higher the activity number, the higher the strength of the clay at a given cement–moisture ratio.
- Using the introduced empirical β and α parameters has enabled the prediction of the undrained shear strength, vertical yield stress, and the slope of the compression line for artificially cemented clays.
- Undrained shear strength of cemented clays is independent of the confining pressure of the soil, unless the yield stress has been exceeded. Therefore, the pre-yield behaviour of cement-treated clays is predominantly dependent on cementation bonds, rather than friction.
- Pozzolanic reactions are believed to be responsible for the observed behaviour of cemented clays. A matrix of hardened material is thought to form in the soil body due to hydration of cement. However, the hydration process does not necessarily produce bonds between the particles. The pozzolanic reactions generate cementing bonds between soil particles and within aggregates and are apparently responsible for the overall strength of the cement-aggregate network.
- The hardening of cemented clay takes much longer than that of concrete, perhaps due to the slow rate of the pozzolanic reactions. In addition, more active clays, which are believed to react more rapidly and intensely with the hydration products and therefore produce more cementing bonds, gain higher amount of strength due to artificial cementation.
- It should be noted that the suggested predictive relationships have been obtained based on information from specimens with a β parameter less than 0.4. The proposed relationships may be used only with great caution for organic clays or smectitic soils, with an activity number higher than 1.2.

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