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TECHNICAL NOTE

Drained strength of bentonite-enhanced sand

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KEYWORDS: clays; laboratory tests; sands; shear strength.

INTRODUCTION

Barriers with a low hydraulic conductivity are used as part of waste containment systems to prevent groundwater contamination by liquids from the waste. Commonly barriers are either a geomembrane (usually an HDPE sheet), a mineral layer or a combination of the two. Recently there has been increasing interest in the use of bentonite–sand mixtures as the mineral layer, in both landfill liners and vertical cut-off walls, partly because they are less susceptible to frost damage and desiccation cracking than compacted clay (Dixon *et al.*, 1985; Kraus *et al.*, 1997). Currently there is uncertainty about the strength and bearing capacity of these materials. This note reports drained strength data for bentonite–sand mixtures and proposes that trends in these data are mainly the result of variations in the relative density of the sand.

MATERIALS

The materials used in the study were Conquest grade Wyoming bentonite supplied by Colin Stewart MinChem, and Knapton Quarry sand. Conquest grade Wyoming bentonite originates from Newcastle, Wyoming, and is ground, dried and air classified during processing. X-ray diffraction spectroscopy and chemical analysis (Mollins *et al.*, 1996) indicate that it is predominantly a sodium montmorillonite with some quartz and trace cristobalite. Knapton Quarry sand is a slightly silty, fine, angular quartz sand (containing some iron oxides and silicate minerals). It was sieved to remove particles greater than 2 mm (the fraction removed represented less than 1% by weight). Other properties of these materials are given in Table 1.

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TEST PROCEDURE

Triaxial specimens of Knapton Quarry sand measuring approximately 76 mm × 38 mm in diameter were prepared by tamping dry sand into a mould. Once the specimens were confined in the triaxial cell, the pore air was displaced by the upward flow of de-aired water.

Mixtures containing 5, 10 and 20% bentonite by dry weight were compacted into a Proctor mould by the modified Proctor method at optimum moisture content, and 76 mm × 38 mm diameter specimens were cored. These were saturated in a triaxial cell by a back pressure of 300 kPa, with radial and end drainage. Atkinson *et al.* (1985) have shown that radial drains can cause non-uniformities in clay specimens subjected to either undrained loading followed by consolidation, or rapid drained loading, but as such conditions were avoided, the use of radial drains was considered satisfactory. Specimens were kept under back pressure until the pore pressure parameter B indicated a high degree of saturation (Black & Lee, 1973). Typically this took 2 months.

Mixtures containing 10 and 20% bentonite by dry weight were tested in a standard Wykeham Farrance 38 mm triaxial cell equipped with a 3 kN submersible load cell and an 'Imperial College type' volume change unit. Mixtures containing 5% bentonite by dry weight were tested in a 38 mm Bishop-Wesley 'stress-path' triaxial cell equipped with a 5 kN submersible load cell and a rolling-diaphragm type volume change unit. Sand specimens were tested in both apparatuses.

All specimens were subjected to a consolidated drained triaxial compression test. A back pressure of 300 kPa was used in all tests except CD-05-2 and CD-05-3, where 200 kPa was used, and CD-05-1, which was tested without back pressure so that the specimens could be failed without exceeding the capability of the Bishop-Wesley cell. Mixtures tested in the standard cell were compressed axially at a rate of $1{\cdot}5 \times 10^{-4}$ mm/min, and mixtures in the stress-path cell were tested at variable rates of stress increase to achieve approximately the same compression rate. Typical rates were of the order of 0·02 kPa/min once the axial compression had exceeded 3 mm. The average degree of

Table 1. Properties of Wyoming bentonite and Knapton Quarry sand

Wyoming bentonite	Knapton Quarry sand
Liquid limit = 407% Plastic limit = 48% Specific gravity* = 2·76 Moisture content† ~ 13·5% (as supplied)	Percentage fines = 7% Effective size (D_{10}) = 0·07 mm Specific gravity* = 2·67 Moisture content ~ 4·5% (as supplied) Maximum void ratio = 0·978 Minimum void ratio = 0·451

* Determined by the small-pyknometer method on clay oven-dried at 105°C.
† Measured by oven drying at 105°C.

consolidation at failure was typically 98% (Mollins, 1996). Sand specimens were compressed axially at a rate of approximately 0·3 mm/min. All specimens were compressed until either the peak strength or 20% axial strain were exceeded, and the data were corrected for the membrane and radial drains following the recommendations of Bishop & Henkel (1962).

RESULTS

The angle of repose of Knapton Quarry sand measured by the method recommended by Cornforth (1973) was 34°. Cornforth suggested that this angle is roughly equal to ϕ_{crit} (for quartz sand, $31° \leqslant \phi_{crit} \leqslant 35°$; Bolton, 1986). Table 2 shows the results from drained triaxial compression tests on Knapton Quarry sand. On average, Knapton Quarry sand at an initial void ratio of 0·69 mobilized a ϕ_{peak} of 38·2° at $\varepsilon_a = 7{\cdot}2\%$ and dilated at failure.

Table 3 and Fig. 1 show the results from consolidated drained triaxial tests on 5, 10, and 20% bentonite–sand mixtures. Two different void ratios are reported for each mixture, the clay void ratio e_c (= volume of water/volume of clay particles), which characterizes the state of the bentonite (i.e. reflects the stress required to confine the bentonite), and the sand void ratio e_s (for a saturated mixture, e_s = volume of clay and water/volume of sand), which is a measure of the packing of the sand particles.

The 5% mixtures had an average ϕ_{peak} of 46·7° at an average ε_a of 4·3% and dilated strongly at failure. The average initial sand void ratio e_s of the 5% mixtures was 0·48. The 10% mixtures had an average ϕ_{peak} of 43·5° at an average ε_a of 5·2%. All the 10% mixtures dilated at failure, although there was an overall volume decrease in test CD-10-4, where the initial e_s was 0·67 (the other 10% mixtures had an initial e_s around 0·55). The 20% mixtures had an average ϕ_{peak} of 29·8° at an average ε_a of 13·3%. Specimens CD-20-1 and CD-20-2, with an initial e_s of 0·81 and 0·86, respectively, dilated at failure, whereas specimens CD-20-3 and CD-20-4, with an initial e_s of 1·01 and 1·23, respectively, compressed until they sheared at roughly constant volume.

In summary, the strength of bentonite–sand mixtures decreases with increasing initial e_s, which corresponds to increasing bentonite content. The axial strain to failure increases with increasing

Table 2. Details of the drained triaxial compression tests on Knapton Quarry sand

Test	σ'_3: kPa	e_0	$(\sigma'_1 - \sigma'_3)_{peak}$: kPa	ϕ_{peak}: degrees	ε_a, peak strength: %	ε_v, peak strength: %	$\left(-\frac{d\varepsilon_v}{d\varepsilon_1}\right)_{max}$
CD-S-1*	51	0·74	169	38·6	8·0	−1·6	0·404
CD-S-2†	51	0·68	170	38·7	8·0	−0·9	0·359
CD-S-3†	51	0·68	176	39·3	7·5	−1·9	0·446
CD-S-4*	52	0·74	197	40·9	5·2	−2·0	0·576
CD-S-5†	87	0·66	264	37·1	6·6	−1·3	0·495
CD-S-6*	101	0·73	343	39·0	6·6	−1·9	0·425
CD-S-7*	101	0·71	350	39·4	6·7	−2·0	0·516
CD-S-8†	200	0·66	588	36·5	9·3	−0·5	0·347
CD-S-9†	200	0·65	602	36·9	7·3	−0·8	0·374
CD-S-10†	201	0·66	602	36·8	7·1	−0·9	0·392
CD-S-11†	201	0·67	617	37·3	8·3	−1·0	0·426
CD-S-12*	203	0·72	697	39·2	5·8	−1·1	0·407

* Bishop-Wesley 'stress-path' cell.
† Standard Wykeham Farrance cell.

Table 3. Details of the consolidated drained triaxial compression tests on bentonite–sand mixtures

Test	Bentonite content: % by dry weight	σ_3': kPa	e_s	e_c	$(\sigma_1' - \sigma_3')_{peak}$: kPa	ϕ_{peak}: degrees	ε_a, peak strength: %	ε_v, peak strength: %	$\left(-\frac{d\varepsilon_v}{d\varepsilon_1}\right)_{max}$
CD-05-1*	5	187	0·49	8·35	983	46·4	4·2	−1·9	1·17
CD-05-2*	5	103	0·48	8·16	473	44·2	4·0	−1·8	0·90
CD-05-3*	5	103	0·50	8·46	558	46·9	4·0	−2·5	0·94
CD-05-4*	5	51	0·45	7·61	253	45·5	5·4	−3·0	1·07
CD-05-5*	5	25	0·48	8·06	156	49·2	4·6	−3·1	1·22
CD-05-6*	5	25	0·46	7·65	143	47·8	3·8	−2·6	1·24
CD-10-1†	10	197	0·52	3·68	807	42·2	4·7	−0·8	0·71
CD-10-2†	10	98	0·57	4·11	460	44·5	2·3	−0·9	0·90
CD-10-3†	10	50	0·55	3·96	269	46·8	4·8	−2·9	1·02
CD-10-4†	10	24	0·67	5·00	89	40·5	8·8	1·9	0·49
CD-20-1†	20	195	0·81	2·22	373	29·3	8·8	−4·3	0·54
CD-20-2†	20	99	0·86	2·44	208	30·8	10·1	−3·0	0·42
CD-20-3†	20	47	1·01	3·03	80	27·4	19·0	2·9	−0·06
CD-20-4†	20	21	1·23	3·90	46	31·5	15·4	3·8	0·01

* Bishop-Wesley 'stress-path' cell.
† Standard Wykeham Farrance cell.

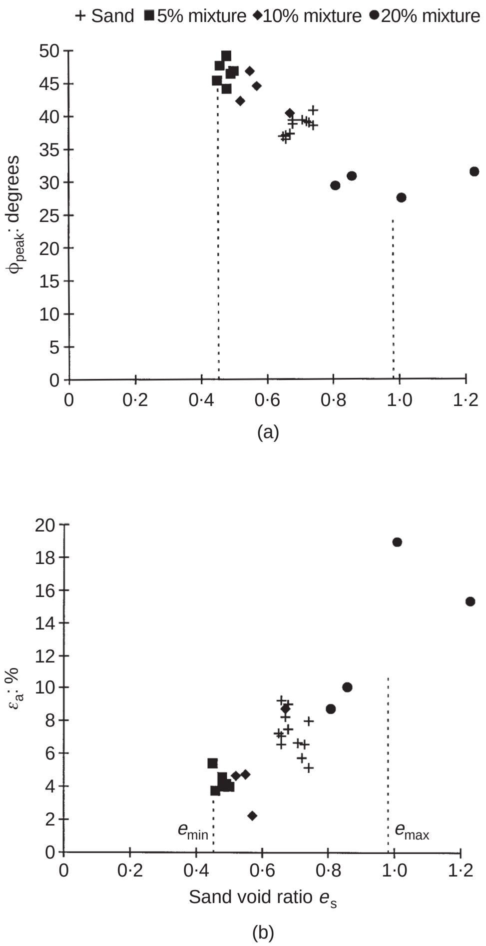

Fig. 1. (a) Peak angle of shearing resistance and (b) axial strain to failure against void ratio of the sand for Knapton Quarry sand and Wyoming bentonite/Knapton Quarry sand mixtures

initial e_s. These trends are illustrated by Fig. 1. The drained strength of Knapton Quarry sand prepared at an initial void ratio of about 0·69 was intermediate between the drained strengths of the 10% mixtures (average e_s 0·58) and the 20% mixtures (average e_s 0·98).

DISCUSSION

The stress–dilatancy relationship of Rowe (1962) relates the maximum effective stress ratio supported by an assembly of irregular rigid particles in contact to the friction between the particles and the rate of dilatancy. When written in terms of ϕ_{peak}, this equation is

$$\tan^2(45^\circ + \phi_{peak}/2) = \tan^2(45^\circ + \phi_f/2) \times \left(1 - \frac{d\varepsilon_v}{d\varepsilon_1}\right)_{max} \qquad (1)$$

where ϕ_f depends on the mechanism of energy dissipation, that is, either particle sliding associated with the average angle of shearing resistance between particle surfaces ϕ_μ or turbulent shear associated with the critical-state angle of shearing resistance ϕ_{crit} (Rowe, 1972). Substitution of the drained triaxial compression data for Knapton Quarry sand (Table 2) into equation (1) yields $\phi_f = 30{\cdot}0^\circ$. Data presented by Rowe (1962) suggest that $\phi_\mu \approx 29^\circ$ for fine quartz sand shearing across a quartz block, whereas for Knapton Quarry sand, $\phi_{crit} \approx 34^\circ$, so a value of $\phi_f = 30{\cdot}0^\circ$ indicates a predominantly sliding mode of shearing at peak strength. Such behaviour is characteristic of a medium dense sand (Rowe, 1972).

Figure 2 shows the variation in peak strength of the bentonite–sand mixtures with the maximum rate of dilation. Also shown in Fig. 2 are the sand data and a line representing equation (1) with $\phi_f = 30{\cdot}0^\circ$. The data for mixtures containing 5 and 10% bentonite fall close to the line representing Rowe's stress–dilatancy relationship, suggesting that it is the interaction of the sand particles in these mixtures that controls the behaviour (i.e. minimal work is done on, or released by, the clay during shearing). Rowe's stress–dilatancy theory cannot be used predictively, as it does not specify the relationship between the peak rate of dilation and the initial state of a soil. Bolton (1986) reviewed extensive data on the strength and dilatancy of numerous types of sand at a range of densities and confining stresses. Bolton reported that the rate of dilation of sand during shearing is a function of relative density and mean effective stress, and proposed a relative dilatancy index I_R that correlates with the dilatant component of strength:

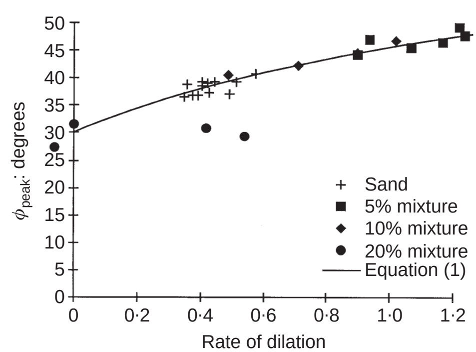

Fig. 2. Peak angle of shearing resistance against maximum rate of dilation for Wyoming bentonite/Knapton Quarry sand mixtures

$$I_R = I_D(10 - \ln p') - 1 \tag{2}$$

where I_D is the relative density ($I_D = (e_{max} - e)/(e_{max} - e_{min})$) and p' is the mean effective normal stress in kN/m^2.

Bolton proposed a correlation between I_R and the peak rate of dilation,

$$\left(-\frac{d\varepsilon_v}{d\varepsilon_1}\right)_{max} = 0{\cdot}3 I_R{}^\circ \tag{3}$$

and another between I_R and the dilatant component of strength,

$$\phi_{max} - \phi_{crit} = 3 I_R{}^\circ \tag{4}$$

which were validated in the range $0 < I_R < 4$ for a wide range of sands. Equations (2) and (3) predict that a sand will dilate when $I_D \geqslant (10 - \ln p')^{-1}$.

Figure 3 shows the maximum rates of dilation and ϕ_{peak} of the sand and the 5 and 10% mixtures plotted against the value of I_R based on $(I_D)_s$, the relative density calculated using e_s. Figs 3(a) and 3(b) show clear trends of increasing dilation and increasing ϕ_{peak}, respectively, with increasing I_R. Lines representing equations (3) and (4) (assuming $\phi_{crit} = 34°$) are shown on Figs 3(a) and 3(b), respectively, and are good fits to the experimental data.

The mixtures containing 20% bentonite all gave a ϕ_{max} of about 30°, and therefore the data from those tests where there was significant dilation at failure (CD-20-1 and CD-20-2) fall significantly below the line representing equation (1) in Fig. 2. The application of equation (1) to a bentonite–sand mixture implicitly assumes that the net work done by the external forces is dissipated as friction between the sand particles. The initial clay void ratios of specimens CD-20-1 and CD-20-2 were lower than in the other tests (see Table 3) so the effective stress confining the bentonite will have been significantly larger, and therefore ignoring the frictional work dissipated by the clay will be inappropriate for these tests. As the effective stress required to confine the clay at a particular void ratio is heavily dependent on stress history, no attempt has been made to correct the data for this effect.

The two 20% bentonite mixtures confined by the lowest effective cell pressure (CD-20-3 and CD-20-4) swelled significantly during 'consolidation' in the triaxial cell to reach initial sand void ratios of 1·01 and 1·23 (i.e. higher than the maximum void ratio for the sand alone). These specimens had maximum angles of shearing resistance of 27·4° and 31·5°, respectively. Kenney (1977) measured the direct shear strength of clay–sand mixtures and observed that there was a transition in the ultimate angle of shearing resistance of the mixture ϕ from the critical-state angle of the sand $(\phi_{crit})_s$ at a sand void ratio of around unity, to the residual angle of the clay $(\phi_r)_c$, at large sand void ratios. Kenney used the relative residual strength R_ϕ to describe this transition, where

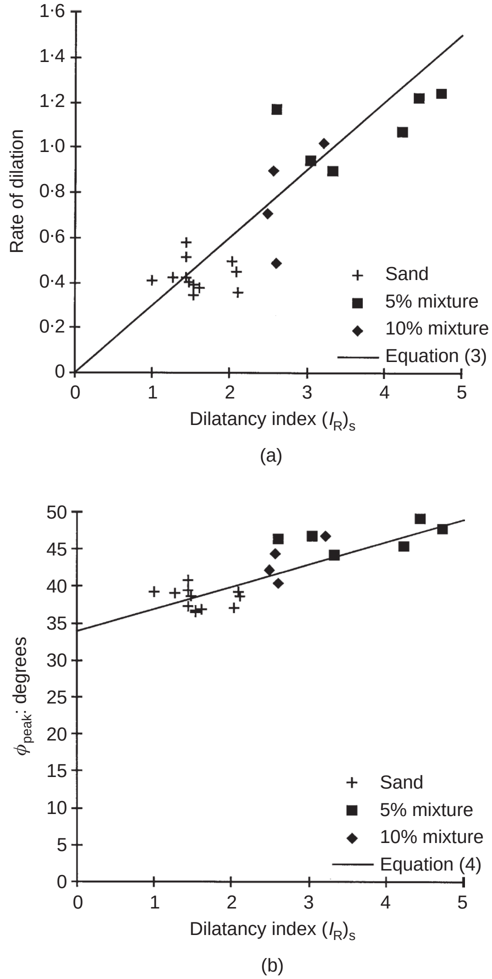

Fig. 3. Variation of (a) maximum rate of dilation and (b) peak strength with dilatancy index for the sand

$$R_\phi = \frac{\tan\phi - \tan(\phi_r)_c}{\tan(\phi_{crit})_s - \tan(\phi_r)_c}$$

Lupini *et al.* (1981) presented data for bentonite–sand mixtures where R_ϕ varied from unity at a sand void ratio of about 0·9 to zero at a sand void ratio of about 4. They also reported that when R_ϕ was greater than about 0·7, the mode of shearing was turbulent (i.e. the mode of shearing exhibited by sand at the critical state). Mixtures CD-20-3 and CD-20-4 had residual strength ratios of 0·74 and 0·90 (based on $(\phi_r)_c = 4°$) and would seem to have been in a state of turbulent shear, as suggested by Lupini *et al.*, and at, or near, a critical state.

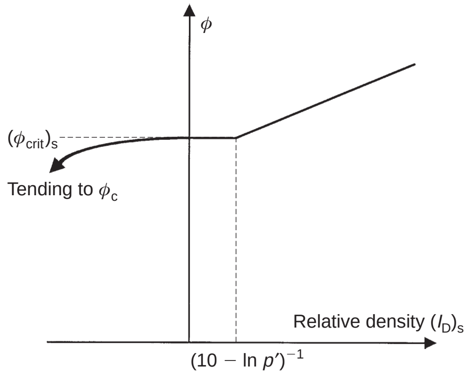

Fig. 4. Schematic graph showing the variation in the drained strength ϕ of bentonite–sand mixtures with the initial relative density of the sand in the mixture $(I_D)_s$

In summary, the drained strength of bentonite–sand mixtures can be idealized by three modes of behaviour, as shown in Fig. 4, over the range of sand relative densities that may be encountered in landfill liners and environmental barriers. Above a sand relative density of $(10 - \ln p')^{-1}$, bentonite–sand mixtures will exhibit a peak strength and dilate at failure in a manner indistinguishable from sand alone, except when the bentonite in the mixture supports an appreciable proportion of the stress. The strength of such mixtures can be estimated using the dilatancy index proposed by Bolton (1986). When $0 \leqslant (I_D)_s \leqslant (10 - \ln p')^{-1}$, the strength of a bentonite–sand mixture will be equal to the critical state strength of the sand. As $(I_D)_s$ reduces below zero (i.e. the sand void ratio exceeds the maximum for sand alone), the strength of the mixture reduces from the critical-state strength of the sand alone towards the residual strength of the clay.

CONCLUSIONS

The ultimate shear strength of a bentonite–sand mixture depends primarily on the relative density and critical-state friction angle of the sand within the mixture. Over the range of bentonite contents used for environmental barriers, the bentonite has very little influence on drained strength unless it supports a significant proportion of the stress (indicated by a low clay void ratio). However, at very high bentonite contents (and consequently high sand void ratios) the strength of the mixture will tend towards the strength of the clay.

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