Drained Shear Strength of Lime-Clay Mixes

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Lime-stabilized clay is being used for novel applications such as slope stabilization and appropriate proof tests are required. The testing of lime-stabilized clay is described in the context of different site applications and recommendations are made. Two British clays of different mineralogies have been tested in unconfined compression and in quick-undrained and slow-drained triaxial tests to examine the difference in both strength and stiffness. The results clearly demonstrate that lime-stabilized clay is a remarkably frictional material. Significant differences were found between the drained and undrained triaxial tests, whereas stiffnesses were approximately similar. In particular the frictional component of strength is shown to be more dominant in drained tests. Unconfined compression tests, which are unable to take account of frictional behavior, are shown to be of value only as index tests to establish whether lime treatment is feasible. The detailed results show that the clays can be treated with relatively low lime contents for use as bulk fill in drained applications. Atterberg Limits are reported for lime-clay mixes between 0 and 56 days after compaction and implications for reaction progress are discussed. The authors conclude that failure envelopes obtained for simulated site conditions should be used for design purposes and that other aspects, such as strain compatibility, should be considered.

Lime has been used extensively to improve the properties of clay soils worldwide, the first records of the process dating back to Roman times. The lime reacts with water contained within the clay, thereby releasing calcium cations (Ca^{2+}) and hydroxyl anions (OH^-) into solution. The calcium-saturated solution surrounding the clay mineral particles results in cation substitution and particle flocculation and agglomeration, thereby modifying the clay. Under conditions of a high pH, slower, longer-term stabilization reactions occur, resulting in gel formation and subsequent crystallization. Modification of the clay changes its nature, from essentially cohesive to cohesionless, and stabilization results in a brittle, cemented material being progressively formed. The rate and extent of property change vary with many factors, including temperature, water content, lime content, and, most important, clay mineralogy. All of this is well known to the engineering community, and yet problems still appear to occur in the interpretation of data from the testing of lime-clay mixes. The latter problem is addressed in this paper.

It is clear from experience of the technique that the properties of a lime-stabilized clay are very different from those of the original clay. For example, there are changes (improvements) in plasticity, strength, stiffness, and volume stability. It is therefore natural that engineers should adopt lime stabilization for ground improvement in preference to the direct and indirect environmental costs of importing crushed rock or other granular fill. The most widespread application must be for road subgrade stabilization, although increasing use is being made of lime stabilization for bulk fill operations, embankments, and cutting slope repair and as a bearing stratum for lightly loaded foundations. This is in addition to the more novel techniques such as lime slurry pressure injection, lime columns and lime piles for ground improvement (*1*).

Recent experience in the United Kingdom (UK) has included many successful applications of the technique together with a few, highly publicized failures. One of these, at Saxmundham in Suffolk, concerned a case in which compaction took place to the dry of optimum water content. Thus, although the dry density was within specification because of the considerably flattened compaction curve, the void content was relatively high. The compacted material was left during the winter, water penetrated via the voids, and freezing ruined the coherence of the stratum. This occurred as a result of a fundamental misunderstanding of the material characteristics and, perhaps, a reluctance to add water at compaction to a clay material. The point here is that the material is no longer a clay when mixed with lime and if the same engineer were to be asked to add water to a granular material to reach the optimum water content, no such reluctance would exist. The authors believe that the same symptoms are prevalent in the approach to lime-clay testing.

The most notable UK failure occurred on the M40 Birmingham-to-Oxford motorway, but this is believed to have concerned sulphate concentrations in the natural soil. The study of clay mineralogy, clay chemistry, and the lime-clay reaction processes is not yet fully complete and lies without the scope of this paper. Guidelines have been adopted in the UK to avoid such problems in the future. Perhaps the most important change, however, is that lime stabilization is being considered from the start of the planning and design process, that is at the site investigation stage, instead of being introduced as an alternative at the tender stage only. This is both encouraging and important, and yet is potentially only of full value if appropriate laboratory testing is carried out as part of the investigation.

It has often been stated that lime-stabilized clay can achieve considerable strength (*2*). However, it is often the case that these very high strengths are not required in practice and engineers are not designing lime-clay mixes to create the properties that they require for the particular purpose. For example, clay modification alone is being increasingly specified. It is apparent, however, that the testing of lime-clay mixes has not progressed in the same way and standard tests, producing little more than index values, are still being specified. Lime-clay testing should run parallel to the design process and attempts to recreate applied inherent stress conditions and site loadings should be made. Presented in this paper are data from three standard tests to illustrate the need for attention to this point.

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TESTING OF LIME-STABILIZED CLAY

Designers necessarily assume material properties for their designs and specify minimum properties for materials to be used on site. Testing of those materials, whether in the laboratory or in situ, is carried out for proof of purpose. Testing should necessarily simulate site conditions, whether of stress or strain, if proof of purpose is to be achieved with efficiency. It should only be necessary to fall back on index values if it proves impossible or impractical to simulate site conditions.

A good example is the case of a lime-stabilized clay road foundation. The foundation has several purposes, which include the need to

1. Provide a suitably stiff platform on which to compact the overlying layers;
2. Act as a haul road during the construction operation (i.e., to carry construction traffic); and
3. Support the stresses transmitted by the overlying layers when the pavement is in use.

The second purpose involves direct trafficking by—albeit relatively few passes of—heavy vehicles. The prime functions of the material itself are to resist permanent deformation (rutting) and to spread the load sufficiently well so that the underlying subgrade does not become overstressed. For this the material must be both sufficiently strong (most important) and stiff, respectively. For the third purpose the same requisite characteristics apply, although resilient stiffness under repeated load applications has most importance. It is apparent, therefore, that the test required here is the repeated load triaxial test: pace, the hollow cylinder test with principal stress rotation. The sample should be confined to match the ambient stress conditions and should be subjected to both relatively few (say 50 to 100) applications of a high deviator stress and a large number (millions) of applications of a relatively low deviator stress. The rate of application, in terms of frequency and duration, of these stress pulses should similarly be representative of site conditions within reasonable limits of practicality. Instead an index test is used, in the form of the California Bearing Ratio (CBR), and correlations are attempted among the requisite parameters and the CBR. The inaccuracy and inefficiency of this approach is amply illustrated by Brown et al (*3*).

For the bulk fill applications, such as slope repair and bearing strata for light foundations, the loading regime is altogether different. In these cases the load application is typically extremely slow and very slow shearing tests should be conducted.

At this point, therefore, it would be useful to examine what is meant by strength. Traditional soil mechanics defines the Mohr-Coulomb envelope as the boundary in shear stress-normal (total or effective) stress space above which the soil will fail in shear. In the case of testing under effective stress, no pore water pressures are reflected in the data, the samples under test being tested very slowly in a fully drained manner. The resulting effective stress envelope, defined by effective cohesion (C') and effective angle of internal friction (ϕ'), would thus apply to the bulk fill applications previously mentioned. A second particular type of test, the quick undrained triaxial test, is used for saturated clays to produce a total stress plot in which porewater pressures are permitted to influence the results. Interpretation of undrained tests is based on the assumption that a saturated clay has a small, often negligible, undrained angle of internal friction (ϕu) and thus a mean horizontal line can be drawn parallel with the axis of normal stress to create an intercept on the shear stress axis of undrained cohesion (Cu). A third variation is the unconfined compression strength (UCS) test, which uses the arguments behind the quick undrained triaxial test to dispense with confining pressure altogether, the mean maximum shear stress in the test representing Cu.

Strength measurement for lime-stabilized clay is often quoted in terms of UCS or Cu, and yet a lime-stabilized clay in practice is generally not saturated and does not behave like a clay. These results must surely, therefore, represent index values of strength under a certain set of arbitrary conditions. If the strength characteristics of a lime-clay mix vary with the rate of testing, as will be demonstrated hereafter, then the rate chosen for testing should match that in practice. In addition a lime-clay mix, which is widely known to have a particulate nature and will be shown to have a significant frictional component of strength, will necessarily be subject to some degree of confinement because of its placement and compaction in addition to site confinement. Thus the argument for selecting UCS for design is invalid, and indeed can result in an overestimation of strength at relatively low confining stresses, most notably in the fully drained (long-term) case. Thus considerable care should be exercised in the interpretation of data from such tests.

RESEARCH PROGRAM

Research Philosophy

The three test techniques already described were applied to two British clays of different characters and mineralogies. Lower Lias clay, which outcrops in the midlands of England, is an illitic silty clay having a liquid limit (LL) of 59 percent and a plastic limit (PL) of 29 percent. The second clay is a refined kaolinite, known as English china clay, derived from the weathering of granite in the south-west of England and has an LL of 60 percent and a PL of 33 percent.

The primary aim of these tests was to consider the behavior of the lime-clay mixes around the lime fixation point, with the aim of assessing their performance in a drained, bulk-fill, possibly slope-stabilization application. Accordingly a relatively high water content was chosen for the initial clay so that, when mixed with lime, the water content would remain slightly above the optimum water content at compaction. Immediate strength gain was not considered important although the development of strength with time was. A further aspect of the results that was of interest was the change in stiffness and brittleness of the material, because differential stiffness in a slope has implications for progressive failure mechanisms. In addition, it has been reported that the strain at failure of the stabilization products (calcium silicate hydrate and calcium aluminate hydrate, CSH and CAH respectively) is approximately 1 to 2 percent (*2*,*4*).

Experimental Procedure

Both the lower Lias and English china clay were air dried and mixed with water to achieve a liquidity index of 0.1. This was found from previous research (*4*) to be the best method for correlation among different soils, instead of using arbitrary moisture

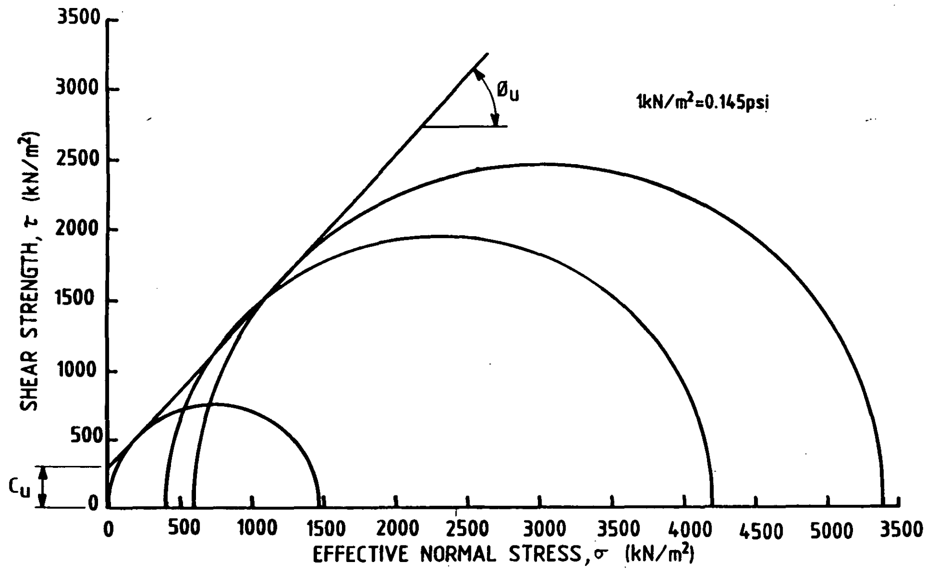

FIGURE 1 Mohr-Coulomb plot for an undrained test on china clay with 5 percent Quicklime at 28 days.

contents that could lead to misinterpretion. Three lime contents were used for the tests: the Eades and Grim value (*5*), and 2 percent above and 2 percent below this value for each clay. Once again this facilitated a true comparison among the clays. The lime-clay batches were mixed in a large pan mixer for 12 min, a time that was found from preliminary trials to ensure intimate mixing. The lime-clay mixes were then stored for 48 hr in heavy-duty polythene bags at 21±2°C to allow for mellowing, for example completion of the modification reaction. This was characterized by changes in Atterberg Limits (measured at 0 and 8 hours, and 1, 2, 7, 28, and 56 days) followed by stability at a reduced plasticity. Once mellowed the modified material was compacted into 100-mm (4-in.)-diameter tubes of 500-mm (20-in.) length in accordance with the British Standard heavy-duty compaction test (*6*). Thirty-eight-mm-diam specimens 76 mm in length were sampled immediately after compaction to avoid damage to the rapidly strengthening samples.

Four sets of 27 samples were prepared (one set of each clay type for drained and undrained testing), again sealed individually, further sealed in groups of three, and stored at 21 ± 2°C. The samples were tested in standard triaxial test machines in accordance with British Standards Institution Report BS1377: Part 8 (*7*) at 7, 28, and 56 days. The undrained tests used a shearing rate of 1.000 mm/min (0.0400 in./min) and the drained tests 0.009 mm/min (0.00035 in./min). At each stage of testing three undrained and three drained specimens were tested at confining pressures of 0, 400 and 600 kN/m^2 (0, 58, and 87 psi), one undrained sample thus being tested under unconfined compressive stress conditions. In addition, untreated clay samples were dried, mixed with water, and stored in the same manner, and were tested at cell pressures of 0, 100, and 200 kN/m^2 (0, 14.5, and 29 psi) after 28 days. Additional samples were prepared for each clay and lime content for replicate testing to both confirm experimental data trends and the veracity of unexpected results. The large confining stress range for the lime-treated clay was chosen to facilitate the production of accurate best-fit lines on the Mohr-Coulomb plots (see Figure 1 for a treated clay in comparison with Figure 2 for an untreated clay). From the test data drained and undrained shear strength parameters were obtained, secant moduli calculated, failure strains recorded, and unconfined compressive strength values calculated.

EXPERIMENTAL RESULTS

General Observations

The test program proved successful and although some (expected) scatter of the data occurred, the trends in the results followed expected patterns (*2,4*) and were reasonably well defined. Supplementary data, such as Atterberg Limits, not reported in detail herein, confirmed that the clays were modified as expected. Good compaction of the lime-clay samples was achieved after mellowing, as evidenced by the measured densities (which were consistent with the water content measurements of the specimens given in Table 1), the shear strength results and the appearance of the samples. The Atterberg Limits for the china clay LL = 60 percent, PL = 33 percent showed a rise in LL and PL of 11 to 12 percent and 4 to 8 percent, respectively, after 48 hr. The LL reduced and PL increased thereafter causing a reduction in Plasticity Index (PI) by 5 percent to +4 percent in relation to the PI of the untreated clay for 1 percent lime, and by 9 percent to −4 percent and 18 percent to −13 percent for 3 percent and 5 percent lime, respectively. A similar trend was noticed for Lias Clay (LL = 59 percent, PL = 29 percent) with LL rising by 0 to 13 percent and PL by 12 to 14 percent after modification. The PI reduced by 11 percent to −12 percent, 5 percent to −17 percent, and 10 percent to −23 percent for 4 percent, 6 percent, and 8 percent lime respectively. The approximate equivalence of 5 percent lime addition to china clay and 4 percent to Lias clay was demonstrated.

The Eades and Grim test (*5*) demonstrated that the nearly pure kaolinite required 3 percent lime for fixation, whereas the Lias

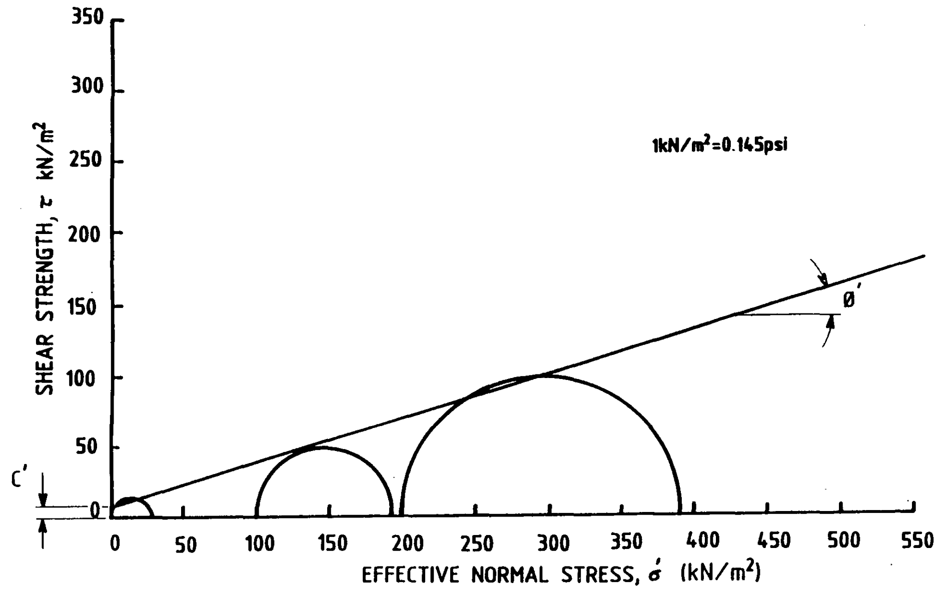

FIGURE 2 Mohr-Coulomb plot for a drained test on untreated china clay at 28 days.

clay required 6 percent lime. However, the undrained and drained shear strengths, shown in Table 2 and discussed in subsequent sections, show that the higher lime contents used for the Lias clay resulted in considerably higher strength gains. There are several factors that influence the magnitude and rate of strength gain, the most important being clay mineralogy, and these undoubtedly account for the observations. Nevertheless, it is important to note that the strengths achieved when mixing lime with clay at the fixation point are likely to be considerably different because some engineers unfamiliar with the process have been misled in this respect.

A further general point to note is that the lime-clay mix is particulate before compaction and that cementation occurs within the flocculated, agglomerated structure. Although cementation tends to produce a coherent structure, the use of 38-mm-diameter samples, out of practicality, will almost certainly result in some variation in strength measurements, depending on the location of the shear plane in relation to the flocs. This will, in part, explain some of the scatter noticed in the data.

Undrained Shear Strength

Lime stabilisation improves both the (apparent) cohesive (Cu) and frictional (ϕu) components of the undrained shear strength, as shown in Table 2. Only small amounts of lime are required to radically improve the strength, whereas increasing lime content and time tend to improve the parameters further. It should be noted that the strength derives from a combination of Cu and ϕu, and thus the two values should be considered together. The impossibility of precise determination of Cu and ϕu, especially when high strengths have been reached, should not be forgotten in subsequent interpretation. Indeed it is clear from the Mohr-Coulomb plots that even the relatively high confining pressures used in this study were inadequate for accurate determination of Cu and ϕu and that values of 0, 2000, and 4000 kPa (or, say, 0, 300, and 600 psi) or more should be used for the high lime contents. From inspection of the Mohr-Coulomb plots (e.g., Figure 1) it is noticed that the angle of friction (ϕ) will be more reliable than the cohesion intercept (*C* perhaps better representing "cementation"

TABLE 1 Moisture Contents of Lime-Clay Mixes After Compaction (%)

Soil Type	Lime Content (%)	Time (days) 0	0.33	1	2	7	28	56
China Clay	0						33	
	1	37	37	37	36	39	39	39
	3	37	37	37	38	38	37	36
	5	40	38	39	38	35	35	35
Lower Lias Clay	0						32	
	4	32	32	32	32	31	31	31
	6	24	23	25	26	26	29	29
	8	26	26	26	26	26	28	28

than "cohesion") because slight changes in ϕ will cause radical changes in C.

The results show that in general Cu increases with lime content, Cu increasing typically twofold for the 4 percent increase in lime content from below to above the Eades and Grim value. Cu also increases with time, the difference between 7 and 56 days being typically threefold for lower Lias clay and somewhat less on average for china clay. Any variations from this trend can be explained by imprecision in data interpretation. The changes within 7 days are attributed primarily to modification and compaction, whereas at 56 days the changes reflect also the development of the pozzolanic (stabilization) reactions. ϕu has also increased substantially above that for untreated clay in the first 7 days for both clays. In the case of English china clay, ϕu has reached a value of a good quality granular material after 7 days even with a lime content of only 1 percent. This trend, confirmed by other recent UK work, is attributed to the fact that the clay has been modified. The trend of increasing ϕu with time and lime content (Figure 3) is perfect for the china clay with the exception of 5 percent lime at 56 days, when cementation (Cu) increased dramatically. The conclusion here would appear to be that ϕu increases as stabilization proceeds (i.e., the Mohr-Coulomb envelope rotates with time, and rises with lime content). Once the stabilization reactions are largely complete, however, this trend appears to break down as very large Cu values are accompanied by lower (though still high in absolute terms) ϕu. These findings have been confirmed by other work at Loughborough, but warrant further work to examine their universality (especially between clay minerals).

For lower Lias clay with its significantly higher lime contents, ϕu is high, even for a granular material, after 7 days implying exaggerated dilational behaviour. At 28 days ϕu falls in all cases, although Cu rises sharply and at 56 days ϕu starts to rise again, although such fine distinctions cannot be concentrated on for the reasons already mentioned. Taking the results together, it can be concluded that ϕu rises as the stabilization reactions progress, but when significant cementation is complete, a moderate reduction in ϕu is accompanied by high (greater than 1000 kPa or 145 psi) Cu. Thus only the 5 percent, 56-day English china clay sample has reached that point, whereas only the Lias clay samples at 4 percent and 6 percent lime at 7 days have not reached that point. These conclusions have also been reached by Lees et al. (*8*) in their work on kaolinite and montmorillonite.

Drained Shear Strength

Similar increases in C' and ϕ' occurred in drained tests (Table 2), with the largely consistent trends already explained. With one exception, ϕ' is higher in the drained condition than in the undrained condition for both treated and untreated soil. For untreated soil this occurs because the drained test measures essentially only frictional behavior. That the same observation is true of treated soil implies that pore water, or fluid, pressure generation occurs in the undrained tests on these materials. Drained shear test data should thus be used for slope or bearing applications.

Unconfined Compressive Strength

With only a few exceptions, UCS increases with time and lime content (Table 2). The UCS for china clay at 5 percent lime content and 56 days, and for all lime-treated samples of lower Lias clay, is high (>2200 kPa or 139 psi), whereas otherwise is less than 1010 kPa (146 psi), adding evidence to the conclusion that only in these samples have the stabilization reactions been substantially completed. Noting that UCS should be halved for comparison with Cu, and by extension with C', it is apparent that overestimation of Cu is only significant when significant stabilization has occurred (say UCS>2000 kN/m^2 or 290 psi), whereas considerable overestimation of C' occurs in all cases.

Therefore, using UCS in the design process would underestimate the strength at high confining pressures, because the frictional component of strength is de facto ignored in any interpretation using UCS, providing a large factor of safety. This would be uneconomic—hence poor engineering—but not dangerous. However, at low confining pressures, UCS would overestimate the

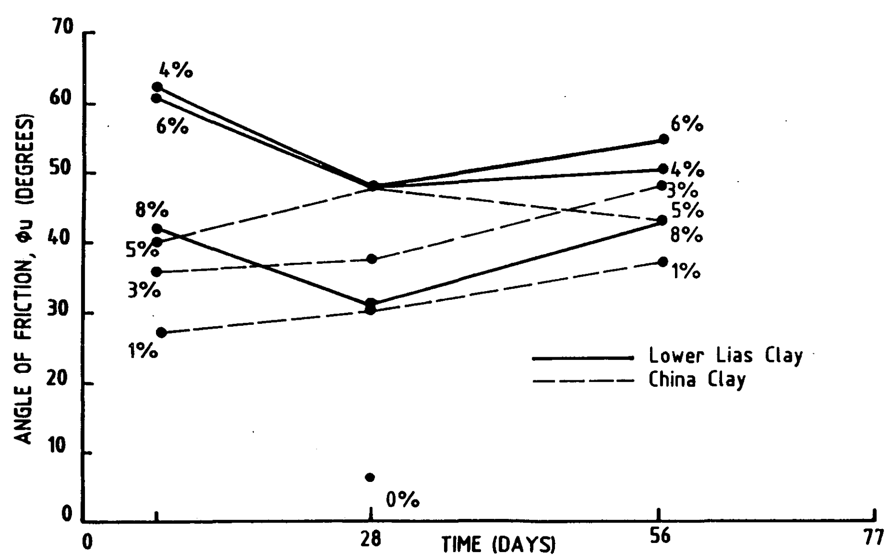

FIGURE 3 Undrained angle of friction versus time for lower Lias clay and china clay.

strength and this could be unsafe. When the loading is not applied rapidly, these problems are exacerbated and unsafe designs could easily result.

Elastic Modulus

Lime stabilization causes considerable improvement in the elastic modulus, changing a soft, plastic material into a stiff, brittle material. This change occurs with both time and lime content (Table 3, Figure 4), and is approximately the same, although with some scatter, for the drained and undrained tests. Comparison with untreated soil leads to the conclusion that lime stabilization is very effective when used to improve, for example, subgrade properties, albeit that repeated load triaxial tests at appropriate stress levels and application rates should be conducted.

The elastic modulus was much higher with lower Lias than English china clay and certainly far higher than could be attributed to any variation in the properties of the remolded clays, as evidenced by the data for the untreated clay. This suggests that either the pozzolanic reaction was more advanced in the lower Lias clay, or that the flocs and their bonds were stiffer. Data presented earlier confirm the former explanation. A typical set of curves, in this case for the undrained tests on lower Lias clay with 4 percent quicklime addition, illustrated the increase in both strength and stiffness with time for the unconfined compression test data (Figure 5). It is clear that the failure strains have reduced from approximately 4 percent to 2 percent between 7 and 28 days. It is equally evident that the brittle behavior of a stabilized soil (i.e., one in which stabilization reactions are substantially complete), characterized by failure strains of 1 to 2 percent and a rapid fall off in strength post peak, is exhibited by the two later curves. A further point, once more typically found from the experimental data, is that when high confining pressures are applied, the stiffness remains approximately constant (evidenced by the gradient of the stress strain curve ignoring zero drift), and yet the strain at failure increases from approximately 2 percent to 4 to 6 percent or more (again ignoring zero drift). This was consistently evident throughout the test program and thus accounts, at least in part, for the increase in strength at higher confining stresses (i.e., frictional

TABLE 2 Strength of Lime-Clay Mixes

Soil Type	Lime Content %	7 days	28 days	56 days
Undrained Shear Strength Cu, ϕu (kN/m^2, degrees)				
China Clay	0	-	45,6°	-
	1	230,27°	170,30°	270,37°
	3	120,36°	360,38°	410,46°
	5	475,40°	300,48°	1000,42°
Lower Lias Clay	0	-	41,7°	-
	4	720,62°	1860,48°	2450,50°
	6	600,61°	1800,48°	2300,54°
	8	1670,43°	3130,31°	5100,42°
Drained Strength C´, ϕ´ (kN/m^2 degrees)				
China Clay	0	-	7,16°	-
	1	125,48°	140,51°	200,50°
	3	125,52°	120,64°	240,60°
	5	275,60°	300,61°	310,63°
Lower Lias Clay	0	-	7,18°	-
	4	1025,43°	950,55°	1500,58°
	6	1175,56°	1225,57°	1400,62°
	8	1450,43°	1800,55°	3500,48°
Unconfined Compressive Strength (kN/m^2)				
China Clay	0	-	82	-
	1	317	269	526
	3	225	750	957
	5	1009	725	2245
Lower Lias Clay	0	-	83	-
	4	3132	4853	6601
	6	2330	5050	7449
	8	3803	5613	6584

TABLE 3 Secant Elastic Modulus (N/mm²)

Soil Type	Lime Content %	Time (days) Drained 7	28	56	Undrained 7	28	56
China Clay	0	-	1.45	-	-	1.51	-
	1	67	123	52	78	50	182
	3	95	82	106	46	68	225
	5	204	214	248	80	223	251
Lower Lias Clay	0	-	3.18	-	-	1.48	-
	4	300	339	612	266	455	624
	6	276	332	549	330	521	627
	8	422	566	794	356	566	720

1 N/mm² = 145 psi

behavior as denoted by ϕu or ϕ') because of the need for significant dilation of the sample before shearing being possible. This finding has considerable relevance for geotechnical design because strain compatibility between stabilized and untreated soils will need to be designed.

One final point is that a great improvement is seen in the English china clay with 1 percent lime. This must be caused by flocculation and agglomeration (i.e., modification) as very little, if any, of the lime contributes to pozzolanic reaction (*9*). These and earlier observations demonstrate that a substantial amount of the strength and stiffness improvements associated with lime are caused by modification alone.

Failure Strains

Many authors have stated that the ultimate strain of crystalline CSH and CAH lies between 1 and 2 percent (*2,4,9*), and nearing these failure strains indicates completion of the pozzolanic reaction within a stabilized soil. The smallest recorded average failure strains in the tests, shown in Table 4, are approximately 3 percent, apparently indicating that, although substantial cementing has occurred, further reaction is necessary before cementation is complete. However, when consideration is given to the detailed results illustrated in Figure 5, presented in Table 4, and discussed in the previous section, it is clear that full (or at least substantial) stabilization has occurred in many of the samples. The data for no confinement in the undrained tests indicate that all of the Lias clay samples excepting the two lower lime contents at 7 days fail at or below 3 percent axial strain. Similarly the china clay samples with 3 and 5 percent lime contents at 56 days have low undrained failure strains. This trend would be completely defined if it were not for the low failure strains for the china clay with 1 percent lime at 28 and 56 days, the reasons for which are not clear. The higher average strains thus clearly result from larger strains at higher normal effective stresses, and this is clearly borne out by the results in Table 4. These effects are exaggerated in the case of drained shear testing (Table 4), in which the failure strains are

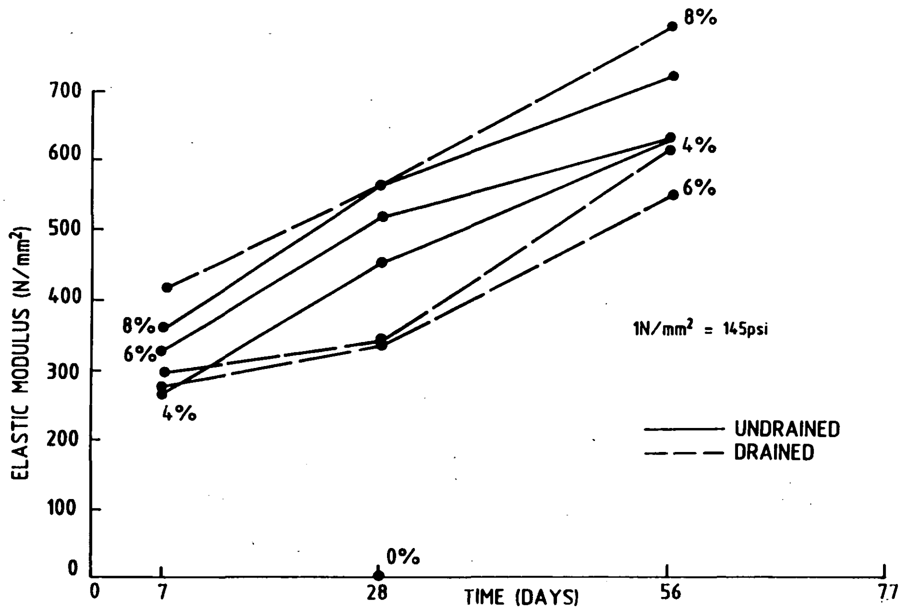

FIGURE 4 Secant elastic modulus versus time for lower Lias clay.

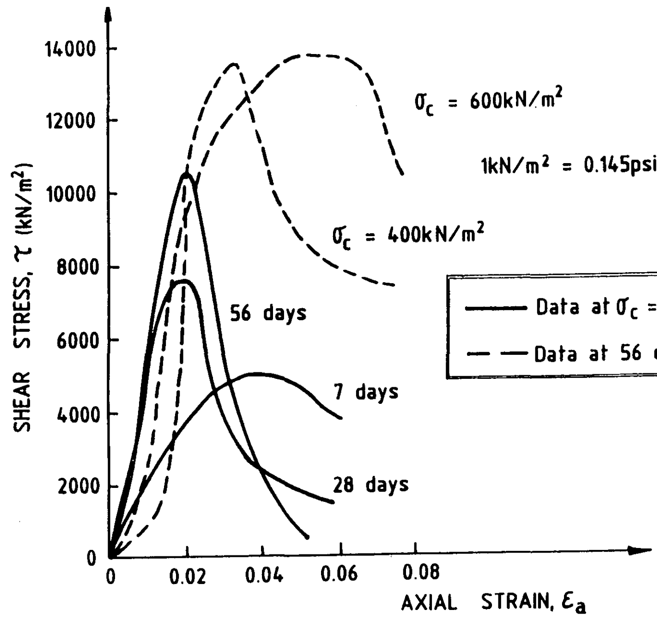

FIGURE 5 Graph of shear stress against axial strain for undrained triaxial tests on lower Lias clay with 4 percent lime.

TABLE 4 Strains at Failure

Soil Type	Lime Content %	7 days $\sigma_c = 0$	7 days 400	7 days 600	7 days av	28 days $\sigma_c = 0$	28 days 400	28 days 600	28 days av	56 days $\sigma_c = 0$	56 days 400	56 days 600	56 days av
Strains at Failure in Undrained Tests													
China Clay	0*					0.110	0.133	0.158	0.134				
	1	0.190	0.200	0.200	0.197	0.020	0.038	0.054	0.037	0.013	0.027	0.052	0.031
	3	0.073	0.079	0.093	0.082	0.042	0.104	0.092	0.079	0.016	0.072	0.072	0.053
	5	0.039	0.072	0.099	0.070	0.040	0.032	0.027	0.033	0.020	0.048	0.052	0.040
Lower Lias Clay	0*					0.142	0.185	0.132	0.153				
	4	0.040	0.071	0.080	0.064	0.018	0.054	0.045	0.039	0.020	0.033	0.055	0.036
	6	0.038	0.053	0.062	0.051	0.030	0.045	0.040	0.038	0.027	0.046	0.066	0.046
	8	0.020	0.040	0.058	0.039	0.020	0.038	0.027	0.028	0.025	0.052	0.059	0.045
Strains at Failure in Drained Tests													
China Clay	0*					0.050	0.110	0.080	0.080				
	1	0.078	0.078	0.150	0.102	0.014	0.080	0.026	0.040	0.027	0.200	0.144	0.124
	3	0.150	0.150	0.070	0.123	0.020	0.126	0.156	0.101	0.020	0.085	0.122	0.076
	5	0.103	0.076	0.088	0.089	0.012	0.096	0.080	0.063	0.013	0.064	-	-
Lower Lias Clay	0*					0.030	0.105	0.130	0.088				
	4	0.036	0.049	0.033	0.039	0.033	0.060	0.023	0.039	0.025	0.038	0.073	0.045
	6	0.042	0.043	0.053	0.046	0.026	0.065	0.094	0.062	0.020	0.050	0.070	0.050
	8	0.053	0.025	0.033	0.037	0.020	0.033	0.065	0.039	0.032	0.039	0.052	0.041

* Confining pressures for zero lime contents were 0, 100, 200 kN/m^2
1 kN/m^2 = 0.145 psi

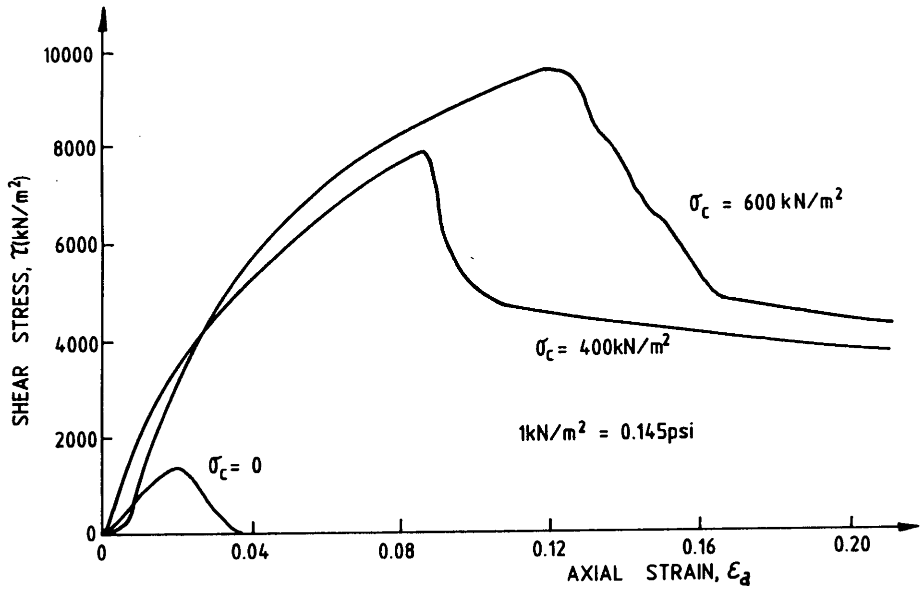

FIGURE 6 Graph of shear stress against axial strain for drained triaxial tests on English china clay with 3 percent lime at 56 days.

typically larger than those of undrained testing. A graphic illustration of the trend for higher strengths and failure strains with higher normal stresses is given in Figure 6, in which the different shear stress-strain behavior of china clay is also demonstrated. These data are crucial for good geotechnical design in certain practical cases, and particularly where slow, or relatively slow, loading is applied.

CONCLUSIONS

From the data presented herein, which are consistent with recent UK thinking on the subject, it is clear that lime improves substantially both the "cohesive" and "frictional" components of shear strength, and only small lime contents (1 to 3 percent) are required for considerable strength and stiffness increases. The "frictional" behaviour of lime-stabilized clay, both immediately on modification and during progressive stabilization, has been clearly demonstrated and must be accounted for in design. "Frictional" behavior can only be accurately determined using high confinement pressures in laboratory testing.

As more applications of lime-stabilized clay are being introduced, greater consideration of proof testing is required. Distinct differences have been demonstrated between drained and undrained behavior, and thus magnitude, rate, frequency, and duration of applied loading should simulate those prevalent in practice if accurate results are to be obtained. In this respect, the quick undrained triaxial test can be considered arbitrary only and thus indicative of relatively rapid loading. The unconfined compressive strength test, which is unable to account for frictional behavior, has been shown to be variously overly conservative and unsafe for design. This test should only be used as an index test to demonstrate whether a clay can be modified and stabilized. The Atterberg Limits also provide a good indication of the progress of lime-clay reactions and can be considered to be valuable index tests.

The two clays tested have been shown to be suitable for drained applications such as slope stabilization, even at lime contents below the Eades and Grim values. The strength and stiffness of the treated materials are sufficiently high after 7 days for operations to continue at an economic rate and the data show that the parameters, particularly ϕ, will increase significantly with time. However at sufficiently high lime contents for full stabilization to occur, very high (cementation) strengths (typically greater than 1000 kN/m^2, or 145 psi) will be obtained and the value of ϕ' will become less important. The failure strains conformed to the expected pattern in the unconfined case, but at high cell pressures higher failure strains were observed and these were considered to explain, at least in part, the "frictional" component of strength because of the need for sample dilation. Further work on lower Lias Clay to examine the performance at lower lime contents is warranted.

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