Deep-Slope Stabilization Using Lime Piles

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Lime piles have been successfully used in practice in several countries worldwide but their mechanism of operation has not been adequately explained. The literature on the subject has been briefly reviewed and a list of suggested stabilizing processes has been compiled. Programs of experimental work to investigate each process are described and the results are presented. Lime piles, which usually consist of columns of quicklime placed in pre-formed holes in clay soils, draw in water during hydration and expand. No lateral consolidation of the surrounding clay occurs, however, because of the stoichiometry of the hydration reaction. The water loss in the surrounding clay causes an increase in undrained shear strength. The associated pore water pressure reduction, or suction, caused by the hydration reaction is shown to be significant in the short term to increase the stability of the slope and in the long term by overconsolidation of the shear plane. Migration of calcium and hydroxyl ions from the piles has been shown to be limited to less than 30 mm (1.2 in.), as might be expected from considerations of clay liners. The pile itself is shown to have significant strength. The laboratory work has been supported by the initial findings of a program of field trials.

The problem of failing slopes has been addressed by engineers over millennia and, with the need for reduced land-take in the United Kingdom (UK), is increasing. Slopes were formed for the waterways, then the railways, and more recently have been formed for the ever-increasing miles of motorways. Failed slopes are a current problem in each of these transportation networks, but have recently been quantified in terms of UK motorways by Perry (*1*), who surveyed 570 km (354 mi) of motorway slopes covering all of the principal geologies. Although the study focused on both cutting and embankment slopes, only the latter will be considered here. The conclusions of this work make recommendations for side slopes to be adopted in new embankment construction in different materials and provide criteria to facilitate recognition of slopes that are at risk of failure. The survey revealed significant incidence of shallow slope failure (a total of more than 17 km or 10.5 mi of embankment slope and 5.5 km or 3.5 mi of cutting slope) and it was estimated that at least three times as many failures will occur in the future in the area surveyed as have occurred over the past 25 years.

It was reported that remedial works in the case of failed embankment slopes almost invariably took the form of excavation, sometimes in benches, to below the failure surface and replacement of the excavated material with compacted granular, free-draining material such as gravel, crushed rock or brick rubble. Topsoil is sometimes placed over the repaired area to permit vegetation to become re-established. This remedial process will typically require closure of part of the transport route, even if only the hard shoulder on a motorway for example, and requires excavation plant and significant equipment movements to both dispose of excavated material and deliver the granular fill. It is apparent that remedial methods that require small plant and minimal associated traffic would be of advantage. Lime piles provide one such solution.

The technique of using lime to improve the engineering properties of clay soils is reasonably well established if not yet fully understood. It is well known that lime reacts chemically with some of the constituents of clay to first modify and then stabilize the clay (*2*). Modification takes place within approximately 24 to 72 hr of mixing lime and clay together and causes flocculation and agglomeration of clay particles, changing the cohesive nature of the material. The clay becomes friable and granular in nature, and its strength and stiffness are improved (Rogers and Lee, in a paper in this Record). Stabilization occurs more slowly and concerns the long-term strength development brought about by the development of cementitious compounds from the reaction of lime with the siliceous clay components.

Lime has been widely used in road construction by intimate mixing with clay subgrades to improve workability, shear strength, and bearing capacity through the reactions already described, and is now being used for many applications. The use of lime as a deep stabilizing technique is a more recent development. In Sweden, China, Japan, and Singapore, lime, in the form of intimately mixed lime-clay columns created in situ, has been used to prepare soft ground for foundations. The stabilization reactions that subsequently occur produce columns of material of greater strength, and arguably increased permeability, than the surrounding untreated clay. The technique does, however, require large plant to carry out. Lime slurry pressure injection has been established as a stabilizing technique in the United States. As the name suggests, lime in the form of a slurry is injected into the ground using specialist, although relatively small, equipment. It has been used to treat expansive clay soils, silts, and other soft ground and to stabilize failed embankment slopes, although the problem of increased pore water pressures induced by this technique could be seen as a drawback with this latter application.

The final technique, and the subject of this paper, is lime piles, which are holes in the ground filled with lime. Ingles and Metcalf (*3*) show one method of construction as illustrated in Figure 1, in which a hollow tube is pushed into the soil to the required depth of pile and quicklime is forced into the tube under pressure as it is withdrawn. The pressure forces open the end of the tube allowing the lime to fill the cavity below. After each meter is filled, the end of the tube is closed and used to compact the lime. In stiff or strong clay soils holes are formed using a powered auger and the lime is fed in and compacted once the auger has been withdrawn. It is believed that lime piles are potentially a simple solution, being both cheap and easy to install using small plant, even in the most remote places. As a result a major research program is being carried out at Loughborough University of Technology in the UK (*4*) to investigate how lime piles work and how they can be designed and installed. The aim of this paper is to outline their potential and to provide supporting experimental data.

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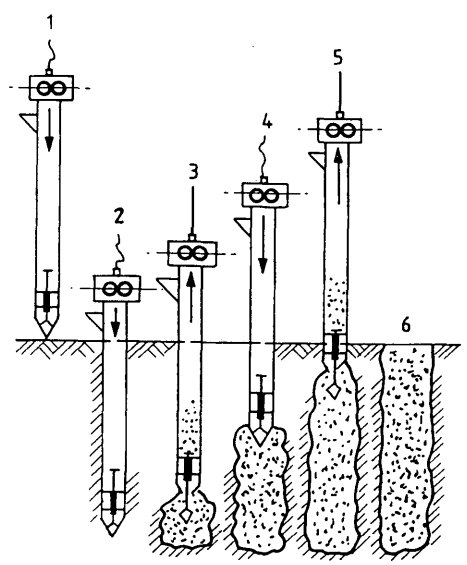


FIGURE 1 Procedure for construction of lime piles in soft clay soils (*4*).

LITERATURE ON LIME PILES

The use of lime piles has been documented in many countries although no consensus on their mode of operation has been reached. Two distinct themes emerge concerning the mechanism of stabilization and authors largely discuss one independent of the other. The first of these is the idea of lime pile expansion and surrounding clay dehydration. Quicklime has a great affinity for water and thus draws in water from the surrounding soil to be consumed by the (expansive) reaction. The piles then expand and are said to cause lateral consolidation of the surrounding clay, although because the reaction is highly exothermic and large volumes of quicklime are used, it is likely that significant quantities of water are removed in the form of steam. In general, the authors who propose this mechanism are using the lime piles to improve the bearing capacity and settlement characteristics of soft ground. The most notable exponents of this technique are the Japanese (*5*), who describe the use of quicklime piles of diameters up to 1 m (39 in.) placed at spacings less than 1.5 m (5 ft) in clays of very high water content (up to 400 percent). The changes produced appear to be proportional to the initial water content of the soil. Wang (*6*), describes similar work being carried out in China. Laboratory studies have been described by Chui and Chin (*7*), at the National Taiwan University, using clay (Taipei silt) compacted into boxes in which small quicklime piles are formed. They also believed that initial moisture content was the key to the success of the process, but mention the possibility of chemical reaction between the lime and the surrounding soil.

The suggested principle of direct clay stabilization by lime is that calcium ions from the piles migrate outward into the surrounding soil and react beneficially in the manner described earlier. Several U.S. authors describe the use of 150-mm (6-in.)-diameter lime piles in slope stabilization, with the review by Lutenegger and Dickson (*8*) being indicative of the ideas being expressed. In general, quicklime is used and water added after placement. Montmorillonitic soils are favored for this treatment. Although several case studies are described and the idea of stabilization through lime-clay reaction presented, few scientific data are given. Water flow was believed to promote calcium ion migration.

Considerable attention has been given to the use of lime piles in India, both in the field and in the laboratory. Model tests similar to those described by Chui and Chin (*7*) have been carried out to investigate such variables as clay type, pile shape, and pile spacing. Presentation of results is, however, rather poor. This means that although the results are interesting, quantitative information cannot be retrieved.

The use of lime piles for improvement of slope stability has been documented in Austria (*9*) and Thailand (*10*). Both papers pursue lime migration as the major stabilization mechanism.

A summary of possible stabilizing mechanisms taken from the literature follows:

1. Lateral Consolidation: Quicklime draws in water from the surrounding ground and reacts to form slaked lime, which has a lower density than quicklime and hence expansion occurs. The diameter of the pile can increase by 30 to 70 percent. This expansion is said to cause lateral consolidation of the ground surrounding the pile.
2. Water Content Reduction: The slaking reaction "uses up" some water from the soil surrounding the piles and produces a significant amount of heat. These two processes in combination cause a significant water content reduction.
3. Clay-Lime Reaction: Lime migrates from the piles to a considerable distance. This lime then reacts with, and stabilizes, the ground in a manner described earlier.
4. Reduction in Pore Water Pressure: The slaking reaction in the piles causes a net flow of water toward the pile. This suction reduces pore water pressures in the ground.

RESEARCH AIMS AND PHILOSOPHY

On the basis of the various suggested principles of operation, an experimental program was devised to study the various stabilization processes independently of one another, where possible. Experiments were designed and initial results were analyzed in order to further develop tests, thus using an iterative approach. In this way it was considered that the mechanisms of operation of lime piles could be determined and quantified.

The technique is to be tested in two full-scale field trials toward the end of the project and a second aim of the laboratory investigations is to provide sufficient data to facilitate design. The field trials should not only provide supportive data on pile performance but allow experimentation with installation techniques. A small-scale field trial on one of the sites has already provided useful data. Following the fieldwork, recommendations will be made concerning site investigation and testing programs necessary for design. A design guide, based on a parallel theoretical study, will be produced, and installation methods and equipment will be recommended, so that the technique may be used in practice with confidence. The first link in the chain is described in this paper.

LABORATORY INVESTIGATION

Migration

It appeared from the literature that the most significant stabilizing process would be the transportation of calcium ions from the pile into the surrounding soil and the subsequent lime clay cementitious reaction. Pile models were created in an 0.4 by 0.4 by 0.4 m (16 in.) rigid box filled with a predominantly illitic lower Lias clay, similar to that of previous researchers (*11,12*). To facilitate compaction into the box, achieve homogeneity, and create minimal voids, the clays were first mixed to a high moisture content (w = 40 percent; liquidity index = 0.5). Compaction was effected by a small manual "sheepsfoot compactor," designed specifically for this purpose, in approximately 50-mm (2-in.) layers. After placement of the first layer, two manometer tubes were positioned on opposing faces of the box. The ends of the tubes were wrapped in geotextile and placed in a sand reservoir to prevent ingress of clay particles. Once filled, the box was covered in polythene and a static stress of 40 kPa (6 psi) was applied to the surface. The system was then allowed to equilibrate, as dictated by the pore water pressure readings, over a 3-week period. A central pile was then created using a 50-mm (2-in.) -diameter hand auger for excavation and quicklime was compacted using a tamping rod into the resulting hole. The surface stress was then reapplied. Samples were taken using the same hand auger at various distances from the pile over the next 8 weeks and tested for free calcium ions.

No real evidence for migration of lime was found at the positions tested. An acid base indicator (phenolphthalein) was painted onto the clay surface after progressive excavation in order to locate the distance of migration (a lime solution is highly alkaline because of hydroxyl ions). However, no evidence of ion migration further than approximately 20 mm (0.8 in.) from the pile was recorded at any depth. It was believed likely that the successes of the previous authors in testing for lime in a similar experiment were attributable to the scatter of lime on the surface of the clay, because edge contamination is inevitable when forming the pile, and possibly the fluctuating free calcium ion concentration in natural clay, which was the parameter measured. These problems combined to cause the apparently larger migration distances reported by Rogers and Bruce (*12*).

A further set of experiments examined the effects of moisture content and clay type on the rate and distance of ion migration. Perspex tubes, 32 mm (1.3 in.) in diameter, were filled with clays mixed with the acid-base indicator at different moisture contents. A 6-mm (0.24-in.) -diameter quicklime pile was placed at the center of each tube and the time taken for the ions to reach the edge of the tube (13 mm or 0.51 in.), evidenced by the change of color of the indicator, was recorded. This was achieved by the integration the acid base indicator into the clay, which changed color as the alkaline lime passed through it.

It was concluded in this study that 20 to 30 mm (0.8 to 1.2 in.) migration was all that could be expected in the conditions provided. Higher moisture contents increased the rate of migration, and clay mineralogy also influenced the rate. The latter point could be attributed to increased porosity. It was also noted that migration would occur preferentially along paths of least resistance (i.e., a slip plane that typically has a higher moisture content) as long as moisture continuity was provided. Similar conclusions were reached by Fohs and Kinter (*13*), investigating lime migration into small blocks of clay.

The question of the effect of a flow of water on migration still remains, particularly when associated with a slip plane. The successful migration of lime slurry under a hydraulic head in a fissured shale (*10*) is understandable. However, the migration recorded by the American authors [e.g., Lutenegger and Dickson (*8*)] reporting on lime piles in the field was difficult to explain, especially as no indication is given of what physical signs were used to detect the lime. One explanation considered was that of water flow through the ground. Because of the low permeability of clay, water flow over a significant distance would have also to be accompanied by zones or features of higher permeability, while retaining sufficient clay content for the clay-lime reaction to occur. A reasonable explanation could therefore be that preferential migration occurred due to the increased permeability created by the shear zone of the slip itself. Although attempts to achieve ion migration by this means have been made, in both specifically designed box experiments and an adapted permeability apparatus, they have proved unsuccessful. Other possible influences on ion migration such as vapor, suction, and osmotic pressures are recognized. However, such a detailed study lies beyond the scope of this investigation. These influences will be considered within a study of chemical movement in the ground, which is on the point of commencement.

Despite the possibility of increased migration produced by a water flow, it now seems unlikely that this forms the major stabilization process and so additional processes to explain the undoubted success of the technique had to be considered carefully.

Expansion

It is possible to calculate the volume of a pile after hydration simply using the equation for slaking

$$CaO + H_2O \longrightarrow Ca(OH)_2$$

and the relative densities of quicklime and slaked lime.

As mentioned previously (*12*), the additional volume of the slaked lime in relation to the quicklime is less than the volume of water required by the reaction. This phenomenon was illustrated by radial cracks in the clay surrounding the piles formed in model tests in the laboratory (described later) caused by the slaking reaction. These penetrated a considerable distance down the length of the pile and could only indicate a reduction in volume in the lime-clay system as a whole because no steam was given off and the system was sealed.

Dehydration

Again, from the stoichiometry of the slaking reaction it is possible to calculate the expected moisture content of the soil after treatment with quicklime. From the equation below the molar masses of the reactants and products are

$$CaO + H_2O \longrightarrow Ca(OH)_2$$

$$56\ g + 18\ g \longrightarrow 74\ g$$

Given piles of known dimensions and spacing and assuming a density for compacted quicklime, it is relatively straightforward to calculate the volume of water consumed in the entire reaction. This is readily converted into a change in moisture content by assuming an area of treatment and a clay density.

This calculation was carried out for a soil of 30 percent moisture content with 3-m (10-ft) -deep piles of 0.1 m (4 in.) diameter at 1-m (39-in.) spacing. Assuming that all of the material in the piles reacts, this would give a final moisture content of 29 percent (i.e., very little change).

However, the topic remained troublingly evident in much of the literature, and thus it was considered that an investigation was necessary. A set of tests was carried out on samples of English china clay, which is almost wholly kaolinitic, at different moisture contents and on one lower Lias (predominantly illitic mixed mineralogy) clay sample. Sample of 100-mm (4-in.) diameter were created by compaction at a known moisture content and piles were created using a 12-mm (0.47-in.) rod driven to create a central hole into which quicklime was poured. This was compacted using a smaller rod. The samples were sealed in plastic. After various curing periods, 38-mm (1.5-in.) samples were cut from the clay surrounding the pile and subjected to undrained triaxial compression tests. Moisture contents were also found.

After a curing time of 8 weeks the moisture contents were compared with both the original values and with the calculated value after the water loss caused by slaking. The measured values were significantly lower than those calculated. This was initially attributed to inadequate sealing of the samples, although a subsequent explanation can be postulated. The pile itself will absorb water both during slaking and subsequently into the pore spaces between the particulate slaked lime, and the amount of water that can be absorbed will increase as the pile expands. A very simple calculation of the water required by the pile to come into equilibrium with the original clay moisture content produces values almost identical to those measured.

The significance of this moisture loss was also investigated. Strength improvements associated with these levels of drying were calculated using the formulae of Skempton and Northey (*14*) and compared with the measured strengths from undrained triaxial tests carried out on the samples. The method of calculation is based on the fact that clays at both the liquid and plastic limit fail at the critical state. Experiments carried out by the authors showed that the undrained shear strength at the plastic limit was 100 times that at liquid limit. As shown in Table 1 the measured and calculated values compare favorably.

In practice, this formula may be used to calculate the improved strength of a clay caused by drying, because little additional drying will occur because of evaporation as the top of the piles were sealed with a clay plug. The extent of the affected area remains a problem. In practice, a very small zone surrounding the pile may become very dry, leaving the rest of the clay at the same original water content. This seems particularly likely when cracks form. Careful sampling during excavation of the field trials should provide evidence on this subject.

TABLE 1 Strength Changes Related to Moisture Content Change

CLAY TYPE	MOISTURE CONTENT (%)		Cu (kPa)		
	INITIAL	FINAL	INITIAL	FINAL	CALC
LOWER LIAS	42	40	25.5	36.5	36.9
ENGLISH CHINA CLAY	35	33	85	124	126.8
	45	43	26	34	37.9
	55	51	18	27	28.4

Note: 1 kPa = 0.15 lbf

Pile Strength

Clay-lime pile models were set up in order to study several different factors, pile strength being one of them. A 1.0-m by 1.0-m by 0.5-m (39-in. by 39-in. by 20-in.) deep steel box was fabricated with the facility of a "water-bag" arrangement within its lid to supply an even, maintainable pressure to its contents (Figure 2). The box was divided internally into four equal compartments so that different clay and pile types could be studied simultaneously. The box was filled with an almost wholly kaolinitic English china clay at a moisture content of 48 percent (equivalent to a liquidity index of 0.5) in layers using hand compaction, as in the previous model. The clay was allowed to consolidate under a normal load of one bar (14.5 psi). Manometers and thermocouples were placed in the positions shown during the compaction process. Piles were formed, as before, with diameters of 100 and 150 mm (4 and 6 in.) at two different degrees of lime compaction, a plastic sheet being spread over the clay surface to prevent lime scatter. The 100-mm (4-in.) -diameter piles were compacted in four layers using 10 blows of a 2.5-kg (5.5-lb) compaction hammer, 20 blows being applied to the layers for the 150-mm (6-in.) -diameter pile. A 40-mm (2-in.) -thick clay "cap" was placed on the top of each pile. After the piles were formed the normal stress was reapplied and the system left intact while pore water pressure changes and temperature fluctuations were monitored. Measurements were taken until a state of equilibrium had been established (i.e., cessation of changes in the readings).

Ultimately the individual piles were excavated and 38-mm (1.5-in.) samples tested to destruction to obtain a value for pile strength. The strength of piles was found to be between 300 kPa (44 psi) and 500 kPa (73 psi).

Pore Water Pressure Changes

The box study previously described considered the changes in pore water pressure both close to, and somewhat removed from, the edge of the pile. These distances are different for the two pile diameters and are illustrated in Figure 2. Two readings were taken close to the piles but only one, at the base, was taken near the edge of the box. The reductions in pore water pressure, or suctions, measured for the 150-mm (6-in.) -diameter uncompacted pile were 8.2 and 10.7 kPa (1.19 and 1.55 psi) close to the pile and 2.5 kPa (0.36 psi) at the edge. The data for the 100-mm (4-in.) -diameter uncompacted pile were 16.2, 12.6, and 3.6 kPa (2.35, 1.83, and 0.52 psi) and for the 100-mm (4-in.) -diameter compacted pile were 13.0, 11.0, and 5.1 kPa (1.89, 1.60, and 0.74 psi), respectively.

Allowing for experimental error, no particular trend relating pore water pressure change to either pile size or degree of compaction has emerged from these or subsequent data, although the pile sizes chosen may be insufficiently dissimilar to discern a difference. It is, however, apparent that the magnitude of the reduction is lower at the edge of the box when compared with that close to the pile in clays of relatively high water content.

To create a better idea of the rate of change across the box, some additional data were retrieved from the original small-box tests. The manometer had been placed 50 mm (2 in.) from the base of the box and 100 mm (4 in.) away from the 50-mm (2-in.) -diameter pile. This was placed in lower Lias clay of the same liquidity index as the china clay previously mentioned. After a similar time period to the readings already mentioned, a reduction of 8.5 kPa (1.23 psi) was reached. If this is compared with the data from the 100-mm (4-in.) -diameter compacted pile at the base of the box, the reduction appears approximately linear with distance away from the pile.

A feature that has been noted in all tests is the formation of radial cracks around the pile. In the large model tests, cracks were observed to develop within 3 days of pile placement, although without subsequent enlargement. The most pronounced cracks were recorded for the 150-mm (6-in.) -diameter compacted pile, in which four 3- to 5-mm (0.12- to 0.2-in.) -wide cracks developed at 90 degrees to one another and extended from the pile to the edge of the box. The size and extent of cracking appeared to be directly related to pile size and degree of compaction, with no cracks being observed in the 100-mm (4-in.) -diameter uncompacted pile. This is likely to be related to the amount of water required by the lime to react and the pressure on the clay from the expanding pile. It may also be a function of the effect of

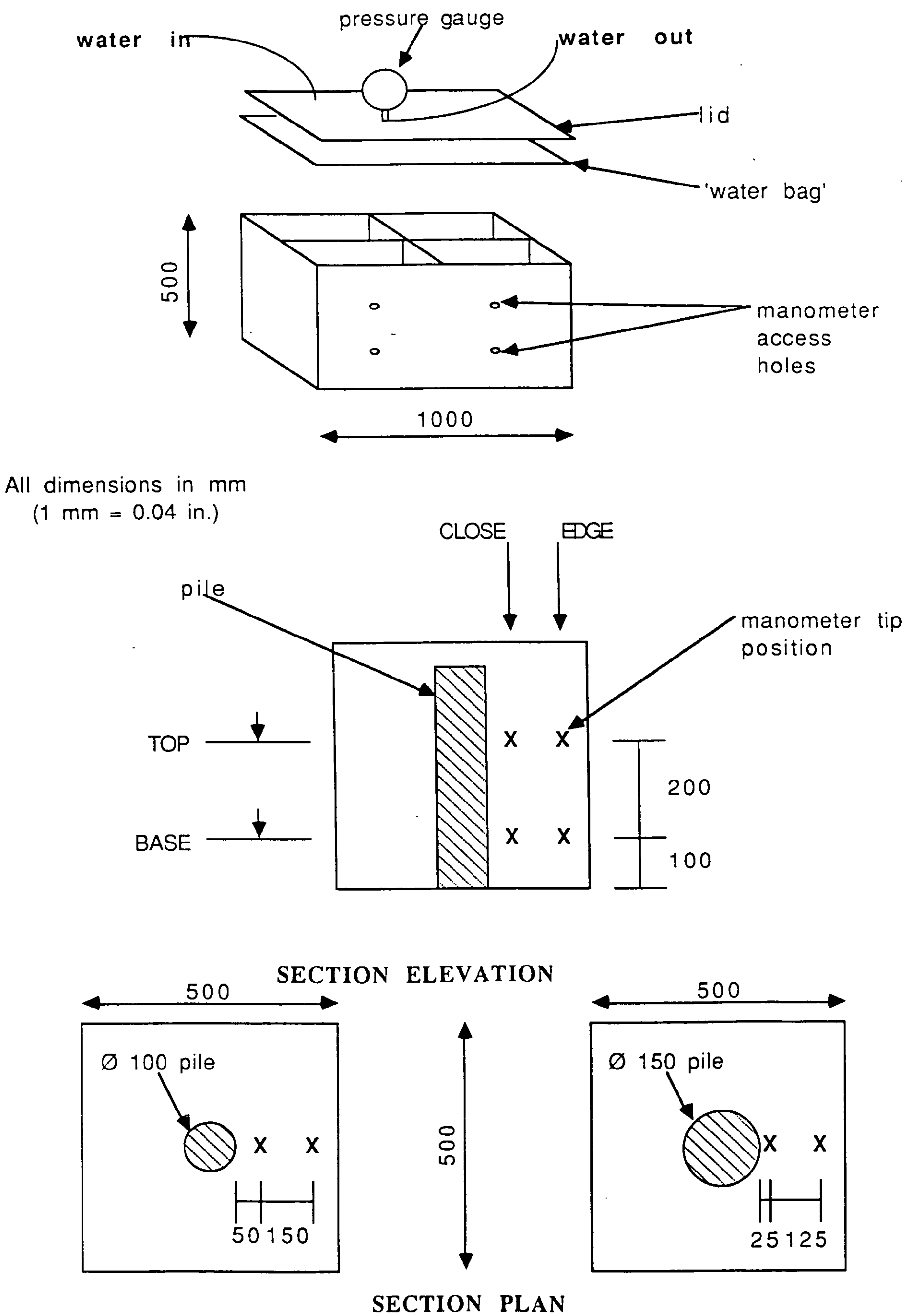


FIGURE 2 Laboratory setup of pile models.

temperature rise, because the recorded temperatures follow a similar pattern to that of the formation of the cracks (i.e., greatest for the largest well compacted pile). The formation of cracks may be quite significant when considering the radius of influence of the reduction in pore water pressure and of moisture content reduction. Careful observations will be made of the development of cracks and their influence in the field trial.

Overconsolidation of the Shear Plane

The idea of pore water pressure reduction as a benefit to a failing slope was first raised by Rogers and Bruce (*12*), although its effects were then only considered to be of temporary benefit before the longer-term reactions took place. Its consequences were thus only considered over the time in which it was present and at best the effect could last until pore water pressures naturally resumed initial values over a number of years. In the longer term the effect on any shear failures present within a slope could be more crucial, however, if the reduction in pore water pressure is considered as an increase in effective stress. Such an increase in stress would lead to consolidation of the remolded zone associated with a shear failure and raise the shear strength in this area above residual. This rise in shear strength may be sufficient to prevent further failure when combined with small additional physical or chemical improvements from the processes previously mentioned.

This phenomena was studied in English china clay in the direct shear box. The residual strength parameters for the clay were determined using normal effective stresses of 150, 300, and 400 kPa (22, 44, and 58 psi) Shear strengths of 39, 69, and 80 kPa (5.66, 10.01, and 11.61 psi) were derived, respectively, thus giving a residual friction angle ($\text{Ø}_r'$) value of 11°. This is in agreement with published data.

Both normally consolidated and overconsolidated (ranging from slightly to highly) samples were tested for comparative purposes, the latter being typical of cutting slopes studied by Perry (*1*). The shear surface was formed by rapid winding of the box, initial tests proving that five passes were sufficient to reach the residual state. The sample was then allowed to reconsolidate for 24 hr before being sheared under fully drained conditions at a rate of 0.036 mm/min. (0.0014 in./min.) until a consistent shear stress was achieved. The box was then stopped and allowed to rest overnight. The normal effective stress was then increased by 20kPa (3 psi) to simulate pore water pressure reduction and the sample was again left overnight. [It should be noted that this value is greater than the suctions measured in the laboratory, but that these refer to relatively wet clays, and an initial field trial produced suctions of approximately 20 kPa (3 psi) 700 mm (28 in.) away from 63-mm (2.5-in.) -diameter piles constructed on a grid in a clay near to its plastic limit]. The 20-kPa (3-psi) increment was then removed and the sample allowed to swell, again overnight, before shearing was resumed. Stress changes were noted at every juncture.

Stoppage alone caused a small but rapid immediate reduction in shear stress in all cases after half an hour, followed by a much slower relaxation recorded after leaving the sample overnight. If shearing was resumed after half an hour, the stress rapidly resumed its initial value, whereas if the sample was left overnight, a small rapid increase in stress over and above that of the original value was generally observed before steady state resumed. The results for a sample overconsolidated to 300 kPa (44 psi) and allowed to swell back to 15 kPa (2 psi), giving an overconsolidation ratio of 20, are presented in Figure 3 and illustrate this trend. It was considered that the small initial reduction in stress could be attributed to pore water pressure dissipation, the pore water pressure being rapidly rebuilt on shearing. The rate of loading was chosen, however, following preliminary testing, to ensure fully drained conditions. The longer-term reduction could be made up of several components, including relaxation of the machine itself and consolidation of the shear plane or shear zone. Most significantly under low normal stresses, the shear surface will be uneven with the platy kaolin crystals tumbling over one another (termed turbulent shearing) rather than being fully aligned (as in sliding shear). It is believed that cessation of shearing could promote a degree of "knitting" of the shear zone, which requires an increase in stress to disrupt when shearing is resumed (the shear surfaces could not be separated when taken out of the box).

The effects of overconsolidation are similar to those previously described for cessation of shear, although they are more pronounced. One important difference shown in Figure 3, however, is that the baseline shear stress rises after overconsolidation and requires considerable further shearing before it falls to the original value. Predictably the effects of overconsolidation were most pronounced in the sample sheared at the lowest normal effective stress, having the most disrupted shear plane, with little or no effect being observed for a normally consolidated sample tested at a high normal effective stress. Such a sample was found to have a smooth easily separable shear surface.

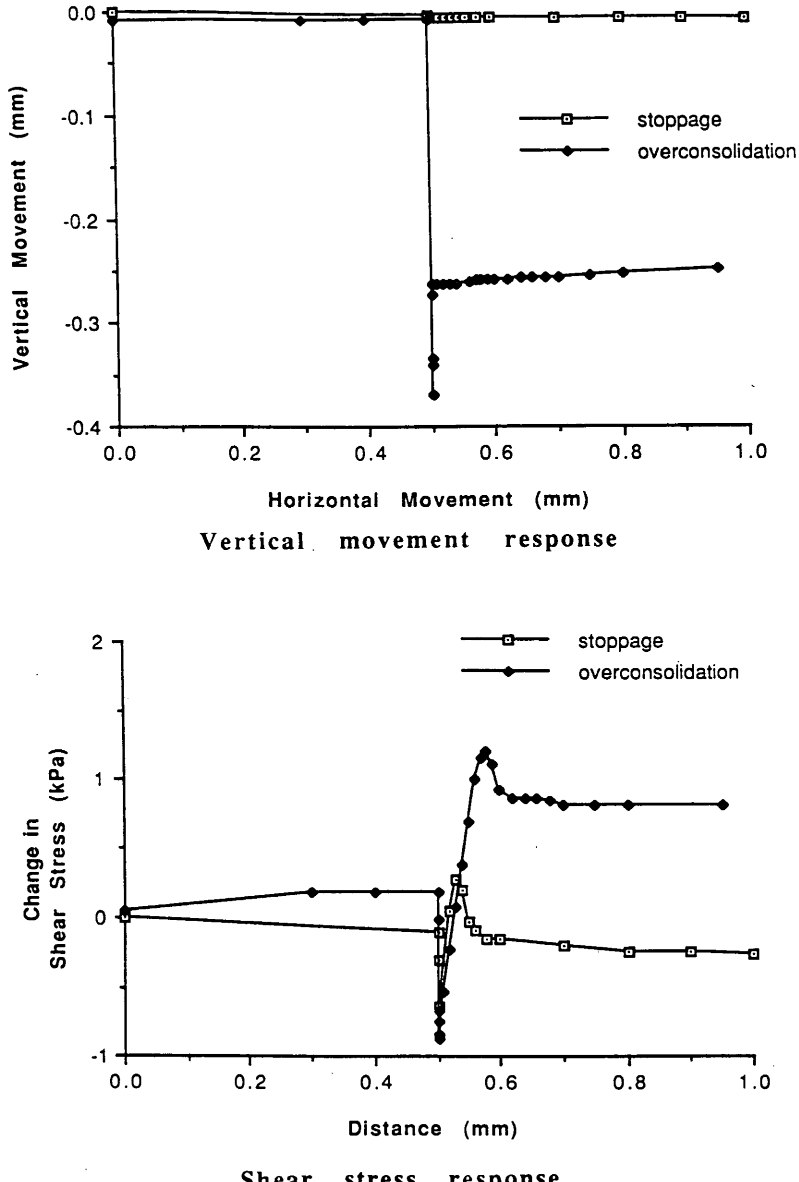
Measurement of vertical movements (Figure 3) show small compressions associated with consolidation for stoppage and much larger compressions (although still small in absolute terms), followed by a small dilation once shearing is resumed for overconsolidation of the shear plane.

A summary of the results obtained to date is given in Table 2, in which the improvements have been quantified in terms of increases in shear strength and equivalent increases in Ø′. The dashed entries for the normally consolidated samples indicate that no clear, significant increase in strength was obtained and the result has been assumed to be zero until further investigations have been conducted.

Because of complications caused by the limited travel and stress changes at the end of the travel of the shear box, ring shear tests have also been conducted. The effect of overconsolidation on a normally consolidated sample with a normal effective stress of 300 kPa (44 psi), if observed over a similar shearing distance to that of the shear boxes, is remarkably similar in shape, whereas vertical movements indicate the creation of a smoother shear surface. However, over a longer distance (10 mm or 0.4 in. as opposed to 1 mm or 0.04 in.) the pattern looks very different. A net rise in shear strength of 1.39 kPa (0.20 psi) is observed before the pre-stop value is resumed.

CONCLUSIONS

The literature, although providing valuable information about the successful use of lime piles for several applications along with some details of laboratory experimentation, does not adequately prove the modes of operation of lime piles. A list of possible processes leading to successful stabilization was compiled, although it was mostly based on authors' speculation rather than experimental evidence. This formed the basis for the study reported herein, which examines the viability of each theory and



Note: 1 mm = 0.04 in.
1 kPa = 0.15 lbf

FIGURE 3 Response of direct shear box.

quantifies their relative effects and their mutual influences. The stabilization processes have thus now been isolated and quantified. The second phase of the project, involving further laboratory experiments to confirm the efficacy of the processes under different conditions and, predominantly, field trials, is under way. The initial results of the field work support the findings reported herein.

An allied research project is being carried out to develop a computer model for slope stabilizing techniques, the use of which will allow lime pile treatment of failing or failed slopes to be modeled and designs to be carried out.

The processes that contribute to lime pile stabilization of slopes are as follows:

1. Reduction in pore water pressure;
2. Overconsolidation of the slip-plane, as a consequence of the reduction in pore water pressure, leading to increased strength of the clay along the slip plane;
3. Pile strength;
4. Increased strength of the clay surrounding the piles in a small annular zone because of lime-clay reaction; and
5. Dehydration of the clay, causing a rise in undrained shear strength.

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TABLE 2 Direct Shear Box: Strength Change Caused by Overconsolidation

Consolidation Stress (kPa)	Shearing Stress (kPa)	O C R at Shear	Rise in kPa	Rise in Ø' (degrees)
	30	5	0.54	1.03
150	75	2	0.23	0.18
	150	1	-	-
	15	20	1.01	3.85
300	30	10	0.44	0.84
	300	1	-	-
400	20	20	1.00	2.86

Note: 1 kPa = 0.15 lbf

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