Lime pile remediation of a Gault Clay embankment slope on the M1

Prepared for Geotechnics and Ground Engineering, Highways Agency

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Executive Summary

Shallow failures on the slopes of highway embankments and cuttings, typically in the over-consolidated plastic clays of Southern England, are a common feature of trunk roads and motorways. The frequency of shallow slips is predicted to increase with time and there is a requirement to develop simple preventative and remedial treatments. For this reason, TRL Limited was commissioned by the Highways Agency to assess the potential of using small (≈ 200mm) diameter lime piles for improving the stability of ageing clay slopes. As part of this research programme, an instrumented full-scale trial of the remediation technique of lime piling was carried out on a Gault Clay embankment on the M1 which has a history of shallow failures. Lime piles were installed in each of two 20m long test areas of embankment, in one case the lime piles were unlined and in the other they were lined with waterwell screen. Results from earlier laboratory trials had indicated that, when using pure quicklime, there was much advantage in using plastic waterwell screen pipe to line piles because of the higher undrained shear strengths obtained. The field performance in both cases was compared with that of a control area where no remedial work was undertaken. Site measurements of pore water pressure and ground movements enabled a systematic comparison of behaviour using the different approaches. The construction procedure, cost implications, an evaluation of slope performance during the first 26 months in service, and design recommendations are described in the report.

At each of the instrumented areas the trend of the results confirmed that pore pressures decreased in the summer months with suctions generally developing in the slopes to depths of just over 2m and to deeper depths nearer to the slope toes. Comparison of contour plots in the remediated and control areas also indicated that 5 months after construction, hydration of the quicklime in the remediated areas was still having some effect in drying out the slope as well as providing support through a dowelling action. By the second year in service the hydration of the lime piles was effectively complete.

Changes in lateral movement and pore water pressure were consistent with those measured at other clay slopes where the seasonal cycle of clay swelling and shrinkage because of wetting and drying has also been monitored.

1 Introduction

Shallow failures on the slopes of highway embankments and cuttings, typically in the over-consolidated plastic clays of Southern England, are a common feature on trunk roads and motorways. The frequency of shallow slips is predicted to increase with time and there is a requirement to develop simple preventative and remedial treatments. For this reason, TRL Limited was commissioned by the Highways Agency to assess the potential of using small (≈ 200mm) diameter lime piles for improving the stability of ageing clay slopes.

The first stage of the project was a comprehensive literature review of the principles and practice of lime-stabilised soil columns and lime piles, which included details of a simple design method that could be used (West and Carder, 1997). The next stage was to assess the strengths of laboratory trial mixes of lime-stabilised soils and of pure quicklime (Brookes *et al.*, 1997). During the mixing trial it was established that, when using pure quicklime, there was much advantage in using plastic waterwell screen pipe to line the pile. Piles lined in this way were expected to have an undrained shear strength for design purposes of nearly double that of unlined piles, although further verification of this is needed in a range of clayey soils.

The last phase of the project involved an instrumented full-scale trial of the lime piling technique on the highway network. For this purpose a Gault Clay embankment on the M1 which has a history of shallow failures was identified. Lime piles were installed in each of two 20m long test areas of embankment, in one case the lime piles were unlined and in the other they were lined with waterwell screen. The field performance in both cases was compared with that of a control area where no remedial work was undertaken. Site measurements of pore water pressure and ground movements enabled a systematic comparison of behaviour using the different approaches. This report describes the construction procedure, cost implications and an evaluation of slope performance during the twenty-six months following construction. Data on lime strength and stiffness obtained from tests on sacrificial piles by exhuming them after four, ten, sixteen and twenty-five months are also included.

2 Location and description of site

The site selected for this study was located near to Junction 12 of the M1 motorway approximately 1km north of Toddington Services. Details of the site location and the three test areas employed are shown in Figure 1. Each area of the embankment was about 20m in length and the slope angle to the horizontal was 28^o in all cases, that is approximately 1v:2h. Lined lime piles were installed in area 2 where the embankment height was 6.95m, unlined piles were installed in area 1 where its height had reduced to 6.1m nearer to the northbound on-slip. A control area, which was untreated, separated the two areas where remedial works were undertaken.

This section of the M1 (Luton to Ridgmont) was opened in November 1959 and its design and construction was described by Williams and Williams (1960). The embankment was constructed using Gault Clay excavated from neighbouring areas of the works. An open ditch was used for drainage purposes in front of the toe of the slope and there is no record of any drainage blanket being used beneath the embankment. In recent years the site had shown some instability with numerous very shallow slips being evident on the grassed slope giving it an undulating appearance. Near to area 2, more significant movement had occurred some years ago causing the nearby motorway lighting column to tilt about 9^o towards the slope, although the slope had since appeared to restabilise.

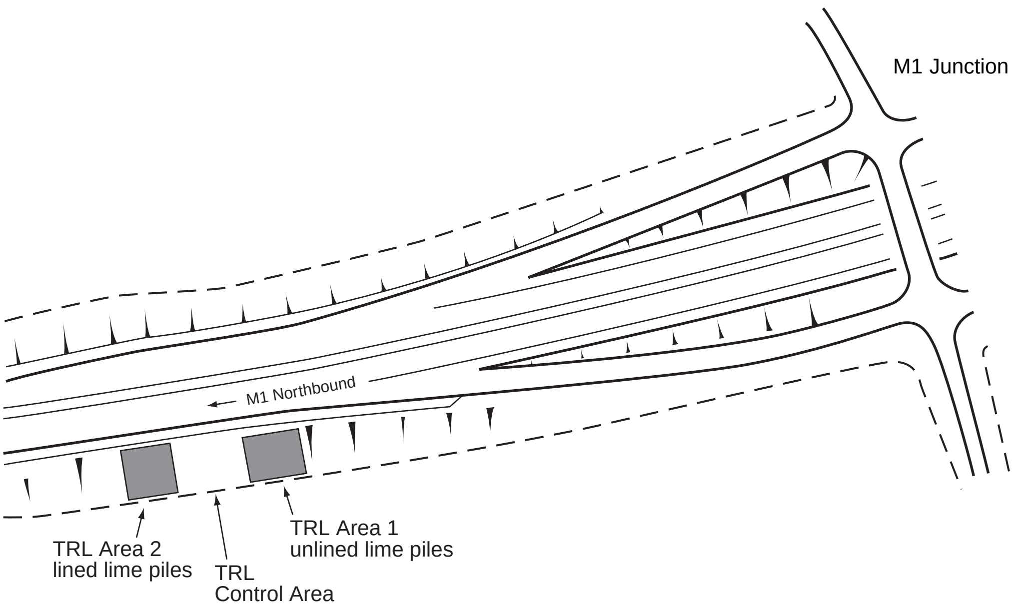


Figure 1 Site location

3 Ground conditions

Results from the ground investigation (Foundation & Exploration Services, 1994) carried out for the proposed M1 widening near the location of the trial site indicate that the embankment fill was a firm brown and grey mottled silty clay (Gault Clay). The embankment was constructed on a foundation of cohesive soil comprising a layer of glacial till up to 2m thick overlying Gault Clay which extended to a depth of at least 20m. The Gault Clay was generally a stiff fissured clay which became very stiff at depth, the zone of weathering of the clay varied over the site between about 2m to 6m.

For the purpose of the lime pile trial, the strength of the embankment fill (Gault Clay) was investigated using the Panda penetrometer (Langton, 1999) and the Marchetti flat-bladed dilatometer (Marchetti, 1980). In both cases, the *in situ* testing was carried out after remediation of the slope. All of the tests in remediated areas were however carried out midway between two lime piles where the clay strengths were most likely to be representative of those existing before the current study. A summary of the results from the Panda penetrometer in the three areas is given in Figure 2. In this figure the undrained shear strength of the clay has been determined by dividing the measured cone resistance by twenty: this follows the recommendation of Langton (1999) based on the findings of Gourves and Barjot (1995) and Butcher *et al.* (1995).

The results in Figures 2b and 2c generally indicated an undrained shear strength of just below 100kPa for the clay within about 3m of the slope surface. The corresponding zone appeared slightly stiffer in Figure 2a where the unlined lime piles were installed. In some tests a higher strength was also recorded at about 0.5m depth at the interface between the topsoil and the clay; this is possibly associated with some drying of the clay surface and was particularly noticeable in tests near to the top of the slope. Some increases in undrained strength to values just above 100kPa were observed below about 3m depth at each test location. In cases where the Panda penetrometer tests continued into the clay foundation, there was no evidence of a layer of higher strength which tended to confirm that a drainage blanket had not been installed below the clay fill at the time of embankment construction.

At one particular location, that is near the top of the slope in the control area, the Panda penetrometer results were compared with soil profiling carried out using the Marchetti flat-bladed dilatometer. A comparison of the undrained shear strengths determined using the two techniques is shown in Figure 3a. The results from the Panda penetrometer were the larger and considered more reliable as the Marchetti strengths involve using an empirical relation which is dependent upon effective vertical stress. In determining the effective stress at each depth, the vertical overburden was corrected for the slope situation using the formula given by Poulos and Davis (1974) and the pore water pressure was estimated from the piezometer readings reported later. However, because of these assumptions, this process can at best only be considered a rough approximation. It is nevertheless interesting that stiffer layers are identified at about 0.5m and below 3m depth when using both techniques. Also included in Figure 3b is the profile of dilatometer modulus against depth calculated using the Marchetti dilatometer: modulus may be a more meaningful parameter in this case as it is independent of effective stress.

4 Construction sequence

The sequence of works is summarised in Table 1. Prior to any construction work commencing, the exact location for each unlined and lined lime pile and each instrument was set out in accordance with Figures 4, 5 and 6 respectively.

The first stage of construction consisted of the erection of suitable working platforms from which to operate the Stealth T3000 rotary drilling rig. Figure 7 and Plate 1 show the typical scaffolding arrangement employed throughout the duration of the works which enabled two rows of boreholes to be drilled from a single elevation. As six rows of piles were required in both areas, three working platforms were progressively constructed in each. Platforms were struck behind the drilling operation and reconstructed over the next two rows to be drilled. This avoided the need for extra and unnecessary scaffolding on site. Plate 2 shows the drilling rig operating on one of the platforms in area 1.

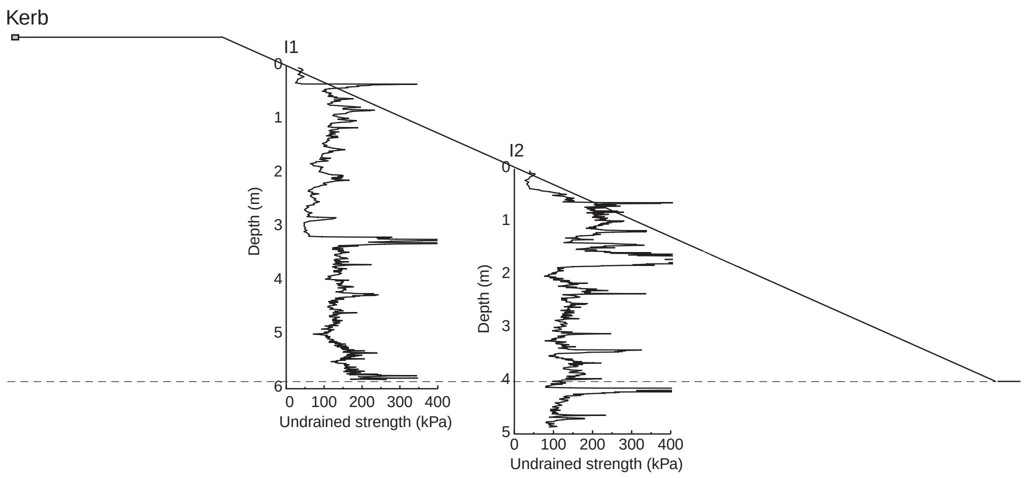
Plant was delivered to site by flatbed lorries (utilising the hard shoulder of the motorway) and transferred directly onto the embankment using a Hi-Ab. This minimised the requirement for lane closures and/or safety barrier removal. The power pack accompanying the Stealth T3000 remained at the top of the slope on similar, but smaller, platforms to those constructed for drilling. The drilling rig climbed up and down the embankment under its own control, but Tirfor and cable were employed to aid the rig where traction was lost.

The process of drilling and installing the lime piles is illustrated in the flow chart in Figure 8. The diameter of each unlined lime pile was approximately 150mm. The lined lime piles were formed within a 165mm external and 150mm internal diameter PVC waterwell screen. For the lined piles a rotary auger of 178mm (nominal 7inch) diameter was used to enable the waterwell screen to be lowered by hand into the borehole. Both the lined and unlined piles were installed in a grid at 2m spacing between pile centres.

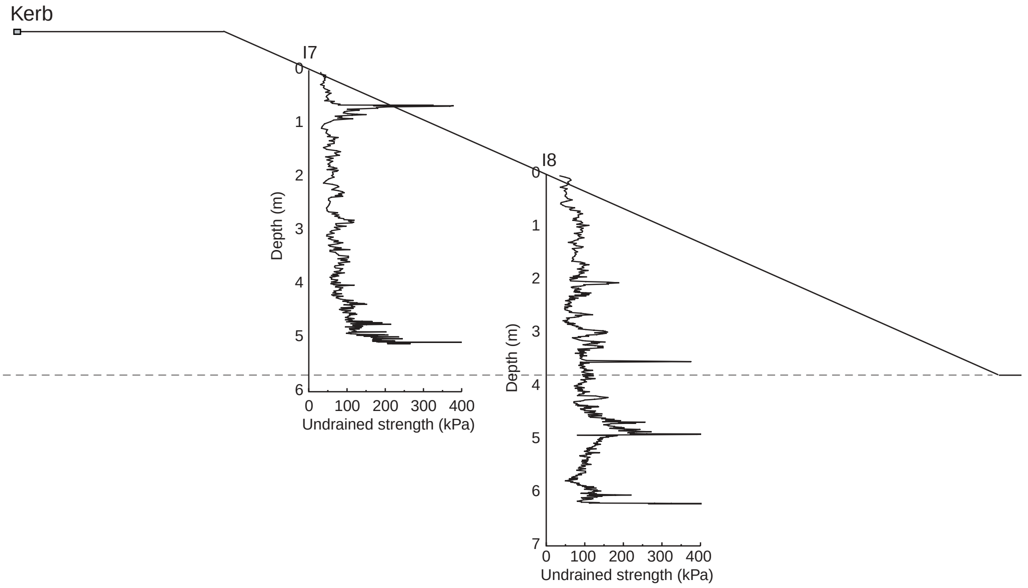
For the construction of both types of pile, granulated quicklime complying with BS 890 was used. When sieved, 100% by mass of the quicklime passed a BS 10mm sieve and at least 95% by mass passed a BS 6mm sieve. The reactivity of the lime when tested in accordance with BS 6463 was such that after 2 minutes it yielded a temperature of at least 50^{o}C. Suitable precautions were taken on site to ensure that the quicklime did not hydrate before use. Compaction of the quicklime within either the borehole or the waterwell screen in the borehole followed the same procedure. The lime was placed into the hole and compacted in approximately 300mm thick layers using a hand-held rammer with a mass of 3kg and a cross-sectional area of about 80% of that of the pile. The lime was compacted to within 400mm of the slope surface with the top of the hole being plugged with clay from the pile arisings.

Eight sacrificial piles (four unlined and four lined) to 1.4m depth were installed at various locations. Pairs of piles were subsequently exhumed 4, 10, 16 and 25months after completion of the construction to investigate the state of hydration of the lime.

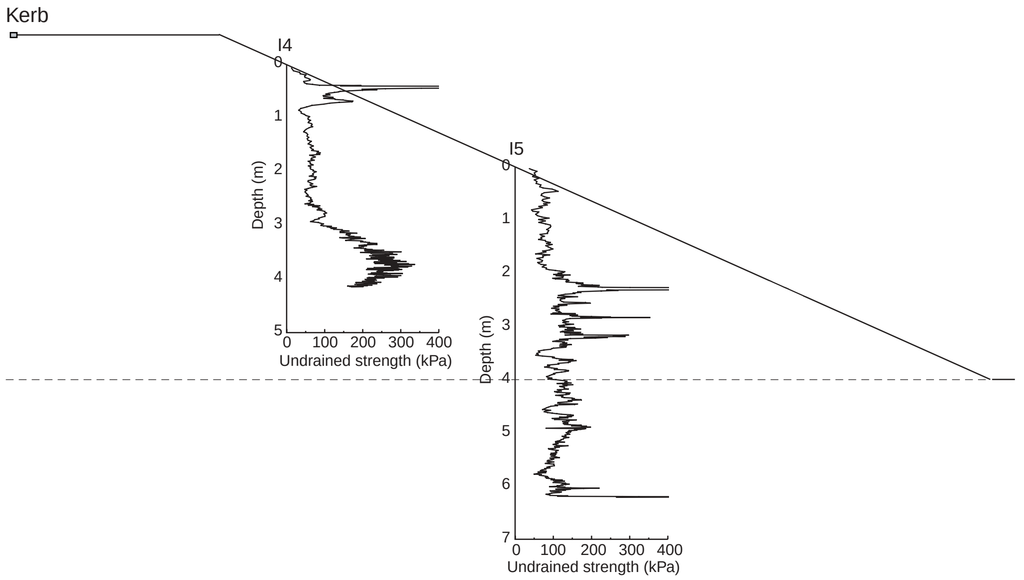
A breakdown of the component costs of lime pile installation at this site is given in Appendix A.



(a) Area 1: Unlined lime piles



(b) Area 2: Lined lime piles



(c) Control area

Figure 2 Panda penetrometer strength profiles at each area

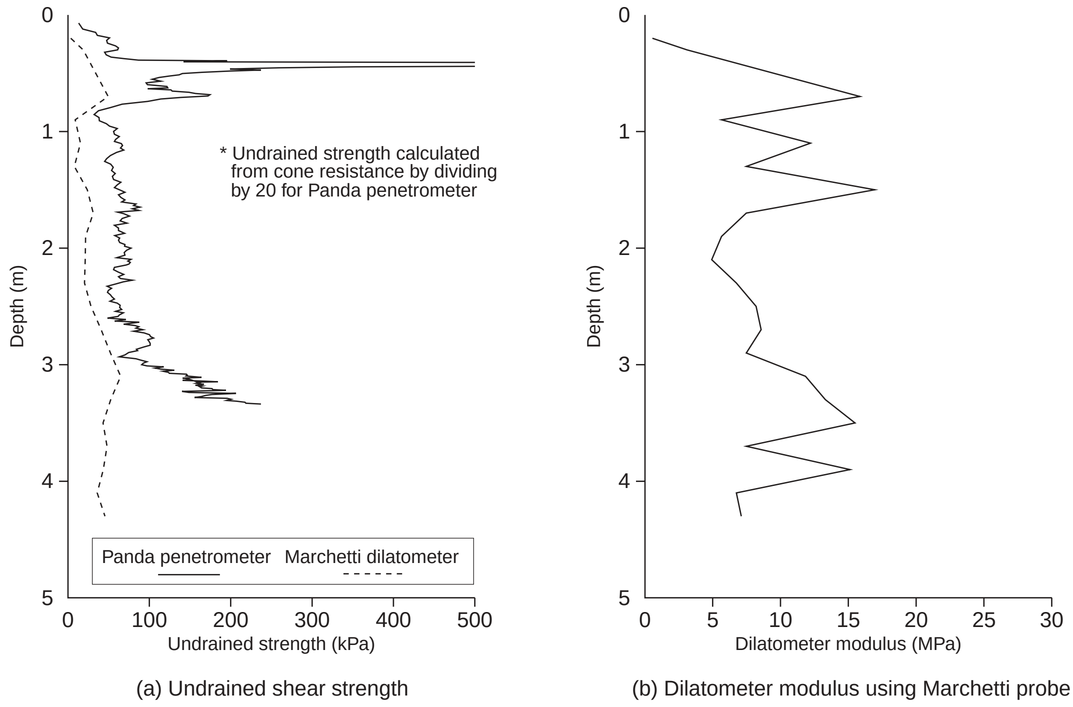


Figure 3 Soil profiling using Panda penetrometer and Marchetti dilatometer (near top of slope in control area)

Table 1 Schedule of construction activities and instrument installation

Date	*Day*	*Construction activity*	*Instrument installation*
15/02/1999	1	Setting out of all borehole locations, scaffolding delivered to site.	
16/02/1999	2	First half of lower lift of scaffolding completed, area 1.	
17/02/1999	3	Second half of lower lift of scaffolding completed, materials to site.	
19/02/1999	5	Lower lift scaffold (area 1) boarded out, storage container to depot.	
22/02/1999	8	Lower lift of scaffolding started on area 2, plant to site.	
23/02/1999	9	Area 1, Row 6; borehole 1 drilled but left open overnight.	
24/02/1999	10	Additional platform erected for power pack. Area 1, row 6; piles 1 to 9,1A & 1B.	
25/02/1999	11	Area 1, row 5; piles 1C,1D,10 to 7. Area 1, row 5; piles 6 to 3. Additional platform erected for power pack.	P10 to 12, I3
26/02/1999	12	Area 1, row 5; piles 2 & 1. Strike half of lower section of scaffolding (area 1), move to middle level.	
01/03/1999	15	Middle lift of scaffolding completed (area 1). Area 1, row 4; piles 1 to 9.	
02/03/1999	16	Area 1, row 3; piles 10 to 4. Commence upper lift of scaffolding, area 1.	P7 to 9, I2
03/03/1999	17	Area 1, row 3; piles 3 to 1. Area 1, row 2, piles 1 to 7. Commence upper lift of scaffolding, area 1. Board out top level of scaffolding, area 1.	
04/03/1999	18	Area 1, row 2; piles 8 & 9. Area 1, row 1; piles 10 to 7. Strike middle section of scaffolding (Area 1), move to middle level of area 2.	P1 to 6, I1
05/03/1999	19	Area 1, row 1; piles 6 to 1. Half of middle lift of scaffolding completed, area 2.	
08/03/1999	22	Move drilling rig from area 1 to area 2. Trial borehole for lined pile.	
09/03/1999	23	Commence upper lift of scaffolding, area 2.	P31 to 36, I8 & I9
10/03/1999	24	Strike scaffolding upper level on area 1. Erect upper and lower platforms on control area.	P25 to 30, I7
11/03/1999	25	Move drilling rig to control area.	P13 to 21, I4 & I5
12/03/1999	26	Excavate service trench (area 1). Continue erection of middle and upper platforms, area 2.	P22 to 24, I6
15/03/1999	29	Move drilling rig to area 2. Area 2, row 6; piles 1 to 9,2A & 2B. Excavate service trench (control area).	
16/03/1999	30	Area 2, row 5; piles 2C,2D,10 to 1.	
17/03/1999	31	Area 2, row 3; piles 1 to 10. Strike lower and complete middle platform, area 2.	
18/03/1999	32	Area 2, row 4; piles 9 to 1. Area 2, row 1; piles 1 & 2. Complete striking of lower and half of middle platform, area 2. Complete erection of upper level, area 2.	
19/03/1999	33	Area 2, row 1; piles 3 to 10. Area 2, row 2; piles 9 to 5. Clear area 1, strike middle platform on area 2.	
22/03/1999	36	No installation works, excavate service trench, area 2. Cast plinths on area 1 and control area. Continue to strike middle and upper platforms, area 2.	
23/03/1999	37	Area 2, row 2; piles 1 to 4. Cast plinth area 2. Scaffolding to top of embankment for collection.	
31/03/1999	45		Full set of readings

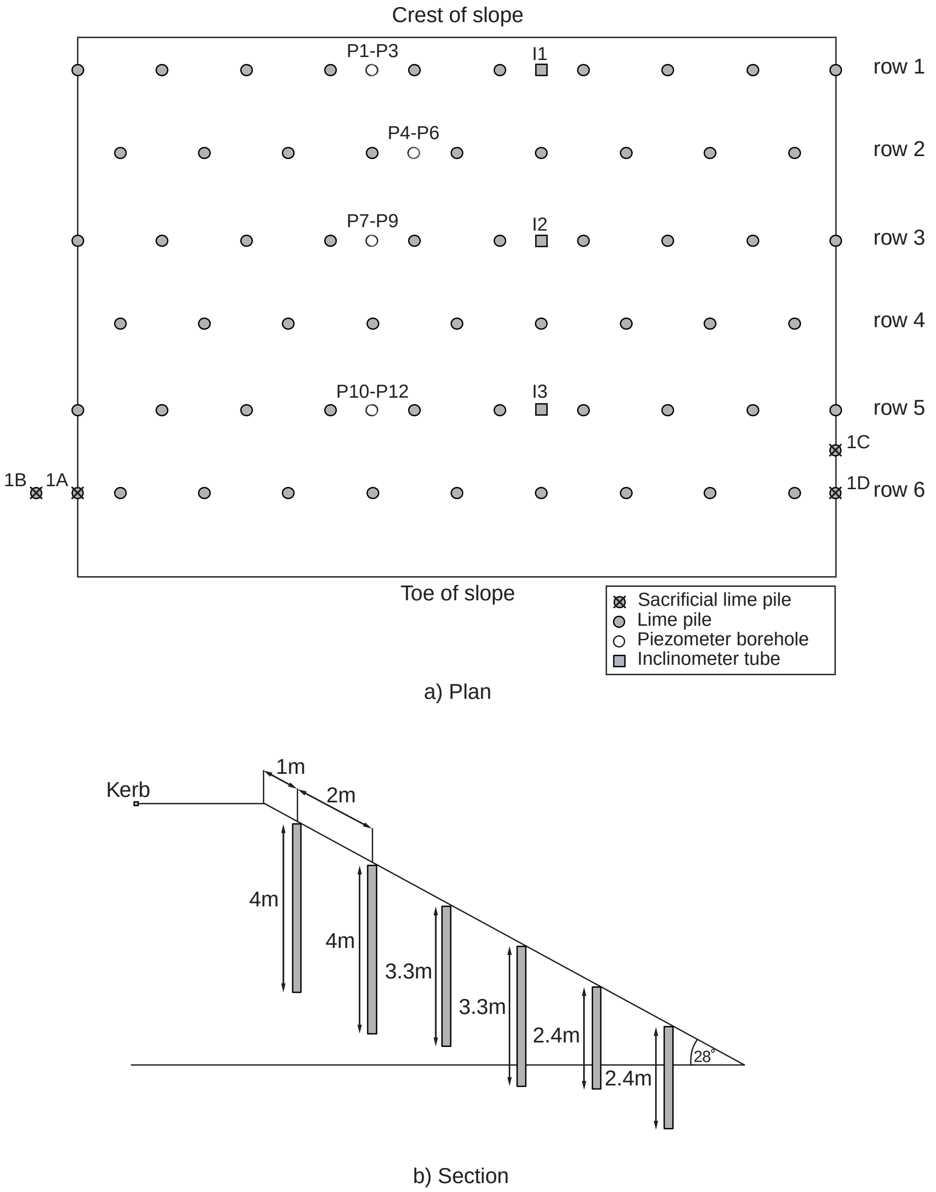


Figure 4 Plan and section views of area1: unlined lime piles

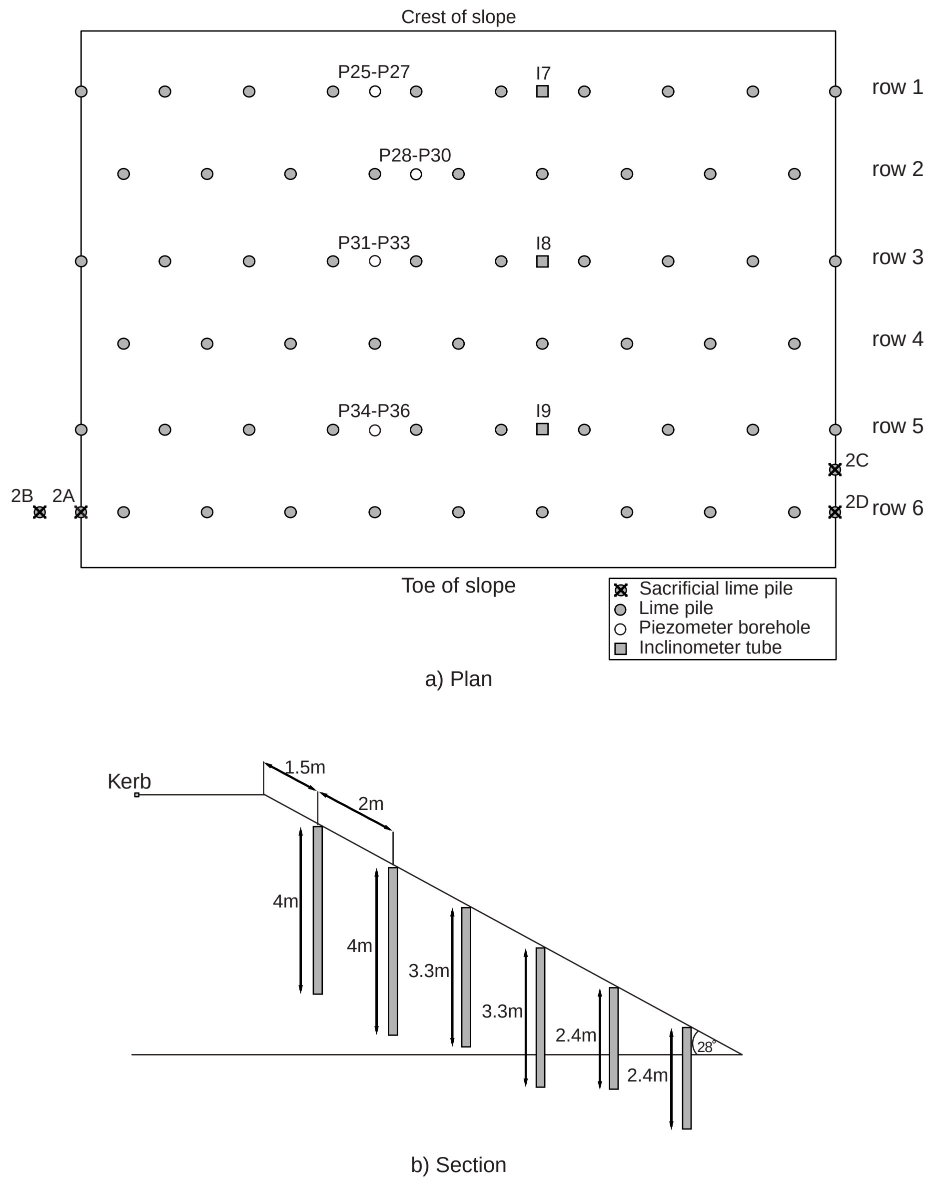


Figure 5 Plan and section views of area 2: lined lime piles

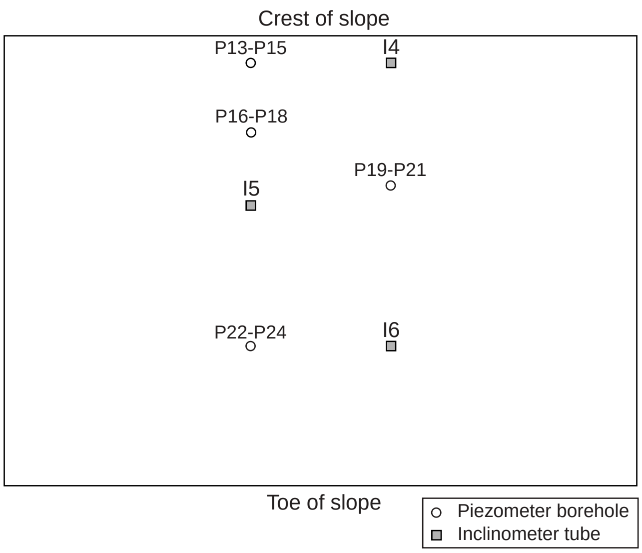


Figure 6 Plan view of control area

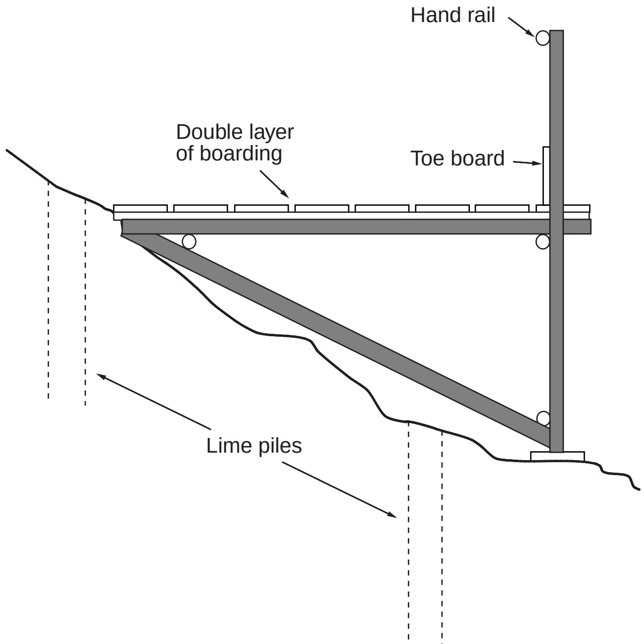


Figure 7 Typical scaffolding layout

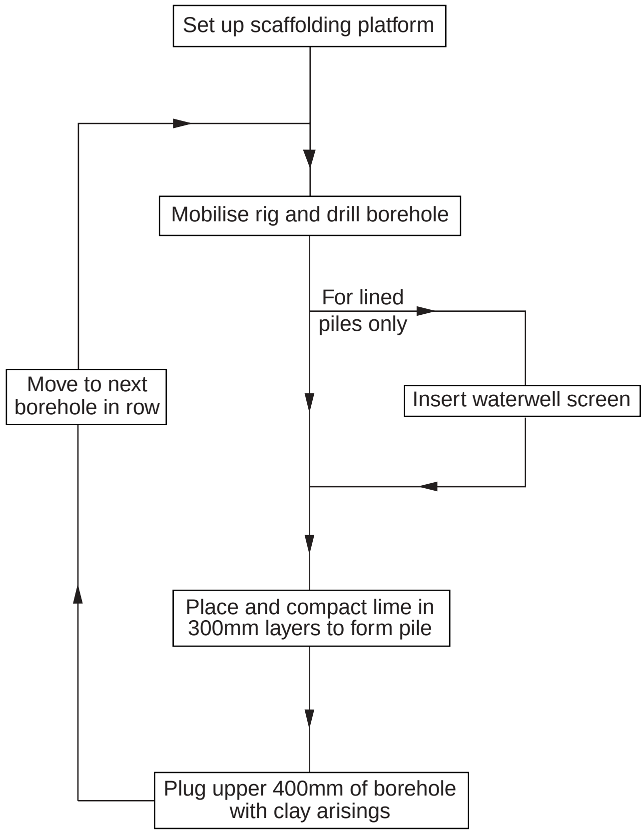


Figure 8 Drilling and pile installation process

5 Instrumentation

The instrumentation used on the slope was designed to monitor:

i pore pressures developed in the slope;

ii lateral movements of the slope;

iii vertical movements of the slope.

Plan views of the instrumentation layout have been shown in Figures 4 to 6. Section views showing the instrumentation within the slope in test area 1, area 2 and the control area are shown in Figures 9, 10 and 11 respectively.

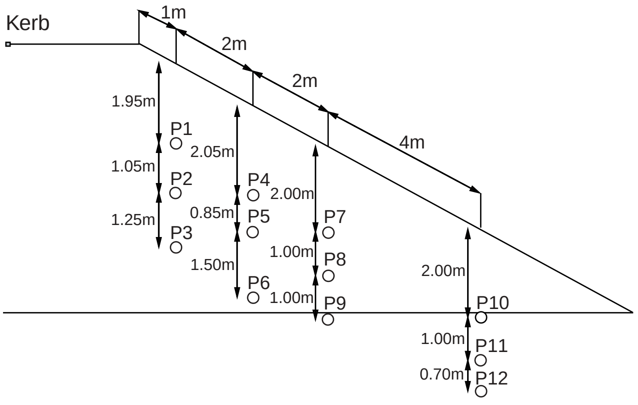
5.1 Pore water pressure measurements in the slope

The pore water pressures within each test area of the slope were monitored using a series of piezometers installed in boreholes. These boreholes were drilled at 2m intervals down the slope generally commencing at a distance of 1m from the top of the slope although, for site specific reasons, the first borehole for area 2 commenced at a distance of 1.5m. Three vibrating wire piezometers, with high air entry tips, were installed at approximately 2m, 3m and 4m depth in each borehole. Each tip was encased by a nominal 100mm long sand cell and the remainder of the borehole was impermeably sealed with bentonite pellets.

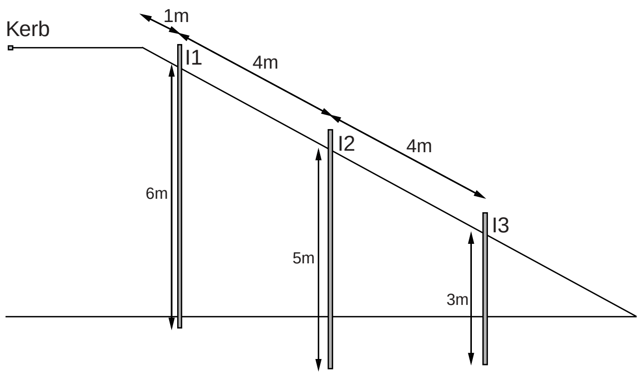
The pore pressures were measured using vibrating wire transducers incorporated in the piezometer tips. Because some suctions were anticipated in the slope, the piezometer tip also incorporated hydraulic twin tubes for de-airing purposes. All cables and tubes for the piezometers were ducted to

Plate 1 Scaffolding arrangement

Plate 2 Drilling rig operating on a platform

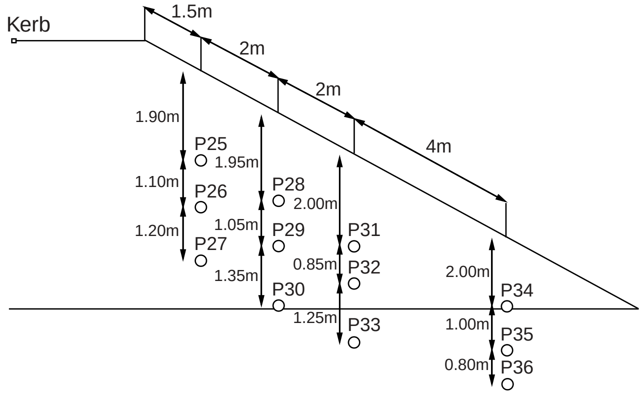


a) Piezometer layout

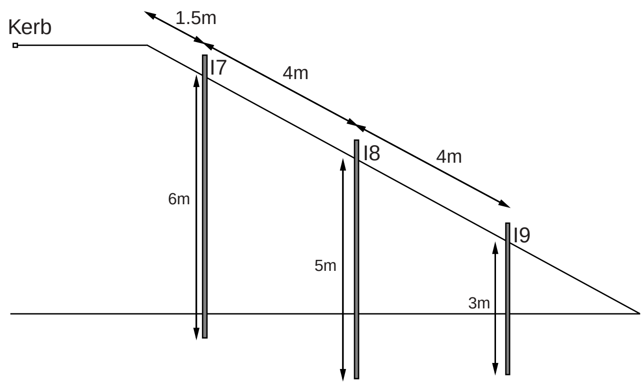


b) Inclinometer layout

Figure 9 Section showing instrumentation in area 1: unlined lime piles



a) Piezometer layout



b) Inclinometer layout

Figure 10 Section showing instrumentation in area 2: lined lime piles

instrument cabinets in each of the instrumented areas.

Resistance temperature devices were also incorporated in the piezometer housing so that the temperature of the ground at each tip location could be recorded.

5.2 Lateral movements of the slope

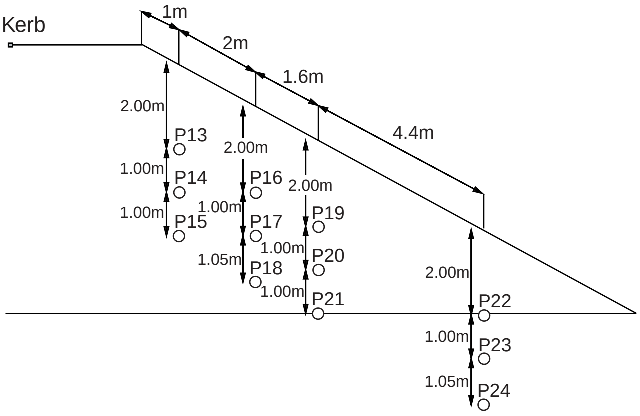
A proprietary sleeve jointed plastic inclinometer tube was installed at three locations within each test area of the slope. These tubes were generally positioned at 1m, 5m and 9m down the slope from the crest. The installation depth of each tube was as detailed in Figures 9, 10 and 11 for the different test areas. For ease of monitoring, each tube was installed so that approximately 0.5m of the tube was above the slope surface.

Inclinometer surveys were taken using a biaxial inclinometer torpedo at various stages during construction and in the longer term. Interpretation of the results enabled the lateral movements at each location to be determined assuming base fixity of the inclinometer tubes in the foundation material beneath the embankment.

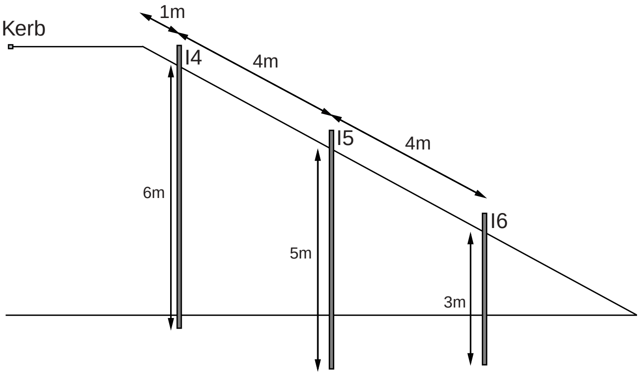
After 8 months of monitoring, there was some evidence of more deep-seated movements in test area 2 and for this reason further instrumentation was installed to independently measure surface lateral movement at the locations of each inclinometer tube. From the measurements, any movement of the base of each inclinometer tube could then be determined. This system was established during August 2000 and readings are available thereafter. These measurements were taken by mounting a high precision electronic distance measuring system (Geomensor) on a fixed pillar at about 240m away from the slope and sighting on a target reflector positioned in turn onto a machined socket installed on a short ground anchor within 0.5m of each inclinometer tube. The accuracy of the measurements using this system was considered to be ±0.5mm.

5.3 Vertical movements of the slope surface

In addition to measuring the lateral movements of the slope surface, the vertical movements of the slope surface were monitored by precise optical levelling (using an invar staff). For this purpose, the same machined sockets used for the Geomensor measurements were also employed as levelling stations. The precise levelling measurements commenced at the end of May 2000.



a) Piezometer layout



b) Inclinometer layout

Figure 11 Section showing instrumentation in control area

6 Observations of pore pressures in the slope

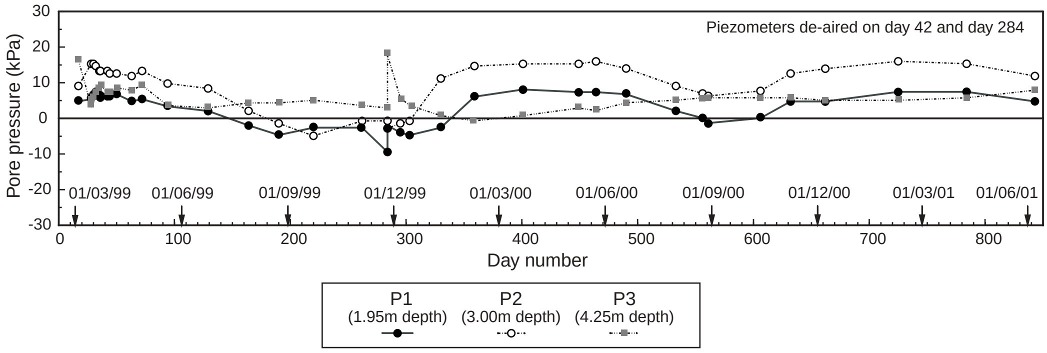
During the construction period and the following twenty-six months, monitoring of the vibrating wire piezometers took place at regular intervals, so that the development of pore water pressures in the slope could be identified. Figures 12, 13 and 14 show the results obtained for the test areas with unlined and lined lime piles and the control area respectively.

Generally the piezometer readings stabilised within 5 days or so after their installation and showed pore pressures at the end of lime pile construction in the range of +20kPa to –10kPa. As the construction work was completed in March when pore pressures were expected to be near their seasonal maxima, the piezometers were de-aired on day 42 to ensure that the magnitude of any suctions recorded in the summer months were not affected by air in the sealed hydraulic lines. A further de-airing was carried out on day 284 although little change in the pore water pressures was detected. At each of the instrumented areas the trend of the results confirmed that pore pressures decreased in the drier summer months of 1999 and 2000 with a tendency for small suctions to develop on the shallowest piezometers at 2m depth, especially those towards the middle and toe of the slope. Monthly rainfall totals from a meteorological station about 3km to the south of the site are shown in Figure 15 and changes in pore pressures can be broadly correlated with them. It is noticeable that April and May 2000 were particularly wet months and pore pressures generally remained slightly positive. Small rises in pore pressures were also recorded, especially on the shallowest piezometers, associated with the particularly wet winter and following spring of 2001.

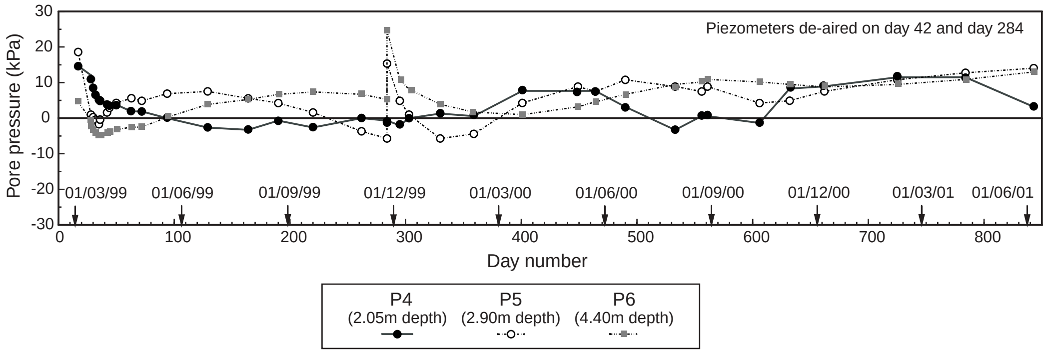
Comparisons of the variation in pore pressures with depth for the different test areas are shown for days 129 (June 1999) and 190 (August 1999) in Figures 16 and 17 respectively. Figures 16a and 16b indicate that pore pressures in the control area at both 1m and 3m down the slope were higher than in the treated areas. This was not unexpected, as short term drying out of the slope is likely as water is absorbed to hydrate the quicklime. Pore pressures at 5m down the slope in the lined lime pile area were considerably below those measured in the other areas (Figure 16c), whilst the pressures near to the toe of the slope shown in Figure 16d were similar in all cases. By day 190 (August 1999), the results in Figure 17 show that suctions were developing at shallow depths and the results were more variable in nature. Figures 18 and 19 show similar profiles of results plotted at day 561 (August 2000) and day 725 (February 2001) when pore pressures were approximately at a minimum and maximum respectively. No consistent pattern in the differences between the pressures in the treated and control areas could be identified and it was concluded that by the second year in service the hydration of the lime piles was effectively complete and no further water was being absorbed from the slope.

A closer examination of the distribution of pore pressures within each test area was carried out by contour plotting the results. Figures 20, 21 and 22 illustrate the seasonal development of pore pressure in the slope within the unlined and lined lime pile areas and the control area respectively. Generally the contour plots confirm the trends described earlier. In all areas the pore pressures reduced significantly during the summer of 1999 (e.g. day 190) with suctions generally developing in the slopes to depths of just over 2m and to greater depths near to the slope toes. Comparison of contour plots in the remediated and control areas also illustrate that at day 190 a pore pressure contour of magnitude 10kPa could only be plotted for the latter. This tends to suggest that 5 months after construction, hydration of the quicklime in the remediated areas was still having some effect in drying out the slope as well as providing support through a dowelling action. This is not unexpected as laboratory tests carried out by Brookes *et al.* (1997) showed that some strength gain due to the chemical reaction and hydration of the lime was still occurring after a similar period. By day 449 a build-up in the positive pore pressures was evident in all areas (Figures 20e, 21e and 22e) and the contours of 10kPa were far more extensive. This correlated with the period of heavy rainfall experienced in April and May 2000. At this time there was also little difference in the pore pressure contours determined in the remediated and control areas, which confirmed that absorption of water by hydration of the lime was virtually complete.

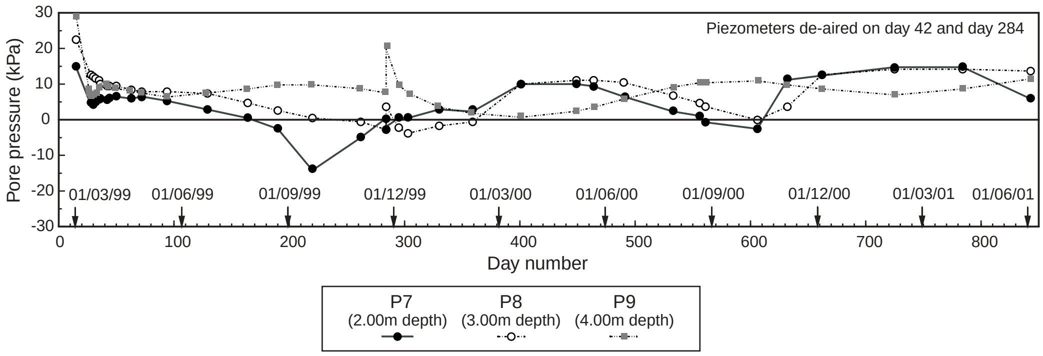
Pore pressure contours on day 561 (August 2000) showed the seasonal drying out in all three instrumented areas, whilst those for day 725 (February 2001) showed the highest pore pressures measured at this site after a prolonged period of heavy rain.



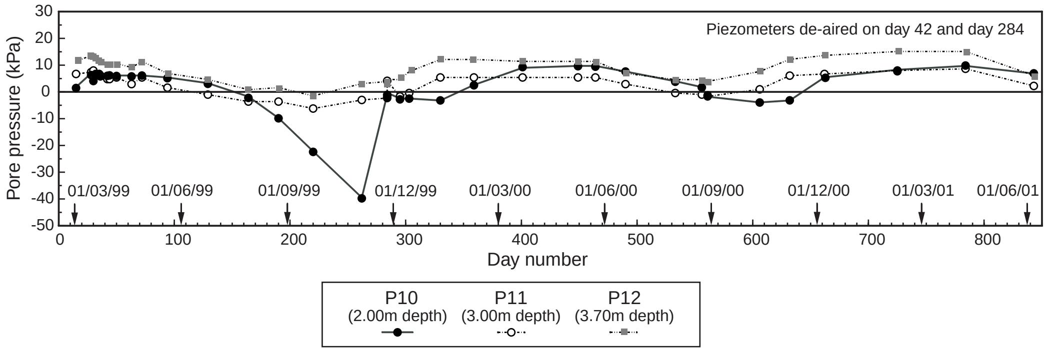
(a) 1m down the slope



(b) 3m down the slope

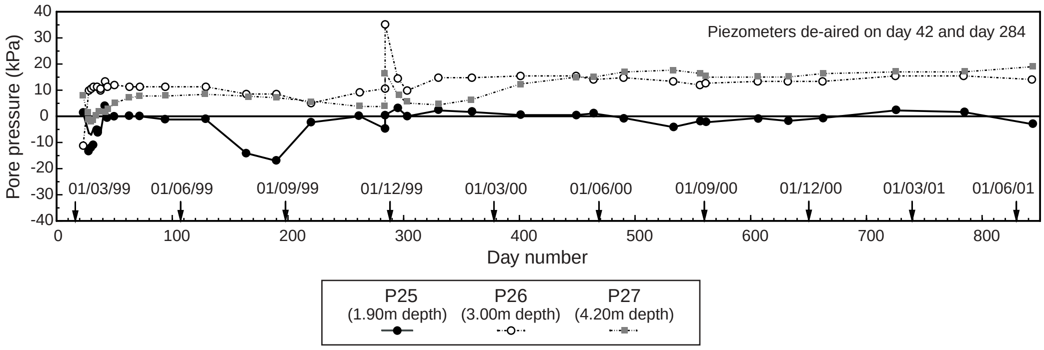


(c) 5m down the slope

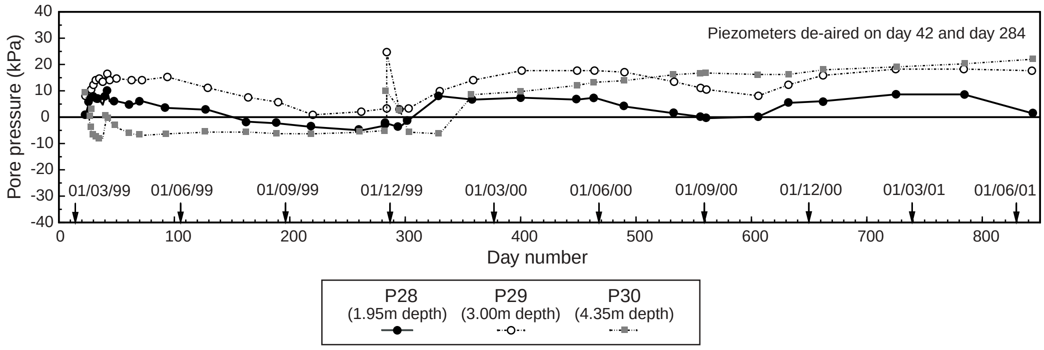


(d) 9m down the slope

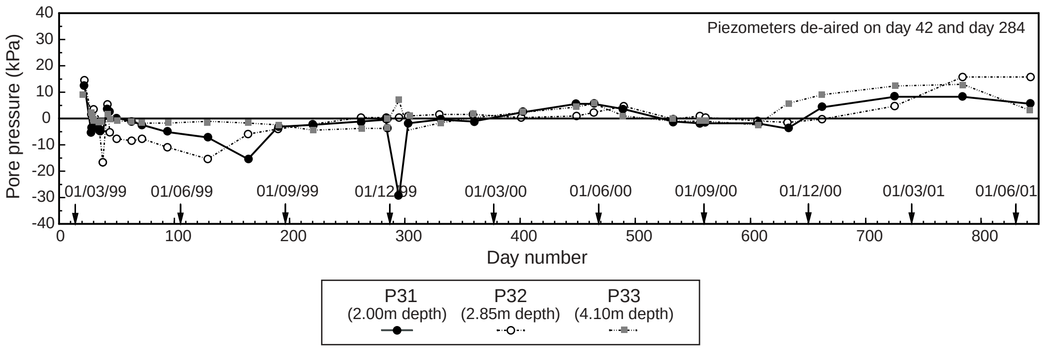
Figure 12 Variation of pore pressure with time: unlined lime piles



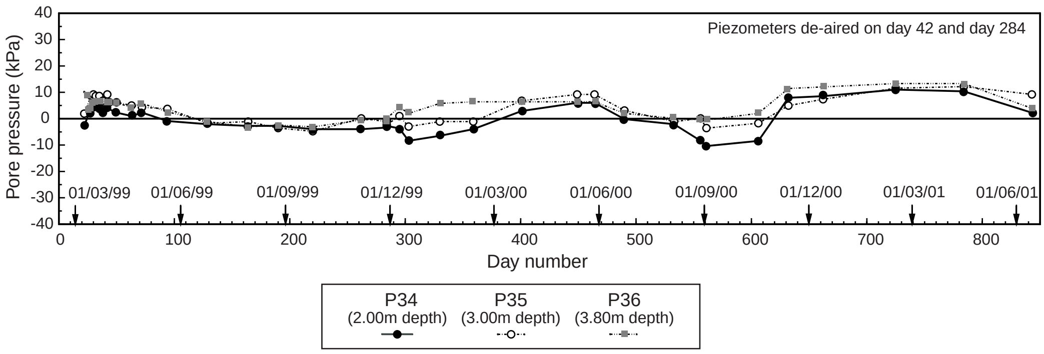
(a) 1m down the slope



(b) 3m down the slope

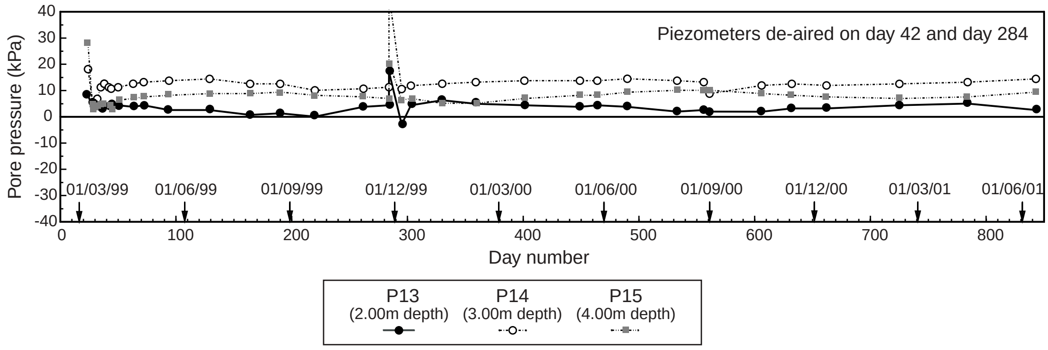


(c) 5m down the slope

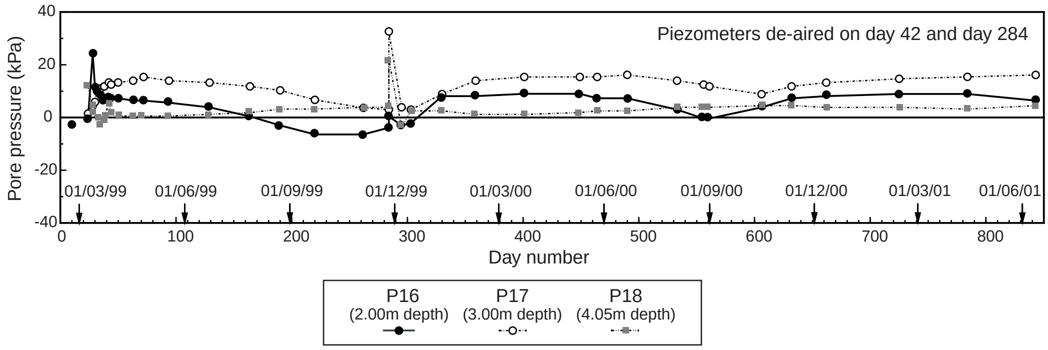


(d) 9m down the slope

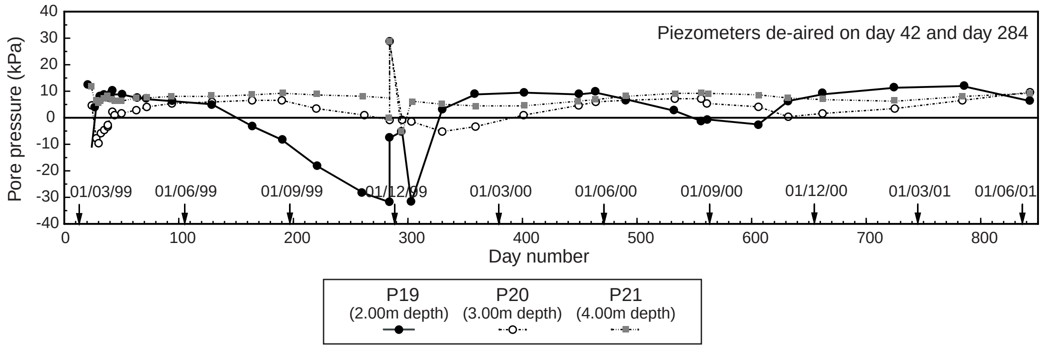
Figure 13 Variation of pore pressure with time: lined lime piles



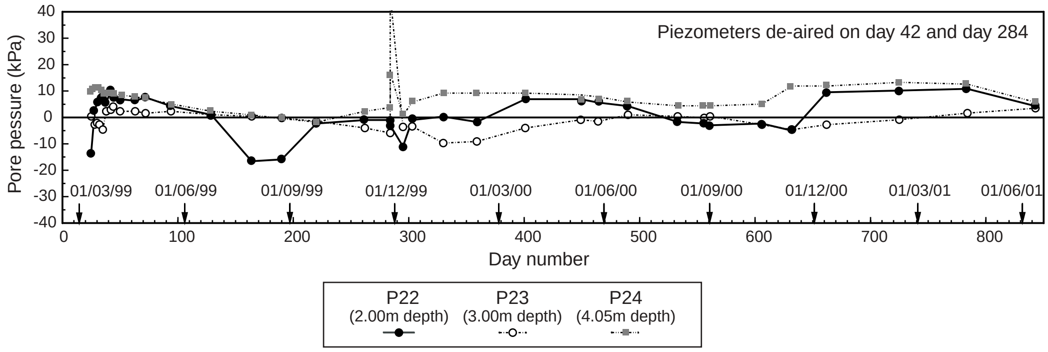
(a) 1m down the slope



(b) 3m down the slope



(c) 5m down the slope



(d) 9m down the slope

Figure 14 Variation of pore pressure with time: control area

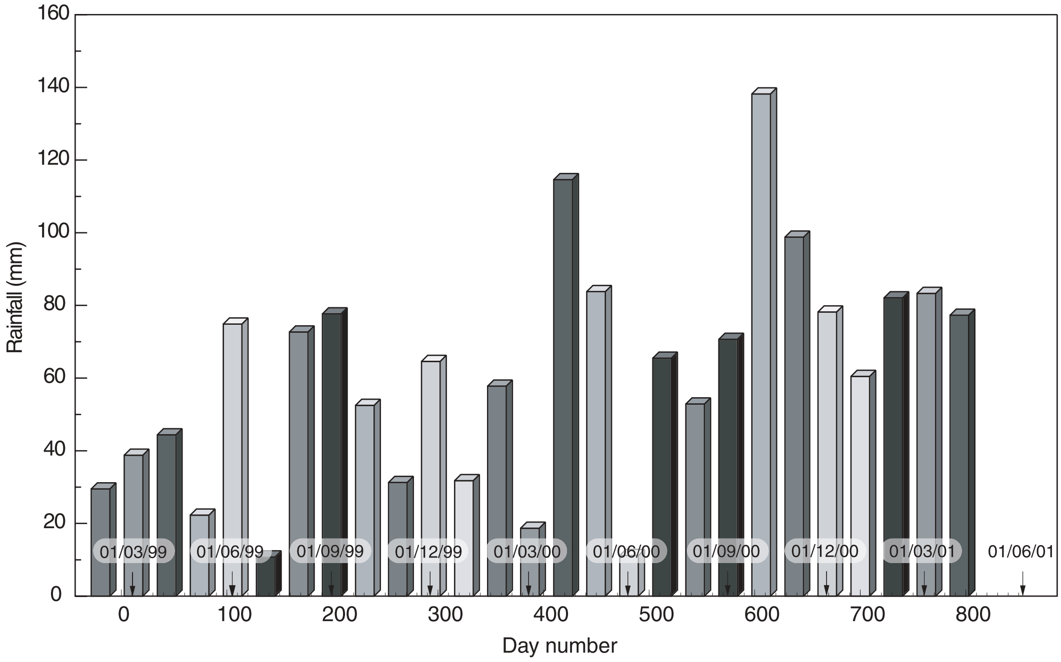
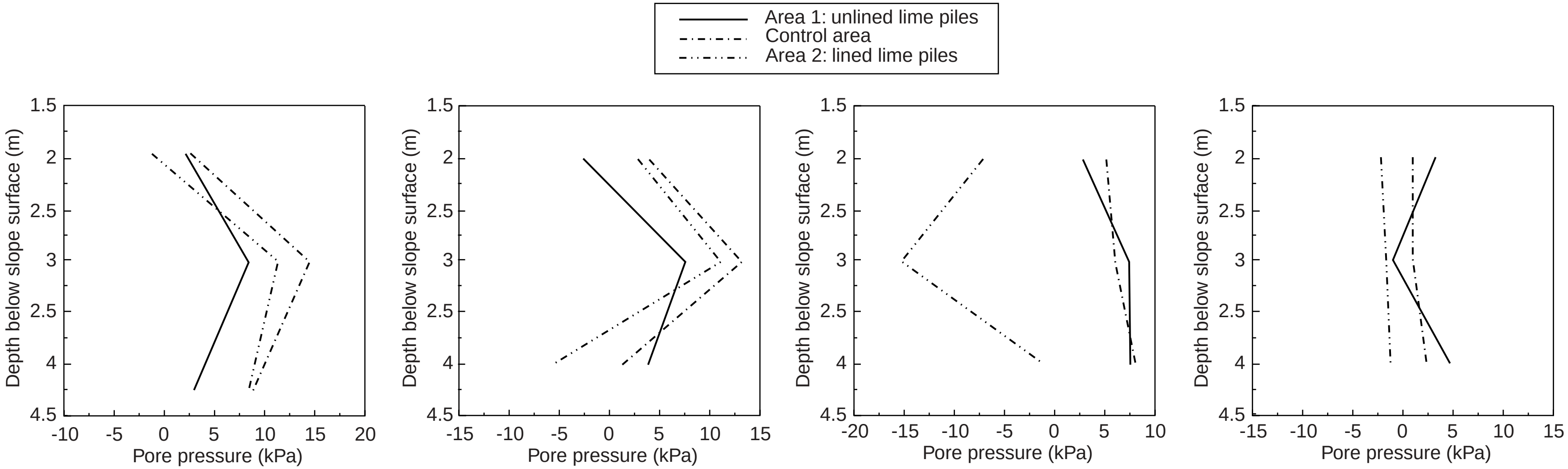
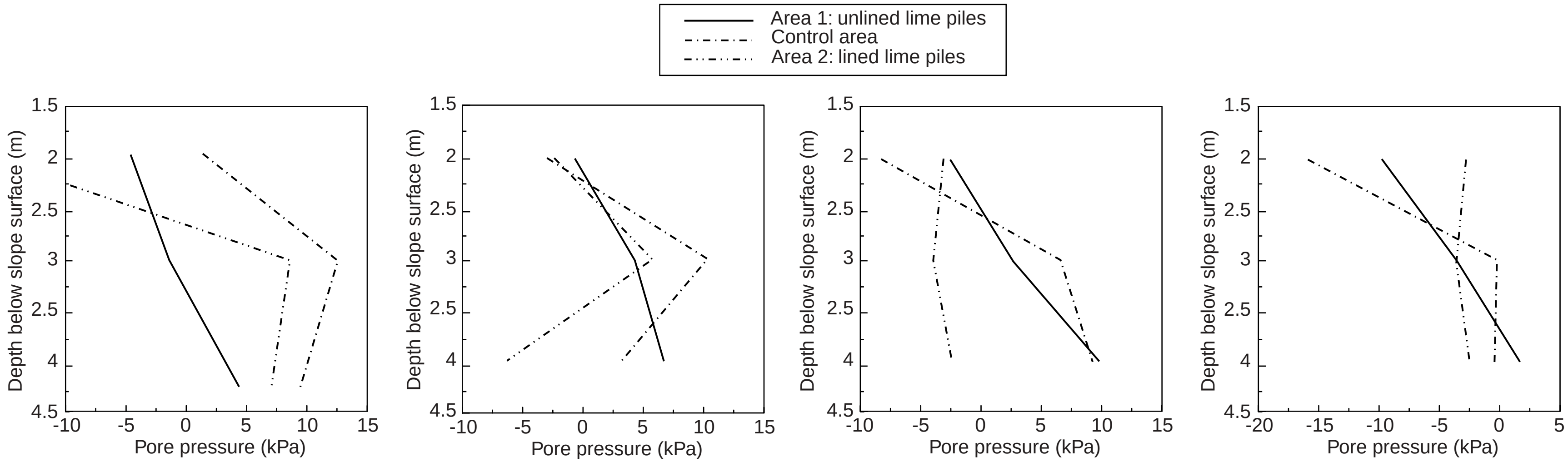


Figure 15 Monthly rainfall data from meteorological station 3km to the south of the site



(a) 1m down the slope (b) 3m down the slope (c) 5m down the slope (d) 9m down the slope

Figure 16 Variation of pore pressures with depth at day 129 (June 1999)



(a) 1m down the slope (b) 3m down the slope (c) 5m down the slope (d) 9m down the slope

Figure 17 Variation of pore pressures with depth at day 190 (August 1999)

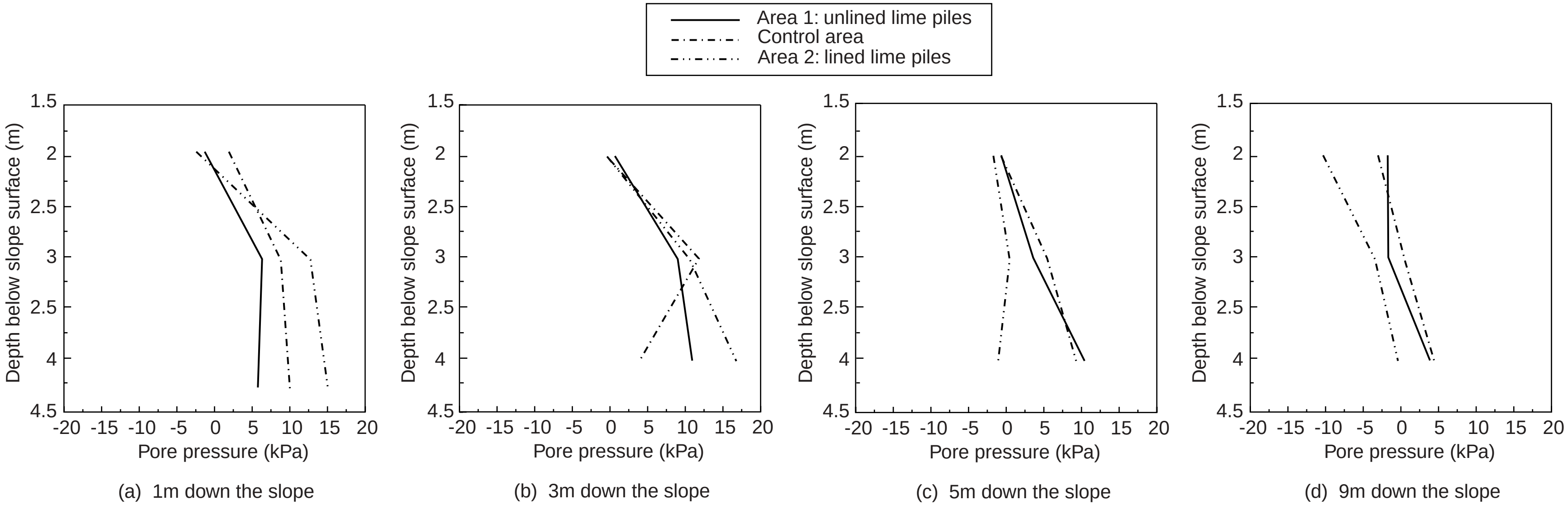


Figure 18 Variation of pore pressures with depth at day 561 (August 2000)

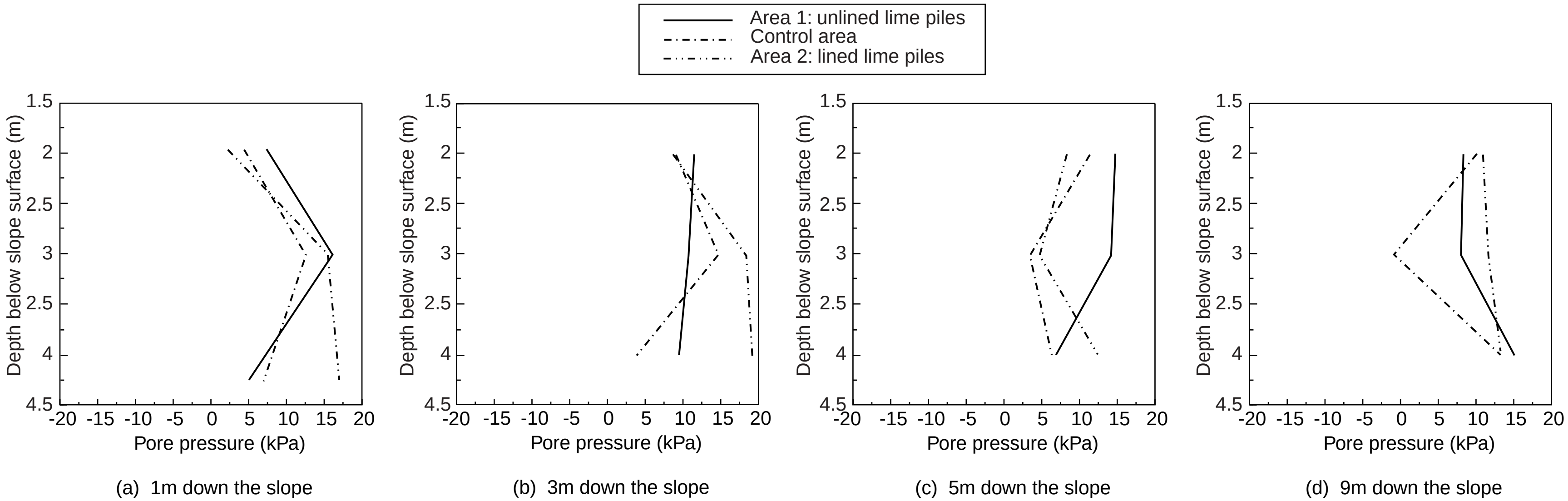
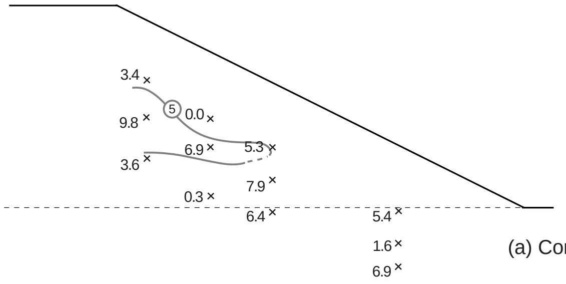
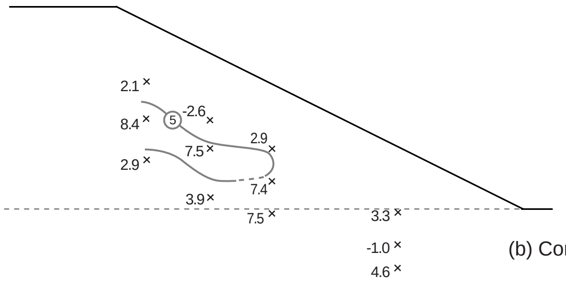


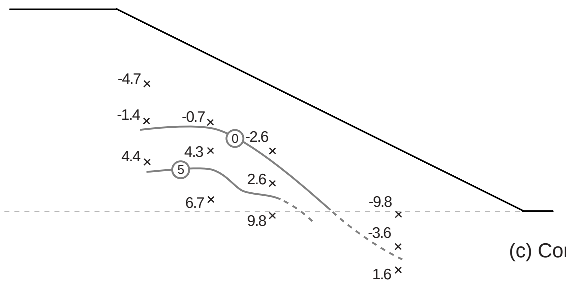
Figure 19 Variation of pore pressures with depth at day 725 (February 2001)



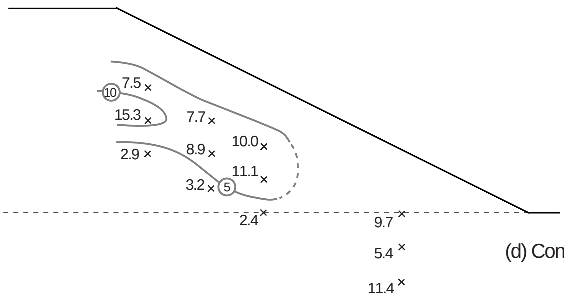
(a) Contour plot for day 94 (May '99)



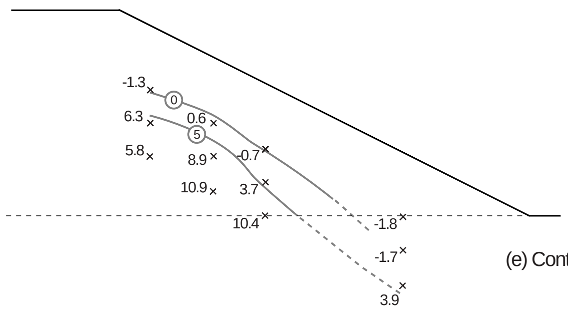
(b) Contour plot for day 129 (June '99)



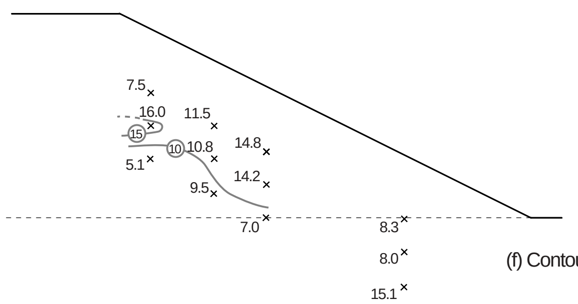
(c) Contour plot for day 190 (August '99)



(d) Contour plot for day 449 (May '00)

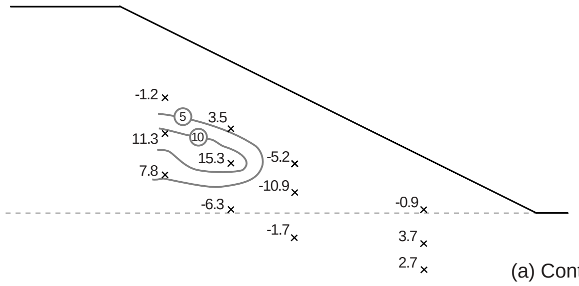


(e) Contour plot for day 561 (August '00)

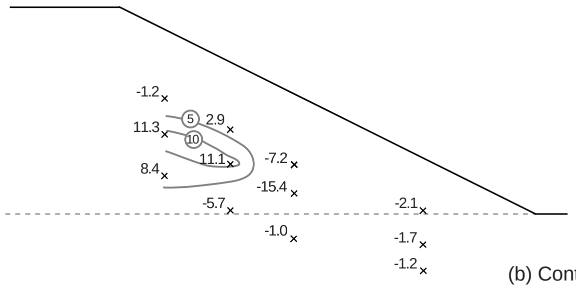


(f) Contour plot for day 725 (February '01)

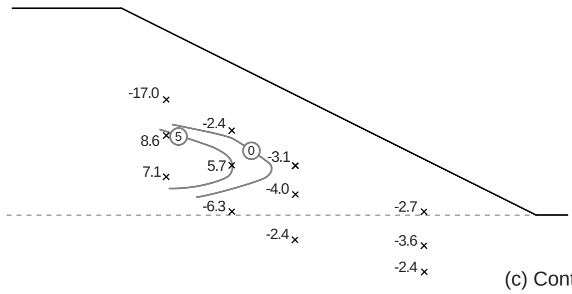
Figure 20 Contour plot of pore pressures within area 1: unlined lime piles



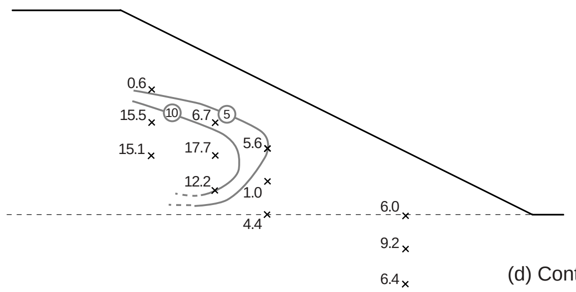
(a) Contour plot for day 94 (May '99)



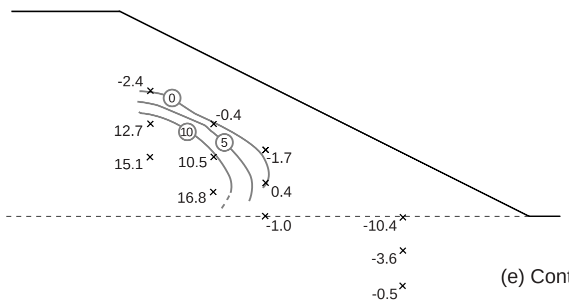
(b) Contour plot for day 129 (June '99)



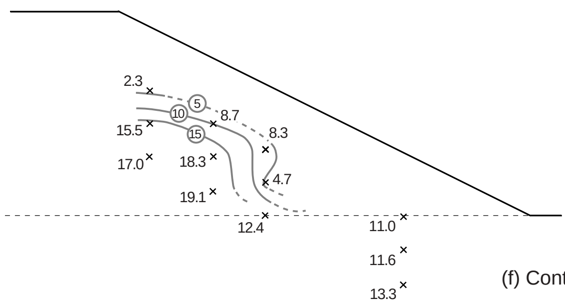
(c) Contour plot for day 190 (August '99)



(d) Contour plot for day 449 (May '00)



(e) Contour plot for day 561 (August '00)



(f) Contour plot for day 725 (February '01)

Figure 21 Contour plot of pore pressures within area 2: lined lime piles

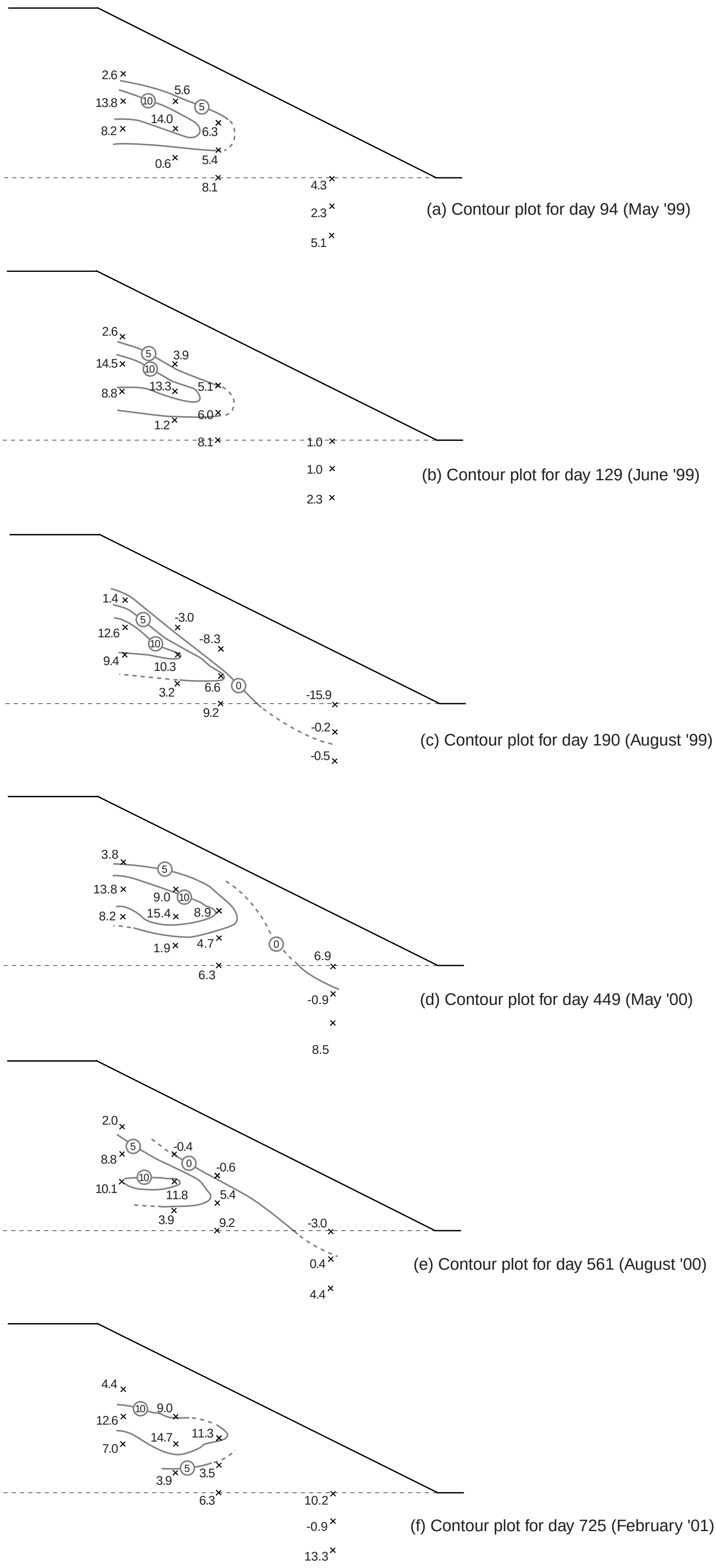


Figure 22 Contour plot of pore pressures within control area

7 Observations of movements with time

7.1 Lateral movements

Inclinometer surveys during construction showed only small movements of generally less than 2mm and for this reason a datum for all the readings was established on March 23rd 1999 when all the construction work was finished. In all cases the lateral movement profiles were determined from the inclinometer tubes by assuming base fixity of each of the tubes. This was considered a reasonable initial assumption as the inclinometer tubes were founded in the soil foundation below the embankment.

The development of lateral movement of the slope measured by inclinometer after completion of construction is shown in Figures 23, 24 and 25 for the unlined and lined lime pile areas and the control area respectively. Some similarities were observed in the trends of movement in all three cases with a seasonal cycle of clay swelling and shrinkage being recorded associated with wetting and drying. The dates that have been plotted in Figures 23, 24 and 25 have been selected to illustrate the seasonal minima and maxima of movement. At the M1 site, the inclinometer datum readings were actually taken during March when swelling would be at a maximum and for this reason shrinkage movement tends to dominate over the first 8 months. As a result between March and November 1999, surface lateral movements of up to about 13mm (towards the centre of the embankment) were measured on the inclinometer tubes mid-slope and towards its toe. Following this, swelling of the clay occurred with outward surface movements of about 10mm in the control and area 1, and about 20mm in area 2, being measured during spring 2000 at the same locations. Further seasonal cycles of swelling and shrinkage of the clay slope then continued.

Generally ground movements were most marked at depths of up to 2m. With the exception of the deep seated movement in area 2 which is discussed below, little movement was measured below this depth. This pattern of behaviour is consistent with the seasonal swelling and shrinkage of the near-surface clay measured at other clay slopes (Crabb and Hiller,1993).

When the lateral movements at the slope surface and at 2m depth are plotted as shown in Figures 26 and 27, it is apparent that movements in the unlined pile and control areas are very similar with considerable lateral swelling and shrinkage at the surface but scarcely any at a depth of 2m. The results for the lined lime piles also show seasonal surface movement but, as is also evident from Figure 24, this mechanism is masked because some continuing outward movement of the clay slope is occurring which is more deep seated. Near to area 2 (lined piles), a significant movement had occurred some years ago which caused the nearby motorway lighting column to tilt about 9^o towards the slope. Although the slope had since appeared to restabilise, these results suggest that some deep-seated movement continued up to about October 2000 although that movement has now slowed. The design of the lime piles was only intended to be effective in arresting shallow failures and whether the dowelling action of the piles in area 2 also served to limit the more deep-seated movements is not clear.

In both of the test and the control areas, Figure 26 shows an overall trend of outward lateral movement of the surface with time whereas the same effect is not so evident at 2m depth. Whether this movement is related to progressive softening of the surface which may eventually lead to shallow failures or due to long term consolidation of the embankment clay is uncertain. Further monitoring over perhaps a decade would be required to confirm the mechanism and to fully ascertain the effectiveness of lime piles in preventing subsequent shallow failures.

Also shown in Figure 26 are limited data obtained using the electronic distance measuring system (Geomensor) sighting from a reference pillar about 240m away in a neighbouring field, as described in Section 5.2. These measurements were only available from August 2000 and broadly confirmed the changes in movement determined from the inclinometer tubes assuming base fixity of the tubes. However during November 2000 excavation of a drainage ditch in the field adjoining the reference pillar adversely affected its stability and results were no longer meaningful.

7.2 Vertical movements

Precise levelling on stations located at the ground surface immediately adjacent to each inclinometer tube commenced in May 2000. The results, which for comparative purposes have been plotted to the same scale as the lateral movements in Figure 26, are shown in Figure 28. In both these figures negative movement represents swelling of the clay and the trends of the changes in lateral and vertical movement are very similar. Comparisons of the relative magnitudes of the lateral and vertical movements over the period in which levelling measurements are available are shown in Table 2.

In Table 2, the resultant movement and its angle with the vertical have been calculated. As the slope angle is 28^o for all three areas, an angle determined from $\tan^{-1}(L/V)$ of the same value would mean that the resultant acts normally to the slope surface. The results show that the swelling is approximately normal to the slope in area 1 (unlined lime piles), whilst a greater angle of between 42^o and 57^o in the control area implies more lateral swelling. On this basis, it is tentatively suggested that the piles may be providing some lateral restraint. Because of the large number of variables, this suggestion is by no means certain particularly as the magnitude of the resultant is slightly larger in the piled area. The same logic cannot be applied to area 2 (lined lime piles) as results are more scattered probably due to the deep-seated movement which is occurring.

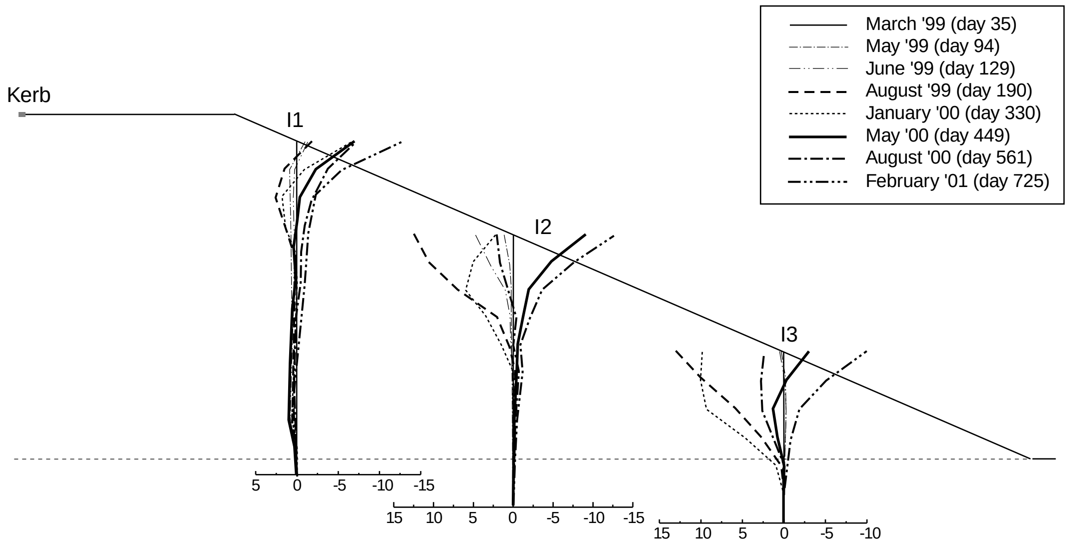


Figure 23 Development of lateral movement (mm) in area 1: unlined lime piles

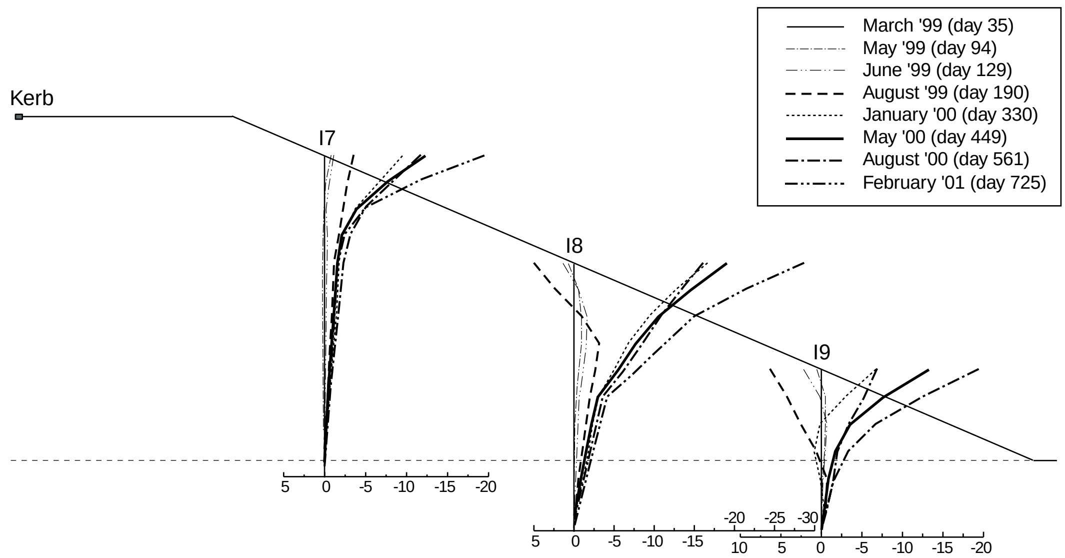


Figure 24 Development of lateral movement (mm) in area 2: lined lime piles

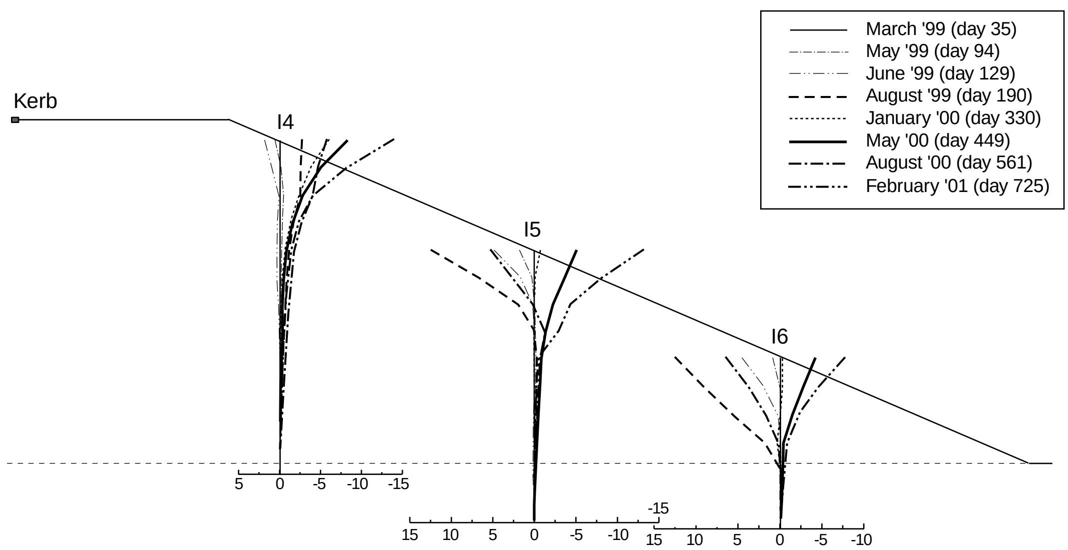


Figure 25 Development of lateral movement (mm) in control section

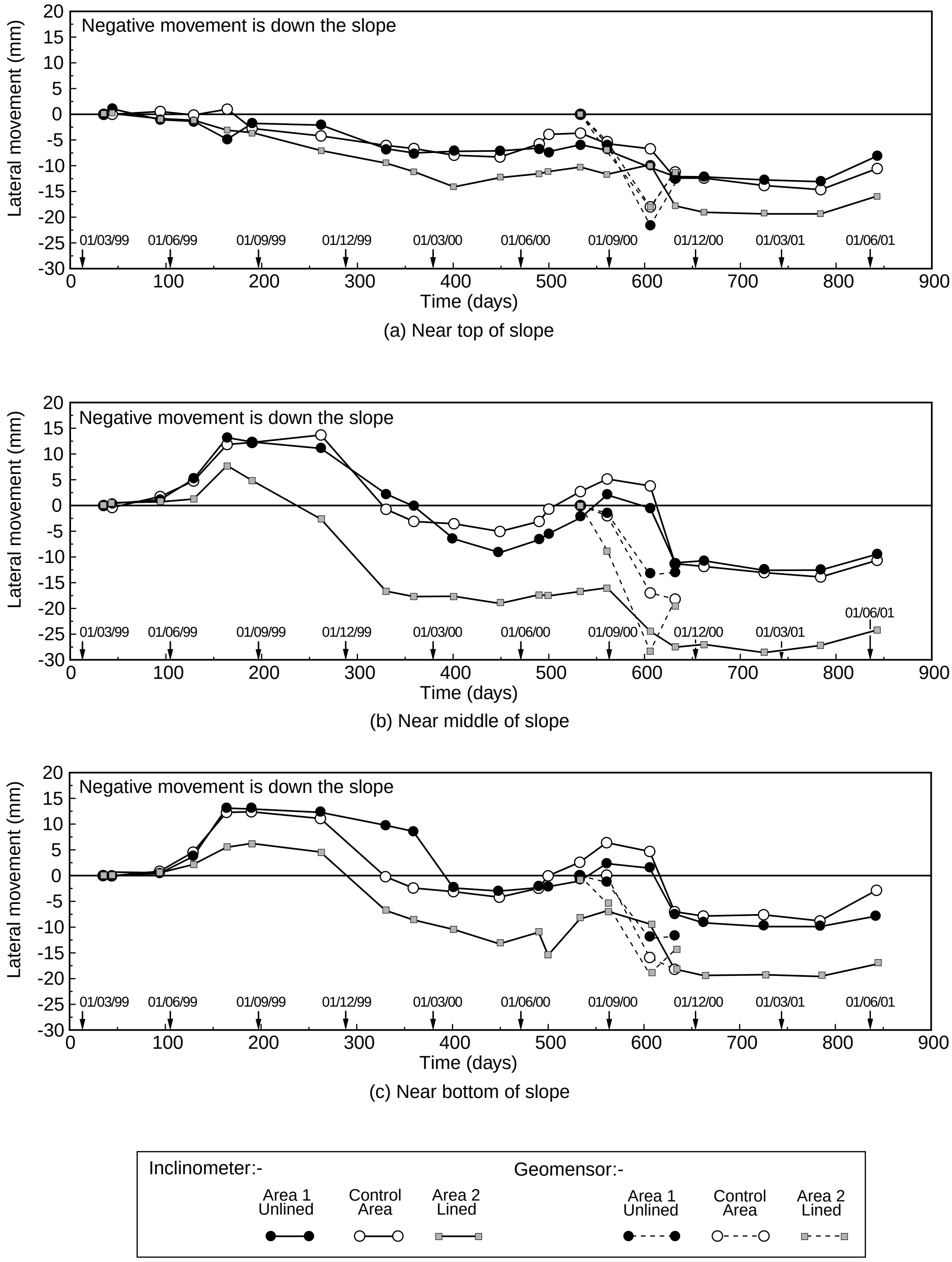
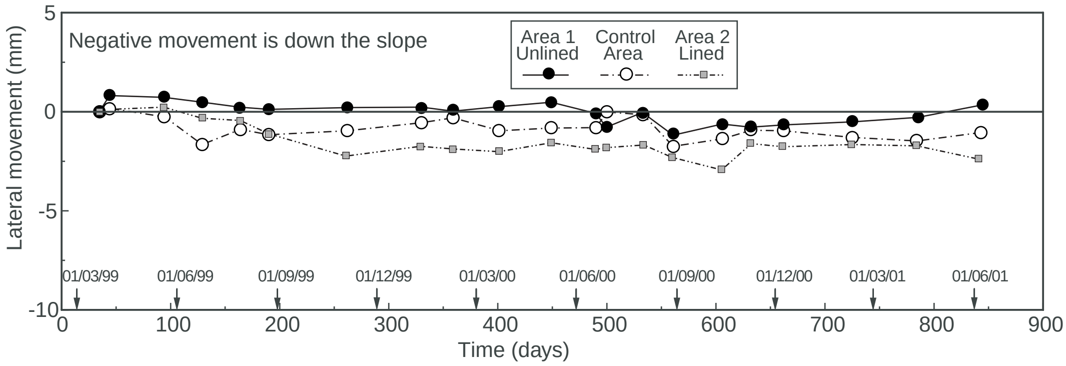
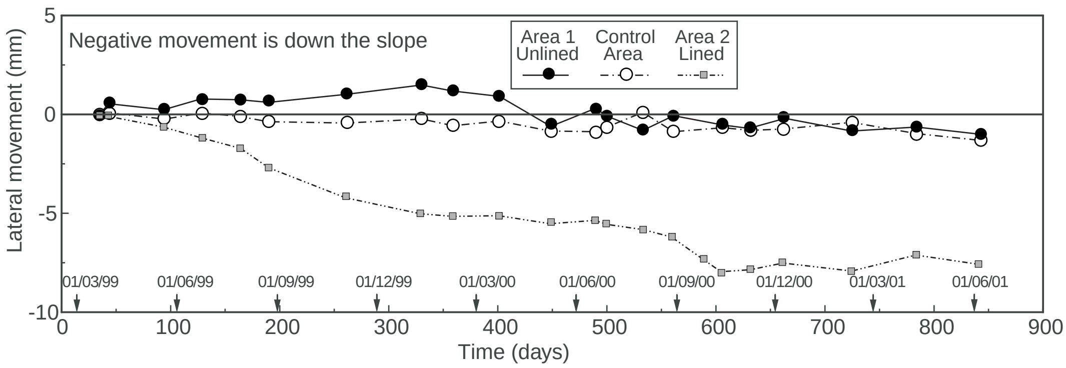


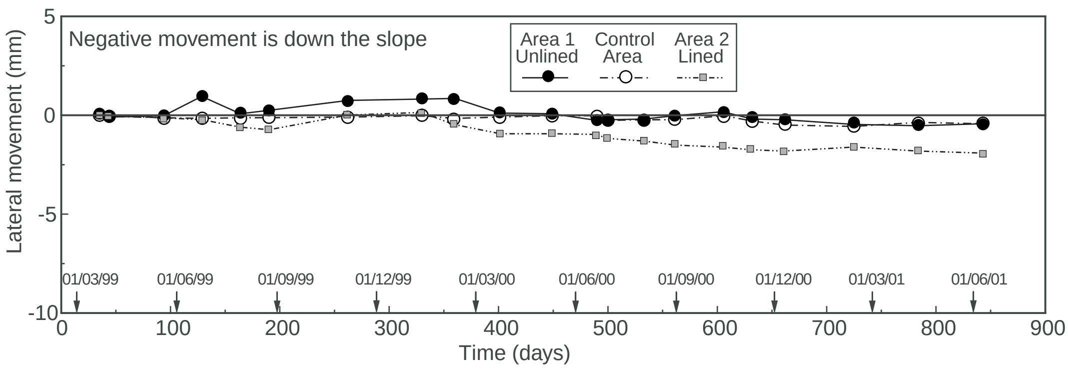
Figure 26 Variation with time of lateral movement at the slope surface



(a) Near top of slope

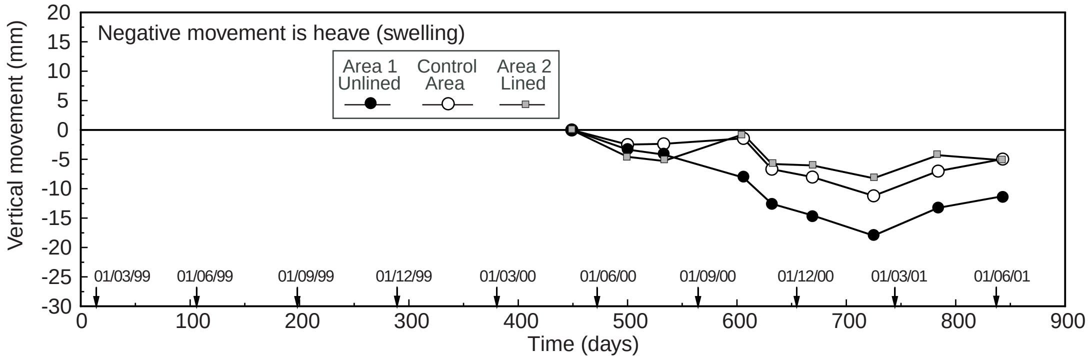


(b) Near middle of slope

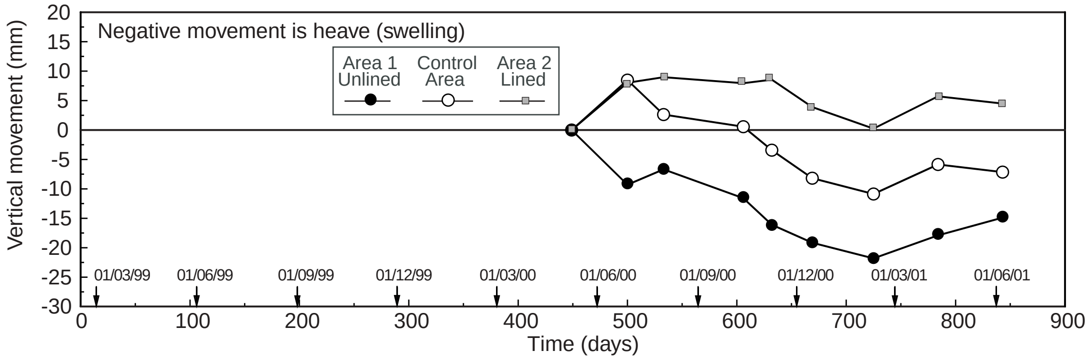


(c) Near bottom of slope

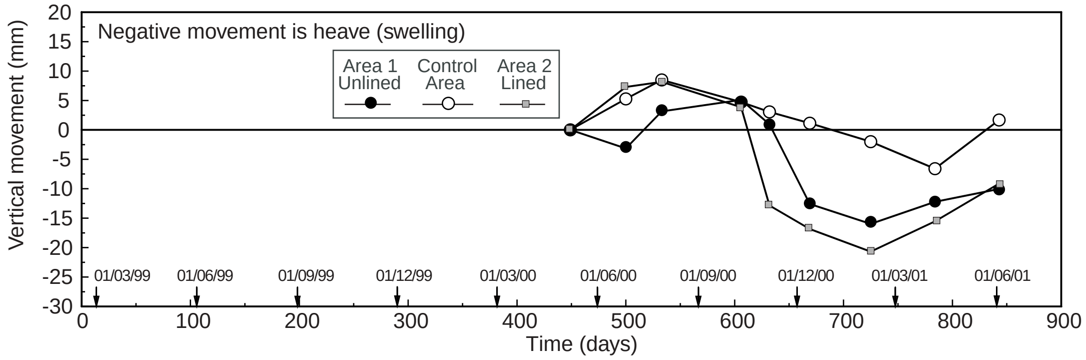
Figure 27 Variation with time of lateral movement at 2m below the slope surface



(a) Near top of slope



(b) Near middle of slope



(c) Near bottom of slope

Figure 28 Variation with time of vertical movement at the slope surface

Table 2 Comparison of the lateral and vertical movements

Location	Lateral movement, L (mm) Day 449	Day 782	Change	Vertical movement,V (mm) Day 782	Resultant movement (mm)	$Tan^{-1}(L/V)$
Area 1						
Top	-7.10	-13.10	-6.00	-13.24	-14.54	24º
Middle	-9.15	-12.55	-3.40	-17.91	-18.23	11º
Bottom	-3.00	-9.90	-6.90	-12.23	-14.04	29º
Area 2						
Top	-12.30	-19.35	-7.05	-4.30	-8.26	59º
Middle	-19.05	-27.20	-8.15	+5.73	-9.96	-55º
Bottom	-13.25	-19.60	-6.35	-15.54	-16.79	22º
Control area						
Top	-8.30	-14.65	-6.35	-6.99	-9.44	42º
Middle	-5.05	-13.90	-8.85	-5.86	-10.61	57º
Bottom	-4.20	-8.80	-4.60	-6.60	-8.04	35º

8 Strength and modulus of exhumed lime piles

To investigate their state of hydration, pairs of lime piles (one unlined and one lined) were exhumed at 4, 10, 16 and 25 months after their installation. A detailed account of the procedure used for their excavation and retrieval, and the results of laboratory testing for strength and modulus are given in Appendix B.

With the unlined lime piles it was concluded that, with the possible exception of the upper 300mm of pile 1A exhumed after only 4 months, the unconfined compressive strength measured using a hand penetrometer always exceeded 450kPa. After being in the ground for more than 16 months, the mean unconfined compressive strength of the lime piles then exceeded 690kPa and at many depths was greater than 890kPa. Generally unconfined compression tests carried out in the laboratory on the recovered 150mm diameter specimens gave compressive stresses at failure which were about four times smaller, this was probably because cracks in the specimens influenced the results whereas the more localised hand penetrometer tests were unaffected.

Generally the unconfined compressive strength of the lime in the lined piles, which was measured by penetrometer following exhumation and removal of the waterwell screen, was greater than 450kPa after only 4 months. After 16 months the mean unconfined compressive strength of pile 2A was 738kPa. This was marginally more than measured after the same period on an unlined pile. Penetrometer strengths of the lime after 25 months were, with one exception, greater than 872kPa. Unconfined compressive strengths on 150mm diameter specimens measured in the laboratory generally exceeded 443kPa throughout. These results were closer to those from the hand penetrometer tests indicating that crack development in the lined piles was limited by the presence of the waterwell screen.

Convolutions of the waterwell screen along its length were observed during exhumation of the lined piles, these were expected to improve their reinforcing capability and further improve the slope stability. The contribution of the waterwell screen to the overall strength of a lined lime pile, particularly in shear, is also expected to be beneficial although this has not been quantified.

9 Summary and conclusions

An instrumented full-scale trial of the technique of lime piling has been undertaken on a Gault Clay embankment (north of Junction 12 of the M1) with a slope of 1v:2h which has a history of shallow failures. Lined lime piles were installed in one 20m long area; unlined piles were installed in a second area. A control area, which was untreated, separated the two areas where remedial works were undertaken. The following main conclusions were reached.

i At each of the instrumented areas the trend of the results confirmed that pore pressures decreased in the summer months with suctions generally developing in the slopes to depths of just over 2m and to greater depths nearer to the slope toes. Comparison of contour plots in the remediated and control areas also indicated that 5 months after construction, hydration of the quicklime in the remediated areas was still having some effect in drying out the slope as well as providing support through a dowelling action. This was not unexpected as laboratory tests carried out by Brookes *et al.* (1997) showed that some strength gain due to the chemical reaction and hydration of the lime was still occurring after a similar period. By the second year in service the hydration of the lime piles was effectively complete and no further water was being absorbed from the slope.

ii Some similarities were observed in the trends of lateral movement in both the lime pile (lined and unlined) and control areas. Surface movements of up to 13mm (lateral shrinkage towards the centre of the embankment) were measured between March and November 1999. Following this, swelling of the clay occurred with outward surface movements of about 10mm in the control and area 1, and about 20mm in area 2, being measured during spring 2000. Further seasonal cycles of swelling and shrinkage of the clay slope then continued with movements being measurable at depths of up to 2m. This behaviour was consistent with that measured at other clay slopes (Crabb and Hiller, 1993) and was considered to be part of the seasonal cycle of clay swelling and shrinkage because of wetting and drying.

iii Lateral movements in the unlined pile and control areas were very similar with considerable lateral shrinkage at the surface but scarcely any at a depth of 2m. In the lined lime pile area, this mechanism was masked to some extent because of a small continuing outward movement of the clay slope which was more deep-seated. In this area a significant movement had occurred some years ago which caused the nearby motorway lighting column to tilt about 9^o towards the slope. Although the slope had since appeared to stabilise itself, these results suggest that some deep-seated movement continued up to about October 2000 although that movement has now slowed. The design of the lime piles was only intended to be effective in arresting shallow failures and whether the dowelling action of the piles in area 2 also served to limit the more deep-seated movements is not clear.

iv Comparisons of the lateral and vertical movements at the slope surface show similar trends of seasonal swelling and shrinkage. Investigation of the direction of the resultant movement showed that it acted approximately normal to the slope surface in the unlined lime pile area. In the control area it acted at an angle of between 42^o and 57^o to the vertical which was greater than the 28^o slope angle. On this basis, it is tentatively suggested that the piles may be providing some lateral restraint. Results were scattered in the lined lime pile (area 2) due to the deep-seated movement which occurred.

v Unlined and lined lime piles were exhumed at 4, 10, 16 and 25 months after their installation in order to investigate their state of hydration. Generally the unconfined compressive strength of the lime measured by hand penetrometer was greater than 450kPa after only 4 months. After being in the ground for more than 16 months, the unconfined compressive strengths from penetrometer tests on the unlined and lined piles generally exceeded 690kPa and 738kPa respectively. Unconfined compression tests carried out in the laboratory on 150mm diameter specimens recovered from the unlined piles gave lower compressive stresses at failure, this was probably because cracks in the specimens influenced the results whereas the more localised hand penetrometer tests were unaffected. Laboratory unconfined compression results on specimens from the lined piles were closer to those from the hand penetrometer indicating that crack development in the lined piles was limited by the presence of the waterwell screen.

vi Convolutions of the waterwell screen along its length were observed during exhumation of the lined piles, these were expected to improve their reinforcing capability and further improve the slope stability. The contribution of the waterwell screen to the overall strength of a lined lime pile, particularly in shear, is also expected to be beneficial although this has not been quantified.

vii Advice on the specification and design of lime pile systems to stabilise clay slopes is given in Appendix C of this report. This advice could be inserted in Section 8 (Reinstatement of slope failures) of HA48 Maintenance of highway earthworks and drainage (DMRB 4.1.3), if so wished.

viii This trial has provided valuable data on the performance of small diameter lime piles installed to stabilise a Gault Clay embankment slope. A second trial to further validate the technique in an over-consolidated clay of different geological history is recommended. In addition to measurements of pore water pressure and ground movement, the innovative use of a subsurface neutron probe would enable moisture migration during hydration of the lime to be investigated. This further study in different soil conditions would give design engineers an increased confidence in using the technique of lime piling to improve the stability of clay slopes.

10 Acknowledgements

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The co-operation of Mr H Greaves of Buxton Lime Industries Limited who arranged the supply of the lime was much appreciated. The construction of the lime piles at the site on the M1 was sub-contracted to Soil Mechanics Limited.

The advice and assistance of Mr C Dowding of Thorburn Colquhuon, the HA Route Manager, is also gratefully acknowledged.

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Design Manual for Roads and Bridges

HA 48 Maintenance of highway earthworks and drainage (DMRB 4.1.3)

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Appendix A: Breakdown of costs of lime pile installation

The installation of piles at the M1 site was carried out as part of a research study and in order to compare relative performance of unlined and lined piles, pile diameters and spacing between centres were similar. In practice because of the higher strength of the lined piles, smaller diameter piles or a larger spacing are likely to be used than with the unlined piles to achieve an equivalent design.

A1 Breakdown of costs by percentage

Figures A1 and A2 give the respective breakdowns in cost by percentage for the unlined and lined piles. The figures show that scaffolding access is a major element (approximately 26% and 21% respectively) of the total cost. The need for scaffolding access of this type is site specific and if it can be avoided has a large impact on the cost effectiveness of the technique. In cases where the clay slope is less than 1v:2h it is anticipated that direct access for a small rotary rig to work on the slope may be feasible. For steeper slopes it is envisaged that use of a small tilting platform beneath the rig may provide a viable option, with the small platform being repositioned for installation of each pile.

Figure A1 shows that apart from the cost of the working platform and the drilling, the next major cost was that of the Supervising Engineer, with the cost of the quicklime being found to be relatively insignificant. However when lined piles were used, the cost of supplying the plastic waterwell screen was significant and amounted to 19% of the overall cost (Figure A2). It must be noted that a short delivery time for the screen was important at this particular site and cheaper variants could have been available given more notice.

A2 Costs per metre run of slope

Given the caveats stated above, the costs for remediating the M1 slope (ie. a slope angle of 28^{o} and an average height of 6.5m) with nominal 150mm diameter piles in a grid at 2m centres was as follows:

- unlined piles - £779 plus £136 VAT per metre run;
- lined piles - £964 plus £169 VAT per metre run.

VAT has been calculated at the current rate of 17.5%.

On this basis the lined piles appear more expensive however, because of their higher strength, the *improvement factor* for stability of the slope (West and Carder, 1997) is likely to have been much higher for the lined pile area. If this is taken into account, the lined lime pile technique is viewed as the more cost effective option.

The overall costs of both techniques is expected to have reduced by up to 20% if an alternative to a scaffolding platform had been available.

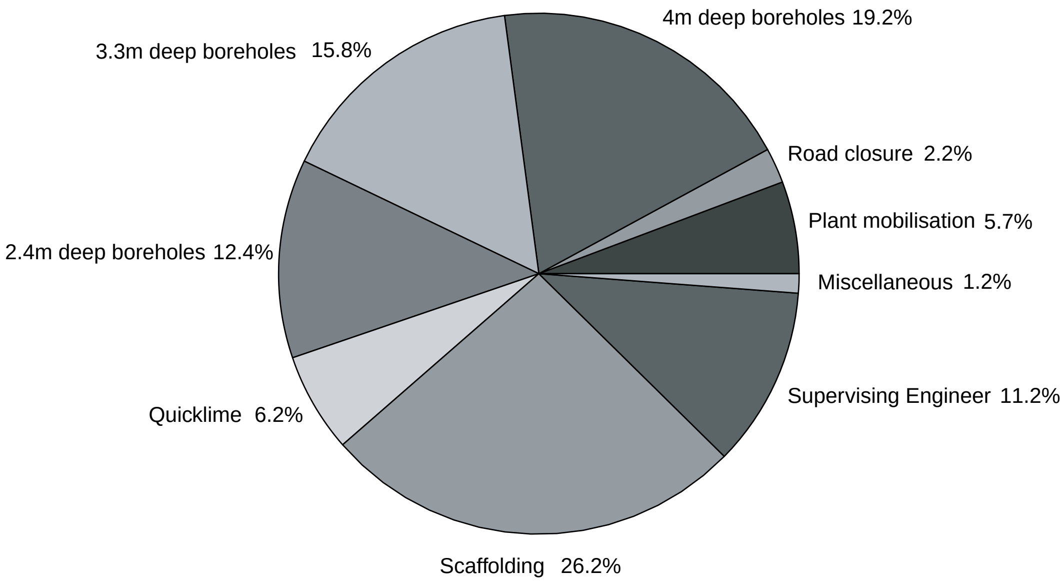

Figure A1 Breakdown of costs of 20m length of slope (unlined lime pile): grid of piles at 2m spacing

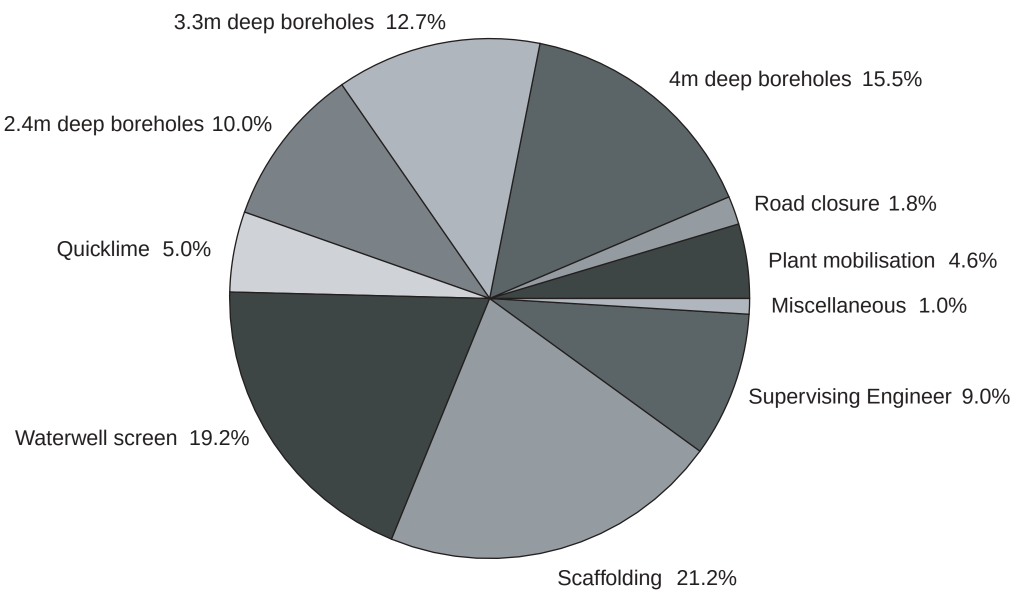

Figure A2 Breakdown of costs of 20m length of slope (lined lime pile): grid of piles at 2m spacing

Appendix B: Exhumation and strength testing of piles

B1 Introduction

Eight sacrifical lime piles, of which four were unlined and four lined, were installed to a depth of 1.4m at the time of slope remediation. The locations of the sacrifical piles near to the toe of the slope are shown in Figures 4 and 5.

Although it was initially intended to exhume all eight of these sacrificial piles, in fact only six were exhumed as it was decided to alternatively exhume two piles from further up the slope to investigate if any difference occurred in properties. These latter piles were exhumed from row 3 (see Figures 4 and 5) of each of the unlined and lined pile areas.

The piles were exhumed at different intervals of time after their installation to investigate their state of hydration and to assess their strength. The sequence of exhumation was as described in Table B1.

Table B1 Sequence of exhumation of lime piles

Pile number	*Installation date*	*Exhumation date*	*Age (months)*
Unlined piles			
1A	24/02/1999	28/06/1999	4
1B	24/02/1999	07/12/1999	10
1C	25/02/1999	10/08/2000	16
Row 3	03/03/1999	18/04/2001	25
Lined piles			
2D	16/03/1999	28/06/1999	4
2C	16/03/1999	01/12/1999	10
2A	15/03/1999	27/07/2000	16
Row 3	17/03/1999	18/04/2001	25

B2 Method of excavation

The tops of the sacrificial lime piles were first exposed by hand excavation. With the lined piles, the clay was carefully removed by spade and pick axe from the front face of the pile down to a level just below the bottom of the water well screen casing surrounding the pile (Plate B1a). A hand trowel was used to remove the soil immediately adjacent to the pile. On exposing the front face of the pile, a series of hand penetrometer and shear vane tests were carried out on the surrounding clay. Soil to either side of the pile was then removed and the pile carefully exhumed. The exhumed pile was then sealed in polythene bags for laboratory testing.

The procedure for exposing the front face of the unlined piles and strength testing of the surrounding clay was similar (Plate B1b). However, whilst the front face of the pile was exposed, *in situ* strength testing of the lime was also carried out using the hand penetrometer. Considerable care was needed in retrieving the samples of unlined pile due to their brittle nature.

In all cases the embankment slope was restored to its original profile following completion of the exhumation.

B3 Field observations and *in situ* testing

For ease of exhumation the 1.4m deep sacrificial piles were installed near to the toe of the slope. After removal of topsoil, the ground conditions generally comprised a firm

(a) Lined lime pile

(b) Unlined lime pile

Plate B1 Exhumation of lime piles

brown clay which became slightly softer with depth. The toes of the piles were generally founded in a stiffer horizon of grey clay which contained abundant flints and chalk fragments. This stiffer clay horizon did not exist where the two piles exhumed further up the slope were located.

B3.1 Unlined lime piles

Results from the hand penetrometer and shear vane tests on the lime and adjoining clay are given in Tables B2, B3, B4 and B5 for piles 1A, 1B, 1C and row 3 respectively.

In Tables B2 and B3 it must be noted that the limit of unconfined compressive strength using the hand penetrometer was 450kPa and higher values which occurred in the lime of the piles could therefore not be recorded. However to extend the range of these readings, a hand penetrometer designed for testing concrete was subsequently used. This penetrometer was of the same diameter as the one for soil testing and an appropriate conversion factor was employed. Higher values of unconfined compressive strength up to 890kPa could then be measured during the exhumation of pile 1C and row 3 (Tables B4 and B5). Generally hydration of the lime proceeded fairly rapidly and, with the possible exception of the upper 300mm of pile 1A, the unconfined compressive strength of the lime pile exceeded 450kPa after only 4 months. With a few exceptions, the unconfined compressive strength of the lime piles then normally exceeded 690kPa after 16 months had elapsed (Tables B4 and B5) and at many depths was greater than the 890kPa which could be measured by penetrometer.

In Table B2 the mean unconfined compressive strength of 338kPa (ie. an equivalent undrained shear strength of 169kPa when account was taken of the factor of two which relates these parameters) measured in the adjoining clay using the hand penetrometer was identical to the mean shear strength measured using the vane tester. The correlation was not so good for the results in Table B3 with mean undrained strengths of the clay of 142kPa and 94kPa from the penetrometer and vane respectively. Results taken during exhumation of pile 1C after 16 months (Table B4) were more scattered although generally undrained strengths of the clay exceeded 190kPa. This apparent gain in strength of the adjoining clay after 16 months may be the result of the migration of lime from the pile but probably is accounted for by the lower moisture content in the clay around pile 1C which was exhumed in the summer. During the first exhumation of an unlined pile after 4 months there was visual evidence of a 10mm thick zone of modified clay around the pile, although this was not observed in the later exhumations. Strength testing of the clay in the vicinity of the row 3 pile, located further up the slope from the other piles, indicated that the clay was more weathered in nature and of a lower strength than that nearer the toe. Vane tests in the clay gave a mean shear strength of 60kPa over the depth of the exhumation. Corresponding hand penetrometer tests gave unconfined compressive strengths which exceeded the ratio of two which relates these parameters.

The moisture contents of the clay near to each pile taken during the various pile exhumations are summarised in Table B6. No particular trends in the variation of moisture content with distance from the pile could be identified at any of the test locations. Generally moisture contents of the clay near pile 1C were lower than those near piles 1A and 1B: this is probably because the pile was excavated in the summer months and a nearby mature bush may also have helped to draw the water table down. The moisture contents were high at shallow depth near the row 3 pile and this was probably related to the high rainfall during April 2001 as shown in Figure 15.

Generally the retrieval of samples of unlined lime piles for laboratory strength testing proved difficult because of the frequency of both horizontal and vertical cracks along their length. The condition of the pile in row 3, further up the slope than the other exhumed piles, proved better although vertical cracks were still present.

B3.2 Lined lime piles

After excavation and retrieval of the lined lime piles, the waterwell screen was cut along its length using an angle grinder and the lime pile removed for testing as shown in Plate B2.

Tables B7, B8, B9 and B10 give the results from the hand penetrometer and shear vane tests on the lime and adjoining clay for lined piles 2D, 2C, 2A and row 3 respectively. Generally the unconfined compressive strength of the lime in the piles, which was measured by penetrometer following exhumation and removal of the waterwell screen, was greater than 450kPa after only 4 months. After 16 months the mean unconfined compressive strength of pile 2A was 738kPa (Table B9). This was marginally more than measured after the same period on an unlined pile. Compressive strengths of the lime after 25 months (Table B10), with one exception, were greater than 872kPa.

Tests using the hand penetrometer and shear vane on the clay within 200mm of the pile surface generally indicated that its undrained shear strength was of the order of 200kPa at 4 and 10 months after pile installation. Penetrometer results after 16 months given in Table B9 also indicated strengths of this order, that is a mean unconfined strength of 458kPa (equivalent undrained shear strength of 229kPa), although the mean vane strengths of 132kPa near the upper part of the pile were less. Strengths of the clay near the row 3 pile exhumed at 25 months were much reduced: mean results were an unconfined strength (penetrometer) of 158kPa and an undrained strength (shear vane) of 69kPa. These strength results again illustrated the dependence of clay strength upon moisture content. Measurements of moisture content of the clay adjoining the various piles are shown in Table B11. Particularly high moisture contents were obtained when excavating the pile in row 3 following an exceptional monthly rainfall.

When fully exposed, some convolutions along the surface of the waterwell screen were observed associated with the swelling of the lime during its hydration. Measurements of the changes in external diameter of the waterwell screen are given in Table B12 and demonstrate the convoluted nature of the exhumed piles.

Maximum swelling, which in these cases is particularly noticeable at about 300mm depth, tends to occur where the waterwell screen is weakest that is at the midpoint of the

Table B2 Field observations and testing of unlined lime pile 1A (age 4 months)

	Hand penetrometer – Unconfined compressive strength (kPa)		*Vane test – Undrained shear strength (kPa)*	
Depth (m)	*Lime pile*	*Clay*	*Clay*	*Comments*
0.1	200			Top of pile very crumbly. Exterior surface (~5mm) of pile soft, easily indented using thumb. 10mm thick zone of modified material surrounding lime pile.
0.2	400			
0.3	200	150 (at d=50mm) 450 (at d=100mm) 375 (at d=200mm)	153 (at d=50mm) 153 (at d=100mm) 189 (at d=200mm)	Pile exterior still very soft. At depths greater than 0.4m unable to penetrate lime pile with penetrometer. Section of pile removed at 0.5m depth. Some cracking observed on remainder of pile. Remainder of pile split into three sections on subsequent removal.
0.4	425			
0.5	>450			
0.6	>450			
0.7	>450	325 (at d=50mm) 350 (at d=100mm) 375 (at d=200mm)	161 (at d=50mm) >190 (at d=100mm) 168 (at d=200mm)	
0.8	>450			

d = distance from the surface of the pile

Table B3. Field observations and testing of unlined lime pile 1B (age 10 months)

	Hand penetrometer – Unconfined compressive strength (kPa)		*Vane test – Undrained shear strength (kPa)*	
Depth (m)	*Lime pile*	*Clay*	*Clay*	*Comments*
0.1	400			Top of pile very crumbly. Exterior surface of pile firm to touch.
0.2	>450			
0.3	>450	450 (at d=50mm) 225 (at d=100mm) 225 (at d=200mm)	110 (at d=50mm) 88 (at d=100mm) 82 (at d=200mm)	Pile surface still firm. Section of pile removed to 0.7m depth. Cracking observed along the length of the pile. Water exuding from pile at approximately 1m depth. No evidence of the zone of “modified” clay which was found in the previous exhumation.
0.4	>450			
0.5	>450			
0.6	>450			
0.7	>450	450 (at d=50mm) 125 (at d=100mm) 225 (at d=200mm)	128 (at d=50mm) 82 (at d=100mm) 73 (at d=200mm)	
0.8	>450			

d = distance from the surface of the pile

Table B4 Field observations and testing of unlined lime pile 1C (age 16 months)

	Hand penetrometer – Unconfined compressive strength (kPa)		*Vane test – Undrained shear strength (kPa)*	
Depth (m)	*Lime pile*	*Clay*	*Clay*	*Comments*
0.1	771			Top of pile very crumbly. Exterior surface of pile firm to touch. Pile cracked off at 0.2m depth.
0.2	436	838 (at d=0mm)		
0.3	>890	905 (at d=0mm) 671 (at d=50mm) >890 (at d=100mm) >890 (at d=150mm)	 190 (at d=50mm) >190 (at d=100mm) >190 (at d=150mm)	Pile surface still firm. Section of pile removed from 0.2m to 0.6m depth. No evidence of the zone of “modified” clay found in some of the previous exhumations. Clay behind pile had a vertical crack of width ~10mm to at least 0.6m depth. Pile founded in flint at depth.
0.4	>890	510 (at d=0mm)		
0.5	402	>890 (at d=0mm)		
0.6	671	>890 (at d=0mm) 550 (at d=50mm) 724 (at d=100mm) 590 (at d=150mm)	 >190 (at d=50mm) >190 (at d=100mm) 109 (at d=150mm)	
0.7	>890	>890 (at d=0mm)		
0.8	570			

d = distance from the surface of the pile

Table B5 Field observations and testing of unlined lime pile from row 3 (age 25 months)

	Hand penetrometer – Unconfined compressive strength (kPa)		*Vane test – Undrained shear strength (kPa)*	
Depth (m)	*Lime pile*	*Clay*	*Clay*	*Comments*
0.1	482	123 (at d=0mm)		Top of pile very crumbly. Exterior surface of pile firm to touch. Pile cracked off at 0.1m depth.
0.2	>890	172 (at d=0mm)		
0.3	>890	147 (at d=0mm)	45 (at d=50mm) 53 (at d=100mm) 47 (at d=150mm)	Pile surface still firm, although exterior surface (~3mm) was soft between 0.5 to 0.6m depth. Section of pile removed from 0.1m to 0.8m depth. No evidence of the zone of “modified” clay found in some of the previous exhumations. Lime pile had long vertical cracks from 0.1m to 0.65m and from 0.1m to 0.4m depth: these cracks were of widths <10mm and <3mm respectively. There were numerous smaller cracks below 0.6m depth.
0.4	>890	319 (at d=0mm)		
0.5	241	172 (at d=0mm)		
0.6	429	392 (at d=0mm)	47 (at d=50mm) 64 (at d=100mm) 80 (at d=150mm)	
0.75	>890	>890 (at d=0mm)	55 (at d=50mm) 69 (at d=100mm) 76 (at d=150mm)	

d = distance from the surface of the pile

Table B6 Moisture contents in the adjoining clay

Depth of sample (m)	*Distance from pile surface (mm)*	*Moisture content of clay*			
		Near pile 1A	*Near pile 1B*	*Near pile 1C*	*Near row 3 pile*
0.30	50	20.0%	20.0%	15.2%	32.4%
	100	18.9%	20.8%	14.8%	28.1%
	200	16.4%	20.9%	14.2%	30.3%
0.50 - 0.60	50	n/a	25.5%	18.0%	17.2%
	100		21.5%	15.4%	17.4%
	200		17.2%	18.1%	17.0%
0.70	50	18.8%	n/a	n/a	21.2%
	100	n/a			18.8%
	200	19.1%			21.2%

Plate B2 Removal of lime pile from its casing

Table B7 Field observations and testing of lined lime pile 2D (age 4 months)

	Hand penetrometer – Unconfined compressive strength (kPa)		Vane test – Undrained shear strength (kPa)	
Depth (m)	Lime pile	Clay	Clay	Comments
0.1	200			Lime surrounding casing of pile, possibly from time of installation.
0.2	>450	400 (at d=0mm)		
0.3	>450	300 (at d=0mm) 425 (at d=50mm) 450 (at d=100mm) >450 (at d=200mm)	>190 (at d=50mm) >190 (at d=100mm) >190 (at d=200mm)	Pile casing cracked vertically to a depth of 0.6m. Also a horizontal crack was observed at 250mm from the top which extended 90mm around the casing. Some convolutions along the casing were observed caused by the swelling of the lime. With the exception of the top 50mm the pile appeared very rigid.
0.4	>450	>450 (at d=0mm)		
0.5	>450	350 (at d=0mm)		
0.6	>450	425 (at d=0mm)		
0.7	>450	325 (at d=0mm)		
0.8	>450	350 (at d=0mm) 325 (at d=50mm) 250 (at d=100mm) 350 (at d=200mm)	Vane unable to penetrate clay.	

d = distance from the surface of the pile

Table B8 Field observations and testing of lined lime pile 2C (age 10 months)

	Hand penetrometer – Unconfined compressive strength (kPa)		Vane test – Undrained shear strength (kPa)	
Depth (m)	Lime pile	Clay	Clay	Comments
0.1	>450	>450 (at d-0mm)		Lime surrounding casing of pile, possibly from time of installation.
0.2	>450	>450 (at d=0mm)		
0.3	>450	>450 (at d=0mm) >450 (at d=50mm) >450 (at d=100mm) >450 (at d=200mm)	>175 (at d=50mm) >175 (at d=100mm) 152 (at d=200mm)	Pile casing cracked vertically from a depth of 0.15m to 0.88m. With the exception of the top 50mm and areas exposed to the crack in the casing, the pile appeared very rigid. Some convolutions along the casing were observed caused by the swelling of the lime. Exterior of the pile was very hard, interior softer and damp.
0.4	>450	>450 (at d=0mm)		
0.5	>450	>450 (at d=0mm) >450 (at d=50mm) >450 (at d=100mm) >450 (at d=150mm)	Vane unable to penetrate clay. .	
0.6	>450	>450 (at d=0mm)		
0.7	>450	>450 (at d=0mm) >450 (at d=50mm) >450 (at d=100mm) >450 (at d=200mm)	Vane unable to penetrate clay	

d = distance from the surface of the pile

Table B9 Field observations and testing of lined lime pile 2A (age 16 months)

	Hand penetrometer – Unconfined compressive strength (kPa)		*Vane test – Undrained shear strength (kPa)*	
Depth (m)	*Lime pile*	*Clay*	*Clay*	*Comments*
0.0	838	429 (at d=0mm) 503 (at d=50mm) 503 (at d=100mm) 402 (at d=150mm)	150 (at d=50mm) 166 (at d=100mm) 128 (at d=150mm)	Lime exposed about 200mm above top of casing and firm to touch. Top of pile very crumbly and brittle. Lime surrounding casing of pile, possibly from time of installation.
0.1	402	335 (at d=0mm)		
0.2	671	537 (at d=0mm)		Pile casing cracked vertically from its top to a depth of 0.8m. With the exception of the top 200mm and areas exposed to the crack in the casing, the pile appeared very rigid. Some convolutions along the casing were observed caused by the swelling of the lime. Exterior of the pile was very hard, interior softer and damp.
0.3	771	637 (at d=0mm) 872 (at d=50mm) 268 (at d=100mm) 470 (at d=150mm)	126 (at d=50mm) 120 (at d=100mm) 99 (at d=150mm)	
0.4	805	302 (at d=0mm)		
0.5	771	402 (at d=0mm)		
0.6	805	235 (at d=0) 402 (at d=50mm) 436 (at d=100mm) 268 (at d=150mm)	Vane unable to penetrate clay.	
0.7	838	402 (at d=0mm)		

d = distance from the surface of the pile

Table B10 Field observations and testing of lined lime pile from row 3 (age 25 months)

	Hand penetrometer – Unconfined compressive strength (kPa)		*Vane test – Undrained shear strength (kPa)*	
Depth (m)	*Lime pile*	*Clay*	*Clay*	*Comments*
0.0	>890	98 (at d=0mm)	86 (at d=50mm) 70 (at d=100mm) 54 (at d=150mm)	Lime exposed about 150mm above top of casing and firm to touch. Top of pile very crumbly and brittle. Lime surrounding casing of pile, possibly from time of installation.
0.1	>890	147 (at d=0mm)		
0.2	>890	147 (at d=0mm)		
0.3	872	123 (at d=0mm)	69 (at d=50mm) 70 (at d=100mm) 54 (at d=150mm)	Pile casing cracked vertically from 0.2m to 0.8m depth. The pile appeared very rigid and had some convolutions along the casing caused by the swelling of the lime. Exterior of the pile was very hard, interior softer and damp.
0.4	>890	147 (at d=0mm)		
0.5	>890	147 (at d=0mm)		
0.6	>890	172 (at d=0mm)	70 (at d=50mm) 85 (at d=100mm) 67 (at d=150mm)	
0.7	392	147 (at d=0mm)		
0.8	n/a	294 (at d=0mm)		

d = distance from the surface of the pile

Table B11 Moisture content of adjoining clay

Depth of sample (m)	Distance from pile surface (mm)	Moisture content of clay Near pile 2D	Near pile 2C	Near pile 2A	Near row 3 pile
0.30	50	17.2%	16.2%	22.3%	36.0%
	100	16.9%	15.0%	23.2%	37.1%
	200	16.4%	15.4%	15.5%	37.4%
0.60	50	17.0%	13.8%	22.1%	33.6%
	100	17.3%	15.3%	23.4%	35.4%
	200	16.4%	n/a	19.7%	32.7%

Table B12 Measurements of external diameter of waterwell screen

	Pile 2D (age 4 months)		Pile 2C (age 10 months)		Pile 2A (age 16 months)		Row 3 pile (age 25 months)	
Depth (mm)	Swell 1 (mm)	Swell 2 (mm)	Swell 1 (mm)	Swell 2 (mm)	Swell 1 (mm)	Swell 2 (mm)	Swell 1 (mm)	Swell 2 (mm)
50	10	7	-1	-3	8	0	-3	0
100	12	7	-1	-3	10	6	-3	0
150	11	15	0	-1	19	11	-2	1
200	12	19	15	10	23	18	1	7
250	15	24	27	29	21	20	1	13
300	15	25	31	29	23	22	1	18
350	15	23	32	24	24	19	2	18
400	12	21	32	25	23	19	2	22
450	9	18	32	25	18	19	2	20
500	6	13	31	28	15	14	2	20
550	5	8	27	28	12	9	3	21
600	3	6	23	23	7	5	6	22
650	3	7	19	18	5	0	7	22
700	5	7	9	15	3	-1	10	26
750	2	3	7	9	-1	-1	13	28
800	2	2	9	5	-2	-2	n/a	n/a
850	1	1	3	6	-3	-1	n/a	n/a
Mean	8.1	12.1	17.4	15.7	12.1	9.2	2.8	15.9

Swell 1 and 2 are the changes in diameter measured orthogonally

slotted sector and away from the joints. In most cases this swelling was sufficient to cause a vertical crack in the waterwell screen which ran from the top of the casing to depths of up to 0.8m. This crack did not affect the general rigidity of the lined piles which remained intact on exhumation: this can be contrasted with the brittle nature of the unlined piles which tended to shear on exhumation.

B4 Laboratory testing

Where possible specimens of the lime were recovered from the piles for laboratory tests. In most cases only unconfined compression tests on the nominal 150mm diameter samples could be carried out because of their irregular surface finish. In one case, it proved possible to machine the sample and carry out an undrained triaxial test. A summary of the results is given in Table B13.

For both the unlined and lined piles installed near to the toe of the slope, there was a general trend in the unconfined compression tests of increasing modulus with age of the pile. For the lined piles this was accompanied by an increase

Table B13 Summary of laboratory test results on lime piles

Pile number	Age (months)	Unconfined compression tests Modulus (MPa)	Strain to failure (%)	Compressive stress at failure (kPa)
Unlined piles				
1A	4	17	0.8	136
1B	10	Specimens disintegrated during trimming		
1C	16	74	0.2**	155**
Row 3	25	3	5.0	168
		1	7.0	81
Lined piles				
2D	4	31	1.1	463
		166*	0.8*	1088*
2C	10	Specimens disintegrated during trimming		
2A	16	98	0.75	564
Row 3	25	18	3.0	443

* *Results from undrained triaxial tests carried out at a confining stress of 20kPa.*

** *Flaw in specimen caused premature failure.*

in the compressive stress at failure, no such comparison was available for the unlined piles because of the premature failure of the flawed specimen from pile 1C. In the case where an undrained triaxial test was carried out at a low confining stress of 20kPa, a much higher compressive stress at failure was measured than in the unconfined tests. With the exception of the test on pile 1C, all specimens failed at a strain of between 0.75% and 1.1%.

Results on the row 3 piles showed different behaviour with modulii being much lower and strains to failure being as high as 7%. This apparent anomaly may be the consequence of a lower confining stress in the ground during hydration of the lime which may have resulted in piles of a lower density. The effect of density had previously been noticed during the laboratory testing reported by Brookes *et al.* (1997). Nevertheless the compressive stresses at failure for the row 3 piles appeared similar to those recorded on piles installed near to the toe of the slope.

Compressive stresses at failure for the lined lime piles exceeded 443kPa throughout whereas, with one exception, stresses for the unlined piles exceeded 136kPa. Generally, a period of 4 months appeared to be adequate for the piles to gain strength. It must be noted that these compressive stresses can be considered as worst case values as the process of exhuming the piles may have induced some cracking and weakening of the piles, furthermore the confining pressure from the surrounding clay will result in higher *in situ* failure stresses.

Appendix C: Design advice

The following advice on the use of lime piles to stabilise clay slopes could be inserted in Section 8 (Reinstatement of slope failures) of HA48 Maintenance of highway earthworks and drainage (DMRB 4.1.3), if so wished.

C1 Lime piling

The stability against shallow failure of ageing clay slopes of highway embankments and cuttings can be improved by the installation of a grid of small diameter (»200mm) piles. These piles can be formed from either pure quicklime or lime-stabilised soil. The major short-term benefit in stability is gained from a reduction in pore water pressure and dehydration of the clay surrounding the pile as water is absorbed from the slope to hydrate the lime. In the longer term the original ground water levels are likely to be re-established, however by this time various chemical and pozzolanic reactions (West and Carder, 1997) will have occurred resulting in sufficient strength increase for the piles to improve stability by dowelling action.

Simple methods for deciding on the spacing and depth of the piles have been reviewed by West and Carder (1997). These involve calculating an improvement in resistance to shear of the soil on the potential slip surface arising from the installation of the piles, designated the *improvement factor* (IF). The improvement factor can be calculated from:

$$IF = 1+ (0.785x^2/y^2)((P-S)/S)$$

where the pile diameter is x and the spacing between pile centres is y. The shear strength of the unmodified clay is S and the shear strength of the pile is P.

Generally the piles should be deep enough to intercept the potential slip surface and penetrate beyond it into stable ground. For lime piles, vertical installation is generally easier in terms of drilling, placing and compacting the lime. Piles installed normal to the potential failure plane are however equally acceptable, although their horizontal installation is not only impractical but also of limited benefit as the piles are not able to take tension.

C2 Design strengths

The long term undrained shear strength of pure quicklime piles for design purposes has been reported as 400kPa by Rogers and Glendinning (1997) and as a higher value of 700kPa by Brookes *et al.* (1997). These strengths were largely based on laboratory and field testing of piles installed in London Clay; strengths in other clays will depend on their mineralogy being such that the various chemical reactions with the lime occur. Limited testing on lime-stabilised soil piles indicates that design strengths are likely to be lower than achieved with pure quicklime piles and laboratory strength testing is generally required before using this approach. Some improvement is obtained by adding cement to the lime-soil mix but laboratory evaluation of strengths is again necessary.

Exhumation of lime piles after their hydration has shown that, although they are rigid in nature, they are nevertheless very brittle. This gives rise to concerns about their resistance to lateral movement of the clay slope which may result in the piles being subjected to a combination of shear and bending forces. Brookes *et al.* (1997) have demonstrated that this potential problem can be readily overcome by installing a plastic waterwell screen as a close fitting liner to the borehole prior to placing and compacting the quicklime within it. In this process intimate contact of the clay and lime is still maintained through the slotted parts of the screen and as the lime hydrates it both pushes through the slots and also produces convolutions of the screen along its length. The undrained shear strength, for design purposes, of lined lime piles installed in London Clay is of the order of 1300kPa and the piles are not so brittle in nature.

An instrumented full-scale trial of the remediation technique of lime piling was carried out on a Gault Clay embankment slope on the M1 which has a history of shallow failures. The field performance of unlined and lined lime piles was compared with that of a control area where no remedial work was undertaken and the findings reported by Carder *et al.* (2001). Unconfined compression test measurements on the lime using hand penetrometers generally exceeded 690kPa and 738kPa on the unlined and lined piles respectively. Equivalent undrained shear strengths would be one half of these values. Laboratory unconfined compression tests on 150mm diameter specimens gave lower results than the penetrometer because cracks in the specimens influenced the results whereas the more localised hand penetrometer tests were unaffected. This effect was less marked on specimens from the lined piles as crack development was limited by the waterwell screen. In the case where an undrained triaxial test was carried out at a low confining stress of 20kPa, a compressive stress at failure was measured for the lime which was about double that from the unconfined tests.

C3 Site issues

Granulated quicklime may be preferred to powdered quicklime or slaked lime for this application to minimise dust. Granulated quicklime complying with BS 890 with 100% by mass of the quicklime passing a BS 10mm sieve and at least 95% by mass passing a BS 6mm sieve is suitable. The reactivity of the lime when tested in accordance with BS 6463 needs to be such that after 2 minutes it yields a temperature of at least 50^{o}C. Suitable storage and precautions are required on site to ensure that the quicklime does not hydrate before use. All operatives need to be aware of the necessary safety precautions relating to the use of lime and suitably equipped with appropriate protective gear.

Compaction of the quicklime within either the borehole or the waterwell screen is important in producing a high density and stiffness product. This can be achieved using a hand-held rammer. The top of the borehole is normally plugged with clay from the pile arisings to prevent a dust hazard on site.

C4 References

Brookes A H, West G and Carder D R (1997). *Laboratory trial mixes for lime-stabilised soil columns and lime piles.* TRL Report TRL306. Crowthorne: TRL Limited.

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Rogers C D F and Glendinning S (1997). *Slope stabilisation using lime piles.* In: Ground Improvement Geosystems: Proc 3rd Int Conf on Ground Improvement Geosystems, Densification and Reinforcement. London: Thomas Telford. pp174-180.

West G and Carder D R (1997). *Review of lime piles and lime-stabilised soil columns.* TRL Report TRL305. Crowthorne: TRL Limited.

Abstract

An instrumented full-scale trial of the remediation technique of lime piling was carried out on a Gault Clay embankment slope on the M1 which has a history of shallow failures. Lime piles were installed in each of two 20m long test areas of embankment, in one case the lime piles were unlined and in the other they were lined with waterwell screen. Results from earlier laboratory trials had indicated that, when using pure quicklime, there was much advantage in using plastic waterwell screen pipe to line piles because of the higher undrained shear strengths obtained. The field performance in both cases was compared with that of a control area where no remedial work was undertaken. Site measurements of pore water pressure and ground movements enabled a systematic comparison of behaviour using the different approaches. The construction procedure, cost implications and an evaluation of slope performance during the twenty-six months following construction are described in the report.

Related publications

TRL505 *Swell test requirements for lime stabilised materials* by D J MacNeil and D P Steele. 2001 (price £35, code H)

TRL424 *Detailed chemical analysis of lime stabilised materials* by J D McKinley, H Thomas, K Williams and J M Reid. 1999 (price £25, code E)

TRL306 *Laboratory trial mixes for lime-stabilised soil columns and lime piles* by A H Brookes, G West and D R Carder. 1997 (price £25, code E)

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