Analysis of performance of spaced piles to stabilise embankment and cutting slopes

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Executive Summary

The use of a single row of spaced piles for stabilising slopes has been used widely and with success for some considerable time primarily elsewhere in Europe and in Japan. On the UK highway network the technique could provide a permanent and cost effective method of both remediating potentially unstable or failed slopes and of steepening slopes in a widening situation. However, the availability of design guidance on the use of this technique is limited: this is partly because the soil-structure interaction problem is complex and three dimensional in nature. This report discusses the results from three dimensional finite element analyses in which the performance of untreated cutting and embankment slopes was compared with those stabilised using a single row of piles. The study concentrates on the performance of clay slopes and discusses the implication of the findings upon their design.

The finite element evaluation focused on the calculation of ground deformations under working loads, that is pre-failure, although the results provided a useful indicator of the potential rupture surface in clay slopes in the longer term. Particular aspects which received attention included the effect of increasing the slope angle, different pile spacings, softening of the clay, and the influence of different pore water pressure regimes upon the results.

In addition to providing guidance on the various mechanisms which need consideration in the design of the piling system, the report compares the lateral stresses acting on and the bending moments developed in the piles with those predicted using existing semi-empirical methods.

This report on numerical modelling is the third phase of the project. A review of the literature has already been undertaken to establish the state-of-the-art in using spaced piles to stabilise slopes. This was followed by a series of centrifuge model tests on the use of piles to stabilise cutting slopes constructed in clay. A full-scale trial of the technique on the highway network is also planned.

1 Introduction

One of the options for improving the stability of clay slopes is to use a single row of spaced piles to provide support. In the construction and maintenance of highway corridors within existing boundaries, this engineering option may be attractive both in allowing the construction of slightly steeper slopes in widening situations and in providing a method of reinstatement or repair of potentially unstable or failed slopes. The latter application is particularly relevant as on the highway network, many clay slopes which are prone to shallow failures (Perry, 1989) are now reaching a critical age for deep-seated failure to occur. In addition, prolonged delays in dealing with shallow failures may lead to the formation of deeper slips (Geotechnical Consulting Group, 1993).

The piling technique for stabilising slopes has been used widely and with success for some considerable time primarily elsewhere in Europe and in Japan (Carder and Temporal, 2000), although there is a paucity of well documented case histories. Stabilisation of landslides and natural slopes appears to have been more frequent than attempts to stabilise embankments. However, the cases that have been reported suggest that spaced piles may form a useful method of stabilising highway earthworks, provided that realistic design criteria can be developed.

Determination of pile diameter, spacing, penetration depth and location on the slope is however a complex soil-structure interaction problem on which there is currently little design guidance. To provide more information, TRL were therefore commissioned by the Highways Agency to carry out a research study. As part of this study, a literature review (Carder and Temporal, 2000) and a series of centrifuge tests (Hayward *et al.*, 2000) have already been completed. This report describes the next phase of the study which comprises numerical modelling in three dimensions to predict likely performance.

In the analyses, the performances of untreated cutting and embankment slopes were compared with those stabilised using a single row of spaced piles. Particular aspects which received attention included the effect of increasing the slope angle, different pile spacings, softening of the clay, and the influence of different pore water pressure regimes upon the results.

2 Applicability of finite element analysis to slope performance

Griffiths and Lane (1999) have compared the use of traditional methods of slope stability analysis based on limit equilibrium methods with the finite element approach. They found the main advantages were as follows:

a no assumption needs to be made about the shape and development of the failure surface;

b assumptions about inter-slice forces which are required in traditional methods are not needed;

c if realistic soil data are available, finite elements will give information on deformations at working stress levels;

d progressive failure can be investigated using finite elements as time-dependent behaviour can be modelled.

The main difficulty with finite element analysis of slope behaviour is however the possible definitions of failure and this may involve a judgement on when excessive deformations are deemed to have occurred or when the yield zone becomes sufficiently extensive. There is no direct determination of a lumped factor of safety for overall stability of the slope without using complex mathematical algorithms.

For this reason, finite element evaluation has tended to focus on the calculation of ground deformations under working loads. The magnitudes of these deformations at pre-failure, together with a knowledge of shear strain and yield development, are not only essential for serviceability design but also provide a useful (even if not entirely definitive) indicator of the potential rupture surface in clay slopes in the longer term.

The delayed collapse of cut slopes, particularly old railway cuttings, in stiff clay has received much attention over the last forty years (Skempton, 1964; Vaughan, 1994; Potts *et al*., 1997; Cooper *et al.*, 1998). Failures are generally considered to be progressive in nature with peak soil strengths reducing towards residual strengths. Finite element analysis is better suited than conventional slope stability analysis for predicting behaviour in this instance as the construction sequence and stress history can be accurately modelled within a coupled consolidation analysis. The behaviour of a cutting slope in stiff clay has been numerically investigated by Potts *et al.* (1997) who found that progressive failure is generated primarily by the high lateral stresses in the ground prior to excavation and the time taken for excess pore water pressures to dissipate.

Little information is available on finite element modelling of embankments constructed using stiff clay fill, although numerical modelling of embankments on soft clay has often been undertaken (Almeida *et al.,* 1986).

It must be noted that finite element analysis is not generally effective in modelling shallow failures as the influence of weathering, seasonal swelling and shrinkage, and vegetation upon pore water pressures near the surface of a clay slope is complex to model.

3 Finite element analysis and mesh design

Three dimensional finite element analysis was performed using the SAGE CRISP 97 engine, with pre- and post-processing using FEMGV software. Soils were generally considered as elastic perfectly plastic with the yield surface defined by the Mohr-Coulomb criterion.

The performance of both cutting and embankment slopes constructed in clay soils were investigated. These were treated as separate cases due to their completely different nature, ie. the soil parameters and types, construction techniques, geometry and pore water pressure regimes. The meshes were comprised entirely of 1440 linear strain brick elements each of which was 20 noded: consolidating elements (68 degrees of freedom) were employed for the soil strata and non-consolidating elements (60 degrees of freedom) were employed for the bored piles. The geometry of the slopes was based upon ‘typical’ dimensions that would be commonly employed by the Highways Agency in a road scheme.

The initial geometry in the cutting situation, as indicated in Figure 1a, allowed for a 1v:2h slope (ie. a slope angle of about 27°) which was 8m high. Some analyses were subsequently carried out for a steeper slope of 1v:1.5h (ie. a slope angle of about 34°) with the slope height being maintained at 8m. The performance of the slope was investigated with or without the inclusion of 0.8m diameter bored piles whose depths could be varied. The mesh was constructed in plan (Figure 1b) to allow consideration of an unreinforced slope or slopes reinforced with piles spaced at centres of 3 or 6 pile diameters. The mesh boundary was 36m away from the crest of the cutting and this was considered to be sufficiently remote to eliminate edge effects. The centre of the cutting was 15m from the toe of the slope so that a motorway could be

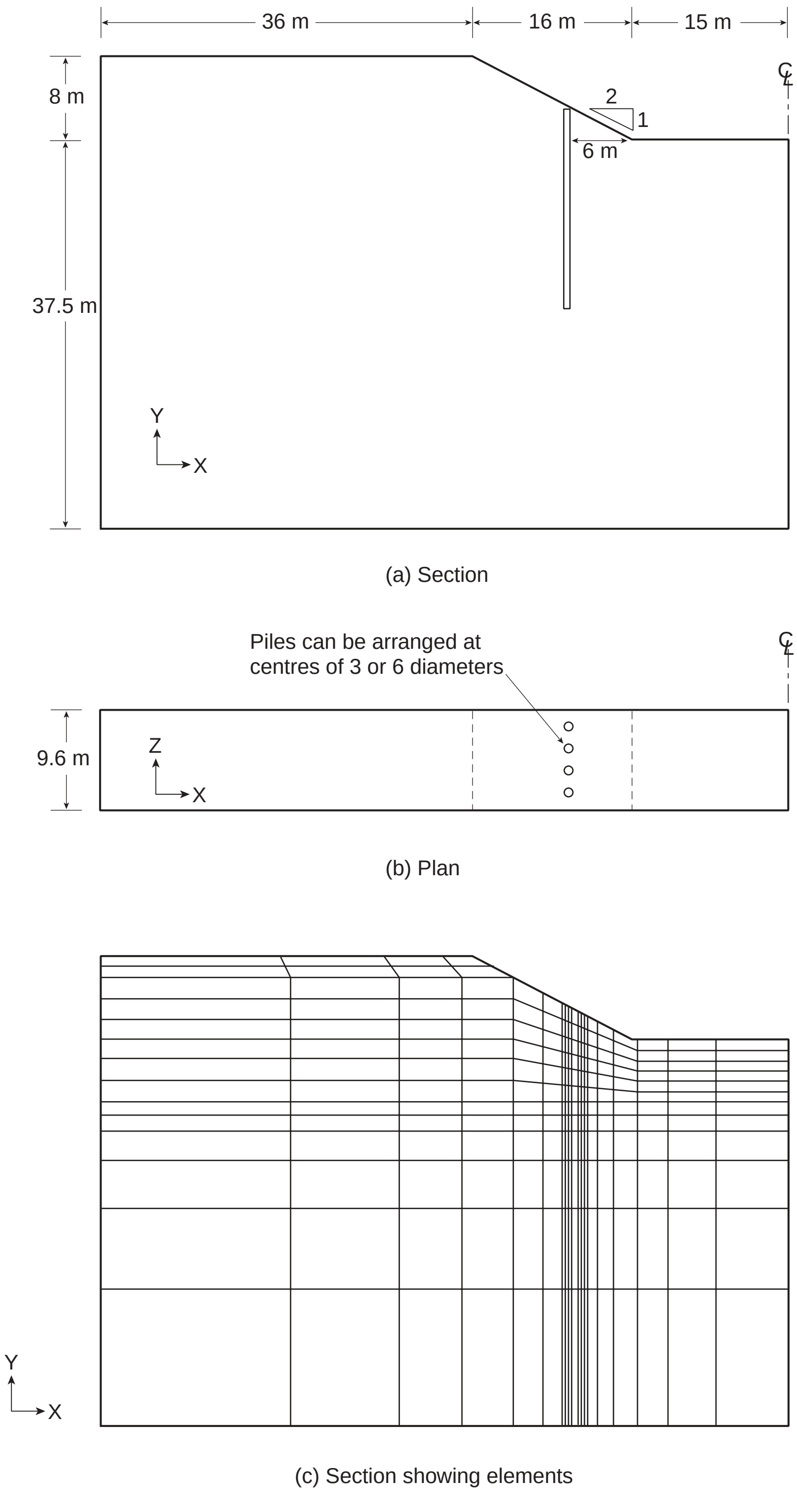


Figure 1 Details of the finite element mesh (cutting slope)

accommodated within the cut. Figure 1c shows a view in the x-y plane of the mesh design and fixities of both the vertical boundaries were such that displacement was only permitted in the y-direction. The appropriate fixity for the x-y plane was separately investigated in preliminary runs. These runs were carried out with displacement constraints in both the x- and z-directions on this plane using a mesh which had been greatly elongated in the z-direction. However this was found to be onerous in computing time and a sufficiently good approximation was achieved using the reduced mesh size shown in Figure 1b and fixity of the x-y plane boundaries in only the z-direction.

The dimensions of the mesh used to analyse the embankment slope were essentially similar to those given in Figure 1 for the cutting slope with the following exceptions. The distance from the mesh boundary to the crest of the embankment was limited to 15m to allow for a motorway construction on top of the embankment. Also the distance to the boundary in front of the toe of the slope was increased to 36m, so that this boundary was remote.

In all cases, the finite element analysis incorporated coupled consolidation. The pore water pressure regimes were however different for the cutting and embankment analyses as follows:

a In the cutting situation, a hydrostatic distribution of pore water pressure from a water table at 1m depth was assumed at the boundary remote from the slope crest. The water table at the bottom of the slope was assumed to be at ground level for the long-term consolidation analysis (although this condition was only applied after excavation was complete). No other pore pressure fixities were assumed and the drawdown of the water table in the slope was that calculated during the consolidation analysis.

b In the embankment situation, a hydrostatic distribution of pore water pressure with depth from the surface was assumed in the *in situ* ground prior to the embankment construction. Following construction, a perched water table at a depth of 1m was assumed beneath the top-surface of the embankment. Pore pressure fixities were also applied on the vertical boundary of symmetry at the centre of the embankment. These allowed for a hydrostatic increase in pore pressure to mid-height of the embankment, below this a reduction in pore pressure occurred because of the presence of the drainage blanket. Beneath the drainage blanket, pore water pressures again built up hydrostatically. Pore pressures elsewhere in the slope were those calculated as a result of the consolidation process. For the embankment case, the adverse effect of a build-up in pore pressures in the slope were also separately investigated as described later.

4 Construction sequence

The performance of highway slopes, both unreinforced and reinforced, will depend on their method of construction and whether it is a cutting or embankment which is under consideration. The construction technique will have a particular influence upon soil parameters such as strength and stiffness and also on the *in situ* stress state: these factors are discussed in Section 5.

In both cases, the performance of the highway slope over its 60 year design life was investigated with the following situations being modelled:

- an unreinforced clay slope;
- a clay slope reinforced by a row of spaced piles at the time of construction;
- a 20 year old unreinforced clay slope subsequently reinforced by a row of spaced piles during remedial works to improve its stability.

Within the general framework outlined above, the effect of an increased slope angle, different pile spacings, and the effect of embedment of the pile toe into a stiffer stratum were also investigated in specific instances. In the case of an embankment slope, the effect of a drainage blanket beneath the embankment received some attention. A general sensitivity analysis of all these parameters was not however possible because of the heavy demands on computing time which would have been required for this number of variables.

Where the slope was reinforced by piles, their installation was modelled by directly interchanging soil elements with pile elements of the same size. In this way their installation could be conveniently arranged either during the slope construction phase or after the slope had been in service for 20 years.

5 Soil properties and *in situ* stresses

Soil parameters were adopted which were typical of those of an over-consolidated clay, such as London Clay, and these are shown in Table 1. Peak shear strength parameters of c_p' from 17kPa to 25kPa and φ_p' of 20^o to 29^o might be expected for unweathered clay in such geology (Cripps and Taylor, 1986). However, in a slope situation, weathering and softening of the clay would occur. For this reason, reduced values with a c' of 12kPa and a φ' of 20^o were adopted for most of the analyses although a limited number were also carried with c' of 5kPa and a φ' of 18^o to simulate further softening. It must be noted that because a strain softening soil model was not being employed for this analysis, the results must be viewed as only providing guidance as to performance pre-failure. After the onset of failure a more significant reduction in strength parameters to residual values would be expected and a limited investigation of this is included by further reducing soil strength parameters.

Table 1 Effective stress soil parameters

Description	*Soil type A (stiff clay)*	*Soil type B (soft clay)*
Elastic modulus, E' (MPa)	6.6	4
Rate of increase with depth, mE' (MPa/m)	3.2	0.5
Angle of friction, φ' (^{o})	20	20
Cohesion, c' (kPa)	12	12
Poisson's ratio, ν	0.2	0.2
Horizontal permeability, k_x (m/year)	0.015	0.015
Vertical permeability, k_y (m/year)	0.003	0.003

The modulus values (E') and their rate of increase with depth (mE') given in Table 1 also take some account of the weathering and softening which are likely to occur in a slope situation. Whereas a high 'small strain' modulus, for example E' of 32MPa and mE' of 8.4MPa/m, calculated from back-analysis may be appropriate for retaining wall problems (Burland and Kalra, 1986), in a slope analysis the strains are larger and the modulii have been accordingly reduced.

Values for soil type A (stiff clay) were calculated to provide an equivalent soil stiffness to that employed if typical Cam Clay parameters (κ=0.0125, λ=0.16, e_o=1.45) were adopted. The equation for increase of modulus with depth for soil type B (soft clay) is similar to that employed by Potts *et al.* (1997) for analysis of the collapse of a cut slope in weathered clay. The variation of modulus with depth for the two soil types is compared in Figure 2.

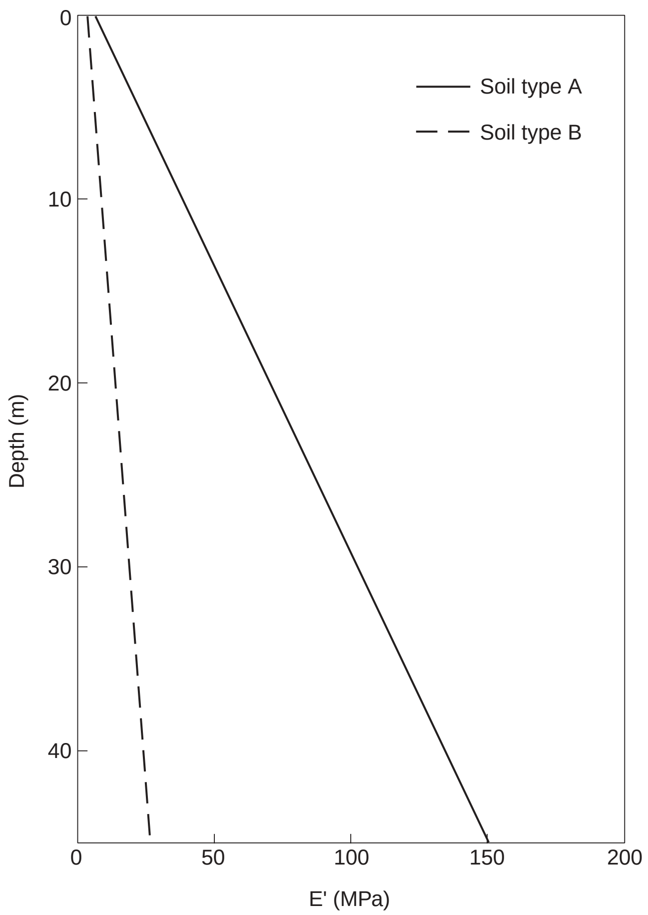


Figure 2 Variation of modulus for the two soil types

In the cutting situation, analyses were carried out with both soil types A and B as slope material. In the embankment case, only the softer soil type B was used to represent the fill over a stiff clay foundation.

The relation of decreasing permeability with depth for an over-consolidated clay, such as London Clay, has been investigated by Dixon and Bromhead (1999) who found that horizontal permeability decreased from about 5×10^{-10} m/s to 1×10^{-11} m/s below about 40m depth. As the major pore pressure changes occur at shallow depths in the slope situation, the former value which is equivalent to 0.015m/year has been adopted for *in situ* clay to provide an upper bound of movement predictions. As clays are generally considered to be more impermeable in the vertical direction, a lower coefficient of 0.003m/year was used for this parameter in the analysis. The clay fill used in the embankment analyses was assumed to be ten times more permeable in both directions than the *in situ* clay.

For the cutting situation, an *in situ* K value (ie. ratio of horizontal to vertical effective stress) of 2 was used which Symons (1992) reports is consistent with that calculated by a number of approaches for an over-consolidated clay with a φ' value of 20°. The use of a high K value of 2 ensures that larger displacements develop although the time to collapse may be increased (Potts *et al.*, 1997). In the embankment situation, a K value of 2 in the undisturbed foundation clay was also assumed. The lateral stresses (and hence K value) in the embankment fill were those generated by the self-weight of fill during its placement.

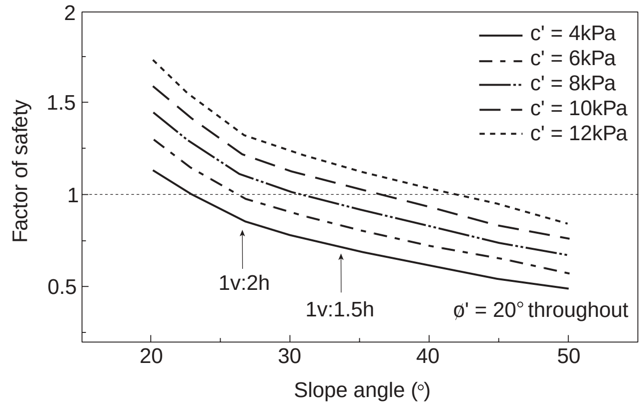
6 Preliminary assessment of slope stability

Prior to carrying out the finite element study the stability of both the unreinforced cutting and embankment slope was investigated using Bishop's Simplified method (Bishop, 1955) and employing parallel inclined interslice forces. Similar parameters were employed for the stability analysis to those already discussed in the previous sections. As the long-term condition was being modelled, drained analyses were carried out using a φ' of 20°. The c' was varied from the pre-failure value of 12kPa to 4kPa so that the consequence of a reduction in this parameter could be assessed. The differences between the results for the cutting and embankment slopes were primarily a consequence of the lower pore water pressures in the latter because a drainage blanket was included beneath the embankment fill. It must be noted that this type of simplified analysis takes no account of stress history.

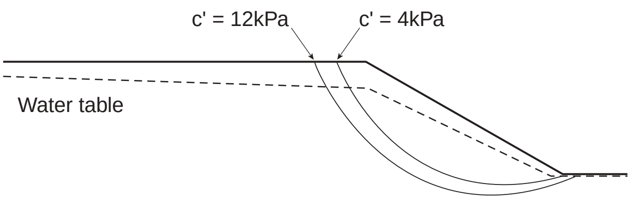
6.1 Cutting slope

A simple drawdown was assumed in the water table from 1m below the surface at the top of the slope to the excavated surface at the bottom of the slope. The results of the analyses are shown in Figure 3.

Figure 3a shows that for a slope of 1v:2h and the c' value of 12kPa used for the major part of the finite element study, the factor of safety against circular failure is 1.32. If progressive failure of the slope occurred with an associated reduction in the effective cohesion, failure would be expected for this same slope angle when the c' reduced below about 6kPa. If the slope angle is steepened to 1v:1.5h, the factor of safety using a c' of 12kPa against a deep-seated circular failure reduces to 1.17. Failure would be expected for the steeper slope angle when c' falls to about 9kPa.



(a) Variation of factor of safety with slope angle



(b) Location of critical slip surface for a 1v:2h slope

Figure 3 Results from drained stability analysis of cutting slope

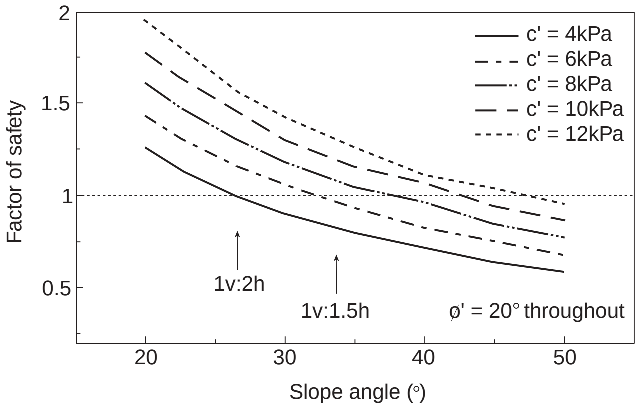
The location of the critical slip surface for a 1v:2h slope is shown in Figure 3b for the upper and lower limits of c' which were investigated. When using the higher c' of 12kPa, the increased yield stress meant that a slightly deeper seated failure was obtained which emerged at the ground surface just in front of the toe of the slope.

This preliminary assessment, based on typical properties for over-consolidated clays, provided some justification of the choice of cutting slope angle for the finite element study. Generally a slope angle of 1v:2h appeared to have a reasonable factor of safety against deep-seated failure even if some softening of the clay occurred. The stability of a steeper slope of 1v:1.5h was more marginal with little contingency if any softening occurred.

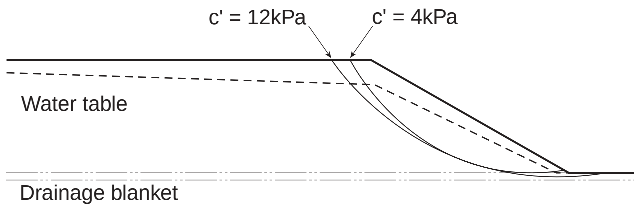
6.2 Embankment slope

In addition to the simple drawdown of pore water pressures assumed in the analysis of the cutting slope, the embankment analysis included a 0.5m thick drainage blanket at foundation level comprising ($c'=0$, $\varphi'=30^o$, $E'=100MPa$) granular material. This resulted in lower pore water pressures in the slope and generally improved factors of safety as shown in Figure 4a. In this figure, there are some small discontinuities in the plots as the slope angle steepens, these tend to occur when the critical slip surface starts to extend slightly into the drainage blanket.

For embankment slopes of 1v:2h, the critical slip surface tends to stop at the drainage blanket and is not as deep-seated as for a cutting slope (Figure 4b). This is an effect which is reproduced in the subsequent finite element analysis.



(a) Variation of factor of safety with slope angle



(b) Location of critical slip surface for a 1v:2h slope

Figure 4 Results from drained stability analysis of embankment slope

On the basis of this preliminary assessment, the factor of safety of a 1v:2h embankment slope against circular failure for a c' value of 12kPa is 1.55 and failure of the slope would be expected after softening had occurred and the c' was about 4kPa. Failure of a steeper 1v:1.5h slope would only be expected when the c' falls to about 7kPa, ie. a greater margin of safety than with the same cutting slope.

7 Results from analyses of cutting performance

The effect of installing a single row of spaced piles to stabilise a cutting slope was investigated analytically for the two situations of (i) the piles being installed at the time of construction and (ii) the piles being installed 20 years after slope construction. In both situations, the results were compared with the predicted performance of a similar unreinforced slope.

7.1 Piles installed at the time of construction

The long-term development in the overall pattern of movement of the unreinforced slope after excavation of the cutting is shown in Figure 5. Vectors of movement are shown together with the deformed shape (not to the same scale) of the slope. The vectors indicate that, as a consequence of the long-term dissipation of negative excess pore water pressures (caused by the unloading due to excavation of the cutting), a significant heave of the ground at cutting level develops. The results also indicate that the potential rupture surface is likely to emerge

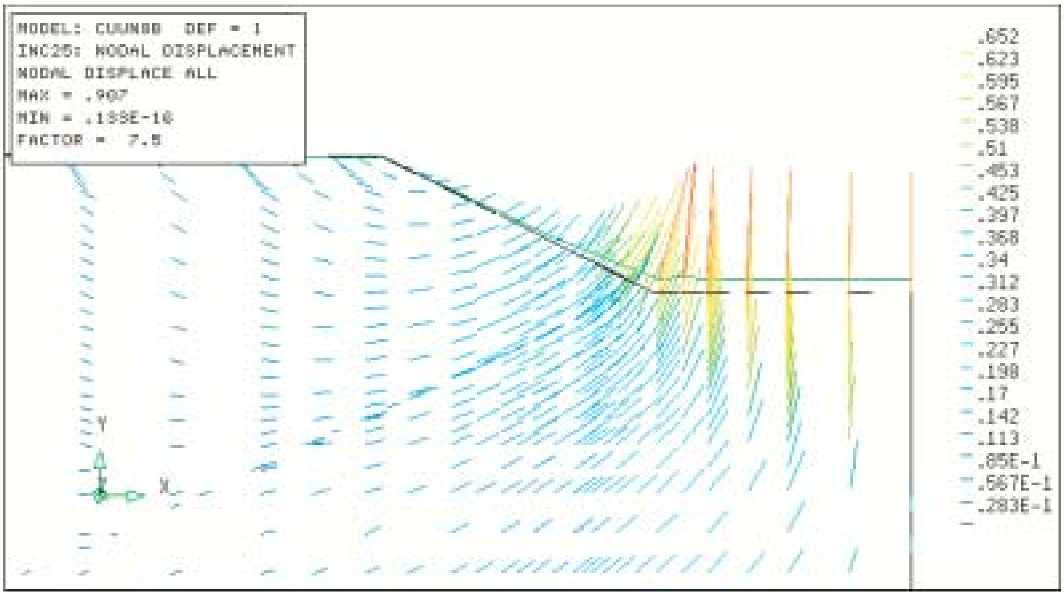


Figure 5 Ground movements after 60 years in service (unreinforced cutting slope in soft clay)

further behind the crest of the slope than would be predicted by conventional slope stability analysis (Section 6). However the latter method takes no account of the *in situ* stress regime and the stress history due to excavation of the cutting.

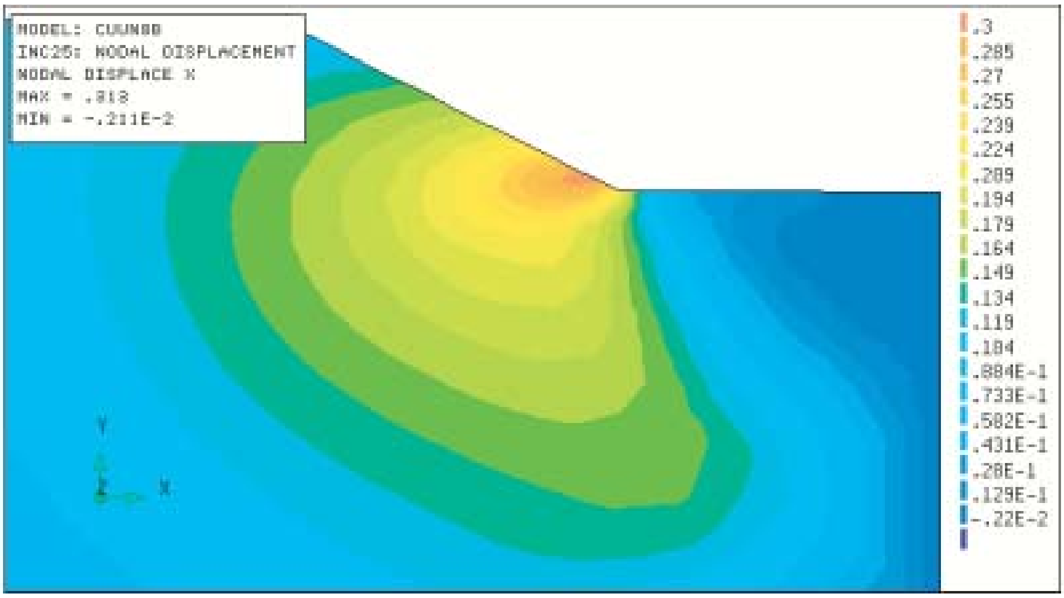
The installation of spaced piles does not significantly constrain the magnitude of ground heave, but will act to limit the lateral displacement of the ground. Figures 6a and 6b compare the contours of lateral displacement predicted for soil type B (soft clay) after 60 years in service for the natural (unreinforced) slope and a slope heavily reinforced by piles at 3 diameter centres. A close comparison of the lateral displacement results shows that slightly larger values are calculated at the toe of the unreinforced slope, but the main difference is that the action of the piles restricts the magnitude and extent of the displacement contours immediately behind the piles.

This effect is better demonstrated in Figure 7 which considers the deviator strains (which are about twice the magnitude of the shear strains) for the same analyses. Higher strains are shown developing beneath the body of the unreinforced slope with some indication of the initial development of a yield surface. In the case of the reinforced slope, the piles act to curtail the development of strain behind the piles although, as might be expected, there is evidence that a smaller slip could still eventually occur in front of the pile. There are early signs of a build-up in strain appearing near the slope surface in front of the piles.

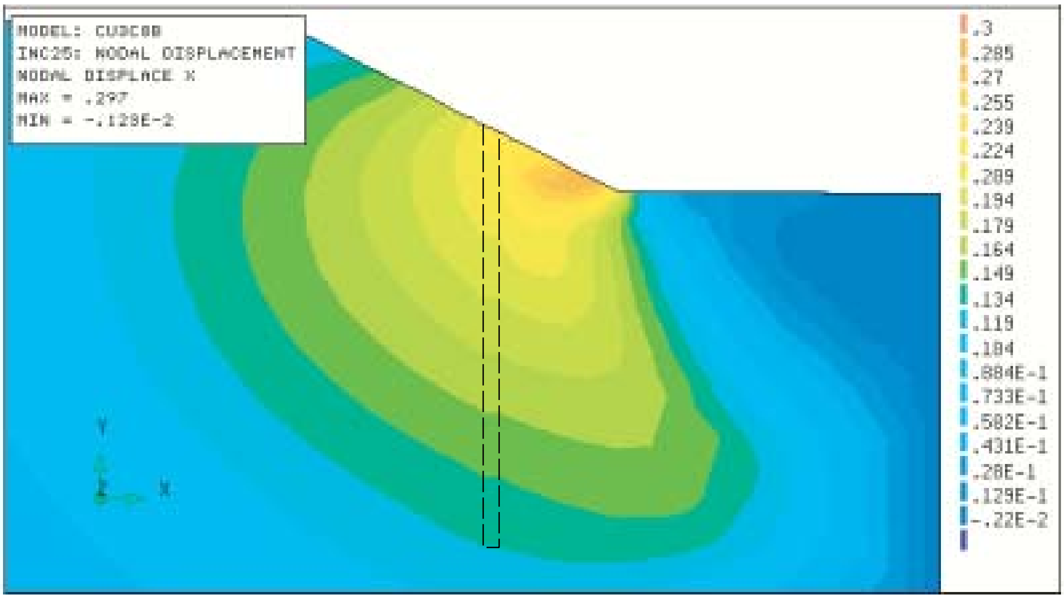
The deviator strains in Figure 7 are predicted after the slope has been in service for 60 years. As discussed previously a large component of these strains is caused by the initial unloading due to excavation of the cutting. If only the strain increment over the last 25 years in service is investigated, the results in Figure 8 are obtained. This type of contour plot has the advantage of only displaying areas where shearing is still continuing. In Figure 8b for example, the zone of likely yielding in front of the piles is more clearly defined. As before, strain in the unreinforced slope is slowly developing from the toe of the slope and would eventually be expected to produce a deep-seated failure which encompasses the complete slope.

The results in Figures 6, 7 and 8 are for the lower modulus clay (soil type B). Similar trends in behaviour are shown by analyses using the stiffer clay (soil type A), although the magnitudes of the displacements and strains developed are smaller. Figure A1 in Appendix A gives the deviator strains developed in the stiffer clay after 60 years in service: these can be compared with the results in Figure 7.

A summary of the lateral displacements of the piles and ground predicted after 60 years in service and for the last 25 years in service are shown in Figure 9. In this figure, pile displacements are compared both with that of the ground midway between two piles and also with that of the ground at the same location in an unreinforced (control) slope. Figures 9a and 9b show the calculated displacements with piles at 3 diameter and 6 diameter centres respectively. Inspection of the results indicates a large lateral movement of the toe of the piles at 19.5m depth. This is considered a consequence of the unloading due to excavation of the cutting and is expected to be less evident if the piles are founded in a stiffer stratum than was used in the analysis: this effect is investigated later. In Figure 9 it is the relative displacements, particularly at around 3m depth (ie. the depth from the top of the piles to cutting level), which are important rather than the absolute values. These analyses are based on a cutting slope in the clay of lower modulus (soil type B), the results for the stiffer clay show smaller displacements and are given in Figure A2 of Appendix A.

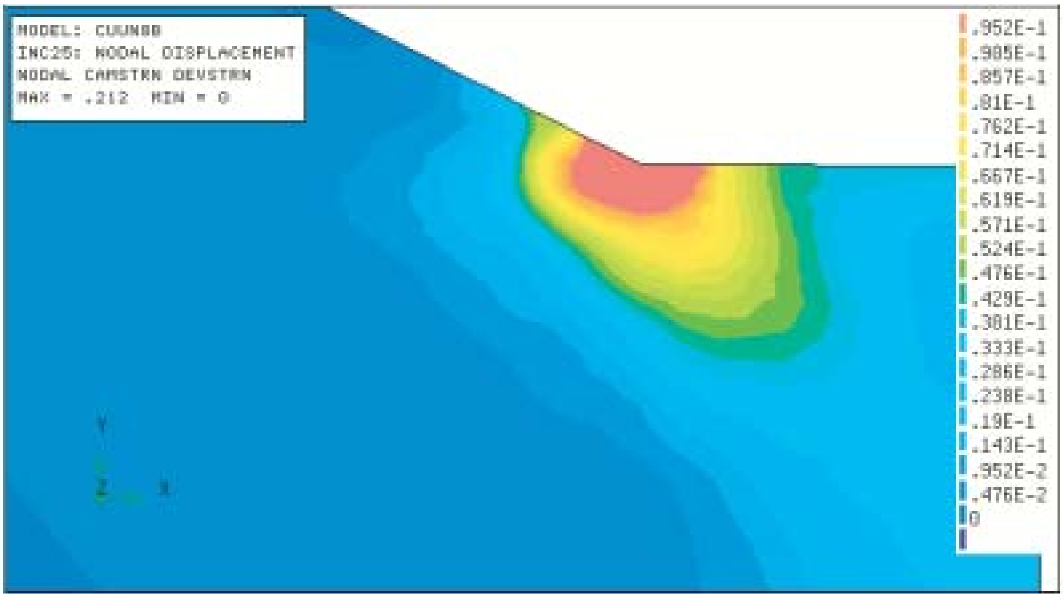


(a) Unreinforced slope

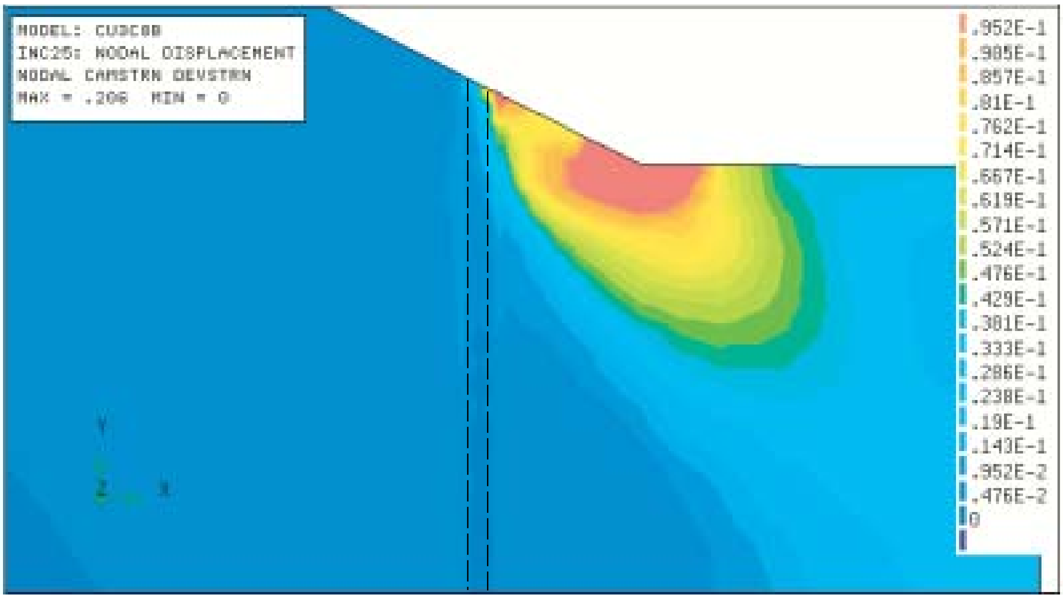


(b) Slope reinforced by piles at 3 diameter centres (cross section through pile line)

Figure 6 Lateral displacement after 60 years in service (cutting slope in soft clay)

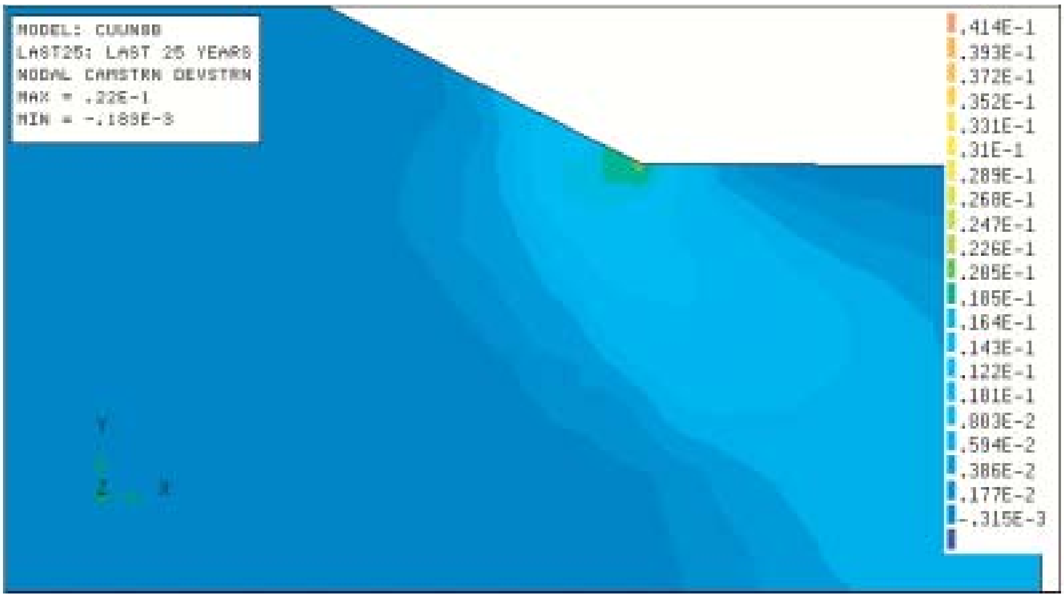


(a) Unreinforced slope

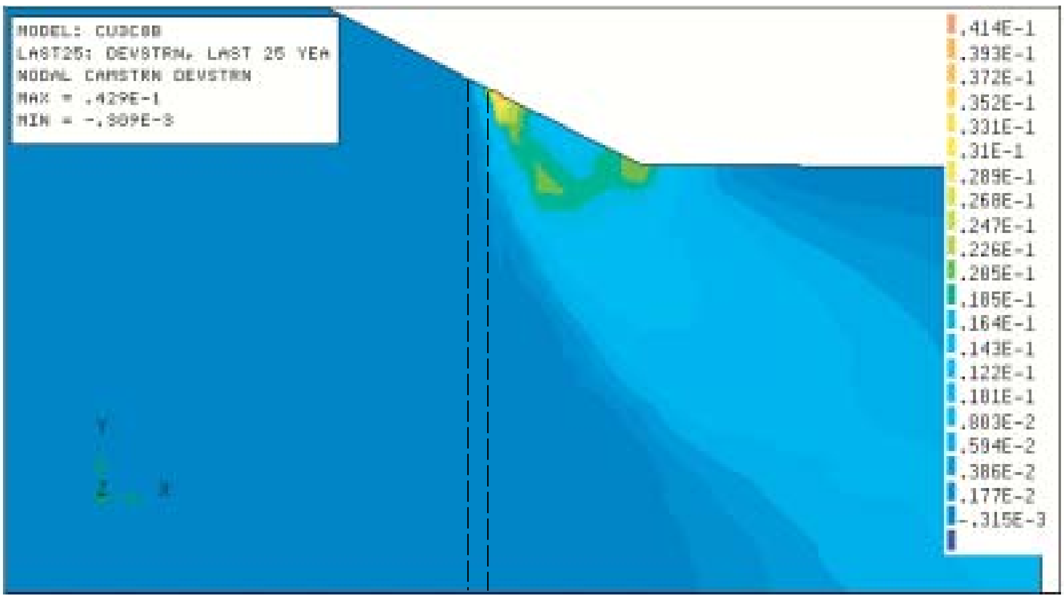


(b) Slope reinforced by piles at 3 diameter centres (cross section through pile line)

Figure 7 Deviator strain after 60 years in service (cutting slope in soft clay)

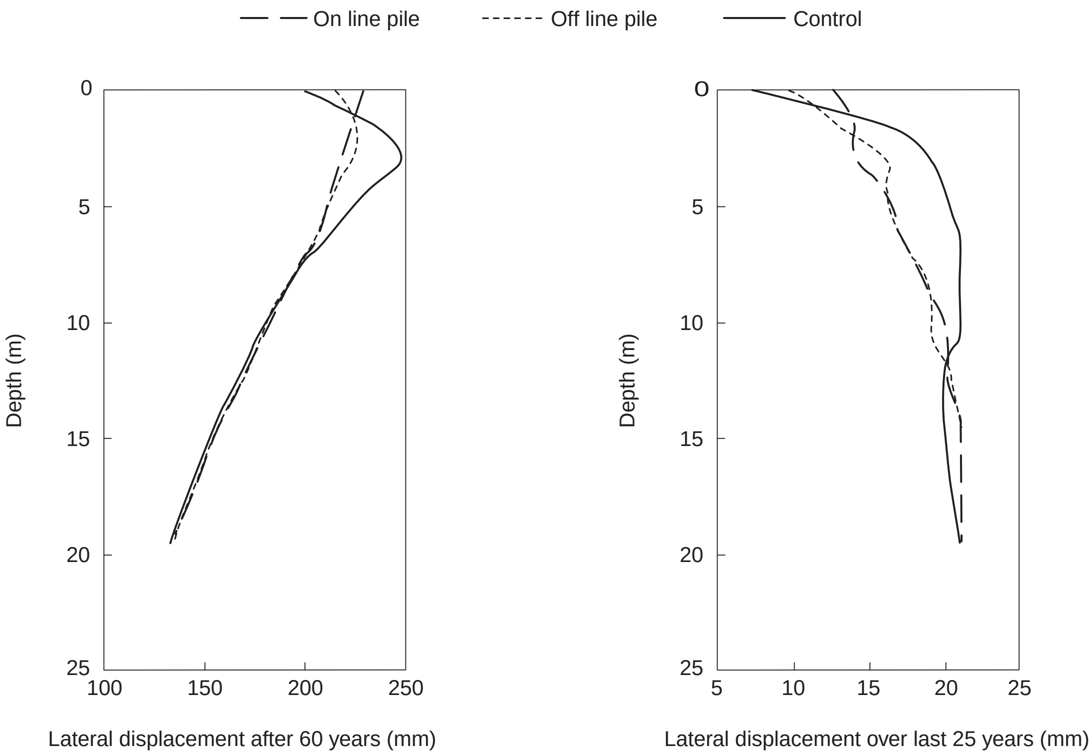


(a) Unreinforced slope

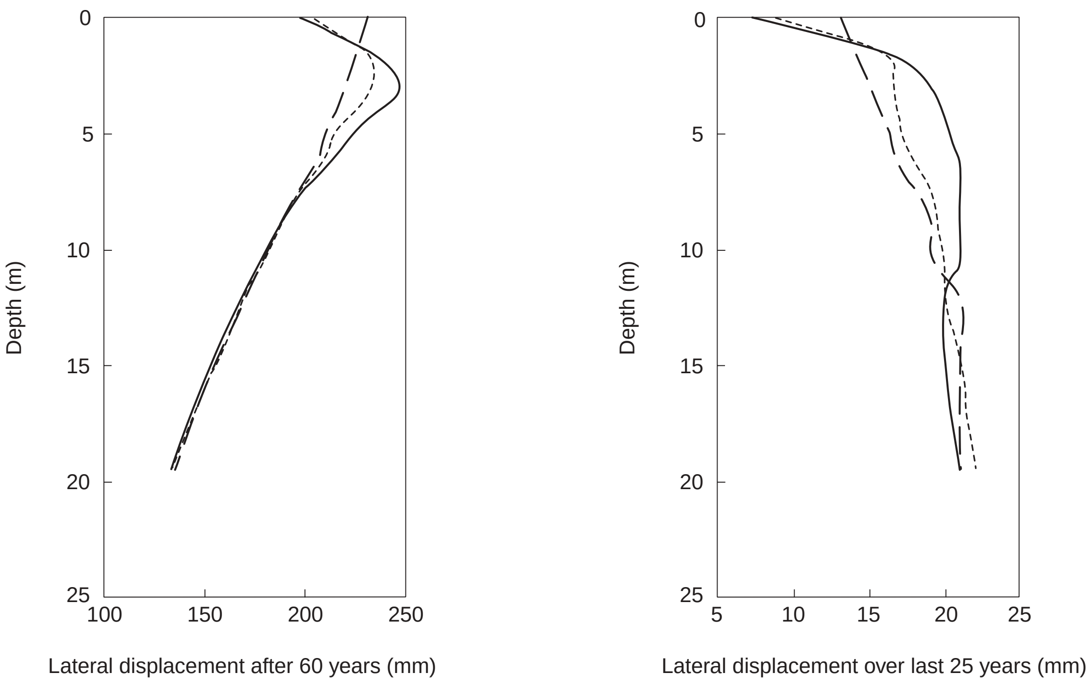


(b) Slope reinforced by piles at 3 diameter centres (cross section through pile line)

Figure 8 Deviator strain developed during the last 25 years in service (cutting slope in soft clay)



(a) Piles at 3 diameter centres



(b) Piles at 6 diameter centres

Figure 9 Displacement plots for cutting slope in soft clay

In Figure 9, lateral displacements after 60 years in service of the unreinforced slope peaked at about 3m depth and were significantly above those calculated for the piles at this depth. The ground displacements at this depth and midway between piles at 6 diameter centres (Figure 9b) were generally between those calculated on the pile line and those for the unreinforced slope. This indicated that at this pile spacing, soil flow between piles was starting to develop and was likely to become more significant if the piles were spaced further apart. When the piles were closely spaced at 3 diameter centres (Figure 9a), there was little differential displacement between the pile and adjoining ground. Also shown in Figure 9 are the incremental displacements over the last 25 years which tend to confirm the same mechanisms although it is more evident that displacements, particularly at depths of up to 10m, are continuing at a faster rate in the unreinforced than the reinforced slopes.

It is interesting to note that displacements near the surface of the slope are generally not a good indicator of overall performance in Figures 9 and A2. This is because a reversal in curvature occurs at shallow depths in the ground displacement plots which is not observed in pile displacement plots.

Figure 10 summarises the bending moment profile with depth for the different soil types and spacing between piles. As would be anticipated much higher bending moments of the pile are developed in the lower modulus clay as the larger ground movements mean that the piles have to do more work. The results in Figure 10 also show that the moment profiles in one particular soil type are very similar whether the piles are spaced at 3 or 6 diameter centres: this is primarily because the potential failure plane in a cutting slope is relatively deep-seated. In general the magnitudes of the bending moments are low and this is because of the pile toe movements discussed above which act to reduce pile moments. Larger moments are developed when the piles are founded in a stiffer stratum at depth and this situation is now considered.

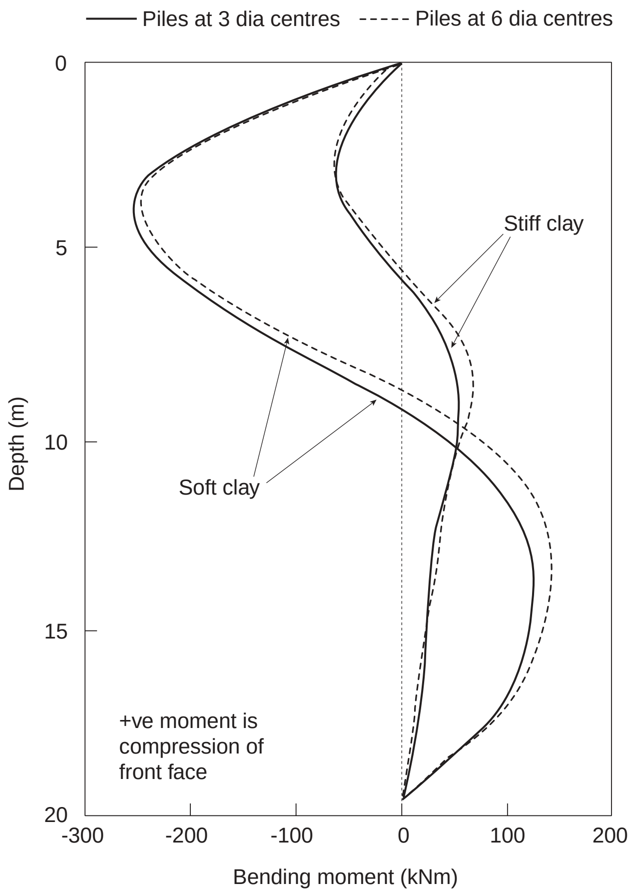


Figure 10 Pile bending moment in cutting slopes

7.1.1 Effect of stiffer stratum at depth

The magnitude of the heave developed because of the unloading during excavation of the cutting is sensitive to the stiffness of the ground below cutting level. Figure 11 shows the displacement vectors calculated for an unreinforced slope after 60 years in service when a stiffer stratum was introduced below a depth of 15.5m from the top of the slope (10.5m from the top of the pile, where present). This stratum was arbitrarily assigned a modulus of 150MPa which was constant with depth, although its other properties remained the same as for the overlying layer. Little ground movement was predicted in the stiffer stratum and vertical movements elsewhere were then generally about two thirds of those shown for homogeneous soil in Figure 5.

The effect of a stiffer stratum at depth was investigated for the piled slope where there was most slope movement, ie. for the soft clay with piles at the larger spacing of 6 diameters between centres. The various displacement plots at the pile and equivalent ground location, with and without the stiffer stratum, are compared in Figure 12. Only small lateral movements of either the pile or ground within the stiffer stratum were calculated. Overall displacements in the soft clay after 60 years in service (Figure 12a) were also significantly reduced with an underlying stiffer stratum. Once again the pattern of behaviour indicated that with piles at 6 diameter centres, some soil flow was starting to develop between the piles. However in general, there was a larger difference in displacements between the piled and natural slope when a stiffer underlying stratum was present in both cases. This is also demonstrated by the results in Figure 12b where the incremental displacements over the last 25 years indicate that movement of the unreinforced slope is proceeding at a much faster rate than that of the piled slope.

The reduction in the magnitude of ground movements that results from founding the piles in a stiffer stratum is significant with more of the perturbing load being borne by the piles and hence a much larger bending moment develops in the pile near to the interface between the soft clay and the stiffer stratum as shown in Figure 13. For example, in the case of piles spaced at 6 diameter centres, the peak bending moment (compression of the front face of the pile) increases from 140 to 2300kNm when the pile is founded in the stiffer stratum. In design, attention is then needed to ensure that failure does not occur with the development of a plastic hinge in the pile.

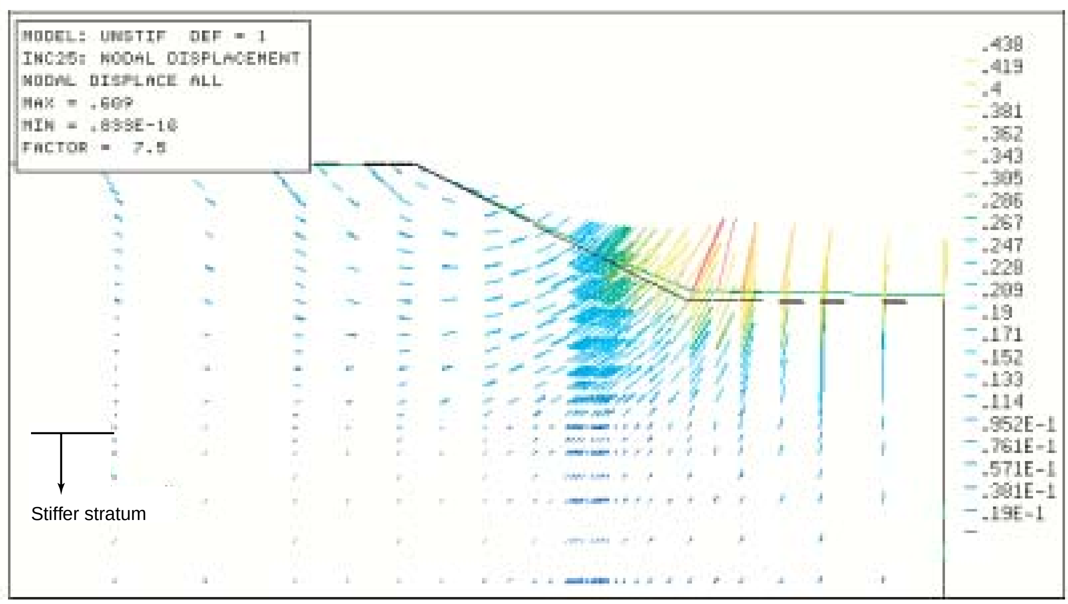


Figure 11 Ground movements after 60 years in service with a stiff underlying stratum (cutting slope in soft clay)

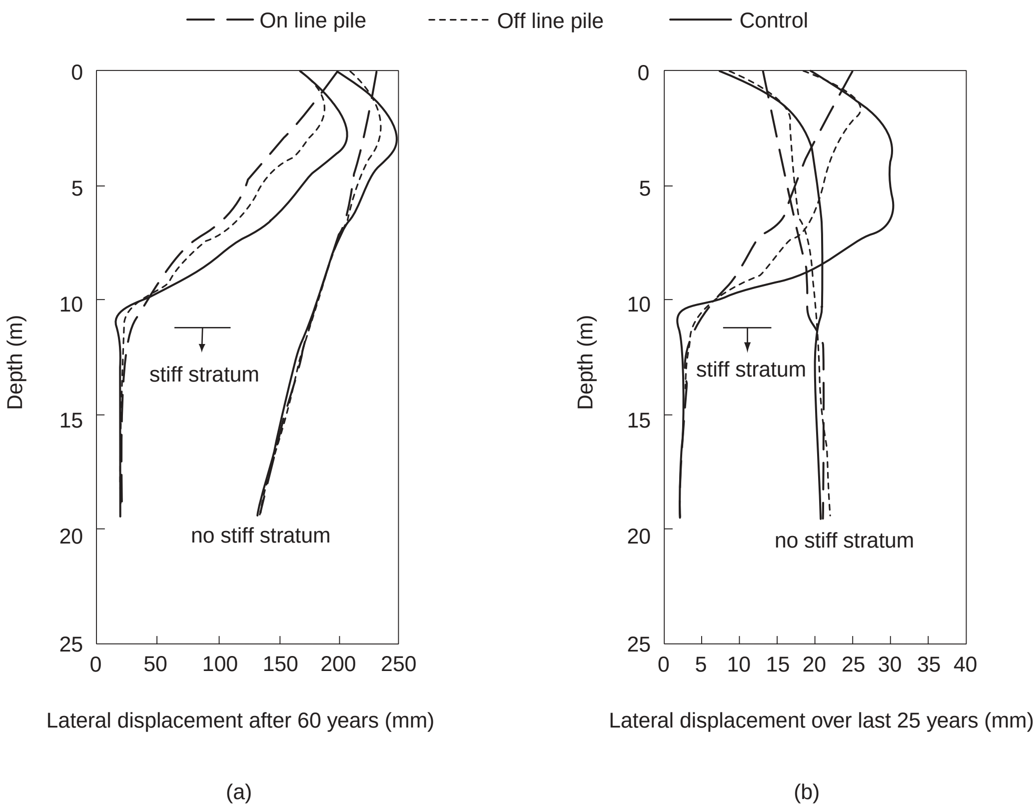


Figure 12 Effect upon displacement of founding the pile in a stiffer stratum (cutting slope in soft clay; piles at 6 diameter centres)

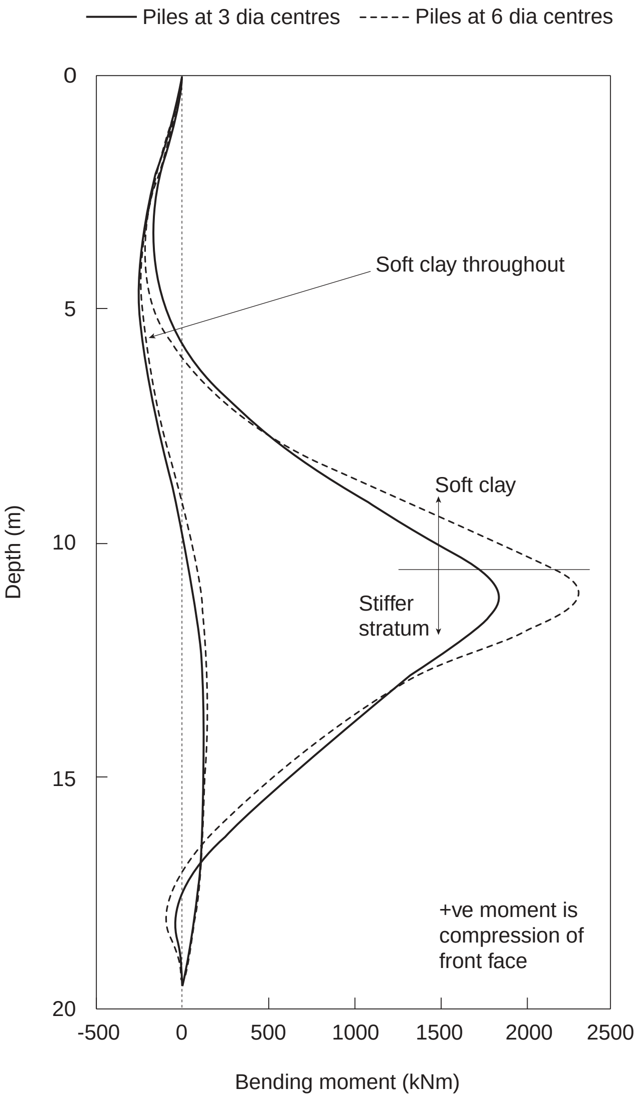


Figure 13 Pile bending moments developed after 60 years in service (cutting slope)

7.1.2 Effect of increasing the slope angle

The sensitivity of the results to slope angle was investigated by repeating selected analyses using a steeper slope angle (about 34^o) corresponding to 1v:1.5h. These analyses assume the soil is homogeneous and do not take into account the effect of a stiffer stratum at depth. A comparison of the magnitude and depth of the peak lateral displacements of the ground are shown in Table 2. The actual magnitudes of the displacements are again influenced by the unloading effect due to excavation of the cutting and it is the relative magnitudes rather the absolute values which are important.

The trends given in Table 2 indicate that for the two slope angles, both the depth to the peak movement and the magnitude of the movement decreased as the number of piles increased. This in itself is a justification of the reinforcing effect which piles will have on a marginally stable clay slope. The results in Section 6.1 where simple slope stability analyses were undertaken suggest that the factor of safety of the natural slope reduces from 1.32 to 1.17 with the change in slope angle. The finite element modelling confirmed that steepening of the slope angle

Table 2 Peak lateral displacements for soft clay cuttings of varying angle

	Slope 1v:2h		*Slope 1v:1.5h*	
	Depth (m)	*Lateral displace-ment (mm)*	*Depth (m)*	*Lateral displace-ment (mm)*
Piles at 3 diameter centres	2.00	225	1.65	255
Piles at 6 diameter centres	2.40	235	1.75	266
Unreinforced slope	2.80	248	1.85	276

Displacements are calculated in the ground midway between piles or at the equivalent location in the unreinforced slope case.

induced more lateral displacement of the ground, although generally more yielding would be anticipated before failure.

The corresponding peak positive and negative bending moments calculated in a pile from the analyses using the two slope angles are given in Table 3. Generally the peak negative bending moments (compression of the back face of the pile) were predicted at about 4m depth, ie. a few metres below the depth at which the maximum lateral movements occurred. Peak positive moments were calculated at depths ranging between 12.6m to 14.6m. The largest peak positive moment of 206kNm was determined with the piles at 6 diameter centres for the 1v:1.5h slope which tended to confirm that, when the piles were at the larger spacing in the steeper slope, they were being called on to provide most bending moment resistance.

Table 3 Peak pile bending moments for soft clay cuttings of varying angle

	Slope 1v:2h			*Slope 1v:1.5h*		
		Peak bending moment (kNm)			*Peak bending moment (kNm)*	
	Depth (m)	*+ve*	*-ve*	*Depth (m)*	*+ve*	*-ve*
Piles at 3	4.2	–	-255	3.8	–	-150
diameter centres	14.6	125	–	13.2	170	–
Piles at 6	4.1	–	-250	3.7	–	-130
diameter centres	13.5	140	–	12.6	206	–

Positive bending moments are compression of the front face of the pile.

7.1.3 Effect of further softening of the clay

It is important to emphasise again that the soil strength parameters generally used in this report relate to pre-failure behaviour as a strain softening model was not used. For the analyses, the operational strength was reduced by using a value of φ' below its peak. However the extent of softening is not easily quantified and soil parameters are expected to lie between the peak and residual strength (Potts *et al.*, 2000). The effect of a further reduction in strength was therefore crudely evaluated by assigning lower soil parameters of c'=5kPa and φ'=18^o to the clay within the slope area whilst all other parameters were kept the same. The general trends of behaviour remained the same as before although the magnitudes of the movements and pile bending moments increased. For example, with a slope of 1v:2h, the maximum displacements calculated in the ground midway between piles (at both 3 and 6 diameter centres) increased by about 25%. The lateral displacement of the ground at an equivalent location in the unreinforced slope increased slightly more, ie. 30%.

It must be pointed out that as the residual strength of the clay is approached, the appropriateness of a continuum model such as finite elements must be treated with caution as discontinuities may develop.

7.2 Piles installed 20 years after construction

When piles are installed as a remedial measure some 20 years after construction, a considerable part of the heave related movement due to the unloading caused by excavation of the cutting has occurred. Figure 14 shows the lateral displacement plots at the pile and equivalent ground location which occur in the following 40 years after pile installation to reinforce the slope. Movements of the natural slope over the same period are larger confirming that the piles provide a restraint to ground movement. These values can be compared with those in Figure 9a for the full 60 years life in service which, as expected, are an order of magnitude larger.

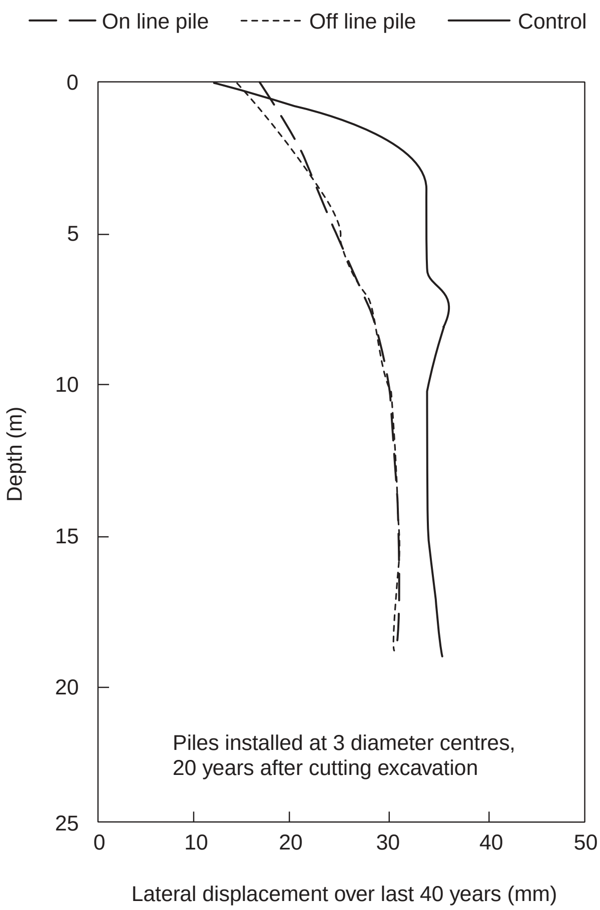


Figure 14 Displacement plot for cutting slope in soft clay (40 years after installing piles)

Changes in deviator strain over the last time increment are similar to those shown in Figure 8 and again show that, although soil strain behind the pile and the possibility of an overall failure are reduced by installing piles, a smaller slip may develop in front of the piles in the much longer term. It was concluded that in impermeable clays the dissipation of negative excess pore pressures may take many decades. For this reason, benefits for cutting slopes of marginal stability will still accrue if reinforcing piles are installed after construction and well into the 60 year service life of the slope.

8 Results from analyses of embankment performance

The stabilising effect of a single row of spaced piles on an embankment slope constructed from clay fill was investigated. Throughout the analyses, the soft clay (soil type B in Table 1) was used for the fill material and the underlying stiff clay foundation comprised soil type A. The effects of installing the piles at the time of construction and 20 years after slope construction were studied separately.

8.1 Piles installed at the time of construction

The overall pattern of movement predicted after an unreinforced embankment has been in service for 60 years is shown in Figure 15. Whereas with the cutting situation the development of movement was dominated by a rotational heave due to the unloading caused by excavation, this does not occur in the embankment model. Settlement occurs mainly in the embankment fill with some in the foundation clay due to the applied surcharge loading. Only very small lateral movements are predicted beneath the granular drainage blanket with indications that potential rupture is confined to the slope above the drainage blanket, this correlates with the findings of the slope stability analysis given in Figure 4. The zone of rupture in the embankment case is generally much smaller than that predicted for a cutting slope (Figure 5).

A closer examination of the contours of lateral movement for an unreinforced and reinforced embankment is given in Figure 16. For the unreinforced embankment (Figure 16a), the cut-off in lateral movement at the drainage blanket and the potential development of rupture from a location near to the toe of the slope are evident. For a slope with piles at 3 diameter centres, the lateral ground movement on a cross-section through the line of one of the piles is shown in Figure 16b. In this case the piles appear effective in minimising movement of the slope behind them. Some localised movement still occurs at the toe of the slope but it is much reduced in magnitude from that which is determined for the unreinforced slope.

A study of the deviator strains (which are about twice the magnitude of the shear strains) confirms this behaviour. Figure 17a shows the strains in the unreinforced slope and indicates that a potential rupture surface is likely to initiate at the toe of the slope and

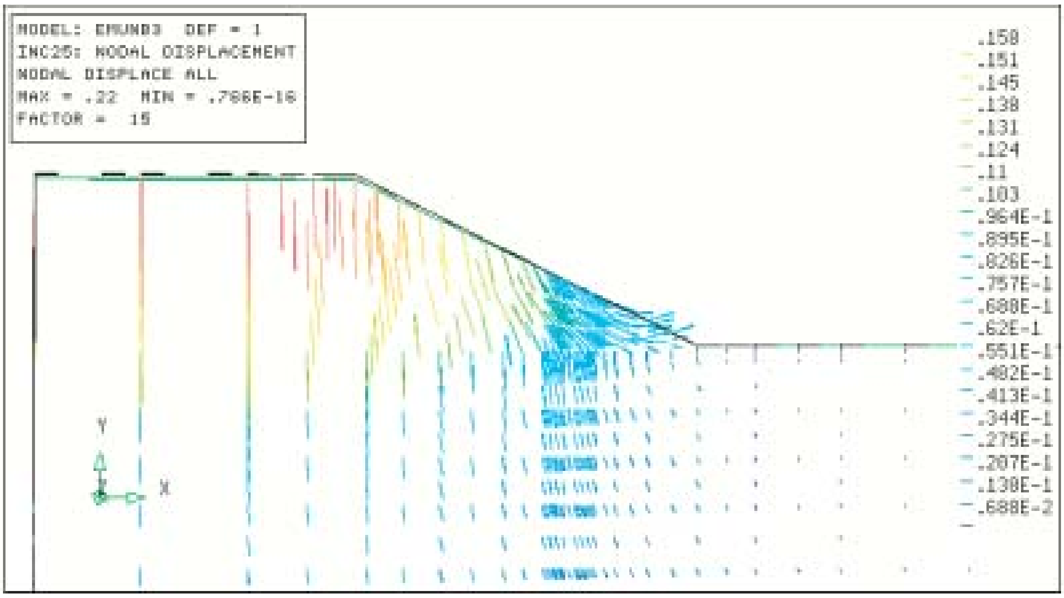


Figure 15 Ground movements after 60 years in service (unreinforced embankment slope)

spread upwards. This mechanism has been reported by other authors (eg. Potts *et al.*, 1997). The effect of pile installation shown in Figure 17b virtually halves the magnitude of the deviator strain at the toe of the slope and there is little sign of any significant strain development upslope of the pile after the embankment has been in service for 60 years.

An overall assessment of slope performance based on an analysis of lateral movements is given in Figure 18. The results demonstrate that lateral movement of the ground midway between piles installed at 3 diameter centres is only slightly more than that of a pile, although both results are considerably less than those of the ground at an equivalent location in an unreinforced slope. Figure 18b suggests that for piles installed at 6 diameter centres there is significantly greater differential movement between the piles and the ground midway between piles. Although piles at this spacing still act to restrain ground movements when compared with those in an unreinforced slope, the increase in differential movements means that, with further softening of the ground, flow of the clay fill between piles may eventually lead to instability.

Figure 19 shows the distributions of pile bending moment with depth obtained for piles spaced at both 3 and 6 diameter centres. In both cases the bending moment (compression of the front face of the pile) increases with depth to a maximum just above the drainage blanket level. The maximum moment for piles installed at the larger spacing is about three times greater than that for piles at 3 diameter centres. This is not unexpected for when the piles are less closely spaced, individual piles carry more load in supporting the slope. Below the drainage blanket (which is assumed to consist of granular filter material), the pile moment reduces rapidly with depth in both cases. In these particular analyses, the small moments developed towards the pile toes indicate that the length of the piles could safely be reduced without affecting slope stability. This was confirmed by a series of analyses with a reduced pile penetration in which it was demonstrated that near identical movement and bending moment profiles to those shown in Figures 18 and 19 were predicted when the penetration was reduced progressively from 19.5m to 10.5m.

8.1.1 Effect of increasing the slope angle

The effect of increasing the slope angle from 1v:2h (about 27°) to a steeper angle of 1v:1.5h (about 34°) was investigated and a comparison of the depth to and the magnitude of the peak lateral movements is shown in Table 4.

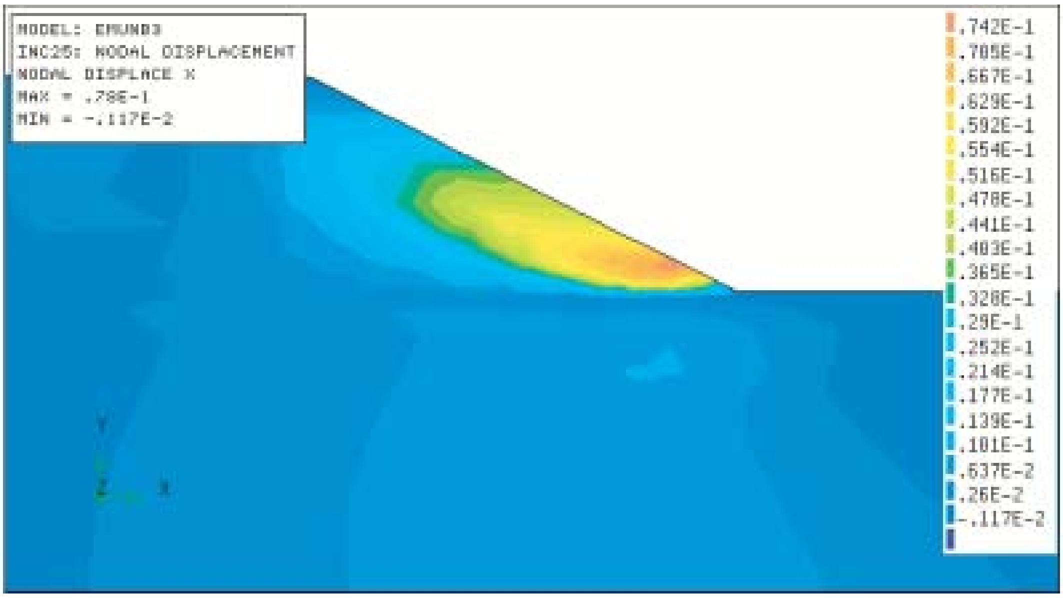
Table 4 Peak lateral displacements for clay embankments of varying angle

	Slope 1v:2h		*Slope 1v:1.5h*	
	Depth (m)	*Lateral displace-ment (mm)*	*Depth (m)*	*Lateral displace-ment (mm)*
Piles at 3 diameter centres	0.0	14.8	0.0	28.6
Piles at 6 diameter centres	1.2	30.0	0.8	54.0
Unreinforced slope	1.8	59.0	1.6	101.0

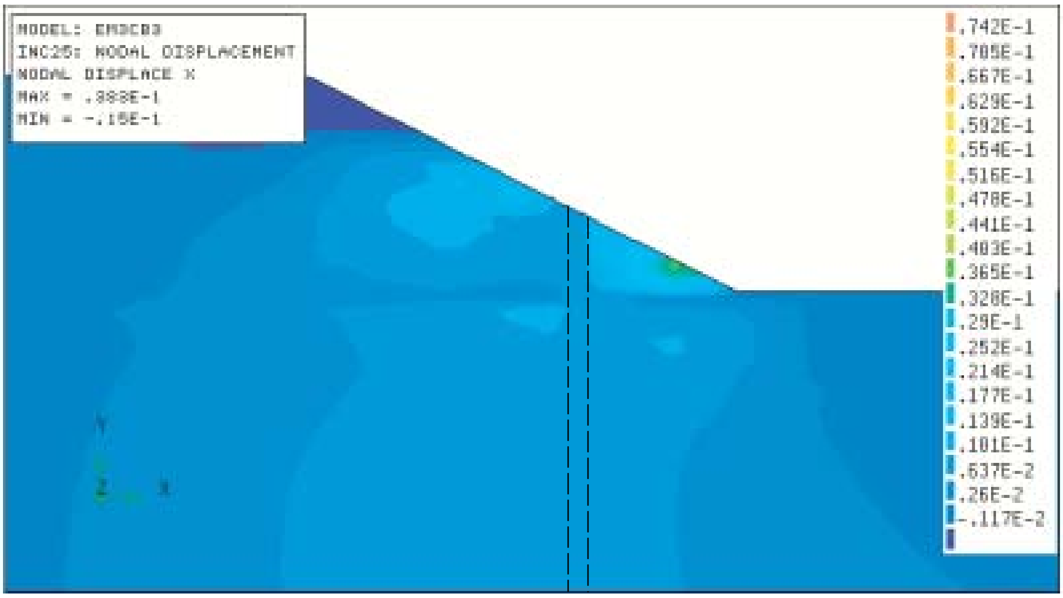
Displacements are calculated in the ground midway between piles or at the equivalent location in the unreinforced slope case.

A number of trends are evident from the results in Table 4. Firstly, the magnitude of the lateral displacements increased significantly with the change in slope angle. Displacements of the unreinforced slope increased by 42mm whilst those of the more heavily reinforced slope increased by about 14mm.

For the same analyses, a comparison of the peak positive and negative bending moments determined in the piles are given in Table 5. The results in Table 5 confirm that a significant increase in pile bending moment

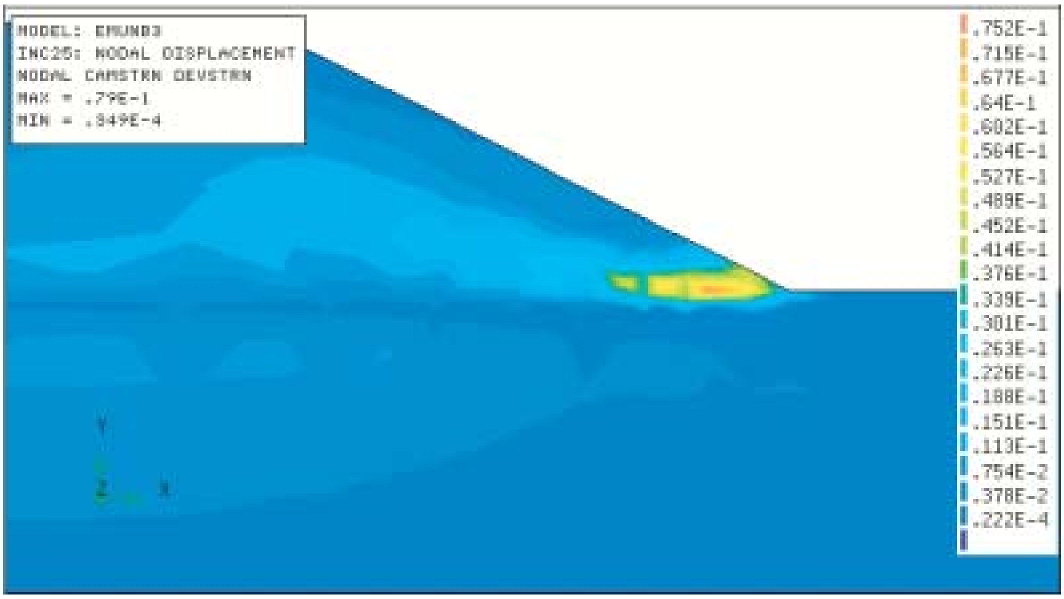


(a) Unreinforced slope

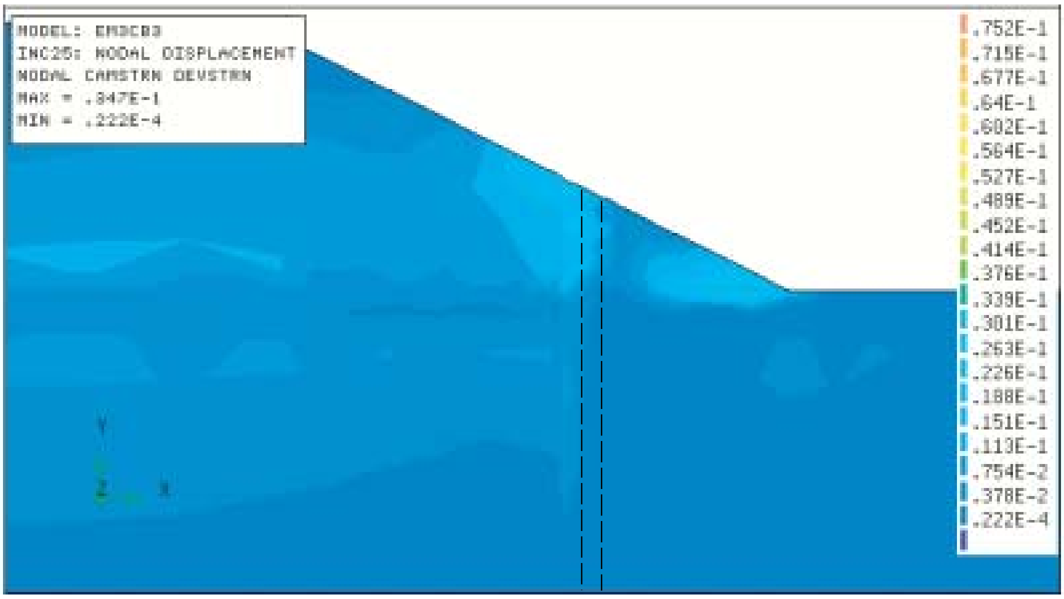


(b) Slope reinforced by piles at 3 diameter centres (cross section through pile line)

Figure 16 Lateral displacements after 60 years in service (embankment slope)

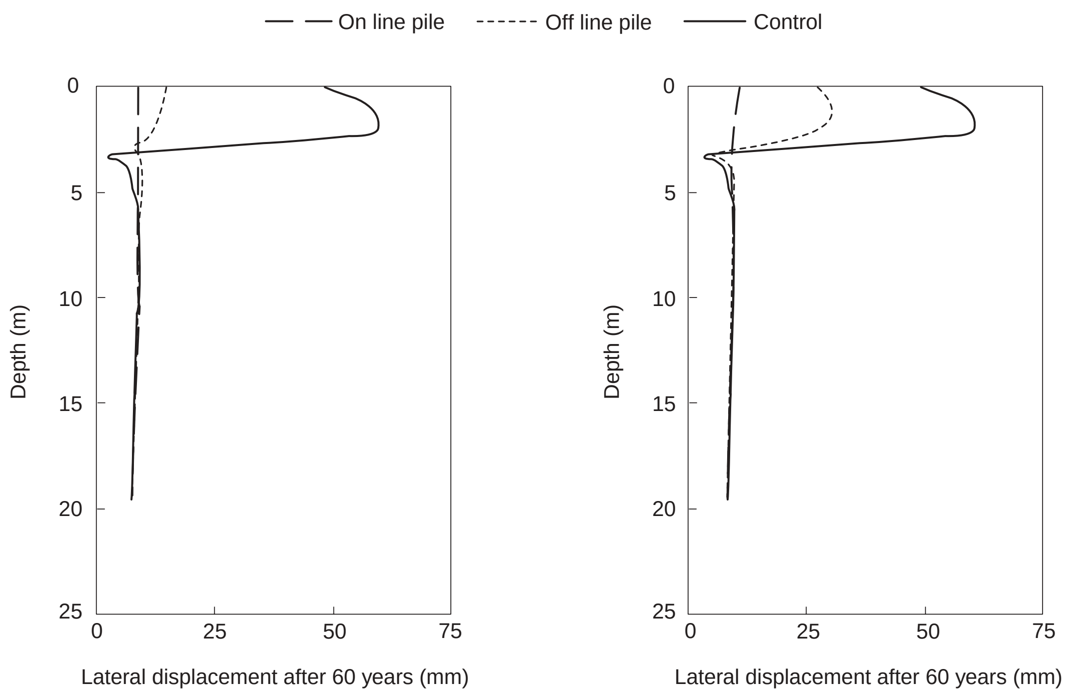


(a) Unreinforced slope



(b) Slope reinforced by piles at 3 diameter centres (cross section through pile line)

Figure 17 Deviator strain developed after 60 years in service (embankment slope)



(a) Piles at 3 diameter centres

(b) Piles at 6 diameter centres

Figure 18 Displacement plots for embankment slopes

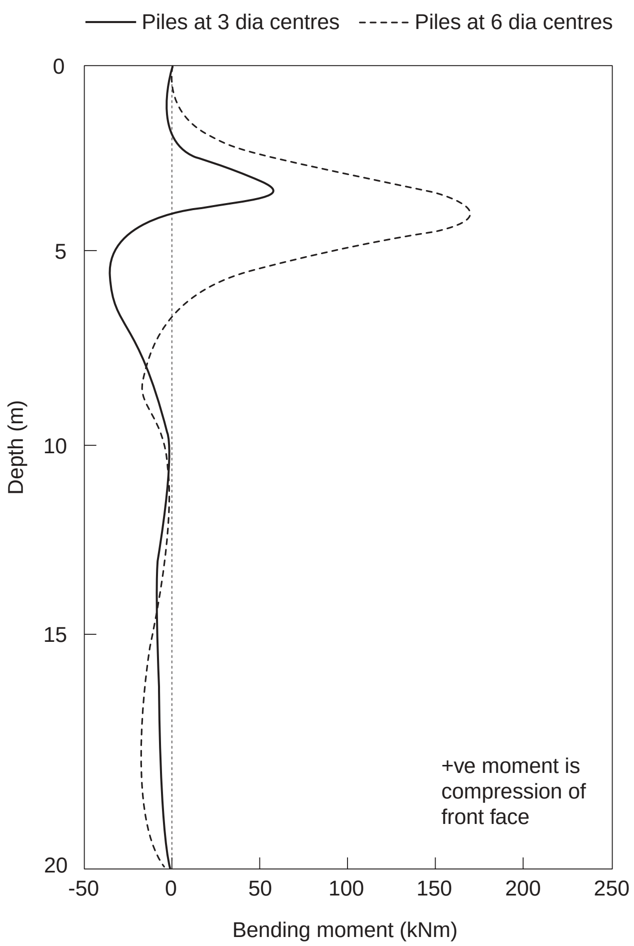


Figure 19 Pile bending moment in embankment slopes

Table 5 Maximum and minimum pile bending moments for embankment slopes of varying angle

	Slope 1v:2h			*Slope 1v:1.5h*		
	Depth (m)	*Peak bending moment (kNm) +ve*	*-ve*	*Depth (m)*	*Peak bending moment (kNm) +ve*	*-ve*
Piles at 3	3.2	57	–	3.3	115	–
diameter centres	5.4	–	-35	5.4	–	17
Piles at 6	3.8	168	–	3.9	225	–
diameter centres	7.8	–	-16	6.3	–	60

i Positive bending moments are compression of the front face of the pile.

ii Only peak positive and negative moments over the top 10m of the pile are given, some small strain reversals occur at depth.

occurred when the slope angle was steepened and that the higher peak positive values were predicted when the piles were widely spaced and carrying more of the perturbing load. In the steepened slope situation, pile bending moments reduced more slowly with depth and this confirmed that a deeper pile embedment would be advantageous for steeper slopes.

8.1.2 Effect of different pore water pressure regimes and the drainage blanket

The major part of the analyses of embankment performance were carried out using the pore water pressure fixities described in Section 3. After long-term consolidation the final pore water pressure distribution is then as shown Figure 20a. This illustrates the perched water table in the centre of the embankment which initially increases with depth and then reduces again at the level of the drainage blanket. Although not shown in the figure, pore water pressures in the foundation clay below the drainage blanket then increase hydrostatically with depth. Under this pore water pressure regime, suctions develop near the slope face as would be expected.

In some situations, vertical ingress of water to the embankment may result in higher pore pressures within the clay fill and the pore pressure fixities were therefore modified to produce the final regime shown in Figure 20b. Analyses using the modified pore pressure fixities showed the same trends of behaviour as those described earlier for the regime in Figure 20a. However, because of the higher pore water pressures within the slope, ground movements were up to 12% greater.

More significant differences in slope behaviour are predicted if either the embankment is constructed directly on its soil foundation without using a drainage blanket or for the cases where the drainage blanket fails to operate satisfactorily. The final pore water pressure regime then follows the pattern shown in Figure 20c with the drawdown in water pressures near to the drainage blanket not being observed. The respective displacements calculated after 60 years in service are shown in Figures 21a and 21b when using piles at 3 and 6 diameter centres. The movements in Figure 21 can be compared with those reported in Figure 18 where a drainage blanket was present. In Figure 21, the higher pore water pressures and absence of the permeable filter material comprising the drainage blanket mean that the ground movements which occur are not only larger but more deep-seated. For this reason the distributions of pile bending moments with depth which are shown in Figure 22 demonstrate much larger maximum moments developing at a greater depth than those shown in Figure 19 for the case where a drainage layer is present.

The results clearly demonstrate the likely effectiveness of a drainage blanket upon both unreinforced and reinforced slope performance. The reinforcing effect of a row of piles remains similar to before, in so far as piles at 3 diameter as opposed to 6 diameter centres limit the potential for flow of the clay fill between the piles.

8.1.3 Effect of further softening of the clay

The influence of water ingress and strain softening upon the strength parameters were found to have a major influence upon embankment performance. When lower strength parameters of $c'=5$kPa and $\varphi'=18^{o}$ were assigned to the clay fill, the predicted contours of lateral movement and deviator strain for the unreinforced and reinforced slopes were as shown in Figures 23 and 24. These contours can be compared with those given earlier in Figures 16 and 17 for the higher strength parameters. Maximum lateral movements with the reduced strengths were generally greater by up to about three times. In the case of the unreinforced slope, the contours of movement (Figure 23a) and strain (Figure 24a) indicate a much more advanced development of the potential rupture surface. In the case of the piled slope there are also some very early signs in Figure 24b of an increase in the deviator (ie. the shear) strain upslope of the piles; this is accompanied by some movement development as shown in Figure 23b. Once again the restraining action of the piles and the enhanced stability of the piled slope are very much in evidence.

8.2 Piles installed 20 years after construction

The effect of installing piles as a remedial measure at 20 years after construction of the embankment was also separately investigated. However evaluation of the movements of both the unreinforced and reinforced slopes over the subsequent 40 years showed only maximum settlements of 4mm and lateral movements of 2mm.

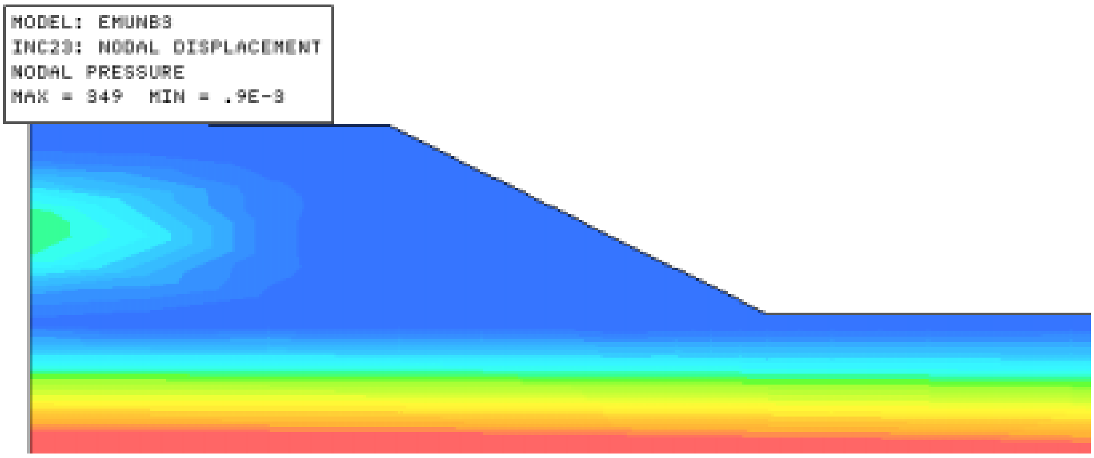
It was concluded that the major part of the movements shown in Figure 16a for the full 60 year service life of the slope therefore developed in the early part of its service. This is primarily because the permeability adopted for the clay fill, ie. ten times more permeable than *in situ* clay, was such that excess pore pressures generated by the embankment loading dissipated over the first few decades.

On this basis, any potential instability in clay embankments would be expected to develop more rapidly than in a cutting situation and installation of bored piles as a reinforcing measure to restrict movements is likely to be more effective the earlier it is carried out. However if, for site specific reasons, embankment instability and excessive movements develop at a later stage, the installation of spaced bored piles will still be beneficial but their performance is not covered by the analyses in this report. Site specific reasons which may lead to delayed collapse are expected to be mainly related to progressive softening of the clay fill, causes of which might include defects or failure of the drainage systems, or possibly a change in the pattern of water flow due to new construction nearby.

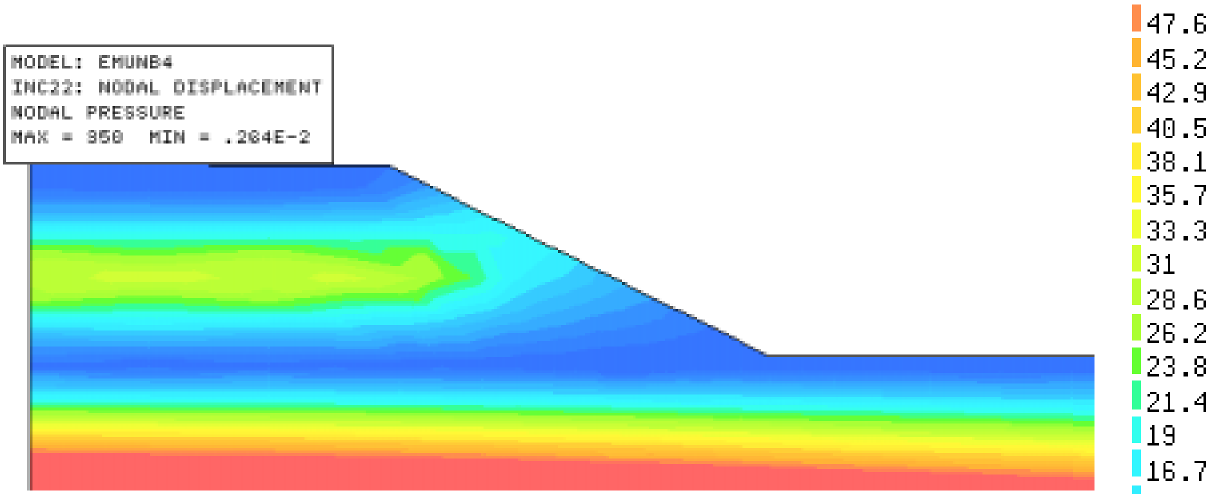
9 Discussion

The finite element analysis of the pre-failure condition has given some guidance on the effect of changes in the spacing between piles and the importance of founding the piles in a firm stratum. The predicted magnitudes of pile bending moments have also been discussed. However, in order to enhance the understanding of the various, complex mechanisms of behaviour, it is worth briefly reviewing the lateral stresses acting on an individual pile in the row.

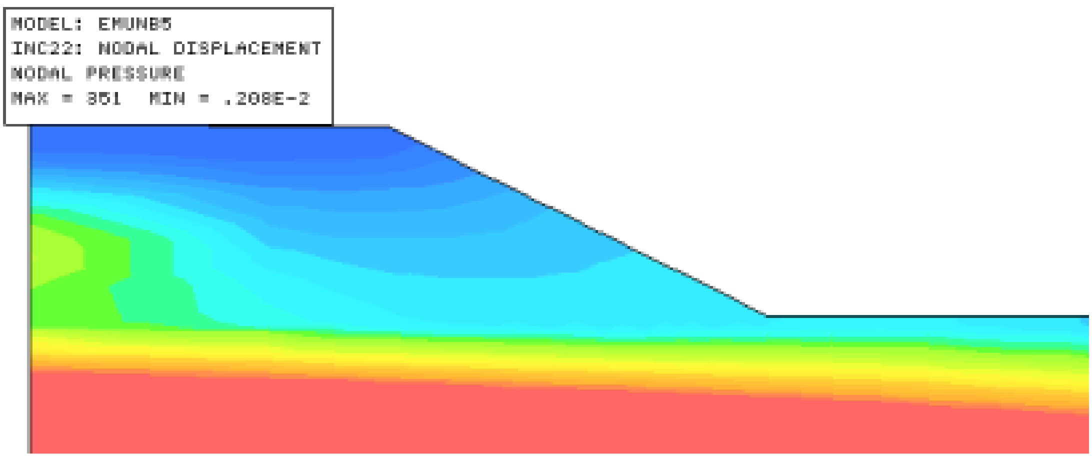
Figure 25 contrasts the very different stress distributions acting on a pile in the cutting and embankment situations. In this figure the contour plots are taken on a cross-section through one of the piles in a line of piles at 6 diameter centres. In the cutting situation (Figure 25a) the long-term development of rotational heave of the over-consolidated clay coupled with the pile action in resisting the development of a deep-seated rupture surface act in such a way that high lateral stresses are developed on the downslope side near the top of the pile. As would be anticipated lateral stresses on the upslope side of the top of the pile are small.



(a) Drainage blanket present



(b) Drainage blanket present plus increased pore pressure in the slope



(c) Case (a) without a drainage blanket

Figure 20 Pore water regime after 60 years in service (embankment slope)

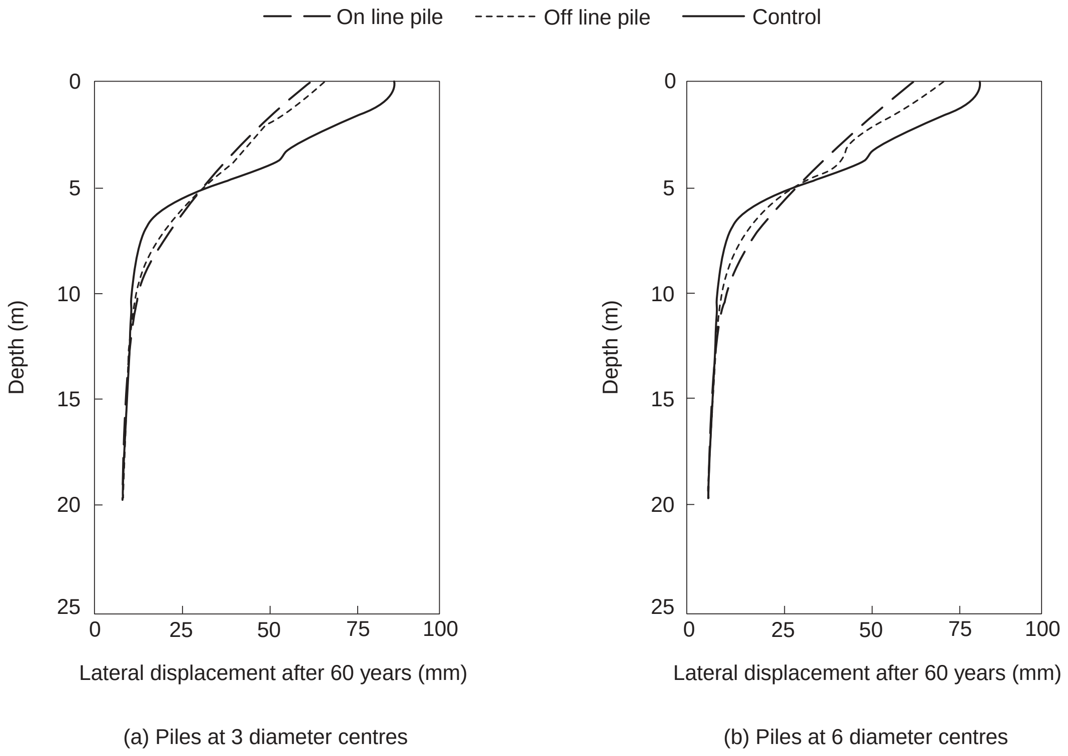


Figure 21 Displacement plots for embankment slopes without a drainage blanket

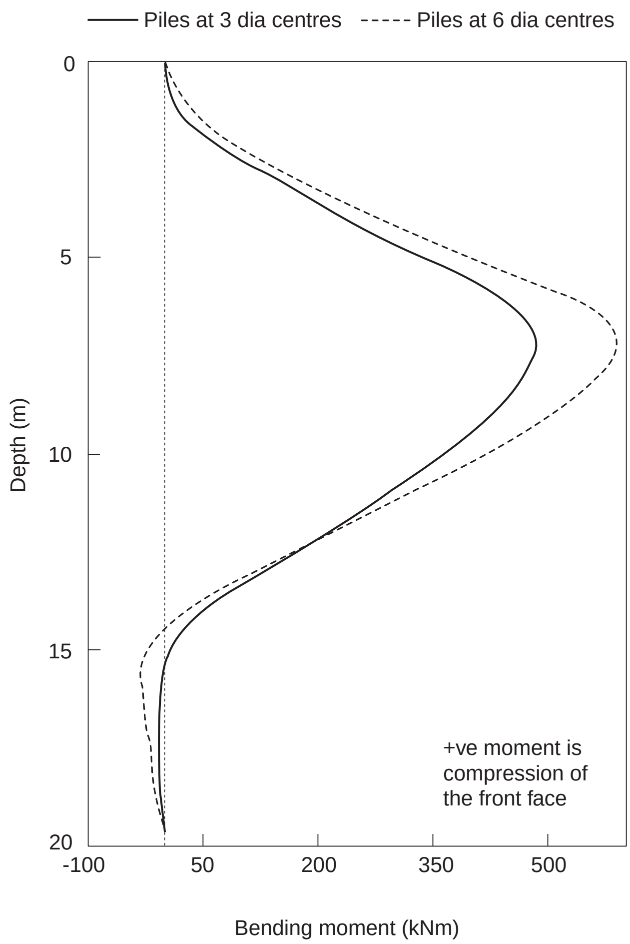
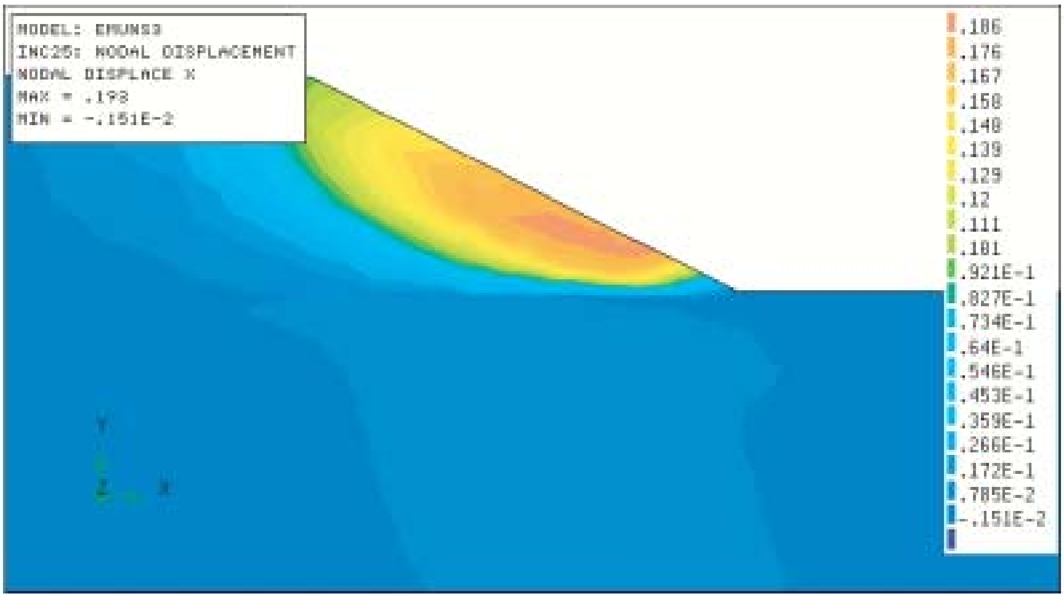
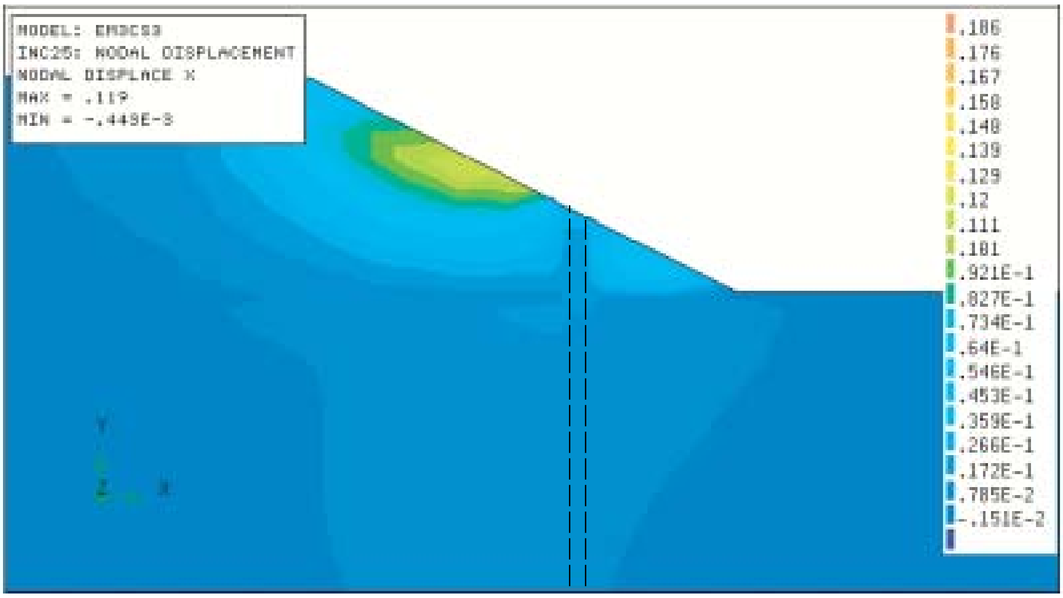


Figure 22 Pile bending moment in an embankment without a drainage blanket

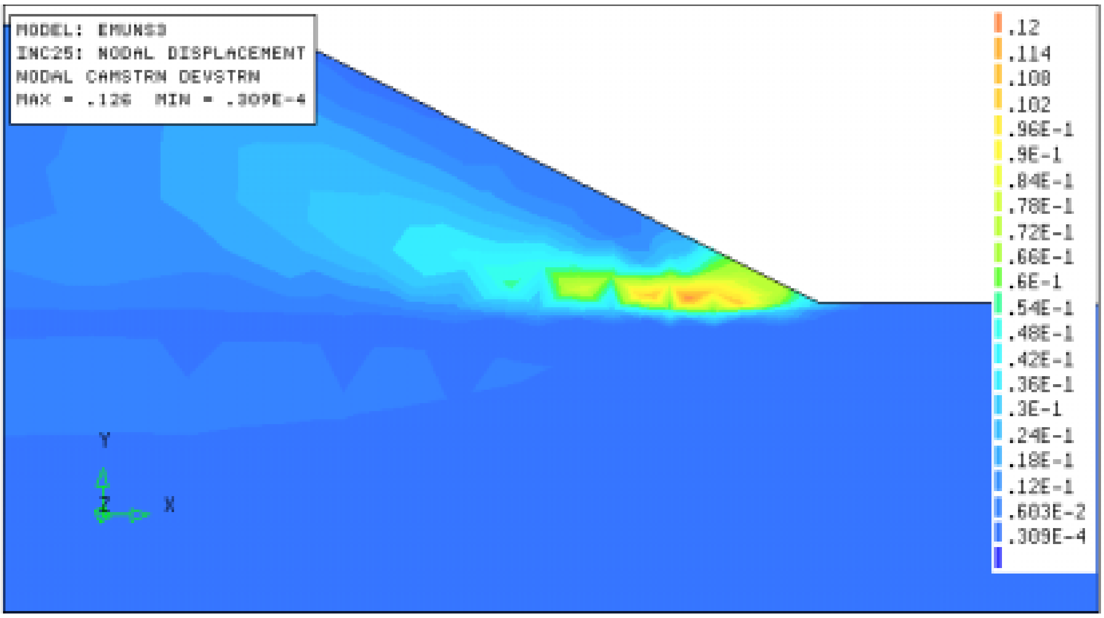


(a) Unreinforced slope

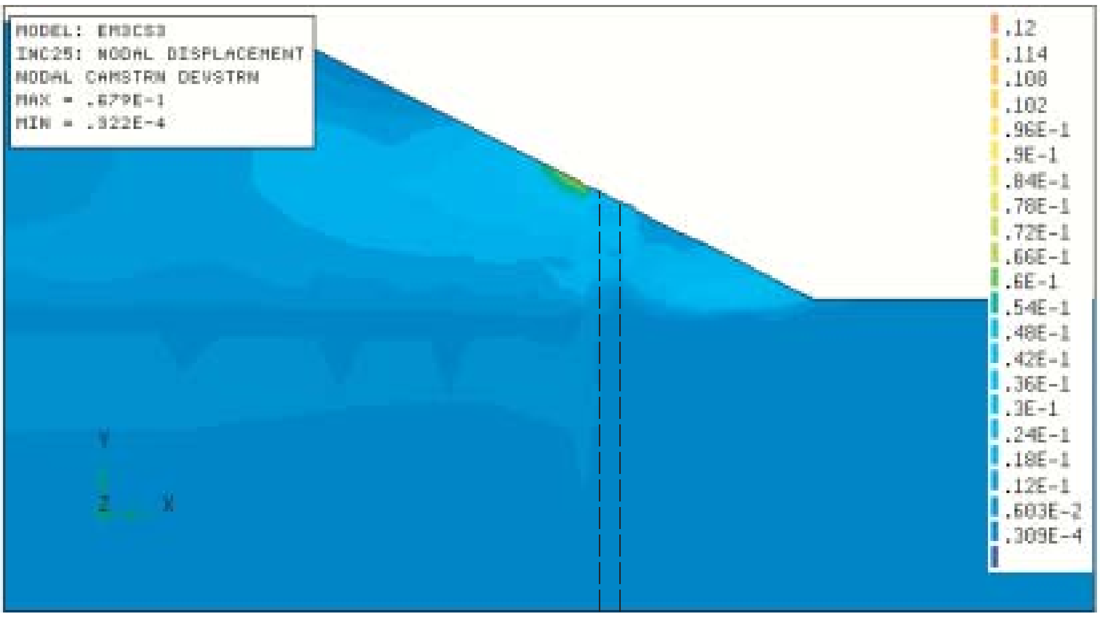


(b) Slope reinforced by piles at 3 diameter centres (cross section through pile line)

Figure 23 Lateral displacement after 60 years in service (embankment slope, softened soil)

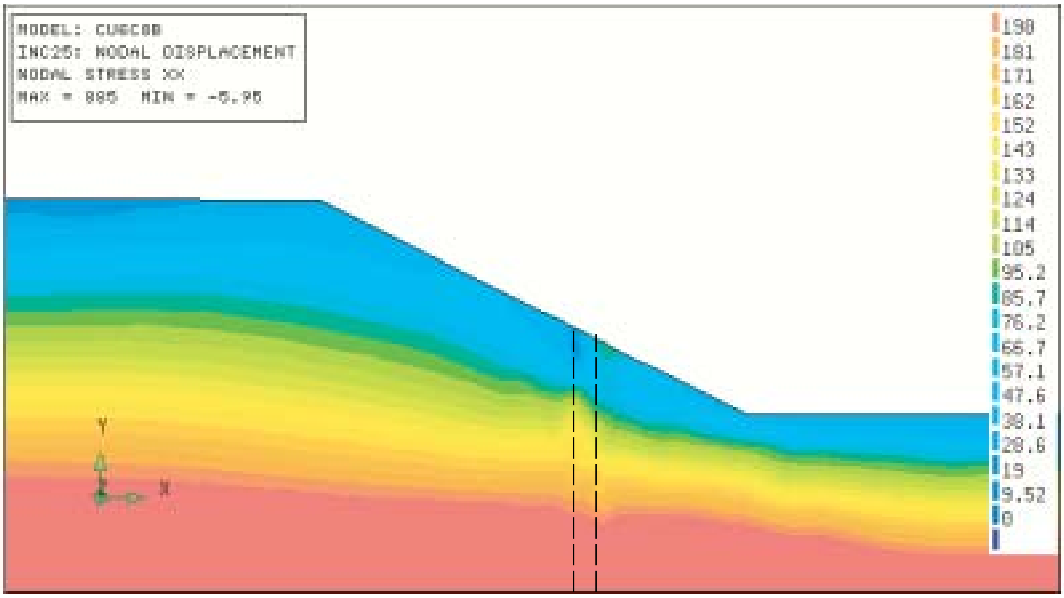


(a) Unreinforced slope

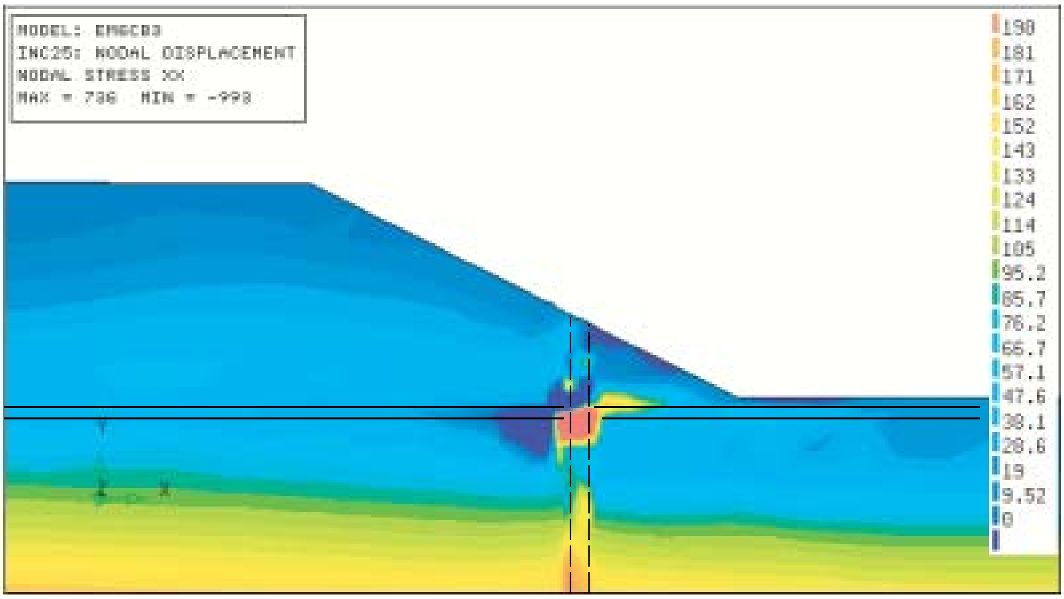


(b) Slope reinforced by piles at 3 diameter centres (cross section through pile line)

Figure 24 Deviator strain developed after 60 years in service (embankment slope, softened soil)



(a) Cutting slope, piles at 6 diameter centres (cross section through pile line)



(b) Embankment slope, piles at 6 diameter centres (cross section through pile line)

Figure 25 Lateral stress developed after 60 years in service

The development of passive pressures on the downslope side of the top of the pile will be limited by the yield pressure of the soil. If the pile was part of a continuous retaining wall and the ground was level, effective stress design would give a passive pressure (p_p') at any depth of

$$p_p' = K_p\sigma_v' + 2c'\sqrt{K_p} \qquad (1)$$

For a single pile the zone of yielding will be larger because of three dimensional effects: for this reason Broms (1964) proposed a limiting soil pressure in non-cohesive soil and for a single pile of $3K_p\sigma_v'd$ where d is its diameter. Fleming *et al.* (1994) suggested from the data of Barton (1982) that the limiting pressure was better given by $K_p\sigma_v'd$ at depths of up to 1.5 diameters, although $K_p^2\sigma_v'd$ was considered a better approximation at depths beyond this. As these formulae were derived for a pile installed in and pushing into level ground, it is not clear whether in sloping ground it is better to evaluate K_p from (1+sin φ')/(1-sin φ') or to attempt to modify K_p by taking account of the slope angle using tables from Caquot and Kerisel (1948). The latter calculation may not always be possible anyway because of instabilities which arise when the slope angle is greater than φ'.

In this report, φ' for the clay has generally been taken as 20° and the corresponding value of K_p is 2.04 if level ground is assumed and frictional effects between the pile and the soil are ignored. Based on the limit pressure methods of Broms (1964) and Fleming *et al.* (1994), and arbitrarily assuming no water pressure over the upper 4m of the pile, the pile bending moments shown in Figure 26 are determined. These values are also compared with the maximum working bending moments calculated from the finite element analysis of cutting slope performance and reproduced from Figure 10. Absolute values of working bending moments were marginally larger than the limit pressures at depths of up to 2m, this was probably because the effect of soil suctions on σ_v' near the slope surface had not been included in the determination of the limit pressures. At greater depths, bending moments (absolute values) from limit pressures exceeded the working moments and this may indicate either that the pre-failure stresses in the finite element analysis were well below the soil yield stresses or that the K_p used in the limit pressure calculations was excessively high because no allowance was made for the sloping ground.

The contours of lateral stress shown in Figure 25b demonstrate a very different mechanism for the case of a piled embankment slope. The lateral stresses downslope of the top of the pile are very small whereas a build-up in stress occurs upslope of the pile. This mechanism of passive pressure being created by the ground pushing onto the piles means that methods such as those of Ito and Matsui (1975) and Wang and Yen (1974), which consider plastic deformation and soil arching between piles respectively, may be more appropriate in evaluating pressure on the piles rather than the limit pressure methods described above. Figure 27 compares the pile bending moments determined from finite elements with those from other methods. For the purpose of this comparison, a spacing of 6 diameters was

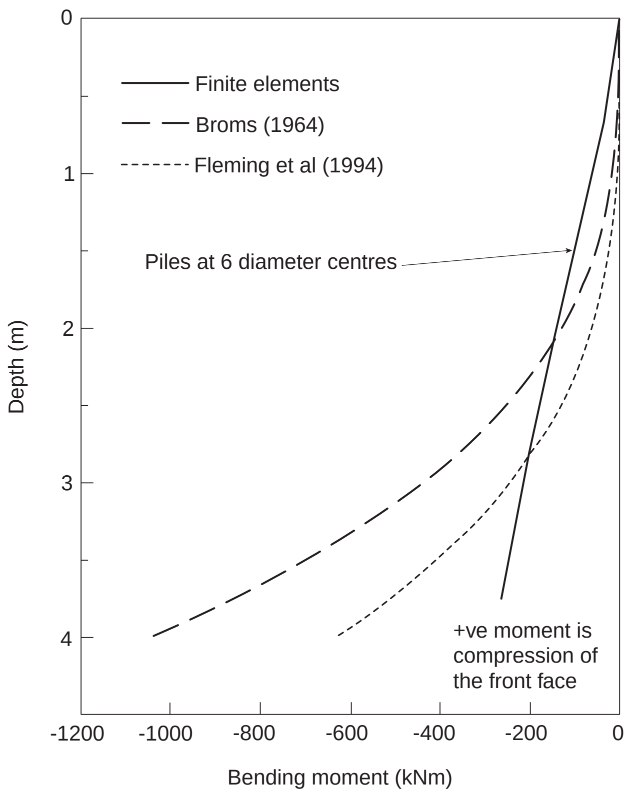


Figure 26 Comparison of pile bending moments from finite elements and limit pressure calculations (cutting slope)

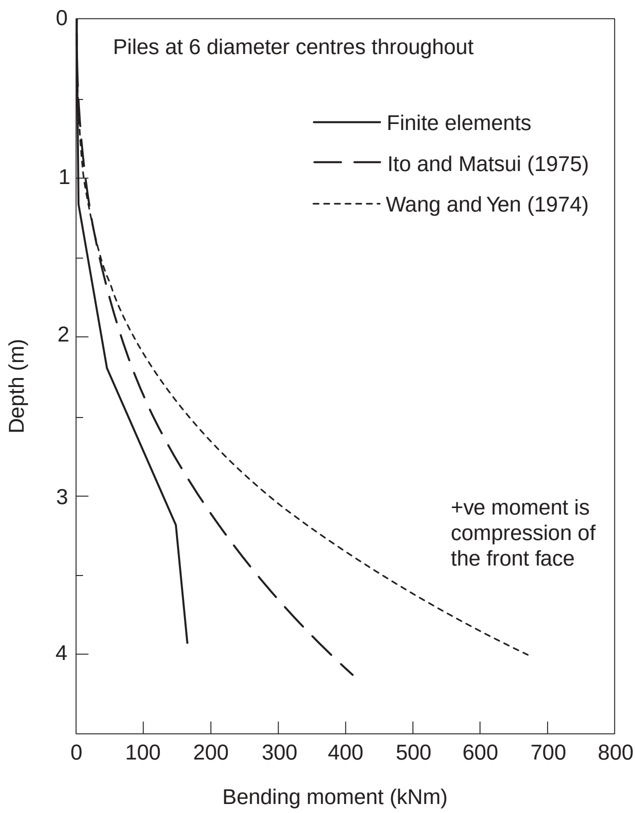


Figure 27 Comparison of pile bending moments from finite elements and other methods (embankment slope)

assumed between pile centres throughout. The working bending moments from finite elements were less than the moments calculated using the other methods which may be regarded as the upper limits determined from the maximum pressures acting on the piles.

In the above discussion, emphasis has been placed on designing the piles to accommodate the build-up in lateral stress on the downslope side in the cutting case and the converse in the embankment situation. This accords with the finite element prediction of pre-failure behaviour, other factors which have not been considered in this report may however have a significant impact on behaviour. For example, the vegetation on the slope may create soil suctions near the slope surface and have a stabilising effect. Alternatively, soil weathering and desiccation of the slope surface coupled with the ingress of water because of a badly designed or failed drainage system are expected to adversely affect slope performance. These and any other site specific factors need to be considered when designing the piling system.

10 Summary and conclusions

The stability of clay slopes can be improved by using a single row of spaced piles to provide support. A series of three dimensional finite element analyses were undertaken to compare the performances of untreated cutting and embankment slopes with those stabilised by piles. The finite element evaluation focused on the calculation of ground deformations under working loads (ie. pre-failure), although the results also provided a useful indicator of the potential rupture surface in clay slopes in the longer term. Particular design aspects which received attention included the effect of increasing the slope angle, different pile spacings, softening of the clay, and the influence of different pore water pressure regimes upon the results. The following conclusions were reached:

10.1 Cutting slopes

i With over-consolidated clay slopes the *in situ* lateral stresses are high and negative excess pore water pressures generated during excavation may take many decades to dissipate leading to the risk of a delayed collapse. This behaviour generated a significant rotational heave towards the cutting in the longer term which dominated the finite element results. For this reason it was considered that the relative magnitudes of the movements of the unreinforced and reinforced slopes were more important than the absolute values.

ii The results of the finite element analysis of the unreinforced slope indicated that the potential rupture surface is likely to emerge further behind the crest of the slope than would be predicted by conventional slope stability analysis which takes no account of the *in situ* stress regime and the stress history due to excavation of the cutting.

iii The analysis indicated that installation of spaced piles at the time of construction does not significantly constrain the magnitude of ground heave, but will act to limit the lateral displacement of the ground. When piles were installed at 6 diameter centres, there was evidence after 60 years in service of differential displacement between the piles and the ground between them indicating that soil flow between piles was starting to develop. When the piles were more closely spaced at 3 diameter centres there was little differential displacement. At both spacings, the piles were effective in limiting ground displacements to less than those of the unreinforced slope.

iv The magnitude of the long-term rotational heave (and hence the lateral movement of the toe of the piles) developed because of the unloading due to excavation of the cutting is sensitive to the stiffness of the ground below cutting level. The reduction in the magnitude of ground movements, that results from founding the piles in a stiffer stratum, is significant with more of the perturbing load being borne by the piles and hence a much larger bending moment develops in the pile near to the interface between the clay and the stiffer stratum. Site investigations for the pile installation therefore need to identify a founding stratum and care is then needed in the design to ensure that failure does not occur with the development of a plastic hinge in the pile.

v The finite element modelling confirmed that either steepening of the slope angle or further softening of the clay induced more lateral displacement of the reinforced slope and also larger pile bending moments. Of these two effects, softening of the clay parameters to near residual values increased displacements (in the ground midway between piles) by about 25%. The lateral displacement of the ground at an equivalent location in the unreinforced slope increased by slightly more (30%).

vi When piles are installed as a remedial or preventative measure after construction, a considerable part of the heave related movement due to the unloading caused by excavation of the cutting has occurred. However, with an impermeable clay, some heave may continue for many decades until the dissipation of negative excess pore pressures is complete. For this reason, benefits for cutting slopes of marginal stability will still accrue if reinforcing piles are installed after construction and well into the 60 year service life of the slope.

vii In the finite element analysis of cutting slope behaviour, a build-up in soil lateral stress was observed as the top of the pile pushed downslope although its magnitude will ultimately be limited by the yield pressure of the soil. Based on the limit pressure methods of Broms (1964) and Fleming *et al.* (1994), the pile bending moments were determined and compared with the maximum working bending moments calculated from the finite element analysis. Absolute values of working moments were marginally larger than those calculated from the limit pressures at shallow depths (<2m); this was probably because the effect of soil suctions on σ_v' near the slope surface had not been included in the determination of the limit pressures. At greater depths, bending moments (absolute values) from limit pressures exceeded the working moments and this may indicate either that the pre-failure stresses in the finite element analysis were well below the soil yield stresses or that the K_p used in the limit pressure calculations was excessively high because no allowance was made for the sloping ground.

10.2 Embankment slopes

viii Whereas with the cutting situation the development of movement was dominated by a rotational heave due to the unloading caused by excavation, this does not occur in the embankment model. Settlement occurs mainly in the embankment fill with some in the foundation clay due to the applied surcharge loading. Only very small lateral movements are predicted beneath the granular drainage blanket with indications that potential rupture is confined to the slope above the drainage blanket: this correlates with the findings from conventional slope stability analysis. The zone of rupture in the embankment case is generally much smaller than that predicted for a cutting slope.

ix When piles are installed at the time of embankment construction, numerical analysis indicates that lateral movements of the ground midway between piles spaced at 3 diameter centres are only slightly more than those of the piles, but still considerably less than those of the ground at an equivalent location in an unreinforced slope. When piles are installed at 6 diameter centres there is significantly more differential movement between the piles and the ground midway between piles. Although piles at this spacing still act to restrain ground movements when compared with those in an unreinforced slope, the increase in differential movements means that, with further softening of the ground, flow of the clay fill between piles may possibly lead to instability.

x The analysis showed that increasing the slope angle from 1v:2h to 1v:1.5h had a significant effect in approximately doubling the lateral ground movements in both unreinforced and reinforced embankment slopes. In the latter case the pile bending moments (compression of the front face) also increased by a similar factor.

xi When lower strength parameters approaching residual values were used for the clay fill, development of a rupture surface was much more pronounced in the unreinforced slope. With the reinforced slope, both movements and pile bending moments were increased although the restraining action of the piles and the enhanced stability of the piled slope were very much in evidence.

xii Significant differences in both unreinforced and reinforced slope behaviour are predicted if either the embankment is constructed directly on its soil foundation without using a drainage blanket or the case where the drainage blanket fails to operate satisfactorily. The higher pore water pressures and absence of the permeable filter material comprising the drainage blanket mean that the ground movements which occur are not only larger but more deep-seated. For this reason much larger pile bending moments are developed if the drainage layer is absent. If an effective drainage blanket is present, the depth of penetration of piles installed to stabilise the slope can be much reduced.

xiii Because the clay fill was assumed to be ten times more permeable than *in situ* clay, the excess pore pressures generated by the embankment loading dissipated over the first few decades and only small ground movements were predicted beyond this time. For the adopted clay permeability, the value of installing piles as a remedial measure at 20 years after construction of the embankment was therefore limited and, in general, installation of bored piles as a reinforcing measure is more effective the earlier it is carried out. However, for site specific reasons, embankment instability may develop at a later stage and subsequent installation of spaced bored piles is still expected to improve the factor of safety. Site specific reasons which may lead to delayed collapse are expected to be mainly related to progressive softening of the clay fill, causes of which might include defects or failure of the drainage systems, or possibly a change in the pattern of water flow due to new construction nearby.

xiv The contours of lateral stress determined for a piled embankment slope show a build-up in lateral stress upslope of the top of the pile and only small stresses downslope of the pile, the converse to the cutting situation. This mechanism of passive pressure being created by the ground pushing onto the piles means that methods such as those of Ito and Matsui (1975) and Wang and Yen (1974), which consider plastic deformation and soil arching between piles respectively, may be more appropriate in evaluating pressure on the piles rather than the limit pressure methods described in (vii). A comparison shows that the working bending moments from finite elements were less than the moments calculated using the other methods which may be regarded as the upper limits determined from the maximum pressures acting on the piles.

10.3 General (cutting and embankment slopes)

xv The optimum location for the row of piles needs to be such that the potential for slip development behind and in front of the piles is minimised. In this report, a location for the piles which was three eighths of the way up the slope was found to be satisfactory. Investigation of the contours of movement and strain after 60 years in service then indicated that the piles acted to restrain the ground upslope although, as might be expected, there was evidence that a smaller slip might still eventually occur in front of the piles. If a small slip of this nature occurs, it can usually be tolerated as it will not impinge on the carriageway and can be easily reinstated. If the piles were located nearer to the toe of the slope, it is anticipated that a more major failure would then occur above them.

11 Acknowledgements

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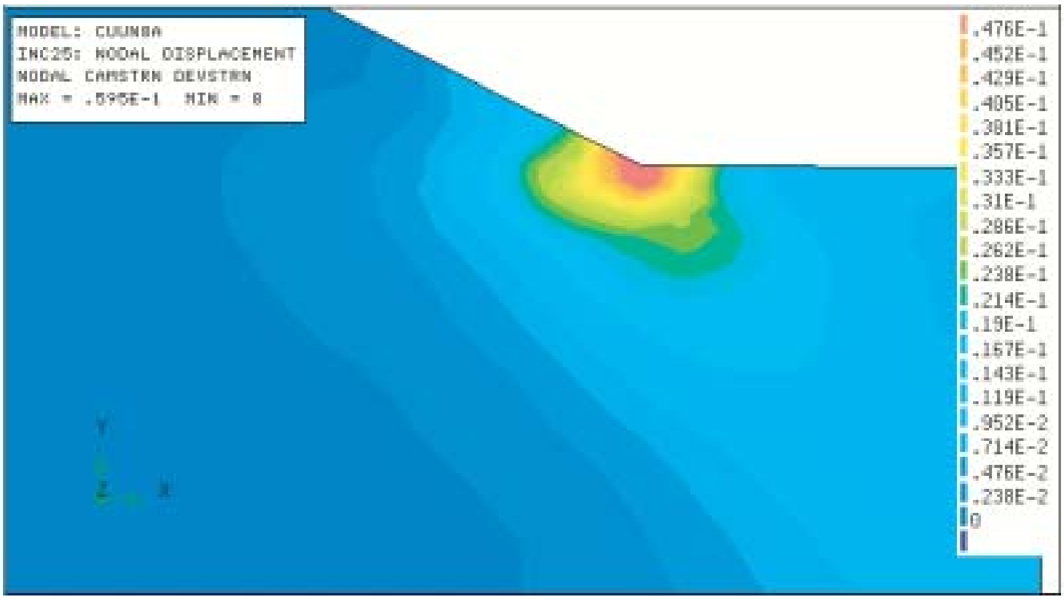
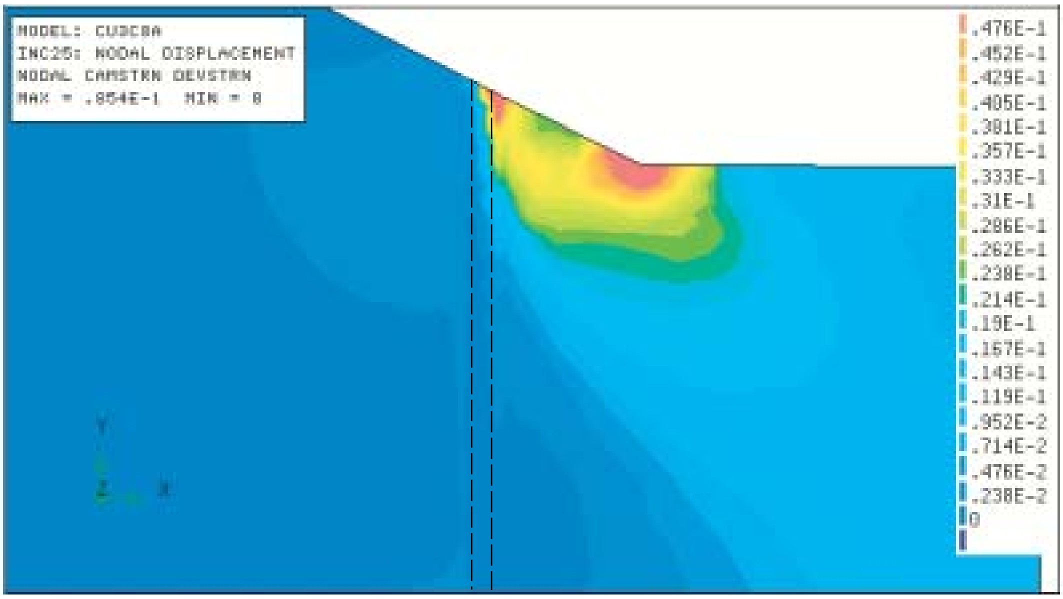
Appendix A: Analyses of performance of cutting slope – results in stiffer clay



(a) Unreinforced slope



(b) Slope reinforced by piles at 3 diameter centres (cross section through pile line)

Figure A1 Deviator strain after 60 years in service (cutting slope in stiff clay)

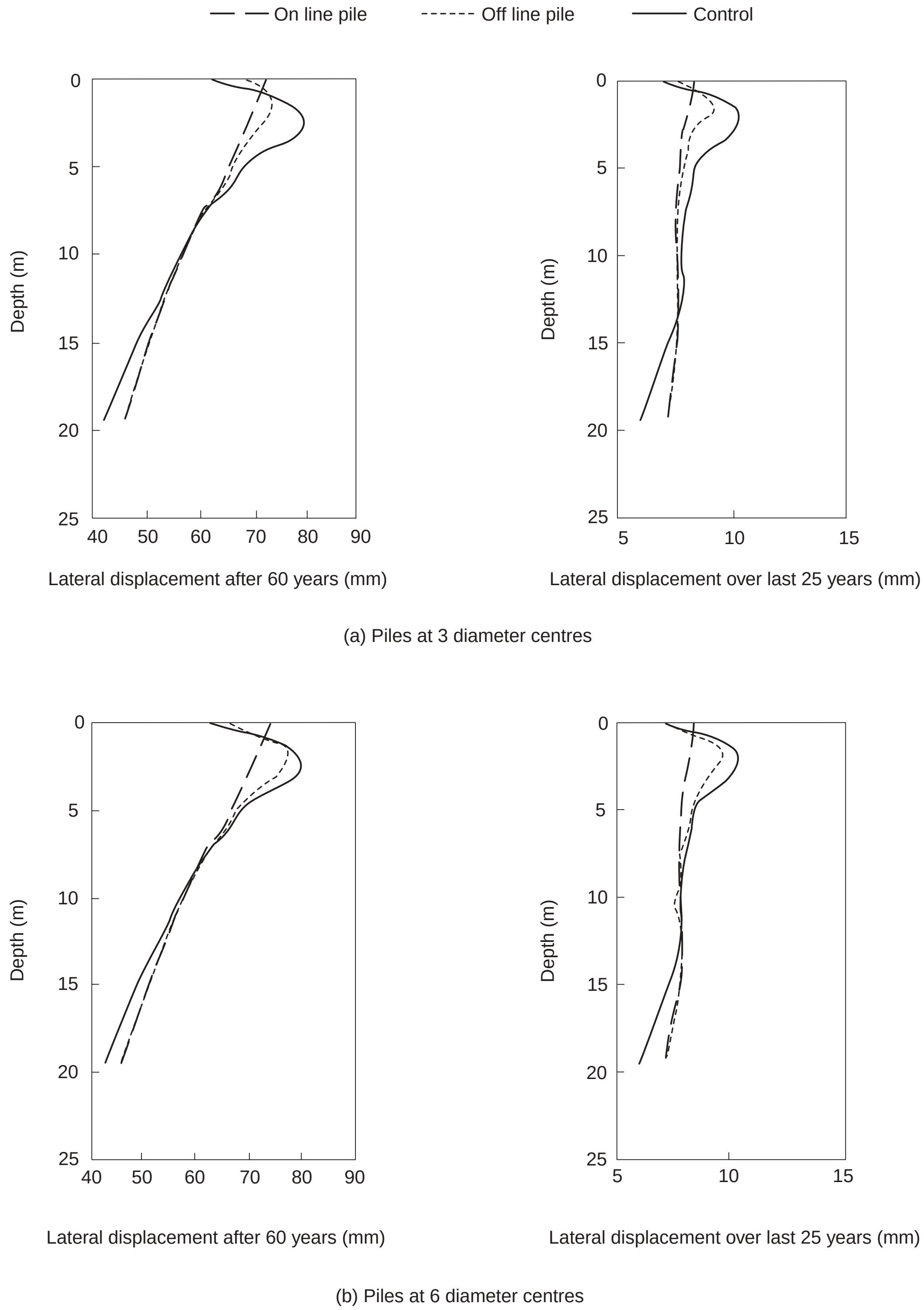


Figure A2 Displacement plots for cutting slope in stiff clay

Abstract

The use of a single row of spaced piles for stabilising slopes has been used widely and with success for some considerable time primarily elsewhere in Europe and in Japan. On the UK highway network the technique could provide a permanent and cost effective method of both remediating potentially unstable or failed slopes and of steepening slopes in a widening situation. However, the availability of design guidance on the use of this technique is limited: this is partly because the soil-structure interaction problem is complex and three dimensional in nature. This report discusses the results from three dimensional finite element analyses in which the performance of untreated cutting and embankment slopes was compared with those stabilised using a single row of piles. The study concentrates on the performance of clay slopes and discusses the implication of the findings upon their design.

Related publications

TRL471 *Centrifuge modelling of a cutting slope stabilised by discrete piles*. Hayward T, Lees A, Powrie W, Richards D J and Smethurst J. 2000 (price £35, code H)

TRL466 *A review of the use of spaced piles to stabilise embankment and cutting slopes* by D R Carder and J Temporal. 2000 (price £25, code E)

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