Improving the stability of slopes using a spaced piling technique

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CONTENTS

Executive summary vi
Abstract vi
1 Introduction 1
2 Literature review 2
3 Centrifuge modelling of a cutting slope 3
  3.1 Centrifuge modelling procedure 3
  3.2 Centrifuge test results 3
  3.3 Summary 4
4 Finite element modelling 5
  4.1 Method of analysis 5
  4.2 Summary of the findings 5
    4.2.1 Cutting slope 5
    4.2.2 Embankment slope 6
5 Case history study on the M25 7
  5.1 Site description 7
  5.2 Construction sequence 8
  5.3 Instrumentation 8
  5.4 Performance monitoring during construction and in the longer term 9
    5.4.1 Lateral movements of the piles and the ground between the piles 9
    5.4.2 Bending moments in the piles 11
    5.4.3 Lateral movement of the ground upslope of the pile line 12
    5.4.4 Pore water pressures 13
  5.5 Summary 14
6 Case history study on the A12 15
  6.1 Site description 15
  6.2 Construction sequence 16
  6.3 Instrumentation 16
6 Case history study on the A12 (cont’d)

6.4 Performance monitoring during construction

6.4.1 Lateral movements of the piles and the ground between the piles

6.4.2 Bending moments in the piles

6.4.3 Lateral movement of the ground upslope of the pile line

6.5 Performance during first two years in service

6.5.1 Lateral movements of the piles and the ground between the piles

6.5.2 Bending moments in the piles

6.5.3 Lateral movement of the ground upslope of the pile line

6.5.4 Pore water pressures

6.6 Summary

7 Design implications

7.1 Site investigation

7.2 Factors for consideration in the design

7.2.1 Spacing and diameter of the piles

7.2.2 Lateral resistance supplied by the piles

7.2.3 Penetration and location of the piles

7.2.4 Overall stability

7.2.5 Differences between cutting and embankment situations

7.2.6 Influence of construction sequence

7.3 Design procedures

7.3.1 Evaluating the restoring shear force to achieve the required factor for slope stability

7.3.2 Evaluating the lateral resistance provided by each pile

7.3.3 Other design procedures

8 Conclusions

Acknowledgements

References
Executive summary

A single row of bored piles spaced at intervals along a clay slope provides an effective remedial measure to improve the stability of a slope that has either failed or is showing incipient signs of failure. The technique can also be used to steepen slopes so that highway corridors can be widened within existing boundaries to accommodate extra vehicular lanes or other modes of transport. For these reasons, the Highways Agency commissioned a major programme of research with TRL to investigate the potential of the piling technique and provide design advice.

This programme of research was completed in a number of phases, which were separately reported. The phases of this research were:

• A literature review of instrumented case history studies of pile-reinforced slopes, analogies drawn with other construction forms where soil flow and arching occurs, and currently available design methods using this technique.
• Centrifuge modelling of a cutting slope to investigate the effectiveness of a single line of piles at different spacings in stabilising a clay cutting slope. The findings were compared with the performance of an unreinforced slope.
• Three-dimensional finite element analyses in which the performances of untreated cutting and embankment slopes were compared with those stabilised using a single row of piles. Particular aspects that received attention included the effect of increasing the slope angle, the use of different pile spacings, softening of the clay, and the influence of different pore water pressure regimes.
• Instrumented trials of remediations of a Gault Clay cutting slope on the M25 and a London Clay cutting slope on the A12. Pile movements and bending moments, together with movements and pore pressures developed in the slopes, were measured both during construction and during their early service life.
• Provision of advice on the form of the ground investigation, the factors for consideration in the design, and suitable design procedures.

This Insight Report provides a comprehensive overview of the main findings from each of the above phases. By bringing these findings together in one report, the engineer is presented with the complete history of the development of the research strategy and the associated design recommendations.

Determination of the optimum design for a piled slope is a complex soil-structure interaction problem. The main factors for consideration include the spacing and diameter of piles, their lateral resistance, the optimum penetration and location of the piles, and the overall stability of the piled slope. The depth of penetration and required lateral resistance of stabilising piles in a cutting are likely to be greater than in an embankment.

The programme of research was aimed at highway slopes. Therefore, its emphasis and recommendations are generally applicable to clay slopes that are typically up to about 10 m in height. For higher slopes, more than one row of piles or anchoring of the pile heads may be considered necessary for stability. It should be noted that this Insight Report concentrates on giving design guidance when only a single row of cantilevered piles is employed, although it is anticipated that some of the philosophies can be used more widely.
Abstract

Discrete piles are used to stabilise infrastructure slopes, especially where there is insufficient land to allow construction of large toe berms or regrading of the slope. In addition to providing a technique that can be used to steepen slopes with minimum landtake, spaced piling can be used for the permanent and cost-effective reinstatement and repair of unstable or failed clay slopes. This is particularly relevant to many motorway slopes, which are prone to shallow failures and are now reaching a critical age for deep-seated failure to occur. The situation is also exacerbated by the increasing incidence of heavy rainfall and flooding that has occurred in recent years due to the effects of climate change, which will add to the risk of softening of the clay. For these reasons, the Highways Agency initiated a comprehensive programme of research with TRL to investigate the potential of the piling technique. This Insight Report discusses the findings from a literature review, centrifuge and analytical studies, and two instrumented case history studies of the remediation of clay slopes on the highway network. The implications of the findings are discussed and appropriate design advice recommended.
1 Introduction

With an integrated transport policy, there may be a need to widen highway pavements within existing corridor boundaries to provide extra space for bus/cycle lanes and other modes of transport — or indeed just to provide a further lane for general traffic. In such situations, there is an increasing demand for embankment and cutting slopes to be steepened, so avoiding additional landtake and increased costs, and associated delays to construction. In many cases, the additional lateral loading that then develops near the toe of the embankment or cutting can be accommodated by installation of a single row of piles spaced at intervals along the slope (Figure 1.1). The same technique can be used for the permanent and cost-effective reinstatement and repair of unstable or failed clay slopes. This is particularly relevant to many motorway slopes, which are prone to shallow failures and are now reaching a critical age for deep-seated failure to occur. The situation is also exacerbated by the increasing incidence of heavy rainfall and flooding, which will add to the risk of softening of the clay that has occurred in recent years due to the effects of climate change. For these reasons, the Highways Agency (HA), which manages, maintains and improves the network of trunk roads and motorways in England, initiated a programme of research with TRL to investigate the potential of the technique and provide design advice.

![Figure 1.1 The use of a single row of bored piles to stabilise highway slopes](image)

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. The technique is, however, now becoming more commonly used in the UK, both on the highway and rail networks, for stabilisation of soil slopes. Determination of pile diameter, spacing, penetration depth and location on the slope is a complex soil-structure interaction problem on which further design guidance is needed.

The TRL research has therefore been at the forefront in providing much encouragement and advice to designers of this type of construction. This Insight Report provides a comprehensive overview of the main findings of the research for the HA, which is primarily based upon the following:

- A literature review of existing case histories and current design methods undertaken by Carder and Temporal (2000)
- Centrifuge modelling of a cutting slope reported by Hayward et al. (2000)
- Three-dimensional finite element modelling discussed by Carder and Easton (2001)
- Instrumented case history studies of slope remediation on the M25 and the A12 reported by Carder et al. (2003) and Carder and Barker (2005a, 2005b)
- Design guidance on the use of the piling technique (Carder, 2005)

Each of the above aspects is now considered.
The review (Carder and Temporal, 2000) drew analogies with other construction techniques where soil flow and arching occur between piles or piers. The principal findings from considering the performance of piled foundations embedded in clays are that a pile spacing of about five times the pile diameter is sufficiently large to minimise the interaction between adjoining piles, while no interaction is expected at a spacing of more than eight diameters. On this basis, it can then be argued that in a slope situation, flow of soil between piles is fairly certain at spacings of more than eight diameters and possible at spacings of more than five diameters. A review of design recommendations for spill-through abutments – where the embankment fill can spill between the piers that support the bridge deck – suggested a similar result, with an upper limit for spacing of eight pile diameters (or widths) for arching to be effective in granular soils. It must be noted that some authors, however, recommended a lower ratio of five diameters.

Current design approaches involve three main stages, which are:

- evaluating the restoring shear force needed to achieve the required factor of safety for stability of the slope,
- evaluating the shear force that each pile can provide to resist sliding, and
- selecting the diameter, spacings, penetration and most suitable location for the piles.

These stages are inter-related and, for site-specific reasons, pile dimensions may well be established first.

The spacing between and diameter of piles must be designed to maximise the arching of the soil between the piles while minimising the flow of soil between them. Various theories have been developed for design purposes, based on a combination of elastic and plastic soil behaviour. Carder and Temporal (2000) discuss methods based on rigid-plastic behaviour (Wang and Yen, 1974; Day, 1999), on plastic deformation (Ito and Matsui, 1975; Ito et al., 1981), and on numerical analyses (Rowe and Poulos, 1979; Chen and Poulos, 1997; Brown and Shie, 1990).

Although most of the above design approaches enable the lateral force acting on each pile to be determined, the lateral resistance supplied by the piles will be limited by the yield pressure of the soil both above and below the potential slip surface. Information on yield pressures is available for the case of laterally loaded single piles pushing through level ground and creating passive pressures. For non-cohesive soils, Broms (1964) used a limiting soil reaction per unit length of pile of $3K_d\sigma^v_d$, where $K_d$ is the passive earth pressure coefficient, $\sigma^v_d$ is the effective vertical stress and $d$ is the diameter of the pile. Fleming et al. (1994) suggested from the data of Barton (1982) that at depths of up to 1.5 diameters, the limiting force per unit length was better given by $K_d\sigma^v_d$. At depths beyond this, $K_d\sigma^v_d$ was considered a better approximation. Although these formulas were derived for a pile installed in level ground, account can then be tentatively taken of the slope angle by modifying the value of $K_d$ according to whether the resistance of the soil to the top of the pile deflecting downslope or the toe fixity of the pile is being determined. For piles in cohesive soil, Fleming et al. (1994) reported that the limiting pressure developed by level ground in front of the pile from the wedge of the soil is approximately twice the undrained shear strength ($c_u$). More complicated expressions from wedge theory have been derived by Ashour et al. (1998) and for piles in sloping ground by Gabr and Borden (1990).

The overall stability of the slope can be investigated by taking account of the extra restoring force provided by the piles during calculations that may be based on limit equilibrium methods such as the friction circle method and the method of slices (Bishop, 1955; Bishop and Morgenstern, 1960; Morgenstern and Price, 1965). This force can be calculated from the theory of ultimate resistance described by Poulos and Davis (1980).

Viggiani (1981) studied the different failure modes that can occur and derived expressions for the maximum shear force exerted by the pile on the slip surface and for the bending moments acting on the pile. These solutions were for the ultimate limit state of purely cohesive soils and their cohesion was assumed to be constant with depth in both the unstable and stable zones. Poulos (1973) developed a more rigorous approach with the restoring force from the piles being calculated from numerical analysis using boundary elements. Hassiotis et al. (1997), following the approach of Ito and Matsui (1975), integrated the formula for plastic deformation of the soil between piles to determine the ultimate load over the upper part of the piles. Their slope analyses based on determination of this force showed that the improvement in the factor of safety of the slope for a particular pile penetration needed to take account of the change in location of the critical failure surface from its original position.

Some of the more obvious conclusions from the review of current design methods are:

- the factor of safety of the slope increases as the spacing between piles decreases and more load is taken by the piles,
- more arching and less flow of the soil between piles occurs as the strength of the soil increases,
- the piles must extend well below the expected critical failure surface to ensure that the failure zone does not increase to encompass the pile toe, and
- a single row of piles is probably better placed one-third to one-half of the way up the slope to avoid significant slope failures behind or in front of the piles.

The construction technique of using bored piles to stabilise slopes is expected to be of value in satisfying the increasing demand for embankment and cutting slopes to be steepened, so reducing the landtake and associated costs and delays to construction. The technique is also seen as a permanent structural solution in preventing or repairing slopes prone to deep-seated failure and as improving the factor of safety against shallow failure. The factor of safety against shallow failure is likely to be further enhanced by the use of connecting beams or walings between the tops of adjoining piles; this technique has been used successfully outside the UK, but not extensively researched.
3 Centrifuge modelling of a cutting slope

The literature review identified that a key issue is the maximum spacing at which the piles can be installed and still support the soil in between them. There is further uncertainty concerning the lateral pressure for which each pile should be designed, and indeed the overall effectiveness of this approach for slope stabilisation in uniform ground (i.e., where there is no clearly identifiable weaker surface layer and a stronger substratum into which the piles can be socketed). An evaluation of behaviour at full scale can be achieved by placing a model in an elevated gravity field, produced by the rotation of a centrifuge. Thus the dimensions and many of the physical processes can be scaled correctly if an Nth scale model is accelerated by N times the acceleration due to gravity. Four 1:60 scale centrifuge model tests were carried out to investigate the suitability of a row of discrete piles for cutting slope stabilisation in uniform ground, and the impact of pile spacing.

3.1 Centrifuge modelling procedure

The models were made from blocks of speswhite kaolin clay of dimensions 200 mm by 550 mm on plan and 350 mm deep. Each clay sample was prepared by one-dimensional consolidation in a press to a maximum vertical effective stress of 800 kPa, followed by swelling back to a vertical effective stress of 80 kPa. On removal from the consolidation press, the clay was trimmed to the dimensions of the model. Also at this stage, a vertical trench was excavated in the clay at the left-hand edge of the model and filled with sand to enable the upslope ground water level to be controlled. For tests incorporating piles, piles were installed in the middle of the cutting slope at the required longitudinal spacing. A typical cross-sectional view of the model is shown in Figure 3.1, with dimensions indicated at prototype scale.

The model was instrumented with miniature pore water pressure transducers, and black plastic markers were installed in a grid pattern on the side face and on the upper surface of the clay block, to enable the measurement of displacements. One model pile was strain-gauged to measure bending moments at seven locations over its depth.

The geometry of the 1:60 scale model was based on a full-size 6 m deep cutting with a 1:2 side slope. The first test was carried out on an unpiled slope. Subsequent tests incorporated 1 m diameter piles installed halfway down the slope with an embedded depth of 9 m, at spacings between pile centres of three, four and six pile diameters.

The model was transferred to the centrifuge and the centrifugal acceleration increased to 60 g. Water was then supplied to the base and side drains, and the behaviour of the slope observed as the pore water pressures increased to an equilibrium state. The duration of each test was generally of the order of 20 hours, which is equivalent to 3000 days (i.e., about eight years) at prototype scale.

3.2 Centrifuge test results

Failure developed in the natural (unpiled) slope (Figure 3.2). The yield planes were identified from the movement of the black markers and are clearly visible.

Displacement vector plots were determined by image analysis of the black markers as viewed through the side face window of the model at various stages during the test. The photograph and vector plot in Figure 3.2 indicate a well-defined failure of the natural cutting slope and the photograph shows two clear failure surfaces, one propagating from the top of the slope and the other starting lower down.

A photograph and displacement vector plot taken at the end of the test with piles installed at a spacing of three pile diameters are shown in Figure 3.3. These suggest that by using stabilising piles at this spacing, the ground movements at the top of the...
cutting are reduced. In this case, the grid of black markers
indicates only small ground movements and no evidence of a
major failure plane developing, although shallow local failure
surfaces are apparent just downslope of the line of the piles.

A key finding was that failure occurred in both the
natural slope and a slope with piles installed at about six
pile diameters, while the slopes with piles installed at three
and four diameters did not. This is demonstrated by an
examination of the plan view displacement vectors for tests at
spacings of three and six pile diameters (Figure 3.4).

The plan view vectors for the test with piles at four-
diameter spacing were near identical to those at three-
diameter spacing, and in both cases illustrate that ground
movements towards the top of the cutting are reduced more
than those downslope of the row of piles. In Figure 3.4b, the
ground movements upslope of the piles are not reduced in
the same way for the increased spacing of six diameters, and
also show signs of some flow between piles.

The maximum pile bending moments that were measured
increased with pile spacing as would be expected. The
magnitudes of the pile moments and the lateral stresses
acting on them were compared with those derived from
existing design methods and a finite element back analysis
(Hayward et al., 2000).

### 3.3 Summary

The results of the centrifuge model tests confirm that the
installation of a single row of discrete piles can help to stabilise
a cutting slope of marginal stability. Both the unreinforced
slope and the slope with piles installed at a spacing of six pile
diameters failed, while the slopes with piles installed at three
and four diameters did not.
4 Finite element modelling

Following completion of the literature review and centrifuge modelling, the next phase of the study comprised numerical modelling in three dimensions to predict likely performance. In the analyses, the performances of untreated clay slopes were compared with those stabilised using a single row of spaced piles. The finite element evaluation focused on the calculation of ground deformations under working loads (ie pre-failure), although the results provided a useful indicator of the potential rupture surface in clay slopes in the longer term. Particular aspects that received attention included the effect of increasing the slope angle, using different pile spacings, softening of the clay, and the influence of different pore water pressure regimes upon the results.

4.1 Method of analysis

The performances of cutting and embankment slopes constructed in clay soils 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). Meshes were comprised entirely of 1440 linear strain brick elements, each of which was 20 noded; consolidating elements were employed for the soil strata. As far as was possible, the selected geometry modelled that of a typical highway slope.

The initial geometry for both the cutting and embankment allowed for a 1:2 (vertical:horizontal) slope (ie a slope angle of about 27 °) and a slope height of 8 m. However, some analyses were subsequently carried out for a steeper slope of 1v:1.5h. The mesh was constructed to allow consideration of an unpiled slope or slopes reinforced with a single line of 0.8 m diameter piles spaced at centres of three or six pile diameters. The depth of the piles and the properties of the soil in which they were founded could be varied.

The pore water pressure regimes were very different for the cutting and embankment analyses. For the cutting, a hydrostatic distribution of pore water pressure with depth from the surface was assumed at the boundary, remote from the crest of the slope, and the water table for the long-term analysis was considered to be at ground level at the bottom of the slope. For the embankment, a hydrostatic distribution of pore pressure was assumed in the in situ ground prior to embankment construction. Following construction, a perched water table at a depth of 1 m was assumed beneath the surface at the centre of the embankment. The effect of a drainage blanket below the clay fill of the embankment was also modelled. Further information on the pore pressure fixities is given by Carder and Easton (2001).

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

- An untreated clay slope
- A clay slope reinforced by a row of spaced piles at the time of construction
- A 20-year-old 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 embedment of the pile toe into a stiffer stratum were also investigated in specific instances.

4.2 Summary of the findings

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; a location for the piles that was three-eighths of the way up the slope was investigated. The contours of movement and strain after 60 years in service 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. A summary of the more detailed findings from the numerical analyses reported by Carder and Easton (2001) is as follows.

4.2.1 Cutting slope

With cutting slopes constructed in over-consolidated clay, 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 (Potts et al., 1997). This behaviour generated a significant rotational heave towards the cutting in the longer term, which dominated the finite element results.

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 six-metre 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 three-metre centres, there was little differential displacement. At both spacings, the piles were effective in limiting ground displacements to less than those of the unpiled slope.

When piles are installed as a remedial or preventative measure some while after construction, a considerable part of the heave-related movement due to the unloading caused by excavation of the cutting has occurred. However, with 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.
4.2.2 Embankment slope

Whereas with the cutting 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 a 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 (Figure 4.1a) in the embankment case is generally much smaller than that predicted for a cutting slope.

A comparison of the contours of lateral movement for an unpiled and piled embankment is given in Figure 4.1, produced using the same colour zoning. In Figure 4.1b, the lateral ground movement is shown on a cross-section through the line of one of the piles, which are spaced at three-diameter centres. 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 unpiled slope. When piles are installed at six-diameter centres, there is significantly more differential movement between the piles and the ground midway between piles than when the piles are installed at three-diameter centres.

Significant differences in the behaviour of all embankment slopes are predicted if either the embankment is constructed directly on its soil foundation without using a drainage blanket or 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 that occur are not only larger, but also 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 the piles installed to stabilise the slope can be much reduced.

![Figure 4.1](image-url) Contours of lateral movement from three-dimensional finite element analysis
5 Case history study on the M25

A unique opportunity to complement these earlier desk, model and analytical studies with an instrumented case history study arose following a deep-seated failure of a Gault Clay cutting slope near Junction 6 of the M25 orbital motorway around London. A 200 m long failure of the clay slope occurred in December 2000 during one of the wettest winters recorded in the UK. The deep-seated failure extended from a steep backscarp about 80 m up the slope to the motorway hard shoulder, which suffered a heave of 150 mm. The technique selected for remediation of the slope was that of a single row of reinforced concrete bored piles accompanied by extensive drainage works.

5.1 Site description

The site is located adjacent to Flower Lane Bridge in the north face of the cutting slope at Flint Hall Farm. Figure 5.1 shows a plan of the site and the extent of the slope failure that occurred in December 2000. Also shown is the location of the single row of piles used to stabilise the slope against future slips and the approximate line of the TRL instrumentation used to monitor its effectiveness.

When the failure was first identified, a site investigation and movement monitoring programme was initiated by the then HA Maintaining Agent (Mott MacDonald). It was found that the principal movement was of a large soil wedge formed by the intersection of a steep backscarp and a near-horizontal main shear surface, which was up to 10 m below ground level.

Generally, the slope comprised stiff fissured Gault Clay with some weathering of the upper zones. In some areas, the clay was overlain by a few metres of poorly sorted head deposits. Gault Clay cutting slopes are known to be a geology associated with a high percentage of shallow failures. On the motorway network, they tend to have a predominant slope angle of about 20 ° to the horizontal. In this case, the relatively deep-seated failure occurred even though the slope angle was only about 14 °.

The weathered Gault comprised firm to stiff grey-brown fissured clays and silts of high to extremely high plasticity. Typical plastic and liquid limits were 32% and 75%, respectively, with natural moisture contents of 35%. Undrained strengths on three samples varied between 37 kPa and 61 kPa. Results from one set of triaxial tests gave peak effective stress shear strength parameters, \( c' \) and \( \phi' \), of 10 kPa and 29 °, respectively. Residual values of \( c_r' \) and \( \phi_r' \) were 1.5 kPa and 13.5 °.

The unweathered Gault comprised stiff dark grey fissured clays and silts. Plastic and liquid limits were typically 36% and 82%, respectively, with natural moisture contents of 29%. Undrained shear strengths were variable but generally increased with depth and were in the range of 35–195 kPa at depths of up to 15 m. Based on these results and those of four previous investigations in the area, Davies et al. (2003) adopted effective stress design values for the Gault Clay of 1 kPa and 24 ° for \( c'_e \) and \( \phi'_e \). Residual values of \( c'_r \) and \( \phi'_r \) of zero and 14 ° were employed in sheared zones.

In the research area, the remedial works involved installing a single line of 1050 mm diameter and 16 m long bored piles at 2500 mm centres.

Figure 5.1 Location of the slope failure
5.2 Construction sequence
The initial construction activities involved the earthworks necessary to construct a working platform and haul road for the piling rig, cranes and concreting lorries. Figure 5.2 shows the rig that was used to install a single line of 1050 mm diameter and 16 m long piles at 2500 mm centres. The pile line was approximately one-third of the way up the failure (ie 25 m upslope from the M25 kerb line) so as to reduce the possibility of any subsequent slope failure above or below the line of piles.

In parallel with the piling operations, construction of a 4 m deep cut-off drainage trench about 40 m upslope of the backscarp took place. Following installation of the single row of spaced piles and trimming of the pile tops, a counterfort drainage system was installed downslope of the piles. For this purpose, 600 mm wide trenches were excavated to a nominal 3 m depth at 6 m centres in front of the piles and the trenches were filled with granular filter material.

5.3 Instrumentation
Instrumentation was installed to monitor lateral movement of the ground and piles, and the bending moments in the bored piles. Section and plan views of the instrumentation layout are shown in Figures 5.3 and 5.4, respectively.

Proprietary sleeve-jointed plastic inclinometer tubes (I1 and I2) were attached to the reinforcing cages for piles 21 and 23 (Figure 5.4) prior to their installation. Each tube was attached to one of the main longitudinal steel bars using specially designed steel fittings that retained it in position but allowed the cage to bend when lifted without the inclinometer tube being damaged.

Inclinometer tubes I3 and I4 were installed in the ground to measure subsurface lateral movements (a) at locations between piles 22 and 23, and (b) at a location about 20 m upslope from the row of piles. The tube I3 was positioned to detect any differences between pile movements and that of the clay between them; any differential movements would indicate flow of the clay between the piles. These ground inclinometer tubes were grouted into 150 mm diameter boreholes sunk to depths of 19.5 m (tube I3) and 16.5 m (tube I4) using a cable tool percussion drilling rig. A bentonite-cement grout mix, designed to have a similar stiffness to that of the surrounding Gault Clay, was used to backfill the boreholes.

Inclinometer surveys at various stages during construction and in the longer term enabled lateral movements of the ground and piles to be determined assuming base fixity of each inclinometer tube.

Two profiles each of 24 vibrating wire embedment gauges were installed on the reinforcing cages of piles 22 and 24. The gauges, which were installed in pairs, were attached to vertical reinforcing bars such that one gauge of each pair was on the downslope face of the pile and the other gauge was on the upslope face. This arrangement enabled both any differences between pile movements and that of the clay between them; any differential movements would indicate flow of the clay between the piles. These ground inclinometer tubes were grouted into 150 mm diameter boreholes sunk to depths of 19.5 m (tube I3) and 16.5 m (tube I4) using a cable tool percussion drilling rig. A bentonite-cement grout mix, designed to have a similar stiffness to that of the surrounding Gault Clay, was used to backfill the boreholes.

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5.4 Performance monitoring during construction and in the longer term

As part of the research for the HA, monitoring was initially carried out from September 2001 to February 2004, which covered both the construction period and the first 14 months in service following the remedial works. The monitoring data were reported by Carder and Barker (2005a).

No further measurements were taken in the instrumented area until 2007, when Mouchel Parkman (HA Managing Agent, M25 Sphere) commissioned TRL to revisit the site to ascertain if the instruments were still functional, which proved to be the case, and to recommence site measurements from July 2007 until March 2008.

5.4.1 Lateral movements of the piles and the ground between the piles

Shortly after installation of the instrumented piles, particularly heavy rainfall occurred, with 41 mm of rain falling on 7 October 2001 and a further 19 mm on the following day. At this stage, the upslope cut-off drain was operational whereas the counterfort drainage system in front of the pile line had not been installed in the instrumented area. The changes in movement recorded on pile inclinometer tubes (I1 and I2) as a result of this rainfall are shown in Figure 5.5. By 15 November 2001, the piles had cantilevered towards the motorway, with lateral movements of 15 mm and 13.5 mm being recorded at the tops of the respective inclinometer tubes and some movement being measured to a depth of about 12 m, corresponding to a level of 140 m above ordnance datum (AOD).

Following these changes towards the end of construction, no significant changes in pile behaviour occurred for about a year, even during a period of intense and persistent rainfall recorded during November 2002. However, after continued heavy rain during December 2002, readings taken on 8 January 2003 indicated that more load was coming onto the piles. A further cantilevering of the piles (by about 4 mm at their top) occurred as they were called on to provide more support to the slope.

Figure 5.5 also shows the development of the lateral movement of the piles in the longer term. The measurements in July 2007 indicated that, since February 2004, the tops of piles 21 and 23 had cantilevered by a further 4.8 mm and 3.5 mm, respectively. During the remainder of 2007, very little further change in pile movements occurred. However, following some periods of exceptionally heavy rainfall in late December 2007 and January 2008, the measurements again indicated that up to 2 mm further movement of the tops of the piles had occurred.

Figure 5.6 shows the results obtained from the inclinometer tube I3 installed midway between piles 22 and 23 to monitor any flow of clay between the piles. It is important to note that during the construction work this tube became operational at a later date than the inclinometer tubes installed in the piles, so comparison of absolute movements is not possible. However, similar trends in movement to those of the piles were observed. An increase in movement at the top of the tube of 5.3 mm was recorded from February 2004 to July 2007. After a fairly static period, further small movements were again recorded in January 2008.
The results in Figure 5.6 also indicate that the assumption of base fixity when determining pile movements was justifiable as tube I3 (between the piles) extends to about 3 m below the pile toes and shows no evidence of any translation at this depth.

A comparison of the trends in movement of pile inclinometers (I1 and I2) and the ground between them (I3) is shown in Figure 5.7. The similarity in the trends indicates that there was no evidence of any significant flow of clay between the piles during the period of monitoring.

The seasonal cyclic effects also show up clearly in Figure 5.7, with increases in lateral movement of the piles (and the ground between) tending to occur around December/January every year. This is considered to be in response to heavy rainfall events occurring when the pore water pressures in the slope are already high. It is important to note that the trend in movements is progressive at this particular slope, with increases in surface lateral movement of about 2 mm/year on the pile line.

**Figure 5.5** Development of pile movements and bending moments

**Figure 5.6** Ground movement between piles 22 and 23 (inclinometer I3)
5.4.2 Bending moments in the piles

The bending moment distributions with depth measured for piles 22 and 24 are shown in Figure 5.8, and the results from the two piles were very similar in both shape and magnitude.

The heavy rainfall, which occurred in early October 2001 shortly after the installation of the instrumented piles, produced the most significant change in bending moment. By 16 October, pile 22 was showing a peak bending moment of 518 kNm at about 142 mAOD (10 m depth), while pile 24 gave a peak moment of 358 kNm at 1 m above this. This behaviour correlated well with the measured cantilevering movement of the piles discussed in Section 5.4.1. The peak moment was measured in the unweathered Gault Clay slightly below the original shear surface at about 146 mAOD. This response to the first significant rainfall after pile installation was not unexpected, as some slope movement was necessary for the piles to deflect and generate sufficient resistance.

By the time measurements were first suspended in February 2004, peak moments of 757 kNm and 656 kNm were recorded in piles 22 and 24, respectively. On the resumption of measurements in July 2007, these peak values had increased to 897 kNm and 764 kNm, respectively. Little further change then occurred until January 2008, when peak moments increased slightly to 937 kNm and 792 kNm, respectively.

Also worth noting in Figure 5.8 are the small negative moments recorded towards the top of the piles at about

Figure 5.7 Time dependence of surface lateral movements on the pile line

Figure 5.8 Development of pile bending moments
149 mAOD. This type of behaviour was predicted from the finite element analyses of cutting slopes carried out by Carder and Easton (2001) and is explained by a localised build-up in soil stress as the top of the pile attempts to move into the ground in front of it.

In Figure 5.8, it is important to note that for both piles the sign of the bending moment reverses at about 138 mAOD, and that this level has remained virtually unchanged since the end of construction. This indicates that the level of the firm stratum in which the piles were founded is slightly above this. It can be argued that any reduction in this level is important as an indicator that toe embedment may no longer be adequate to sustain the load.

As bending moment balance needs to be maintained for each pile, it is anticipated that the measured small increases in positive bending moment must be compensated by the negative bending moment increasing below 138 mAOD. For practical reasons, it was not feasible to install strain gauges near the toe of the pile during construction. Zero bending moment must of course occur at the very toe of the pile.

5.4.2.1 Comparison with original design
Davies et al. (2003) reported that the pile design at this site was based on the method proposed by Viggiani (1981). Slope stability analyses, using the surveyed extent of the slip and the soil parameters derived during the site investigation, established that a restoring force of 300 kN/m was required to stabilise the 8 m deep slip and provide a 20% increase in the factor of safety. This restoring force was provided by using 16 m long piles.

The most probable distribution of bending moment predicted by the designer is shown in Figure 5.9 and is compared with the measured profiles for piles 22 and 24 in February 2004 and March 2008. Small increases in bending moment were measured over the four-year period and it is anticipated that further increases in pile moments will occur in the longer term.

Although the design values shown in Figure 5.9 were about five times higher than those measured, the trends in shape were very similar. This discrepancy can be partly explained by the sensitivity of the slope analysis to the residual angle of shearing resistance (φr) and the need to account for long-term behaviour in the design (Davies et al., 2003). The extensive drainage measures undertaken at this site also increased the stability of the slope with the result that pile loads and moments would be expected to be below the design values predicted from stability analyses.

5.4.3 Lateral movement of the ground upslope of the pile line
The lateral movements of the ground measured using an inclinometer tube about 20 m upslope from the line of the piles are shown in Figure 5.10. Lateral movements of about 6 mm were measured to a depth of about 9 m (148 mAOD) during late October 2001, indicating a block movement of the Gault Clay and overlying head deposits to this depth. The shearing at about 9 m depth appeared to occur on the original shear surface identified during the site investigation. Whereas little or no movements of the piles were observed between November 2001 and November 2002, further movement of the ground at the location of this tube continued to occur throughout this period.

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\[1\] It must be noted that bending moment increases would not necessarily develop for the case when the failure plane passes below the pile toe.
Subsurface lateral movement of the ground has continued 20 m upslope of the pile line, with peaks of movement occurring at the slope surface and at two other depths. By February 2004, 33 mm of movement was measured at the slope surface where there are surficial head deposits, with peaks of 21 mm and 14 mm occurring at the levels of 152 mAOD (within the zone of weathered Gault Clay) and 148 mAOD (in the unweathered Gault Clay), respectively. The shearing at about 9 m depth (148 mAOD) appeared to occur on the original shear surface identified during the site investigation for the scheme (Davies et al., 2003).

The time plots given in Figure 5.11 illustrate the trends in movement at these three selected depths. The trends are very similar to those presented in Figure 5.7 for the piles, with changes being strongly related to the cumulative effect of any persistent and heavy rainfall in the winter period when the water table is high. However, the magnitude of the ground surface movement measured upslope of the pile is approximately double that at the pile location.

The results in Figure 5.11 show that by the end of monitoring in March 2008, lateral movements had increased to 50 mm, 27 mm and 19 mm at the slope surface (ie 156.9 mAOD) – 151.9 mAOD and 148.4 mAOD, respectively. These increases appear to be progressive in nature and indicate the development of some shearing about 20 m upslope of the line of piles.

In Figure 5.11, the measurements at the slope surface indicate that lateral movement is continuing at a rate of about 5 mm/year, so that after a further decade the movement will have increased to about 100 mm. A movement of this magnitude would generally be expected to be accompanied by some crack development in the longer term. However, it must be noted that the rate of movement may slow with time, and this appears to have happened to some extent on the shear planes at 151.9 mAOD and 148.4 mAOD. Continued monitoring is advisable to ascertain if a failure occurs upslope of the piles and if more load is then thrown onto the row of stabilising piles in the longer term.

**5.4.4 Pore water pressures**

Carder and Barker (2005a) compared the variations with time of the pore water pressures measured by the piezometers, installed at the locations and depths shown in Figure 5.3, with rainfall data. There was reasonable correlation and small increases of typically 5 kPa were measured after periods of heavy or prolonged rainfall. Pore water pressures tended to be at their peaks around December/January each year, and this was when the increases in pile movement and bending moments were normally recorded.
5.5 Summary
The method of using a single row of piles to provide a permanent solution to stabilising the Gault Clay slope failure on the M25 has appeared effective during its first five years in service.

Measurements confirm that load came onto the piles following the first significant rainfall after their installation. This response was not unexpected, as some slope movement was necessary for the piles to deflect and generate sufficient resistance. Over the five-year period of monitoring, further load has come onto the piles. As a result, the tops of the piles are continuing to cantilever towards the motorway at a rate of about 2 mm/year and peak bending moments have increased to 897 kNm and 764 kNm in the two instrumented piles.

These findings are not of immediate concern as the toe fixity of the piles has not changed significantly and the measured bending moments remain well below the design moment of 3500 kNm. These progressive changes in movement and moment are considered to be generally related to the adverse effects of prolonged heavy rainfall on the slope during the winter periods. The sign of the bending moment reverses at about 14 m depth (138 mAOD), and this level has remained virtually unchanged since pile remediation was completed.

It is considered that this level is important as a performance indicator, as any future reduction in the level indicates that the fixity of the pile toe in firm strata is not as good as before.

Subsurface lateral movement of the ground has continued 20 m upslope of the pile line, with peaks of movement occurring at the slope surface and at two other depths. By February 2004, 33 mm of movement was measured at the slope surface where there are surficial head deposits, with peaks of 21 mm and 14 mm occurring at depths of about 5 m below the slope surface (within the zone of weathered Gault Clay) and 9 m (in the unweathered Gault Clay). By the end of monitoring in March 2008, these lateral movements had progressively increased to 50 mm, 27 mm and 19 mm at the respective levels. The measurements at the slope surface indicate that lateral movement is continuing at a rate of about 5 mm/year so that, if this rate remains unchanged, an overall movement of 100 mm will be recorded after a further decade. A movement of this magnitude would generally be expected to be accompanied by some crack development. Continued monitoring is therefore advisable in the longer term to ascertain if a failure occurs upslope of the piles and if more load is then thrown onto the row of stabilising piles in the longer term.
6 Case history study on the A12

Although the case history study of the pile-reinforced slope on the M25 reported in Section 5 was important because of the strategic importance of the continued safe operation of the motorway, the slope was not typical in terms of the dimensions and slope angle of the cutting and embankment slopes for which the technique was originally developed by TRL.

An opportunity to validate the use of the piling remedial technique in a more conventional situation became available when the HA Area 6 Maintaining Agent (WS Atkins Consultants Ltd) recommended the use of the spaced piling technique to remediate a cutting slope on the A12 Colchester Bypass. This slope has had a history of failures and been previously reinstated, but failed again by deep-seated slippage in the underlying clay. The most recent failure occurred in December 1997. A more permanent structural solution was therefore sought by using a line of spaced piles to stabilise the London Clay slope.

6.1 Site description

The site is located on the southbound carriageway of the A12 Colchester Northern Bypass between the A134 junction and the B1508 overbridge. The slope in this area is about 300 m long, of maximum height 9 m, with a slope angle of approximately 1v:2h.

The cutting slope is essentially constructed in London Clay, although various repairs have been carried out by reinstating the slope using imported granular material. A number of ground investigations have been reported over the last decade; the findings of these are summarised in a geotechnical review of existing data by WS Atkins Consultants Ltd (2002a).

The plan view in Figure 6.1 shows the extent of the failed zone and the location of the row of piles installed to stabilise the slope. The piles, which were 13 m long and 900 mm in diameter, were installed in a row about halfway up the slope and at 3 m centres. Figure 6.2 gives a typical cross-section in the area and shows the depth of the cemented Dunkirk slag that was used in the last reinstatement. Also apparent in Figure 6.2 is the significant tension crack near the top of the slope and the slumping towards its toe. The slag overlies firm to stiff brown weathered London Clay. Below a depth of about 9 m from the top of the slope, this high-plasticity clay becomes grey and very stiff with depth and is typical of that encountered in unweathered London Clay geologies. Limited data from piezometers installed in 1998 (WS Atkins Consultants Ltd, 2002b) indicate a perched water table in the weathered clay, with the water table in the unweathered clay increasing approximately hydrostatically with depth from the interface between the softened and intact clays.

In their geotechnical report, WS Atkins Consultants Ltd (2002b) assumed effective stress strength parameters of \( c' = 0 \) and \( \phi = 35^\circ \) for the slag fill and \( c' = 1.5 \) and \( \phi = 20^\circ \) for the brown weathered London Clay, and calculated, from back analyses of stability, a factor of safety of 0.97 for the failed slope.

![Figure 6.1 Plan view of the extent of the failed zone (courtesy of WS Atkins Consultants Ltd)](image1)

![Figure 6.2 Section view through the failure (courtesy of WS Atkins Consultants Ltd)](image2)
6.2 Construction sequence
The initial construction activities involved the earthworks necessary to construct a working platform and haul road for the piling rig, cranes and concreting lorries, so that piles could be installed in a row about halfway up the slope. Temporary benching was also constructed (Figure 6.3) to assist in other site operations such as installing the drainage.

All earthworks and piling operations were carried out at night when traffic management could be put in place to divert traffic onto the other carriageway with minimum inconvenience to road users. Figure 6.4 shows the rotary rig used to auger the 900 mm diameter holes for the piles prior to craning the 13 m long reinforcement cages into position. A full-length tremie pipe was used to pour the concrete to the bottom of the pile, and this tremie was progressively shortened during the course of the concrete pour. The concrete mix design was for Class 40/20 concrete (characteristic strength of 40 N/mm$^2$, maximum aggregate size of 20 mm).

Following installation of the piles, fill was placed in layers on the temporary benches and compacted using six passes of a sheepsfoot roller. The final construction activities involved topsoiling and then excavating a trench and backfilling with filter material to form a cut-off drain at the top of the slope. The condition of the existing filter drain at the toe of the slope was also checked to confirm that it was in working order.

6.3 Instrumentation
Instrumentation was installed to monitor the lateral movement of the ground and piles, the strain distribution in the bored piles and the pore water pressures in the ground. Section and plan views of the instrumentation layout are shown in Figures 6.3 and 6.5, respectively.

Inclinometer tubes (11 and 12) were attached to the reinforcing cages for piles 10 and 12 prior to their installation. Each tube was attached to one of the main longitudinal steel bars using specially designed steel fittings that retained it in position but allowed the cage to bend when lifted without the inclinometer tube being damaged. Both inclinometer tubes were installed along the 13 m length of the pile cages with the tube ends only 0.5 m from the pile toe.

Inclinometer tubes I3 and I4 were installed to measure subsurface lateral movements of the ground (a) at locations between piles 11 and 12, and (b) at a location about 10.6 m upslope from the row of piles. The tube I3 was positioned to detect any differences between pile movements and that of the clay between them; any differential movements would

Figure 6.3 Schematic section view through the instrumented area

Figure 6.4 Night-time working of the rotary rig used to auger the pile bore
indicate flow of the clay between the piles. Both ground inclinometer tubes were grouted into 100 mm diameter boreholes sunk to depths of 13.7 m (tube I3) and 15 m (tube I4) below working platforms using a track-mounted rotary drilling rig. The boreholes were backfilled with a bentonite-cement grout mix, which was designed to have a similar stiffness to that of the surrounding London Clay.

Two profiles each of 24 vibrating wire embedment gauges were installed on the reinforcing cages of piles 10 and 12. The gauges, which were installed in pairs, were attached to vertical reinforcing bars such that one gauge of each pair was on the downslope face of the pile and the other gauge was on the upslope face. This arrangement enabled both axial strains and bending strains (perpendicular to the line of the piles) to be separately determined. The gauge pairs were installed at approximately 1 m intervals of depth so that the strain distributions along the length of the two piles could be established.

Axial loads were determined for the 900 mm diameter piles assuming a modulus (E) of \(3.1 \times 10^{5} \text{kN/m}^{2}\) for the concrete, which had a characteristic strength of 40 N/mm\(^2\) at 28 days (Table 3 of British Standard BS 5400-4 (BSI, 1990)). Bending moments per pile were evaluated from bending strains based on an equivalent flexural rigidity (EI) of \(1.3 \times 10^{6} \text{kNm}^{2}\) per pile. In the calculation of I, allowance was made for the main reinforcement of 16 T32 bars. These values assume that the concrete would remain uncracked at the small strain levels involved and are appropriate under short-term loading.

The pore water pressure distribution with depth was monitored using four vibrating wire piezometers, with high air entry pressure tips, installed in each of three boreholes. The uppermost piezometer in each borehole was fitted with twin hydraulic tubes (sealed by taps at the slope surface) to permit water to be occasionally circulated for de-airing purposes. This was considered necessary because of the possibility of pore water suction existing during the summer months.

The piezometers were installed in 100 mm diameter boreholes sunk using the rotary drilling rig. Each piezometer tip was surrounded by a sand cell of 100 mm length with the remainder of each borehole being backfilled with bentonite pellets to ensure a good seal. The locations of the boreholes (Figures 6.3 and 6.5) were such that the pore water pressure distributions close to the piles and in the middle and upper part of the slope were monitored.

### 6.4 Performance monitoring during construction

In interpreting the monitoring data during the construction period, it is worth noting the following points.

- The instrumented piles were installed between 12 and 14 March 2003 and all values on the various pile instruments were referred to a datum date of 19 March so that, in spite of the tight construction schedule, at least some time had elapsed for the pile concrete to cure.
- Ground inclinometer tubes were installed a little later and initial readings on these instruments were taken on 25 March.

![Figure 6.5 Pile and ground movements measured during construction](image-url)
6.4.1 Lateral movements of the piles and the ground between the piles

Figure 6.5 shows the lateral movements of pile 12 (inclinometer tube I2) and those of the ground midway between piles 11 and 12 (inclinometer tube I3), which were developed from the start of backfilling behind the piles on 25 March until completion of slope reinstatement on 2 April. After completion of reinstatement, the measurements on pile 12 showed that the top of the pile had cantilevered out by about 7 mm. No significant movement of the pile occurred below a level of about 28 mAOD and this was confirmed by the peak bending moment location reported in Section 6.4.2. The measured movements of the pile can be compared with the results in Figure 6.5b for the movements of the ground between the piles. Larger movements at the ground surface of up to 15 mm were recorded during backfilling as the clay was squeezed between the piles by the backfilling operations, although movements only extended to about 1 m depth. The smaller pile movements at ground level, but greater depth to which movements occurred, were considered to be a consequence of the flexural rigidity of the pile providing some constraint and re-distributing the load. It must be noted that inclinometer tube I1 in pile 10 ceased to function after 25 March because of damage incurred during backfilling; until that time, results were near identical to those recorded for pile 12.

6.4.2 Bending moments in the piles

The bending moment profiles with depth measured during construction using vibrating wire strain gauges installed on the reinforcing cages of piles 10 and 12 are shown in Figure 6.6. In both cases, a classical profile with depth for a cantilevered pile was obtained. Peak bending moments were measured around 29 mAOD, indicating that the transition between the failed zone of the slope and the stiffer underlying strata was near this level.

At the end of slope reinstatement, the maximum bending moment measured on pile 12 was 647 kNm/pile. This value was slightly higher than the maximum of 472 kNm/pile measured on pile 10, possibly because pile 12 was where the slip was deepest and therefore likely to carry more load.

6.4.3 Lateral movement of the ground upslope of the pile line

The changes in lateral movement of the ground, measured on inclinometer tube I4 at a location 10.6 m upslope of the pile line, are shown in Figure 6.7.

By completion of reinstatement of the slope, a significant downslope movement of 324 mm had been recorded at the top of the tube. This large movement was primarily caused by compaction plant operating near to the inclinometer tube. Movements extended to a level of about 35 mAOD, which corresponded to the excavated level shown in Figure 6.7 prior to the compaction of the fill.
6.5 Performance during first two years in service

Because of the significant changes in lateral movements of the ground and piles that occurred during construction, new datum values were established for the inclinometer readings on 2 April 2003, and changes since this date therefore relate to the in-service performance only. As pile bending moments and pore water pressures are absolute measurements, the original datum dates established after installation of these instruments remained unchanged.

6.5.1 Lateral movements of the piles and the ground between the piles

Figure 6.8 shows the lateral movements of pile 12 and the ground movements between piles 11 and 12 measured after construction. In Figure 6.8a, it should be noted that the uppermost reading on pile inclinometer I2 is actually in the soil covering the top of the pile and slightly more lateral movement is recorded in this area. More movement was also recorded over the upper 1 m of ground inclinometer tube I3 (midpile), although in terms of the overall performance of the remediation technique these movements of the loosely placed topsoil/clay were not deemed significant.

If these top readings are ignored and the measurements at 33 mAOD plotted against time, the results (Figures 6.9a and 6.9b) are very similar for inclinometer tubes I2 and I3. In both cases, there were small increases in lateral movement in 2003, but by the end of 2004 movements had nearly stabilised with about 4 mm of movement being recorded in both cases. These findings suggest that there is little differential movement developing between the pile and the ground between them, ie no significant plastic flow of the clay between piles.
6.5.2 Bending moments in the piles

A comparison of the bending moments measured in piles 10 and 12 is given in Figure 6.10. Measurements on piles 10 and 12 at the end of construction (2 April 2003) gave maximums of 472 kNm and 647 kNm per pile, respectively. Since then, the maximum moments have slowly increased, and after about a further year (26 March 2004) reached values of 567 kNm and 854 kNm per pile, respectively. These values increased very slightly to 595 kNm and 893 kNm per pile by the end of monitoring in February 2005. The development of the peak bending moments with time is shown in Figure 6.11a, and the results confirm that the peak bending moments on pile 10 have stabilised while those on pile 12 have very nearly done so.

For comparative purposes, the lateral movement at the top of pile 12 has been reproduced (from Figure 6.9a) in Figure 6.11b. The graph confirms the conclusion from the bending moment plot that values have currently nearly stabilised, although the fact that the piles may be called on to give more support to the slope if exceptionally adverse rainfall events occurred cannot be completely discounted.

These latest pile bending moment values were slightly below the design values (WS Atkins Consultants Ltd, 2002b), which gave an expected working moment of 940 kNm and the maximum sustainable bending moment of 1090 kNm.
Figure 6.10 Pile bending moments measured after construction

(a) Pile 10

(b) Lateral movement measured in pile 12

(a) Maximum bending moments measured in piles 10 and 12

(b) Lateral movement measured in pile 12 (I2)

Figure 6.11 Development of pile moments and movements with time after construction

(a) Datum: 19/03/2003
14/04/2003
31/07/2003
06/11/2003
26/03/2004
16/06/2004
11/11/2004
16/02/2005
Datum: 19/03/2003
14/04/2003
31/07/2003
06/11/2003
26/03/2004
16/06/2004
11/11/2004
16/02/2005

(a) Pile 10

(b) Pile 12

Figure 6.10 Pile bending moments measured after construction

(a) Maximum bending moments measured in piles 10 and 12

(b) Lateral movement measured in pile 12 (I2)

Figure 6.11 Development of pile moments and movements with time after construction
6.5.3 Lateral movement of the ground upslope of the pile line

Figure 6.12 shows the ground movements at 10.6 m upslope of the pile line measured from the completion of construction (2 April 2003). This suggests that significant movement of about 35 mm occurred at the top of the inclinometer. However, it must be noted that this movement occurred in the first few months after construction and is considered to be related to these operations rather than in-service performance. On this basis, the movements are generally within ±3 mm of the new datum values and, as shown in Figure 6.13, the lateral movements near the top of the tube are then consistent with the small movements associated with seasonal swell/shrinkage of the clay.

6.5.4 Pore water pressures

The variations in pore water pressure with time measured by the piezometers at each of the locations are reported by Carder and Barker (2005b) and compared with the daily rainfall recorded at a nearby meteorological station. Generally, the only significant seasonal changes in pore water pressure were recorded on the shallowest piezometer at a depth of about 2.5 m below the slope surface at each of the three locations. The behaviour of the shallowest piezometer near the top of the slope showed some response to prolonged periods of heavy rain with noticeable rises in pore water pressure of between 5 kPa and 10 kPa. These were considered to be related to temporary build-ups of water in the cut-off drain at the top of the slope, which deals with considerable run-off from the adjoining field.

The variations in pore water pressure with depth measured after slope remediation are shown in Figure 6.14. Generally, the free-draining nature of the Dunkirk slag, which was used for backfilling the temporary benches in the slope, together with the action of the filter drain at the top of the slope, controlled the apparent water table to about 2–3 m depth below the slope surface. There was evidence of a perched water table in the weathered clay of the slope, as pore water pressures increased with depth but then reduced again nearer to the weathered/unweathered clay interface at about 9 m depth below the crest of the slope.

Limited data from piezometers installed during a ground investigation in 1998 (WS Atkins Consultants Ltd, 2002b) indicated that pore water pressures in the unweathered London Clay then increased approximately hydrostatically with depth below this interface, although none of the TRL piezometers were located deep enough to confirm this.

6.6 Summary

At this scheme, access for the piling rig and associated construction plant was obtained by cutting a number of temporary benches into the slope to provide firm working platforms. After installation of the row of piles was completed, placement and compaction of fill to reinstate these benches and restore the slope to its original shape acted to pre-load the piles. During these operations, the tops of the piles cantilevered downslope by about 7 mm, with an associated increase in the maximum pile bending moment to 647 kNm. These peak bending moments occurred near the transition between the sheared zone of the slope and the stiff underlying clay strata, and this was consistent with little lateral movement occurring below this level.

Subsequent to this pre-loading during construction, little change in pile movement and moment was recorded over the following two years in service. Over this period, only a further 4 mm of lateral movement was recorded at the top of the piles, with no evidence of any flow of clay between the piles. An associated increase in the peak bending moment to 893 kNm per pile was measured. It is considered that had the piles not been pre-loaded during construction, the pile movements and moments would have developed more slowly over a period of time.
Figure 6.13 Comparison of lateral ground (10.6 m upslope) with movements of the piles

Figure 6.14 Variation in pore water pressure distribution with depth
7 Design implications

Recommendations within this Insight Report are generally applicable for clay highway slopes that are typically up to about 10 m in height. For higher slopes, more than one row of piles or anchoring of the pile heads may be considered necessary for stability. It should be noted that this Insight Report concentrates on giving design guidance when only a single row of cantilevered piles is employed, although it is anticipated that some of the philosophies can be used more widely.

7.1 Site investigation

When using bored piles to stabilise or strengthen a slope against a potential or active deep-seated failure, there are three fundamental factors to ascertain during the site investigation in order to achieve a safe and economic design. These are:

- to identify the potential or active failure plane or zone,
- to identify any water-bearing or weak strata, and
- to identify whether a stiff founding stratum exists for the piles.

This information can be obtained in the usual way from a combination of desk studies, topographical surveys, soil profiling, knowledge of the ground water and drainage, laboratory and in situ testing, instrumentation and monitoring, and investigation of any site-specific features and environmental issues.

7.2 Factors for consideration in the design

The main factors that need to be considered when carrying out the design of the piling system to stabilise the slope are now discussed.

7.2.1 Spacing and diameter of the piles

The spacing between and diameter of the piles must be designed both to maximise the arching of the soil between the piles and minimise the flow of soil between them. Various theories have been developed for design purposes based on a combination of elastic and plastic soil behaviour, and these are discussed in Section 7.3.

Carder and Temporal (2000) drew analogies with other construction techniques where soil flow and arching occur. The principal finding from a review of the performance of piled foundations embedded in clays is that a pile spacing of about five times the pile diameter is sufficiently large to minimise the interaction between adjoining piles, and no interaction is expected at a spacing of more than eight diameters. On this basis, it can then be argued that in a slope situation, flow of soil between piles is fairly certain at spacings of more than eight diameters and possible at spacings of more than five diameters. A review of design recommendations for spill-through abutments suggested a similar result with an upper limit for spacing of eight pier diameters (or widths) for arching to be effective in granular soils. It must be noted that some authors, however, recommended a lower ratio of five diameters.

Centrifuge tests carried out by Hayward et al. (2000) concluded that a clay slope with piles installed at a spacing of six pile diameters failed, while the slopes with piles installed at three and four diameters did not. Three-dimensional finite element modelling of ground deformations under working loads (ie pre-failure) investigated the differential movement between the piles and ground midway between them (Carder and Easton, 2001). For both embankment and cutting slopes, little differential movement was predicted when piles were spaced at three-diameter centres whereas significant movement occurred when the spacing was increased to six diameters.

On the basis of the above discussion, it is anticipated that the spacing between pile centres in a typical stabilisation scheme for a highway slope is likely to be in the range of three to five pile diameters. Pile diameters will to some extent be dictated by site access and the type of rig that can be mobilised, but are generally expected to be between 600 mm and 1200 mm.

7.2.2 Lateral resistance supplied by the piles

Although most of the existing design approaches enable the lateral force acting on each pile to be determined, the lateral resistance supplied by the piles will be limited by the yield pressure of the soil both above and below the potential slip surface. Information on yield pressures is available both for the case of a laterally loaded single pile pushing through level ground and creating pressures in front of the pile, and for the movement of soil past existing piles. Both cases have similarities with smaller yield pressures near the surface than at depth.

For non-cohesive soils, Broms (1964) used a limiting soil reaction per unit length of pile of $3K\sigma_v'd$ where $d$ is the diameter of the pile. Fleming et al. (1994) suggested from the data of Barton (1982) that at depths of up to 1.5 diameters, the limiting force per unit length was better given by $K_1\sigma_v'd$. At depths beyond this, $K_2\sigma_v'd$ was considered a better approximation. Although these formulas were derived for a pile installed in level ground, account can then be tentatively taken of the slope angle by modifying the value of $K_1$ according to whether the resistance of the soil to the top of the pile deflecting or the toe fixity of the pile is being determined.

For piles in cohesive soil, Fleming et al. (1994) reported that the limiting pressure developed on the front of the pile from the wedge of the soil near the ground surface is approximately $2c_v'd$. It can be conservatively assumed that this value increases linearly to about $9c_v'd$ at a depth of three pile diameters and then remains constant. More complicated expressions from wedge theory have been derived by Ashour et al. (1998) and for piles in sloping ground by Gabr and Borden (1990).

The ultimate lateral resistance of the lower part of the pile below the potential rupture zone is important. If the pile is short because it is founded in an underlying stiff stratum, its lateral resistance will be high and large bending moments will develop on reversal of curvature as the pile passes into the founding layer (Carder and Easton, 2001). With long piles in more homogeneous ground, the lateral resistance will develop more uniformly along its length, and the pile bending moments will generally be lower. The maximum shear resistance of the pile itself also needs consideration to ensure it has sufficient capacity.
7.2.3 Penetration and location of the piles
The decision on the penetration of the piles is primarily dependent upon the findings of the ground investigation. When using a row of piles to stabilise a cutting or embankment slope or to steepen a slope for highway-widening purposes, the piles must extend well below the expected critical failure surface. This ensures that the failure zone does not increase to encompass the toe of the piles and so obviates the potential support gained from them. The fixity of the toes of the piles is particularly important as the larger the restoring force they can sustain, the smaller will be the slope movements that occur.

Little guidance is available in the literature on the optimum location of the piles in the slope. However, if the row of piles is placed near the top of the slope, a significant failure may simply occur in front of the piles. Likewise, if the row of piles is installed near the bottom of the slope, although heave of the carriageway may be prevented, a significant failure may occur behind the piles. Engineering judgement would suggest that the row of piles is probably better placed about one-third to one-half of the way up the slope, although this can be validated for a particular site situation using finite element or other numerical techniques.

7.2.4 Overall stability
Methods for determining overall stability are separately discussed in Section 7.3. The normal factors crucial to a realistic stability analysis, as with an unreinforced slope, are the pore water pressure distribution in the slope and the appropriateness of the soil strength parameters. Residual strengths should be preferred to model behaviour in zones where shearing has already occurred. Back analyses of slope failures by Crabb and Atkinson (1991) have also shown that critical state values, which lie between peak and residual strength, provide a good prediction in over-consolidated clay slopes.

It must be noted that the location of the critical failure surface will change and is likely to become deeper when the piles are installed, and for this reason an iterative procedure needs to be used to identify the new critical surface.

7.2.5 Differences between cutting and embankment situations
There are significant differences in the behaviour of unreinforced cutting and embankment slopes, which have implications for the design of a strengthening system using piles. Firstly, because cuttings are constructed from in situ clay, the permeability of the clay will generally be lower than that of clay fill used for an embankment, and softening will therefore proceed at a slower rate over many decades before a delayed collapse may occur (Skempton, 1964; Vaughan, 1994). If clay fill for an embankment has been excavated from a deep borrow pit, suctions will exist in the clay lumps and their dissipation may lead to significant softening and swelling over a shorter period of time.

The development of the failure zone will also be dependent upon the lateral stress within the clay, and Potts et al. (1997) reported that for cutting slopes, the depth and extent of progressive failures tended to increase with the over-consolidation ratio. Generally, failures with cuttings, although often slower in developing, will be deeper than those with embankments. For this reason, the depth of penetration and the lateral resistance of the stabilising piles generally need to be much larger in a cutting situation. Measurements of pile lateral movements and bending moments showed increases in response to prolonged periods of heavy rainfall as the piles were called on to provide more support to a cutting slope in Gault Clay (Carder and Barker, 2005a). Emphasis in design therefore needs to be placed on the long-term condition.

Carder and Easton (2001) found that for embankment slopes, the influence of the drainage blanket was significant in improving stability – not only because pore pressures are controlled, but also because the potential rupture is confined to above the stiff granular layer forming the blanket. This was the case with the analyses of performance of both the unreinforced and pile-reinforced slopes.

Finite element analyses carried out by Carder and Easton (2001) also indicated that in a cutting slope, a build-up in soil 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. The limit pressure methods of Broms (1964) and Fleming et al. (1994) may therefore be appropriate to determine loads near the top of the pile in this case. In an embankment situation, a build-up in lateral stress was predicted upslope of the top of the pile and only small stresses downslope of the pile. This mechanism of 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 than limit pressure methods near the top of embankment piles.

7.2.6 Influence of construction sequence
As discussed in Section 6, the installation of large-diameter bored piles involves access for a piling rig, which generally needs to operate from a firm platform, ie a bench cut into the slope. After installation of the row of piles is complete, placement and compaction of fill to reestablish the bench and restore the slope to its intended shape may act to pre-load the piles. This effect was reported by Carder and Barker (2005b), who found that the tops of the piles cantilevered downslope with an associated increase in pile bending moments during the reinstatement of significant benching at one particular slope. Subsequent to this pre-loading, little further change in pile movement and moment was recorded over the following two years in service.

Provided that the moment capacity of the pile is not exceeded, this effect is not viewed as a design problem. However, reinstatements using a reduced thickness for fill layers and light compaction plant may be a sensible precaution in some critical cases.

Where the piles are not pre-loaded in this way, some slope movement is anticipated before pile resistance can be mobilised. The development of further pile lateral movements and bending moments may then be in response to prolonged periods of heavy rainfall as the piles are called on to provide more support to the slope (Carder and Barker, 2005a).
7.3 Design procedures

In general, the current design approaches involve three main stages. These are:

i) evaluating the restoring shear force needed to achieve the required factor of safety for stability of the slope,

ii) evaluating the lateral resistance that each pile can provide to resist sliding, and

iii) selecting the diameter, centre-to-centre spacing, penetration and most suitable location on the slope for the piles.

It does not necessarily follow that these stages need to be considered in the above order, as they are inter-related. In many cases, for site-specific reasons, details of the pile dimensions and layout may well be established first.

In stage iii, an important consideration is to ensure that pile spacing is such either that soil arching between piles can be sustained or that soil flow between piles does not occur within the potential or actual failure zone of the slope. This mechanism is not automatically included in the calculation of factors of safety using stability analysis for the pile-reinforced slope, and needs separate consideration.

The alternative fundamental methods of design are now considered in turn, although a hybrid of these methods may prove convenient in many cases.

7.3.1 Evaluating the restoring shear force to achieve the required factor for slope stability

The stability of the slope can be investigated by taking account of the extra restoring force provided by the piles during calculations that may be based on limit equilibrium methods such as the friction circle method and the method of slices (Bishop, 1955; Bishop and Morgenstern, 1960; Morgenstern and Price, 1965). Many computer programs for slope stability analysis now include the option of examining reinforcement of the slope using vertical piles.

The restoring force can be calculated from the theory of ultimate resistance described by Poulos and Davis (1980). Figure 7.1 shows an idealised section through a slope reinforced by a single row of piles where a circular failure is being assumed. Poulos and Davis give the alternative ways in which the maximum value of the restoring force can be ascertained. These are:

i) from the ultimate lateral resistance of a “short” pile, where full mobilisation of soil strength occurs over the lower part of the pile (L_2),

ii) from the ultimate lateral resistance of a “long” pile, where lateral capacity is mainly dependent on the yield moment of the pile,

iii) from the ultimate load that can be developed if the soil flows between the piles over the upper part (L_1), and

iv) from the shear resistance of the pile section itself.

For stability, the least of the above four forces is appropriate. A sensitivity analysis of the stability of the slope, taking account of the restoring force from the piles, then enables decisions to be made on the pile penetration, its location on the slope and the pile diameter. It must be noted that the critical failure surface changes when the piles are installed in the slope, and for this reason an iterative procedure needs to be followed.

7.3.2 Evaluating the lateral resistance provided by each pile

The lateral resistance of each pile needs consideration both below and above the potential or active failure surface. Limit pressure methods of Broms (1964) and Fleming et al. (1994) are generally used to determine the ultimate lateral resistance of the lower part of the pile below the potential rupture zone, and these are described in Section 7.2.2. Viggiani (1981) used a two-layer model and studied the different failure modes that can occur and derived expressions for the maximum shear force exerted by the pile on the slip surface and of the bending moments acting on the pile. These solutions were for the ultimate limit state of purely cohesive soils and their cohesion was assumed to be constant with depth in both the unstable and stable zones. Poulos (1973) developed a more rigorous approach, with the restoring force from the piles being calculated from numerical analysis using boundary elements.

Although limit pressure methods can be used to calculate the soil pressure on the upper part of the pile above the potential rupture zone, they suffer from the disadvantage that they take no account of arching or flow of soil between the piles, and for this reason give no guidance on suitable pile spacing. Methods based on rigid-plastic behaviour and plastic deformation can therefore be used to calculate the soil pressure and determine pile spacing.
3.2.1 Methods based on rigid-plastic behaviour

Various authors have developed methods, which are based both on a yielding layer that is parallel to the slope and on wedge-type failures. Some of the better known methods are now considered.

- The method reported by Wang and Yen (1974) is fairly classical for an infinitely long slope and uses rigid-plastic theory to consider the forces on an element of soil in a yielding layer that is parallel to the slope (Figure 7.2). As would be anticipated, they found more arching occurred as the strength of the soil (i.e. $\phi'$ and $c'$) increased.

For a sandy soil ($c' = 0$) and using the nomenclature as defined in Figure 7.2, Wang and Yen found a critical spacing ($m_{cr} = B/h$) as follows:

$$m_{cr} = \frac{K(K+1)\tan\phi}{\cos i (\tan i - \tan\phi_1)}$$

where $K$ is the coefficient of earth pressure at rest and assumed to be equal to $(1 - \sin\phi')$. At spacings greater than $m_{cr}$, the piles are not likely to provide any stabilisation. An optimum spacing ($m_{m}$) was also derived at which arching is likely to be most effective, and the variation of critical and optimum spacings with $\phi'$ is illustrated in Figure 7.3 for a 1v:2h slope.

![Figure 7.2](image1.png)

![Figure 7.3](image2.png)

**Figure 7.2** Plan view of series of piles: (a) on slope; (b) cross-section; (c) generic element (after Wang and Yen, 1974; by courtesy of ASCE)

**Figure 7.3** Relationship between pile spacing and soil properties for 1v:2h sandy slope (after Wang and Yen, 1974; by courtesy of ASCE)
A similar relationship was developed for cohesive soil ($\phi' = 0$) as follows:

$$m_c = \frac{2 (c/\gamma_h) \cos i}{\cos i \sin i - (c/\gamma_h)}$$

This equation is plotted in Figure 7.4 for $c = c_1$ and a number of slope angles. It must be noted that the diameter of the piles is not directly involved in the above formula as the arching is assumed to be related to the opening between piles and Wang and Yen assumed that pile size would be sufficiently large to enable the arching mechanism to develop.

b) Typical of the wedge-type methods is the approach reported by Day (1999) for drilled piers used to stabilise slopes. Figure 7.5 shows a planar rupture surface inclined at an angle ($\alpha$) to the horizontal. The factor of safety of the slope can then be defined by considering restoring and perturbing forces parallel to the rupture surface using the following equation:

$$\text{Factor of safety} = \frac{c' L + (W \cos \alpha - u L) \tan \phi' + P_i}{W \sin \alpha}$$

where $L$ is the length of the slip surface, $W$ is the weight of the failure wedge material and $u$ is the average pore water pressure along the slip surface.

For a particular factor of safety, the inclined force on the pier ($P_i$) can then be calculated and the lateral design force ($P_L$) for each pier determined using the equation:

$$P_L = S P_i \cos \alpha$$

where $S$ is the spacing between pier centres.

Figure 7.4 Critical spacing of piles in clay slopes ($c = c_1$) (after Wang and Yen, 1974; by courtesy of ASCE)

Figure 7.5 Design of pier wall for wedge slope failure (after Day, 1999; by courtesy of ASCE)
7.3.2.2 Methods based on plastic deformation

Ito and Matsui (1975) and Ito et al. (1981) considered the state of plastic deformation in the ground just around the piles (Figure 7.6) assuming it satisfied the Mohr-Coulomb yield criterion. When piles are placed at intervals along the slope, they have a preventive effect against plastic deformation. A number of equations and design charts were developed for different soil strengths, which enabled the force acting on the pile to be determined. For example, the equation for the lateral force \( p \) acting on a pile per unit thickness of layer is as follows:

\[
p = c A \left\{ \frac{1}{N_\phi \tan \phi} \left[ \exp \left( \frac{D_1 - D_2}{D_2} N_\phi \tan \phi \left( \frac{\pi}{4} + \frac{\phi}{2} \right) \right) \right] - 2N_\phi^{1/2} \tan \phi \cdot 1 \right. \\
+ \frac{2 \tan \phi + 2N_\phi^{1/2} + N_\phi^{1/2}}{N_\phi^{1/2} \tan \phi + N_\phi - 1} \\
- \left. c \left[ D_1 \left( \frac{2 \tan \phi + 2N_\phi^{1/2} + N_\phi^{1/2}}{N_\phi^{1/2} \tan \phi + N_\phi - 1} \right) - 2D_1 N_\phi^{-1/2} \right] \right\}
\]

The constant \( A \) is equal to \( D_1(D_1/D_2)^b \) where \( b = (N_\phi^{1/2} \tan \phi + N_\phi - 1) \) and \( N_\phi = \tan^2[\pi/4 + \phi/2] \). The soil strength parameters are \( c \) and \( \phi \), \( \gamma \) is the unit weight of soil, \( z \) is the depth and \( D_1 \) and \( D_2 \) are the distances as defined in Figure 7.6.

Figure 7.7 shows that in general, for a constant diameter of pile, the lateral force increases as the interval between piles becomes progressively narrower. This is because the soil just around the piles finds it harder to pass through the gap between them and more load is therefore transferred to the piles. On this basis, the factor of safety of the slope would increase as the spacing between piles decreases and more load is taken by the piles. For the same reason, the lateral force on the piles increases as the soil strength parameters, \( c' \) and \( \phi' \), increase. Popescu (1991), Cantoni et al. (1989) and others have used the design approach of Ito and Matsui (1975) with apparent success, and more recently the method has been further rationalised by Hassiotis et al. (1997).

Hassiotis et al. (1997), following the approach of Ito and Matsui (1975), used integration of the formula for plastic deformation of the soil between piles to determine the ultimate load over the upper part of the piles. Their slope analyses based on determination of this force showed that, with the piles in place in both cases, the improvement in the factor of safety of the slope needed to take account of the change in location of the critical failure surface from its original position (Figure 7.8). Although the factors of safety generally increased with \( S \), it should be noted that the optimum location to minimise risk of failures above or below the pile line is likely to be if they are installed about one-third to one-half of the way up the slope.

Figure 7.6 Plastically deforming ground around stabilising pile (after Ito and Matsui, 1975; by courtesy of the authors)
**Figure 7.7** Effect of pile diameter ($D_1 - D_2$) on plastic deformation (after Ito and Matsui, 1975; by courtesy of the authors)

**Figure 7.8** Effect of pile location on factor of safety (after Hassiotis et al., 1997; by courtesy of ASCE)
7.3.3 Other design procedures

Other design approaches included those of more sophisticated numerical analysis and approximate checks using modified design methods for retaining walls.

7.3.3.1 Numerical analyses

Theories representing a Winkler beam on an elastic foundation have been widely used in the design of laterally loaded piles with empirically derived non-linear springs to represent the soil (p-y curves) or a model of the soil as a linear elastic continuum (Matlock and Reese, 1960; Broms, 1964; Poulos and Davis, 1980). However, the accuracy of such solutions depends upon the characterisation of the interaction between the pile and the surrounding ground. A particularly good representation of the soil-pile interaction yields a more realistic solution. Generally, the analysis of a row of piles is based on either superimposing the behaviour of a number of single piles or on extrapolating the solution to cover a row of piles using semi-empirical interaction factors. Most of these analyses also consider the soil to have a horizontal surface and make no provision for the situation where a row of piles is in a slope.

More sophisticated procedures, in some cases involving finite and boundary element analysis and more complex soil models, have been developed (Rowe and Poulos, 1979; Chen and Poulos, 1997). These methods tend to concentrate on the ultimate lateral resistance of the piles rather than approaching spacing design via arching theory. Chen and Poulos (1997) and Yegian and Wright (1973) both concluded that for a single row of piles in a direction perpendicular to the loading, the spacing is generally greater than 2.5 pile diameters and therefore pile interaction has only a small influence on their ultimate lateral resistance.

More complex finite element analyses involving three dimensions and a plasticity model for the soil, which allows gap formation, have been undertaken by Brown and Shie (1990; 1991), and in addition to providing a better model of arching between piles these can give a better overall view of likely performance. It is not practical to give simple rules for the evaluation of the optimum spacing between piles based on finite element analyses because of the large number of variables, although three-dimensional models of this nature are expected to provide an effective design method.

7.3.3.2 Retaining wall methods

An approximate check of the order of magnitude of the bending moments derived using the other methods can be carried out using retaining wall theory. The extra perturbing force on the piles due to the slope can be estimated using two alternative approaches as follows:

i) The extra load from the clay slope is accounted for by increasing the active pressure coefficient (K_a) on the retained side of the wall to a value of 1. This is the maximum tabulated value for \( \beta / \phi' \) of 1, where \( \phi' \) is the angle of internal friction of the soil and \( \beta \) is the slope angle to the horizontal; beyond this, the value for \( \beta / \phi' \) of greater than 1 is indeterminate (Eurocode 7; BSI, 2004).

ii) The height of the clay slope above the tops of the piles is represented by a uniform vertical surcharge equivalent to 10 kN/m² per metre height. This is about half of that which would exist behind the crest of the slope.

The geometry and parameters adopted when using the two approaches are shown in Figure 7.9. In both cases, the benefit of the support of the soil wedge in front of the pile is ignored. The water table is assumed at the top of the pile in the retained ground and at cutting level in front of the wall. The conservative assumptions of no wall friction and adhesion can also be employed, so that the design looks at the worst case scenario.

On this basis of a continuous wall, the factors of safety on mobilised soil strength can then be calculated in the normal way following the recommendations of British Standard BS 8002 (BSI, 1994). The results will be conservative because the restoring force from the wedge of soil in front of the piled wall has been ignored. If desired, some allowance for the stabilising effect of the berm (ie the wedge in front of the piled wall) can be made by applying a uniform vertical surcharge equivalent to about one-third of the berm height (Fleming et al., 1994).

Wall bending moments can also be calculated using this approach, although the results need to be factored up to take account of the spacing between pile centres. This crude evaluation generally gives bending moments of the same order of magnitude as those predicted by other methods.
Figure 7.9 Using retaining wall design approaches
8 Conclusions

The main conclusions from the research can be summarised as follows:

i) Piles spaced at intervals in a row along a clay slope provide an effective remedial measure and improve the stability of slopes that have either failed or are showing incipient signs of failure.

ii) The spaced piling technique can also be used to steepen slopes so that highway corridors can be widened within existing boundaries to accommodate extra vehicular lanes or other modes of transport. This avoids additional landtake and minimises the associated costs and delays to construction.

iii) Validations of the spaced piling technique have been carried out using various methods:
   a) Centrifuge modelling of a cutting slope in uniform clay.
   b) Three-dimensional finite element modelling of both clay cuttings and embankments.
   c) Performance monitoring at a Gault Clay slope on the M25 and a London Clay slope on the A12. Both slopes are in-cut and a further case history study in an embankment situation would be advantageous.

iv) The main design factors for consideration when using a single row of spaced piles to support a slope include the spacing and diameter of the piles, their lateral resistance, the optimum penetration and location of the piles, and the overall stability of the slope. The depth of penetration and required lateral resistance of stabilising piles for a cutting are likely to be greater than for an embankment slope. In addition to overall stability, an important design consideration is to ensure that pile spacing is such that soil arching between piles is sustained without soil flow between piles.

The findings within this Insight Report are generally applicable for clay highway slopes that are typically up to about 10 m in height. For higher slopes, more than one row of piles or anchoring of the pile heads may be considered necessary for stability.

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REFERENCES


Improving the stability of slopes using a spaced piling technique

Discrete piles are used to stabilise infrastructure slopes, especially where there is insufficient land to allow construction of large toe berms or regrading of the slope. In addition to providing a technique that can be used to steepen slopes with minimum landtake, spaced piling can be used for the permanent and cost-effective reinstatement and repair of unstable or failed clay slopes. This is particularly relevant to many motorway slopes, which are prone to shallow failures and are now reaching a critical age for deep-seated failure to occur. The situation is also exacerbated by the increasing incidence of heavy rainfall and flooding that has occurred in recent years due to the effects of climate change, which will add to the risk of softening of the clay. For these reasons, the Highways Agency initiated a comprehensive programme of research with TRL to investigate the potential of the piling technique. This Insight Report discusses the findings from a literature review, centrifuge and analytical studies, and two instrumented case history studies of the remediation of clay slopes on the highway network. The implications of the findings are discussed and appropriate design advice recommended.

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