The performance of a single row of spaced bored piles to stabilise a Gault Clay slope on the M25

Prepared for Safety, Standards and Research (Civil Engineering Division), Highways Agency

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Executive Summary

The technique of using a single row of spaced piles to stabilise landslides and natural slopes has been used widely and with success for some considerable time, however there is a paucity of instrumented case history studies. TRL Report TRL466 (2000) reviewed those that were available. This review was a precursor to a series of centrifuge tests to compare the performance of unreinforced and reinforced cutting slopes (TRL Report TRL471, 2000) and three-dimensional finite analyses to investigate the likely behaviour of both embankment and cutting slopes (TRL Report TRL493, 2001). These studies generally indicated that the determination of suitable pile diameter, spacing, penetration depth and location on the slope is a complex soil-structure interaction problem much influenced by the slope angle, soil strength and the pore water pressure regime.

Following a deep-seated failure of a clay cutting slope near junction 6 of the M25, the technique selected by the designer (Mott MacDonald) to improve the stability of the slope was that of a single row of spaced bored piles accompanied by extensive drainage works. The opportunity therefore existed to complement the earlier desk, model and analytical studies undertaken by TRL with an instrumented case history study.

The 200m long deep-seated failure of the predominantly Gault 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 80m up the slope to the motorway hard shoulder, which suffered a heave of 150mm.

During pile installation for the remedial work, instrumentation was installed to monitor the subsurface lateral movements of the slope and piles, the bending moment distribution in the piles, and the pore water pressure regime in the slope. The main findings from the monitoring carried out both during construction and over the first 26 months in service and the implications of the findings on future designs are discussed in the report.

The method of using a single row of piles to provide a permanent solution to stabilising the slope at Godstone has appeared effective during its first 26 months in service. However because of continued subsurface movement of the ground, about 20m upslope of the piles, and because of recent increases in pile movements and bending moments caused by exceptional rainfall, monitoring of longer term performance may be advisable.

The implications of the findings from this case history study and a further study at Colchester Bypass (A12) will contribute to a future document giving design advice appropriate to the piling technique.
1 Introduction

The technique of using a single row of spaced piles to stabilise landslides and natural slopes has been used widely and with success for some considerable time, however there is a paucity of instrumented case history studies. Carder and Temporal (2000) have reviewed those available at that time. This review was a precursor to a series of centrifuge tests to compare the performance of unreinforced and reinforced cutting slopes (Hayward et al., 2000) and three-dimensional finite analyses to investigate the likely behaviour of both embankment and cutting slopes (Carder and Easton, 2001). These studies indicated that the determination of suitable pile diameter, spacing, penetration depth and location on the slope is a complex soil-structure interaction problem much influenced by the slope angle, soil strength and the pore water pressure regime.

A unique opportunity to complement these earlier desk, model and analytical studies by an instrumented case history study therefore arose following a deep-seated failure of a Gault Clay cutting slope near Junction 6 of the M25. A 200m long failure of the 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 80m up the slope to the motorway hard shoulder, which suffered a heave of 150mm. The technique selected for remediation of the slope was that of a single row of reinforced concrete bored piles accompanied by extensive drainage works comprising the construction of deep counterfort drains downslope of the piles and a cut-off drain upslope of the backscarp.

This report describes the installation of instruments during the construction activities and discusses the monitoring data obtained both during the construction and over the first 26 months in service from October 2001 until December 2003 following the remedial works. Instrumentation was installed to monitor the subsurface lateral movements of the slope and piles, the bending moment distribution in the piles, and the pore water pressure regime in the remediated slope. The results from this case history study are likely to be invaluable in providing improved design guidance for future construction of a similar type.

2 Site description and ground conditions

The site is located adjacent to Flower Lane bridge in the north face of the cutting slope at Flint Hall Farm. The eastbound on-slip road from the A22 joins the M25 at junction 6 just before the bridge. A plan showing the location of the site and the extent of the slope failure that occurred in December 2000 is shown in Figure 1. As previously mentioned the failure was about 200m in length and extended in front of the piled foundations of Flower Lane bridge. Figure 1 also shows the location of the single row of piles subsequently used to stabilise the slope.

Figure 1 Location of slope failure (courtesy of Mott MacDonald)
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 Highways Agency Managing 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 10m below ground level’ (Ground Engineering, 2001). Sections through lines B-B and C-C (see Figure 1) are shown in Figure 2, post-failure inclinometer measurements showed continuing lateral movements of up to about 35mm along the shear planes which were identified. The shape of the failure wedge along the line of the TRL instrumentation was expected to have been somewhere between those recorded on lines B-B and C-C.

The ground conditions identified during the site investigation are also shown in Figure 2. Generally the slope comprised stiff fissured Gault Clay, in some boreholes weathering of the upper zones was readily identified, in others it was not possible to differentiate. 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 (Perry, 1989). On the motorway network they tend to have a predominant slope angle of 1:2.5. In this case, the relatively deep-seated failure occurred even though the slope angle was only 1:4 to 1:5.

The Mott MacDonald geotechnical report (2001) found that 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 and 61kPa. Results from one set of triaxial tests gave peak effective stress shear strength parameters, \(c'_p\) and \(\varphi'_p\), of 10kPa and 29° respectively. Residual values of \(c'_r\) and \(\varphi'_r\) were 1.5kPa and 13.5°.

![Figure 2](image-url) Ground profiles (courtesy of Mott MacDonald)
The unweathered Gault comprised stiff dark grey fissured clays and silts. Plastic and liquid limits were typically 36% and 82% with natural moisture contents of 29%. Undrained shear strengths were variable but generally increased with depth and were in the range of 35kPa to 195kPa at depths of up to 15m.

Based on these results and those of four previous investigations in the area, Mott MacDonald (2001) adopted effective stress design values for the Gault Clay of 1kPa and 24° for c' and \( \phi' \). Residual values for c' and \( \phi' \) of zero and 14° were employed in sheared zones.

3 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 about 25m upslope from the M25 kerb line. The rig which was used to install the 1050mm diameter and 16m long piles at 2500mm centres was a Soilmec 622 (Figure 3). At two locations where headroom was limited, that is beneath Flower Lane bridge and beneath the overhead electricity cables, 310mm diameter and 9m long piles were installed at 750mm centres using a smaller rotary drilling rig. These latter two locations were remote from the instrumented area.

In parallel with the piling operations, construction of a deep cut-off drainage trench upslope of the backscarp took place. North of the instrumented area, this cut-off drain was approximately 4m deep with a width of between 0.7 and 1.2m. The cut-off drain was completed by early September and, in the instrumented area, piling works and other construction activities then took place according to the schedule given in Table 1.

The dates of installation of the various instruments are also summarised in Table 1. These instrumentation operations were co-ordinated with the various construction activities in such a way that no delay to the construction work was incurred.

When installing the strain gauged piles, a full length tremie pipe was used to ensure that damage to the gauges did not occur from free fall of the wet concrete. The tremie was then progressively shortened during the course of the concrete pour. This measure ensured that all gauges remained functional after concreting was completed. The concrete mix design was for C40/20 concrete (characteristic strength of 40N/mm²; maximum aggregate

Table 1 Timetable of construction activities in the instrumented area

<table>
<thead>
<tr>
<th>Date</th>
<th>Day no.</th>
<th>Construction activity</th>
<th>Instrument installation</th>
</tr>
</thead>
<tbody>
<tr>
<td>04/09/01</td>
<td>32</td>
<td>Completion of construction of cut-off drain (upper slope).</td>
<td>Inclimeter tubes I1 (pile 21) and I2 (pile 23) installed.</td>
</tr>
<tr>
<td>06/09/01</td>
<td>34</td>
<td>Piles 21 and 23 installed.</td>
<td>Strain gauge 1 to 26 (pile 22) installed.</td>
</tr>
<tr>
<td>07/09/01</td>
<td>35</td>
<td>Piles 22 and 25 installed.</td>
<td>Strain gauge 27 to 50 (pile 24) installed.</td>
</tr>
<tr>
<td>10/09/01</td>
<td>38</td>
<td>Piles 24 and 20 installed.</td>
<td>Ground inclinometer tube I3 installed between piles 22 and 23.</td>
</tr>
<tr>
<td>01/10/01</td>
<td>61</td>
<td>Ground inclinometer tube I3 installed between piles 22 and 23.</td>
<td>Piezometers P1 to P4 installed near piles 23 and 24. Ground inclinometer tube I4 installed at mid-slope.</td>
</tr>
<tr>
<td>02/10/01</td>
<td>62</td>
<td>Counterfort drains installed near the instrumented area (ie. in front of piles 20, 22/23 and 25).</td>
<td>Piezometers P5 to P8 installed at mid-slope. Piezometers P9 to P12 installed at upper slope.</td>
</tr>
<tr>
<td>17/10/01</td>
<td>77</td>
<td>Clay slope regraded in vicinity of instrumented area.</td>
<td>Instrument cables ducted to cabinets. Instrumentation completed.</td>
</tr>
<tr>
<td>25/10/01</td>
<td>85</td>
<td>Topsoil placed in vicinity of instrumented area.</td>
<td></td>
</tr>
</tbody>
</table>

Figure 3 Rotary augering of pile borehole
size of 20mm), 50% ground granulated blastfurnace slag (ggbs) was used as cement replacement, and the mix had a nominal slump of 150mm which gave it the required workability for use in this application.

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 600mm wide trenches were excavated to a nominal 3m depth at 6m centres in front of the piles. The trenches were filled with type B filter material and connected to a filter drain at the toe of the slope that carried the water away. The locations of both the cut-off drain and counterfort drains in the instrumented area are shown in Figures 4 and 5.

Final construction activities involved regrading, placement of top soil, and reseeding with grass to restore the slope to its original condition so that it could be once again used as pasture.

4 Instrumentation

Instrumentation was installed in the ground and in the piles to monitor:
- lateral movement of the ground and piles;
- strain distribution in the bored piles;
- pore water pressures in the ground.

Plan and section views of the instrumentation layout are shown in Figures 4 and 5 respectively.

4.1 Lateral movement of the ground and piles

Proprietary sleeve jointed plastic inclinometer tubes (I1 and I2) were attached to the reinforcing cages for piles 21 and 23 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

Figure 4 Plan of instrumented area
allowed the cage to bend when lifted without the inclinometer tube being damaged (Figure 6). Both inclinometer tubes were installed along the 16m length of the pile cages with the tube ends at only 0.5m from the pile toe. After the pile cages were lowered into the augered hole, the inclinometer tubes were ballasted with water to prevent uplift pressures during the concrete pours. The tops of the inclinometer tubes were subsequently extended when piling was completed so that they protruded about 0.5m above finished ground level.

![Figure 6 Lifting of instrumented pile cage](image)

Inclinometer tubes I3 and I4 were installed to measure subsurface lateral movements of the ground at locations (a) between piles 22 and 23, and (b) at a location about 20m 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. Both ground inclinometer tubes were grouted into 150mm diameter boreholes sunk to depths of 19.5m (tube I3) and 16.5m (tube I4) using a cable tool percussion drilling rig. A bentonite-cement grout mix was used which was designed to have a similar stiffness to that of the surrounding Gault Clay. The tops of the inclinometer tubes were designed to protrude about 0.5m above final ground level so that convenient access was available for monitoring purposes.

The completed installations were protected from cattle damage by placing a concrete manhole ring on the slope surface around each inclinometer tube. 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. The validity of this assumption is discussed in more detail in Section 5.

### 4.2 Strain distribution in the piles

Two profiles each of twenty-four vibrating wire embedment gauges were installed on the reinforcing cages of piles 22 and 24 respectively. 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 1.1m intervals of depth so that the strain distributions along the length of the two piles could be established. After installation of the instrumented piles and regrading of the slope, the strain gauge cables were subsequently ducted to an instrumentation cabinet.

Each strain gauge incorporated a thermistor for temperature measurement. The temperature data were useful in monitoring the heat of hydration during concrete curing so that appropriate strain gauge datum values could be adopted after the major part of any shrinkage had occurred.

Axial loads were determined for the 1050mm diameter piles assuming a modulus (E) of $31 \times 10^6$ kN/m$^2$ for the concrete, which had a characteristic strength of 40N/mm$^2$ at 28 days (Table 3 of BS5400: Part 4: 1990). Bending moments per pile were evaluated from bending strains based on an equivalent flexural rigidity (EI) of
falling on the 7th October 2001 and a further 19mm on the
particularly heavy rainfall occurred with 41mm of rain
Shortly after installation of the instrumented piles
5.1 Lateral movement of the ground and piles
Table 2 Depth and location of each piezometer

<table>
<thead>
<tr>
<th>2.5m upslope of pile line</th>
<th>20m upslope of pile line</th>
<th>36m upslope of pile line</th>
</tr>
</thead>
<tbody>
<tr>
<td>GL = 151.2m AOD</td>
<td>GL = 156.3m AOD</td>
<td>GL = 158.7m AOD</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Piezo-meter no.</th>
<th>AOD of tip (m)</th>
<th>Piezo-meter no.</th>
<th>AOD of tip (m)</th>
<th>Piezo-meter no.</th>
<th>AOD of tip (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>149.3</td>
<td>P5</td>
<td>153.3</td>
<td>P9</td>
<td>155.7</td>
</tr>
<tr>
<td>P2</td>
<td>147.7</td>
<td>P6</td>
<td>151.8</td>
<td>P10</td>
<td>154.3</td>
</tr>
<tr>
<td>P3</td>
<td>146.3</td>
<td>P7</td>
<td>150.3</td>
<td>P11</td>
<td>152.7</td>
</tr>
<tr>
<td>P4</td>
<td>144.2</td>
<td>P8</td>
<td>148.2</td>
<td>P12</td>
<td>150.7</td>
</tr>
</tbody>
</table>

5 Discussion of results

The measured changes in lateral movements of the ground
and piles, the pile bending moments and the pore water
pressures are now reported for the periods up until the end
of construction and during the first 26 months in service.

The instrumented piles were installed between 6th and
10th September 2001 (see Table 1) and all values on the
various instruments were referred to a datum date of 28th
September so that sufficient time had elapsed for the pile
concrete to cure. Ground inclinometer tubes were installed
in boreholes during early October and initial readings on
these instruments were taken on 16th October.

5.1 Lateral movement of the ground and piles
Shortly after installation of the instrumented piles
particularly heavy rainfall occurred with 41mm of rain
falling on the 7th October 2001 and a further 19mm 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 7. Both piles cantilevered
towards the motorway with lateral movements of 14mm
and 11mm being recorded at the tops of each tube on 16th
October and a few further millimetres by 15th November.
In both cases some movement was measured to about
140mAOD (12m depth). Below this depth the general
shape of the movement plots suggested that the assumption
used in the calculations of base fixity of the tubes was
probably realistic, although the possibility of translation of
the toes of the piles cannot be entirely precluded. 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.

Following this period of rainfall, the pile inclinometer
results plotted in Figure 7 at approximately three monthly
intervals showed little further outward movement even
during the period of intense and persistent rainfall recorded
during November 2002. However, after continued heavy
rain during December 2002, readings taken on 8th January
2003 indicated that more load was coming onto the piles.
A further cantilevering of the piles (by about 4mm at their
top) occurred as they were called on to provide more
support to the slope. This movement was accompanied by
increases in pile bending moments, which are discussed in
Section 5.2. The readings taken on 17th February 2003
showed no significant changes from the readings on 8th
January although, as illustrated by the readings on 24th
December, the lateral movements reduced marginally as
the pore water pressures fell over the following dry
summer and autumn.

Figure 8 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. This
tube became operational on 16th October 2001, although its
datum date was subsequently adjusted to 1st November to
eliminate movements near the surface caused by localised
regrading of the slope. In common with the inclinometer
pipes installed in the piles, very little lateral movement was
then measured from that time up until November 2002.
There was no evidence of any flow of clay between the
piles during the period of monitoring. The results in Figure
8 also indicate that the assumption of base fixity when
determining pile movements was probably justified as tube
I3 (between the piles) extends to about 3m below the pile
toes and shows no evidence of any translation at this depth.
The readings on 8th January, after the period of persistent
rain, showed changes consistent with the recorded pile
movements but no evidence of any flow of the clay
between the piles.

The lateral movements of the ground measured using
inclinometer tube I4 about 20m upslope from the line of
the piles (see Figure 5) are shown in Figure 9. Lateral
movements of about 6mm were measured down to about
148mAOD (9m depth) during late October 2001,
Figure 7 Development of lateral movement of the piles

Figure 8 Ground movement between piles 22 and 23 (inclinometer I3)

Figure 9 Development of subsurface ground movements (inclinometer I4)
indicating a block movement of the Gault Clay and overlying Head Deposits to this depth. This shearing at 9m depth appeared to occur on the original shear surface shown in Figure 2 which was identified during the site investigation for the scheme. 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 tube I4 has continued to occur throughout this period.

In general the development of these movements is strongly related to the cumulative effect of any persistent and heavy rainfall. Changes are particularly evident on the readings taken on 8th January 2003 (day number 525). The rainfall and movement plots given in Figure 10 illustrate this for three selected depths. These depths relate to where the peaks in movement occur (see Figure 9), that is in the superficial Head Deposits, at about 152mAOD (5m depth) within the zone of weathered Gault Clay, and at about 148mAOD (9m depth) in the unweathered Gault Clay. In each case an increase in the lateral movement down the slope can be correlated with the end of periods when significant rainfall occurs. These increases appear to be progressive in nature and indicate the development of some shearing about 20m upslope of the line of piles. The seasonal clay swelling and shrinkage movements related to the pore water pressure changes are also evident in Figure 10. Continued monitoring is advisable to ascertain if more load is thrown onto the row of stabilising piles as a result of the shear progressing with time when further heavy rainfall occurs.

5.2 Strain distribution in the piles
The profiles of bending moment with depth measured on piles 22 and 24 at approximately three monthly intervals are shown in Figure 11. The results from the two piles were very similar in both shape and magnitude throughout the construction stage and during the early months in service.

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 16th October, pile 22 was showing a peak bending moment of 518kNm at about 142mAOD (10m depth) whilst pile 24 gave a peak moment of 358kNm at 1m above this. This behaviour correlated well with the measured cantilevering movement of the piles discussed in Section 5.1 with the peak moment in the unweathered Gault Clay slightly below the original shear surface at about 146mAOD.

Following this initial development in bending moment, only small increases in the moments were recorded over the period of monitoring up until December 2002. Subsequently, following the heavy and persistent rainfall during this month, the readings on 8th January 2003 showed increases in peak bending moments on both piles 22 and 24 to values of 714 and 621kNm respectively. In both cases these increases represented a change of 24% from the previous readings taken 5 weeks earlier. Also worth noting in Figure 11 are the small increases in negative moment recorded towards the top of the piles at about 149mAOD on 8th January 2003. 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.

Strain gauge readings during 2003 (and up until the final reading in December) suggest that the depth to the peak bending moment has reduced for pile 22 and increased for pile 24, although the magnitudes of the peak moments remain similar for both piles. These piles are separated by a distance of 5m and this effect would be consistent with further load coming more onto pile 24 than pile 22 from...
the original slip, which was at an angle to the line of stabilising piles and deeper to the east (Figures 1 and 2). However this interpretation must remain tentative as the lateral movements measured from inclinometers in piles 21 and 23 are similar and do not show this effect.

The magnitudes of the bending moments in the piles are compared with the design predictions in Section 6.

5.3 Pore water pressures in the ground

The variations with time of the pore water pressures measured by the piezometers (installed in profiles with depth at the three locations shown in Figure 5) are given in Figure 12. The results are also compared with rainfall data measured at Kenley Airfield (about 6km north of the site) which were obtained from the Meteorological Office. With the exception of piezometer 10, the trends in the pore pressure changes are very similar with small increases of typically 5kPa being measured after periods of heavy or prolonged rainfall. In comparing rainfall data with piezometer results it must however be noted that, whereas the former is continuous data, the latter are from measurements on visit days only. Piezometer 10 generally showed more fluctuations with time than the other piezometers and it is not clear whether these are a malfunction of this particular instrument or related to variations in the water level in the nearby cut-off drain. Figure 12 also shows that pore water pressures were at a peak on 8th January when changes in pile movement and bending moments were recorded and it is considered likely that further significant changes may occur when pore pressures reach these threshold values again.

Figure 11 Development of bending moment in the piles

Figure 13 shows the variations of pore water pressure with depth at three selected dates. Two of these dates (26th March 2002 and 7th October 2002) were selected to illustrate the normal seasonal range of pore pressure conditions, whilst the values recorded on 8th January are shown for comparison purposes. On the latter date, pore pressures at 20m upslope and those close to the pile line exceeded the normal range of values, whilst any effect was less noticeable at 36m upslope possibly because of the proximity of the cut-off drain.

Also indicated in Figure 13 is the mean level of the water table in the underlying unweathered Gault Clay, below this level the increase in pore water pressure with depth was approximately hydrostatic. At all locations a perched water table existed in the weathered clay and head deposits: this was particularly evident at the location which was 36m upslope. Near the line of the piles, the perched water table was less noticeable as its development was limited by the counterfort drainage system in front of the piles. From the results in Figure 13 it was concluded that the cut-off and counterfort drainage systems were generally effective in controlling the development of excess pore water pressures within the remediated slope.

6 Comparison of measurements with design predictions

6.1 Original design

Davies et al. (2003) report that the pile design at this site was based on the method proposed by Viggiani (1981). In
Figure 12 Variation of pore water pressures with time
Figure 13 Groundwater profile through instrumented area
this way the pile sizes and capacities were designed to ensure that the pile does not develop a plastic hinge and is fixed in the firm underlying soil. If the destabilising forces are increased until failure occurs, the soil will flow around the piles as opposed to a more brittle failure of the pile itself. 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 300kN/m was required to stabilise the 8m deep slip and provide a 20% increase in the factor of safety. This restoring force was provided by using 16m long piles.

The spacing between the pile centres was 2.4 diameters which was well below the spacing of five diameters at which soil flow between the piles becomes likely (Carder and Temporal, 2000).

The most probable distribution of bending moment predicted by the designer is shown in Figure 14 and is compared with the measured profile in January 2003 reproduced from Figure 11. It must however be noted that at this stage of the measurements, further increases in pile moments may still occur in response to heavy rainfall. Although the design values were about 5 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 ($\phi_r$) and the need to include an adequate factor of safety 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.

### 6.2 Other design methods

Ito and Matsui (1975) and Ito et al. (1981) considered the state of plastic deformation in the ground just around the piles 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:

\[
p = c A - \frac{1}{N_\phi \tan \phi} \exp \left[ \frac{D_1 - D_2}{D_2} N_\phi \tan \phi \tan \left( \frac{\pi}{8} + \frac{\phi}{4} \right) - 2 N_\phi^{(1/2)} \tan \phi - 1 \right]
\]

\[
2 \tan \phi + 2 N_\phi^{(1/2)} + N_\phi^{-1/2)
\]

\[
N_\phi^{(1/2)} \tan \phi + N_\phi - 1
\]

\[
- c \left[ \frac{2 \tan \phi + 2 N_\phi^{(1/2)} + N_\phi^{-1/2}}{N_\phi^{(1/2)} \tan \phi + N_\phi - 1} - 2 D_2 N_\phi^{-1/2} \right]
\]

\[
\gamma \frac{A}{N_\phi} \exp \left[ \frac{D_1 - D_2}{D_2} N_\phi \tan \phi \tan \left( \frac{\pi}{8} + \frac{\phi}{4} \right) \right] - D_2
\]

\[
p = K \gamma h^2 (d + B)/2
\]

Figure 14 Comparison of measured and design pile bending moments

where the constants are $A$ which is equal to $D_1(D_1/D_2)^b$ with $b=(N_0^{(1/2)} \tan \phi_0 + N_0^{-1})$ and $N_0 = \tan^2[\pi/4 + \phi/2]$. 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 defined as the distance between pile centres and the spacing between piles respectively.

Based on this equation, the forces and bending moments given in Table 3 were calculated for both peak and residual shear strength parameters for the Gault Clay.

It is interesting to note that in Table 3, higher shears and bending moments are recorded when peak shear strength parameters are employed rather than residual values. This behaviour occurs because, when $c'$ and $\phi'$ are larger, the soils adjacent to the piles are less able to pass between two piles and for this reason the lateral force on the piles increases. In the limiting condition of plastic deformation of the soil near to the piles, the shear and moments calculated using residual strength parameters are expected to be more appropriate.

Wang and Yen (1974) analysed soil arching in slopes between stabilising piles and presented equations for use in both granular and cohesive soils. They reported that a worst case analysis of the total load per pile in the downhill direction could be calculated for cohesive soil using the following formula:

\[
p = K \gamma h^2 (d + B)/2
\]
Table 3 Shear forces and bending moments per pile (after Ito and Matsui)

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Force per pile</th>
<th>Shear moment (kN/pile)</th>
<th>Bending moment (kNm/pile)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Force per pile</td>
<td>Shear moment (kN/pile)</td>
<td>Bending moment (kNm/pile)</td>
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<td>0</td>
<td>0</td>
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<tr>
<td>2.0</td>
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<td>129</td>
</tr>
<tr>
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<td>264</td>
<td>406</td>
<td>417</td>
</tr>
<tr>
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<td>349</td>
<td>713</td>
<td>970</td>
</tr>
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<td>3208</td>
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<tr>
<td>7.0</td>
<td>606</td>
<td>2145</td>
<td>5065</td>
</tr>
<tr>
<td>8.0</td>
<td>691</td>
<td>2793</td>
<td>7527</td>
</tr>
</tbody>
</table>

All values are for 1050mm diameter piles at 2500mm centres.

In this equation \( h \) is the thickness of the yielding layer and if this is assumed to be equal to the depth at which the shear force and bending moment are being determined, the results in Table 4 are obtained. In this formula, the value of \( K \) is assumed to be that for the earth pressure at rest condition and is therefore calculated in the normal way from \( K_a \) (=$1-\sin \phi$), ie. load increases as \( \phi \) decreases. It can be argued that it may be appropriate to take account of the slope angle (\( \beta \)) and use the earth active value (\( K_a \)). If this is carried out for peak strength parameters, then \( \beta / \phi \) is 0.58 and the value of \( K \) is about 10% less than \( K_a \). However for residual strength parameters, \( \beta / \phi \) is 1 and the value of \( K \) is approaching 40% more than \( K_a \) and will underestimate shear loads and bending moments by the same percentage.

Table 4 Shear forces and bending moments per pile (after Wang and Yen)

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Force per pile</th>
<th>Shear moment (kN/pile)</th>
<th>Bending moment (kNm/pile)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0</td>
<td>0</td>
<td>0</td>
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</tr>
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</tr>
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<tr>
<td>8.0</td>
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<td>2519</td>
<td>1207</td>
</tr>
</tbody>
</table>

All values are for 1050mm diameter piles at 2500mm centres.

Comparisons of the predicted shear loads and bending moments using the two methods are shown in Figure 15. The highest values are predicted when using peak shear strength parameters for the clay and the formula of Ito and Matsui (1975), although it can be argued that peak parameters are inappropriate when looking at localised plastic deformations around the piles. When using residual parameters for the methods of Ito and Matsui (1975) and Wang and Yen (1974), the predicted shear loads and bending moments are similar in the upper part of the pile, which is above the slip plane. As would be anticipated with the latter method, the piles carry less load and moment when the shear strength of the soil increases. Both of these methods enable determination of the pile loads and moments in the failed ground above the basal shear plane. Below this depth, a reversal in the curvature of the bending moment profile is anticipated as the pile gains support from its fixity in the unweathered Gault Clay.

The measured distributions of bending moment are compared in Figure 16 with those from the methods of Ito and Matsui (1975) and Wang and Yen (1974) and also with a design check carried out using retaining wall theory (BS8002, 1994). This design check was carried out by conservatively assuming that no lateral support was provided by the soil wedge in front of the piles so that the ground level immediately in front of the contiguous piled wall was effectively that of the M25 carriageway level. Residual strengths (\( c' = 0, \phi' = 14^\circ \)) were assumed for the clay to a depth of 8m and peak strength parameters (\( c' = 1kPa, \phi' = 24^\circ \)) below this. The extra load from the clay slope was accounted for by increasing the active pressure coefficient in the upper 8m layer of the retained ground to a value of 1. This is the value for a clay with \( \phi' = 14^\circ \) and an inclined slope angle of 14° as given in Eurocode 7 (1994). It is worth noting that this approach can only be used for slope angles no greater than the \( \phi' \) angle as beyond this the K value becomes indeterminate. Some allowance for the stabilising effect of the berm (ie. the slope in front of the piled wall) was made by applying a uniform vertical surcharge equivalent to 1.7m soil to the ground in front of the wall: this corresponded to about one third of the berm height (Fleming et al., 1991). The shape and magnitude of the bending moment distribution determined from retaining wall theory was similar both to that used by the designer (Figure 14) and also those determined from the methods of Ito and Matsui (1975) and Wang and Yen (1974).

Generally the values measured in January 2003 were lower than those from the various design calculations. This was not unexpected particularly as the design methods both represent the ultimate limit state condition rather than the working loads that now exist and also make no allowance for the restoring effect of the counterfort drainage system installed immediately in front of the row of piles. Currently the bending moment capacity of the piles is more than adequate for the intended purpose although continued monitoring is required to ascertain whether further increases in pile moments occur in response to significant rainfall in the future.

As discussed in section 5.2, the small negative bending moments which were measured near the top of both piles at about 149mAOA were typical of those predicted for cutting slopes from finite element analyses carried out by Carder and Easton (2001). These arise from a localised build-up in soil stress as the top of the pile attempts to move downslope into the ground in front of it. For a single pile
Figure 15 Comparison of predicted shear and bending moments developed in the piles

Figure 16 Comparison of measured and predicted pile bending moments
pushing into the ground, the zone of yielding will be larger than that for a continuous retaining wall because of three dimensional effects. Broms (1964) proposed a limiting soil pressure of $3K_{\sigma_v}d$ where $d$ is the pile diameter, whereas Fleming et al. (1991) suggested that the limiting pressure was better given by $K_2\sigma_v d$ at depths up to 1.5 diameters and by $K_2\sigma_v d$ at depths beyond this. These limit pressure relationships were based on the piles pushing into level ground and assumptions need to be made if they are to be adopted for use in a slope situation. At this particular site, the magnitude of the effect was very small and limit pressure calculations were not therefore performed.

7 Conclusions

A deep-seated failure of a Gault Clay cutting slope on the M25 occurred in December 2000. The technique selected to improve the stability of the slope was that of a single row of spaced bored piles accompanied by extensive drainage works. During the remediation work, instrumentation was installed to monitor the subsurface lateral movements of the slope and piles, the bending moment distribution in the piles, and the pore water pressure regime in the slope. The main findings from the monitoring carried out both during construction and over the first 26 months in service and the implications of the findings on future designs were as follows:

i Shortly after installation of the instrumented piles particularly heavy rainfall occurred in early October 2001 with 60mm of rain falling over two days. Both instrumented piles cantilevered towards the motorway with lateral movements of 14mm and 11mm being recorded at the tops of each tube. This movement was accompanied by the development of peak bending moments of 518kNm and 358kNm in the two piles. The shape of the bending moment distribution with depth was as expected for a cantilever movement with the peak moments being located in the unweathered Gault Clay slightly below the original shear surface. 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.

ii Only small changes in the lateral movements and bending moments in the stabilising piles were measured from October 2001 until December 2002. However during this period, lateral movements of the ground measured about 20m upslope from the line of the piles continued to increase. In general, the development of these movements was progressive and well correlated to the cumulative effect of any persistent and heavy rainfall. Three distinct peaks in the movement profile were identified at this location namely in the superficial Head Deposits, at about 5m depth within the zone of weathered Gault Clay, and at about 9m depth in the unweathered Gault Clay.

iii Following heavy rainfall in the previous month, which saturated the ground, further persistent rain followed by snow occurred up to the 8th January 2003 at which time the measurements indicated that more load was coming onto the piles. This was characterised by a further cantilevering of the piles (by about 4mm at their top) and a 24% increase in pile bending moments as the piles were called on to provide more support to the slope. Associated increases in subsurface lateral movement of up to 5mm were measured 20m upslope of the line of the piles and high pore water pressures were recorded.

iv Generally the cut-off and counterfort drainage systems appeared reasonably effective in controlling the development of high excess pore water pressures within the remediated slope, although some variations in pore pressure still occurred which were related to the magnitude and duration of any rainfall. Pore pressures were particularly high on 8th January when some slope and pile movements were observed.

v Comparison of the measured bending moments in the piles with predictions using various design methods indicated that the latter were about 5 times higher, although the trends in shape were very similar and consistent with the piles cantilevering. This discrepancy was not unexpected particularly as the design methods represent the ultimate limit state condition and the need to account for long term behaviour whereas the measurements relate to the working loads developed in the short term. Furthermore the design predictions make no allowance for the stabilising effects of the counterfort drainage system installed immediately in front of the row of piles and the other drainage improvements. Currently the bending moment capacity of the piles is more than adequate for purpose although continued monitoring is advisable to ascertain whether further increases in pile moments occur in response to significant rainfall in the future.

vi Design predictions of the pile bending moments using empirical methods and retaining wall theory appeared to agree reasonably. Their use, in combination with slope stability analyses to establish the necessary restoring force for stability and the necessary pile embedment into underlying firm strata, should provide a safe basis for future designs.

vii The method of using a single row of piles to provide a permanent solution to stabilising the slope at Godstone has appeared effective during its first 26 months in service. However, because of continued subsurface movement of the ground about 20m upslope of the piles and because pile movements and bending moments increase following exceptional rainfall, monitoring in the longer term may be advisable.

viii The study at Godstone forms a special case in so far as the 25m height of slope and the 1.4 to 1.5 slope angle are unusual. A height of up to 10m and slope angle of 1:2 are more typical of the many clay slopes on the highway network and it is on these that the TRL development work on the piling technique has focussed. A further case study on a clay slope of more typical geometry is being undertaken at Colchester Bypass (A12) and these results will be separately reported. The two case history studies should provide HA engineers and their designers with more confidence in the use of the technique.
8 Acknowledgements

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9 References


Abstract

Following a deep-seated failure of a Gault Clay cutting slope on the M25, the technique selected to improve the stability of the slope was that of a single row of spaced bored piles accompanied by extensive drainage works. The opportunity therefore existed to complement earlier desk, model and analytical studies on this piling technique undertaken by TRL with an instrumented case history study.

Instrumentation was installed to monitor the subsurface lateral movements of the slope and piles, the bending moment distribution in the piles, and the pore water pressure regime in the remediated slope. This report describes the instrumentation and discusses the monitoring data obtained both during the construction and over the first 26 months in service following the remedial works. The implications of the findings are discussed with a view of improving the design guidance for future construction of a similar type.

Related publications

TRL493  *Analysis of performance of spaced piles to stabilise embankment and cutting slopes* by D R Carder and M R Easton. 2001 (price £35, code J)

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

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

RR199  *A survey of slope condition on motorway earthworks in England and Wales* by J Perry. 1989 (price £20, code C)

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