The performance of a single row of spaced bored piles to stabilise a London Clay slope on the A12

Prepared for SSR Geotechnics Operations Support Division, Highways Agency

D R Carder and K J Barker
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Executive Summary

Following extensive TRL research on the topic, a single row of bored piles spaced at intervals was selected by the HA Maintaining Agent for the remediation of a failed cutting slope on the A12 Colchester Bypass. This London Clay slope had a history of failures and been previously reinstated, but failed again by deep-seated slippage in the underlying clay. A more permanent structural solution was therefore sought by using the piling technique to stabilise the slope.

Monitoring of pile and slope performance was undertaken to provide a unique case history study which is expected to be of value to designers of future constructions of a similar type. This report describes the installation of instruments during the construction activities and discusses the field data obtained both during the construction and over the first two years in service 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.

At this scheme, access for the piling rig was obtained by cutting a temporary bench into the slope to provide a firm working platform. A number of other benches were also excavated in the slope to facilitate movement of associated construction plant. 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. The tops of the piles cantilevered down-slope with an associated increase in pile bending moments during this reinstatement.

Subsequent to this pre-loading during construction, little change in pile movement and moment was recorded over the following two years in service. It is considered that had the piles not been preloaded in this way, the pile movements and moments would have developed more slowly over a period of time.

Over the two year period of measurement, the single row of piles was considered to have provided very effective support to the clay slope and it is anticipated that a permanent solution has been achieved to improve the slope stability.
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 associated delays to construction. In many cases the additional lateral loading which 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. The same technique can be used for the permanent and cost effective reinstatement and repair of unstable or failed slopes. For these reasons, a significant programme of research has been undertaken by TRL Limited on behalf of the Highways Agency to investigate the potential of the technique and provide design advice.

The preliminary stages of this research included a literature review of the use of the spaced piling technique (Carder and Temporal, 2000), 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.

Following the desk and laboratory based studies, an initial instrumented case history study was undertaken when a row of spaced piles was used to stabilise a deep-seated failure of a clay cutting slope near Junction 6 of the M25. Carder and Barker (2005) and Carder, Lung and Barker (2003) report the findings of this study. Although monitoring the performance of the piling technique at this site was essential in terms of ensuring the continued safe operation of this section of the M25, in some ways this particular slope forms a special case. This is because the slope is not typical in terms of dimensions and slope angle of the cutting and embankment slopes for which the technique was originally developed by TRL.

A unique 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 Northern 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.

This report describes the installation of instruments during the construction activities and discusses the field data obtained both during the construction and over the first two years in service 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 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 300m long, of maximum height 9m, with a slope angle of approximately 1v:2h. The location of the slope is shown in Figure 1.

![Figure 1 Location of the slope near Colchester](image)

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 2 shows the extent of the failed zone and the location of the row of piles installed to stabilise the slope. The piles, which were 13m long and 900mm in diameter, were installed in a row about halfway up the slope and at 3m centres. Figure 3 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 3 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 9m 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) indicates 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.

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 in a row about halfway up the slope. Temporary benching was also constructed as shown in Figure 4 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

Figure 2 Plan view of the extent of the failed zone (courtesy of W S Atkins)

Figure 3 Section view through the failure (courtesy of W S Atkins)

Figure 4 Temporary benching used during construction
inconvenience to road users. Figure 5 shows the rotary rig used to auger the 900mm diameter holes for the piles prior to craning the 13m long reinforcement cages into position (Figure 6). 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 40N/mm², maximum aggregate size of 20mm).

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.

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.

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.

Section and plan views of the instrumentation layout are shown in Figures 7 and 8 respectively.

Figure 5 Rotary rig used to auger the boreholes

Figure 6 Lowering of a reinforcement cage into a typical borehole
### Table 1 Timetable of construction activities in the instrumented area

<table>
<thead>
<tr>
<th>Date</th>
<th>Construction activity</th>
<th>Instrument installation</th>
</tr>
</thead>
<tbody>
<tr>
<td>12/03/2003 to 14/03/2003</td>
<td>Piles 10 and 12: strain gauges and inclinometer tube.</td>
<td></td>
</tr>
<tr>
<td>16/03/2002</td>
<td>Installation of piles 10 to 12.</td>
<td></td>
</tr>
<tr>
<td>19/03/2003 to 21/03/2003</td>
<td>Backfilling behind pile line up to pile 9.</td>
<td>Ground inclinometers and piezometers.</td>
</tr>
<tr>
<td>25/03/2003</td>
<td>Some backfilling behind pile line in instrumented area.</td>
<td>Upper slope inclinometer tube extended.</td>
</tr>
<tr>
<td>25/03/2003 to 29/03/2003</td>
<td>Excavation of trench at crest of slope, installation of cut-off drain and backfilling.</td>
<td></td>
</tr>
<tr>
<td>28/03/2003</td>
<td>Excavation of some backfill from behind pile line in instrumented area.</td>
<td>Emergency repairs carried out on inclinometers I1, I2 and twelve strain gauge cables.</td>
</tr>
<tr>
<td>02/04/2003</td>
<td>Backfilling and reinstatement of slope completed.</td>
<td>Readings taken. Inclinometer I2 blocked at 5m.</td>
</tr>
<tr>
<td>07/08/2003</td>
<td>Further attempt at clearing I2.</td>
<td></td>
</tr>
</tbody>
</table>

![Figure 7 Schematic section through the instrumented area](image)

#### 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 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 13m 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. Figure 9 shows the inclinometer tube in position after cage installation but before concreting. The tops of the inclinometer tubes were subsequently extended.

![Figure 8 Schematic plan of instrumentation layout](image)
when piling was completed so that they protruded about 0.5m above finished ground level.

Inclinometer tubes I3 and I4 were installed to measure subsurface lateral movements of the ground at locations (a) between piles 11 and 12, and (b) at a location about 10.6m 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 100mm diameter boreholes sunk to depths of 13.7m (tube I3) and 15m (tube I4) below working platforms using a track mounted rotary drilling rig (Figure 10). 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. The tops of the inclinometer tubes were extended to take account of the backfilling so that they protruded about 0.5m above final ground level to provide convenient access for monitoring purposes.

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 10 and 12 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. Gauges can be seen mounted on one of the instrumented cages in Figure 9. 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 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 instrument cabinet located over pile 12.

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 900mm diameter piles assuming a modulus (E) of 31×10^6 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 1.3×10^6 kNm^2 per pile. In the calculation of I allowance was made for the main reinforcement of sixteen T32 bars. These values assume that the concrete would remain uncracked at the small strain levels involved and are appropriate under short term loading.

4.3 Pore water pressures in the ground
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 suctions existing during the summer months.

The piezometers were installed in 100mm diameter boreholes sunk using the rotary drilling rig. Each piezometer tip was surrounded by a sand cell of 100mm length with the remainder of each borehole being backfilled with bentonite pellets to ensure a good seal. The locations of the boreholes (Figures 7 and 8) were such that the pore water pressure distributions close to the piles and in the middle and upper part of the slope were monitored. The depth and location of each piezometer is given in Table 2.
Table 2 Depth and location of each piezometer

<table>
<thead>
<tr>
<th>Piezometer no.</th>
<th>AOD of tip (m)</th>
<th>Piezometer no.</th>
<th>AOD of tip (m)</th>
<th>Piezometer no.</th>
<th>AOD of tip (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P0</td>
<td>31.15</td>
<td>P5</td>
<td>32.26</td>
<td>P1</td>
<td>34.30</td>
</tr>
<tr>
<td>P10</td>
<td>28.65</td>
<td>P6</td>
<td>29.76</td>
<td>P2</td>
<td>31.80</td>
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<tr>
<td>P11</td>
<td>26.15</td>
<td>P7</td>
<td>27.26</td>
<td>P3</td>
<td>29.30</td>
</tr>
<tr>
<td>P12</td>
<td>23.65</td>
<td>P8</td>
<td>24.76</td>
<td>P4</td>
<td>26.80</td>
</tr>
</tbody>
</table>
5 Performance during construction

The changes in lateral movements of the ground and piles together with the pile bending moments measured during construction are now reported. For this purpose the effective end of construction was considered to be 2nd April 2003, although placement of topsoil actually took place after this date.

The instrumented piles were installed between 12th and 14th March 2003 and all values on the various pile instruments were referred to a datum date of 19th March so that 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 25th March. Measurements of pore water pressures during construction are not specifically reported as the piezometers were only installed during late March and probably had insufficient time to come to equilibrium after their installation. The piezometer readings obtained during construction are however included in Section 6 for completeness.

5.1 Lateral movements of the ground and piles

Figure 11a and 11b show 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 25th March until completion of slope reinstatement on 2nd April. After completion of reinstatement, the measurements on pile 12 showed that the top of the pile had cantilevered out by about 7mm. No significant movement of the pile occurred below a level of about 28mAOD and this was confirmed by the peak bending moment location reported in Section 5.2. The measured movements of the pile can be compared with the results in Figure 11b for the movements of the ground between the piles. Larger movements at the ground surface of up to 15mm were recorded during backfilling as the clay was squeezed between the piles by the backfilling operations, although movements only extended to about 1m 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 25th March because of damage incurred during backfilling, until that time results were near identical to those recorded for pile 12.

The changes in lateral movement of the ground, measured on inclinometer tube I4 at a location 10.6m upslope of the pile line, are shown in Figure 12. By completion of reinstatement of the slope, a significant downslope movement of 324mm had been recorded at the

![Figure 11](image-url) Pile and ground movements developed during construction
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 35mAOD which corresponded to the excavated level shown in Figure 7 prior to the compaction of the fill.

5.2 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 13. In both cases, a classical profile with depth for a cantilevered pile was obtained. Peak bending moments were measured around 29mAOD 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 647kNm/pile. This value was slightly higher than the maximum of 472kNm/pile measured on pile 10, possibly because pile 12 was where the slip was deepest and therefore likely to carry more load.

6 Performance during the first two years in service

Because of the significant changes in lateral movements of the ground and piles which occurred during construction, new datum values were established for the inclinometer
readings on 2nd 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.

6.1 Lateral movements of the ground and piles
Figure 14 shows the lateral movements of pile 12 and the ground movements between piles 11 and 12 measured after construction. In Figure 14a 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 1m 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 33mAOD plotted against time, the results (shown in Figures 15a and 15b) are very similar for inclinometer tubes I2 and I3 respectively. In both cases there were small increases in lateral movement in 2003, but by the end of 2004 movements had nearly stabilised with about 4mm 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, i.e. no significant plastic flow of the clay between piles.

Figure 16a shows the ground movements at 10.6m upslope of the pile line measured from the completion of construction (2nd April 2003). This suggests that significant movement of about 35mm 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. The results have therefore been re-plotted based on a datum date of 26th June 2003 and these are shown in Figure 16b. On this basis the movements are generally within ±3mm of the new datum values and, as shown in Figure 15c, the lateral movements near the top of the tube are then consistent with the small movements associated with seasonal swell/shrink of the clay.

6.2 Bending moments in the piles
A comparison of the bending moments measured in piles 10 and 12 is given in Figure 17. Measurements on pile 10 and 12 at the end of construction (2/04/2003) gave maximums of 472kNm and 647kNm per pile. Since then the maximum moments have slowly increased and after about a further year (26/03/2004) reached values of

![Figure 14](image-url)
Figure 15 Development of pile and ground movements with time after construction
Figure 16 Ground movements 10.6m upslope from the pile line measured after construction

Figure 17 Pile bending moments measured after construction
567kNm and 854kNm per pile respectively. These values increased very slightly to 595kNm and 893kNm per pile by the end of monitoring in February 2005. The development of the peak bending moments with time is shown in Figure 18a and the results confirm that the peak bending moments on pile 10 have stabilised, whilst 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 15) in Figure 18b. The graph confirms the conclusion from the bending moment plot that values have currently near 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, 2002b), which gave an expected working moment of 940kNm and the maximum sustainable bending moment of 1090kNm.

6.3 Pore water pressures in the ground
The variations in pore water pressure with time measured by the piezometers at each of the locations are shown in Figure 19. Figure 19a also provides details of the daily rainfall recorded at a nearby meteorological station for comparative purposes.

Generally the only significant seasonal changes in pore water pressure were recorded on the shallowest piezometer at a depth of about 2.5m below the slope surface at each of the three locations. The behaviour of the piezometer near the top of the slope (Figure 19b) showed some response to prolonged periods of heavy rain with noticeable rises in porewater pressure of between 5 and 10kPa. 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. A more gradual rise and fall in response to seasonal rainfall was recorded on piezometer 5 (Figure 19c) and piezometer 9 (Figure 19d) further down the slope. Elsewhere very little seasonal change in pore water pressures was recorded.

The variations in pore water pressure with depth measured at the various locations after slope remediation are shown in Figure 20. 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 2m to 3m 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 9m 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.

Figure 18 Development of pile moments and movements with time after construction
Figure 19 Variation in pore water pressure with time
7 Comparison of measurements with design predictions

7.1 Original design
Details of the design of the remedial works are given by WS Atkins Consultants Ltd (2002b). The geotechnical parameters assumed in the design to provide a long term solution are given in Table 3.

Stability analyses assumed a 1.5m thick layer of sheared London Clay over intact but softened brown clay. The interface of the brown weathered zone and the underlying grey, unweathered zone in the London Clay is approximately 9m below the crest of the slope. The piles were designed to improve the shearing capacity of the slope by intersecting potential slip surfaces and providing shear resistance to a sliding mass above. The additional shear capacity to bring the slope to a factor of safety of 1.2 was determined using the computer program SLOPE/W.

WS Atkins report that when the shear resistance required to stabilise the deep slides was established, the piles were designed as laterally loaded piles with a shear strength corresponding to the resistance required for the slope to achieve adequate overall stability. The lateral pile analysis was then used to determine the maximum bending moment and required embedment. In the instrumented area, 13m long piles (900mm diameter) at spacings of 3m were found to be required. The piles were calculated as having maximum sustainable bending moment and shear of 1090kNm and 410kN respectively.

The analyses of slope stability carried by WS Atkins conform with sound practice for determining the out-of-balance (ie. the resisting) force that the piles need to ensure stability. For this reason, these calculations have not been independently carried out by TRL and reference should be made to the WS Atkins (2002b) report.

Table 3 Geotechnical design parameters (after WS Atkins Consultants Ltd, 2002b)

<table>
<thead>
<tr>
<th>Strata</th>
<th>Bulk density (kN/m$^3$)</th>
<th>$c'$ (kPa)</th>
<th>$\phi'$ (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Made ground (15.0 above water table)</td>
<td>19.0</td>
<td>0</td>
<td>35</td>
</tr>
<tr>
<td>Brown London Clay (sheared zone)*</td>
<td>18.6</td>
<td>1.5</td>
<td>19.5</td>
</tr>
<tr>
<td>Brown London Clay (long term softened strength for material below sheared zone)*</td>
<td>18.6</td>
<td>1.5</td>
<td>20</td>
</tr>
<tr>
<td>Grey London Clay</td>
<td>18.6</td>
<td>10</td>
<td>25</td>
</tr>
</tbody>
</table>

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 shown in Equation 1 below, where the constant \( A \) is equal to \( D_1(D_1/D_2)b \), with \( b = (N_0 \tan \theta + N_o^{-1}) \) and \( N_o = \tan^2(\pi/4+\theta/2) \). 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.

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:

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

In this equation \( h \) is the thickness of the yielding layer and this has been assumed to be equal to the depth at which the shear force and bending moment are being determined. The value of \( K \) is assumed to be that for the earth pressure at rest condition following the approach of Ito and Matsui (1975) and is therefore calculated as 0.67 in the normal way from \( K_0 = 1 - \sin \phi' \). It can however be argued that it may be appropriate to take account of the slope angle \( (B) \) and use the earth active value \( (K_a) \). If this is attempted then \( \beta/\phi' \) is >1 and indeterminate, although the value of \( K_a \) for \( \beta/\phi' \) is about 47% more than \( K_0 \). Pile shear loads and bending moments will then of course increase by the same percentage.

Based on these two methods, the forces and bending moments given in Table 4 were calculated using the strengths for the London Clay sheared zone given in Table 3.

A comparison of the predicted bending moments given in Table 4 with the last set of measurements taken about two years after completion of construction is shown in Figure 21a. Reasonable agreement was obtained between predicted and measured data down to about 5m depth (29mAOD), below this depth the theoretical methods are not appropriate as the piles are no longer in the sheared zone but in the unweathered London Clay. Measurements demonstrate that below 29mAOD, a reversal in the curvature of the bending moment profile occurs as the pile gains support from its fixity in the unweathered clay. It can however be concluded that in the sheared zone, both the methods of Ito and Matsui (1975) and Wang and Yen (1974) provide a reasonably close prediction of pile moments.

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

(1)

Table 4 Shear forces and bending moments per pile

<table>
<thead>
<tr>
<th>Method of</th>
<th>Force per pile per unit layer (kPa)</th>
<th>Shear (kN/pile)</th>
<th>Bending moment (kNm/pile)</th>
<th>Shear (kN/pile)</th>
<th>Bending moment (kNm/pile)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ito and Matsui (1975)</td>
<td>4.5</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Wang and Yen (1974)</td>
<td>41.5</td>
<td>20.7</td>
<td>7.6</td>
<td>7.1</td>
<td>2.0</td>
</tr>
<tr>
<td>7.6</td>
<td>7.1</td>
<td>2.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>115.7</td>
<td>162.1</td>
<td>168.2</td>
<td>173.3</td>
<td>131.7</td>
<td></td>
</tr>
<tr>
<td>152.7</td>
<td>282.9</td>
<td>387.9</td>
<td>408.2</td>
<td>411.4</td>
<td></td>
</tr>
<tr>
<td>189.8</td>
<td>437.1</td>
<td>745.1</td>
<td>795.1</td>
<td>998.8</td>
<td></td>
</tr>
<tr>
<td>226.9</td>
<td>624.6</td>
<td>1273.1</td>
<td>1371.8</td>
<td>2064.8</td>
<td></td>
</tr>
<tr>
<td>263.9</td>
<td>845.4</td>
<td>2005.3</td>
<td>2176.3</td>
<td>3818.3</td>
<td></td>
</tr>
<tr>
<td>301.0</td>
<td>1099.6</td>
<td>2975.1</td>
<td>3246.7</td>
<td>6506.0</td>
<td></td>
</tr>
</tbody>
</table>

1. All values are for 900mm diameter piles at 3000mm centres.
2. Strength parameters of \( c' = 1.5 \)kPa, \( \phi' = 19.5^o \) used for the London Clay (sheared zone).

7.3 Using retaining wall methods

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 considered to be that of the A12 carriageway level, ie. a retained height of wall of 3.6m.

Soil strengths of \( c' = 1.5 \)kPa and \( \phi' = 20^o \) were assumed for the brown clay to a depth of 9m below the crest of the slope and strength parameters of \( c' = 10 \)kPa and \( \phi' = 25^o \) for the grey clay below this. The extra perturbing load from the clay slope above the pile heads was accounted for in two alternative ways:

a) By increasing the active pressure coefficient in the upper 3.6m layer of the retained ground to a value of 1. This is the value for a clay with \( \phi' \) of 20° and an inclined slope angle of 20° as given in Eurocode 7 (1994).

b) The clay slope with a height of about 2.7m above the tops of the piles was represented by a uniform surcharge on the retained side of the piled wall of 25kPa. This was about half that which would exist further back from the slope.

In both cases, 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 1.2m soil to the ground in front of the wall: this corresponded to about one third of the berm height (Fleming et al., 1991).

The measured bending moments and those predicted using retaining wall theory (BS8002, 1994) are compared in Figure 21b. In calculating the bending moment per pile allowance was made for the spacing of 3m between pile centres.

Both methods (a) and (b) gave bending moment profiles which were similar to those measured and also in reasonable agreement to those determined from the methods of Ito and Matsui (1975) and Wang and Yen (1974). Peak bending moments in the piles of 952 and 1233kNm were calculated using method (a) and (b) respectively.

7.4 Other design issues
The small negative bending moments which were measured in both piles at about 32mAOD 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 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 $3K_p \sigma_v'd$ where $d$ is the pile diameter, whereas Fleming et al. (1991) suggested that the limiting pressure was better given by $K_p \sigma_v'd$ at depths up to 1.5 diameters and by $K_p^2 \sigma_v'd$ 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.

8 Conclusions
Following extensive TRL research on the topic, a single row of bored piles spaced at intervals was selected by the HA Maintaining Agent for the remediation of a failed cutting slope on the A12 Colchester Bypass. This London Clay slope had a history of failures and been previously reinstated, but failed again by deep-seated slippage in the underlying clay. The most recent slip occurred in December 1997. A more permanent structural solution was therefore sought by using the piling technique to stabilise the slope.

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 observations from the monitoring carried out both during construction and over the first two years in service, and the implications of the findings on future design procedures were as follows.

i At this scheme, access for the piling rig was obtained by cutting a temporary bench into the slope to provide a firm platform. A number of other benches were also excavated in the slope to facilitate movement of associated construction plant. 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 down-slope by about 7mm with an associated increase in the maximum pile bending moment to 647kNm. 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.

ii By completion of reinstatement of the slope, a significant downslope movement of 324mm was recorded at the top of the inclinometer tube located 10.6m up from the pile line and near to the crest of the slope. This large movement was mainly caused by compaction plant operating near to the inclinometer tube and was not viewed as geotechnically significant in terms of slope stability.

iii 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 4mm 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 893kNm 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.

iv Lateral movements near to the top of the slope continued with 35mm of movement being measured during the first few months after completion of construction. However these movements were considered to be related to the aftermath of the construction activities rather than the in-service performance. Subsequent observations have only recorded movements of ±3mm consistent with the small movements associated with seasonal swell/shrink of the clay.

v Pore water pressure variations were observed and compared with rainfall data. Generally the only significant seasonal changes in pore water pressure were recorded on the shallowest piezometers at depths of about 2.5m below the slope surface. The piezometer at the top of the slope showed some response to prolonged periods of heavy rain possibly related to temporary build-ups of water in the cut-off drain at the top of the slope. The free-draining nature of the Dunkirk slag used for backfilling the temporary benches in the slope, together with the cut-off drain, controlled the apparent water table to about 2 to 3m below the slope surface. There was also evidence of a perched water table in the weathered clay of the slope.

vi The approach used by WS Atkins in carrying out analyses of slope stability conformed with sound practice for determining the out-of-balance (ie. the resisting) force that the piles need to ensure stability. Comparison of measured and predicted bending moments in the piles carried out by TRL confirmed that the methods of Ito and Matsui (1975) and Wang and Yen (1974) gave a reasonable estimate of moments above the shear plane. Design checks using modified retaining wall methods also proved useful.

vii Over the two year period of measurement, the single row of piles was considered to have provided very effective support to the clay slope and it is anticipated that a near permanent solution has been achieved to improve the slope stability.

viii On the basis of this case history study and an earlier study on the M25 (Carder and Barker, 2005), it is recommended that the technique of using a single row of spaced piles to improve the stability of clay slopes be more widely implemented.

9 Acknowledgements

The work described in this report forms part of the research programme of the Structures Group at TRL and was funded by Safety, Standards and Research (Operations Support Division) of Highways Agency. The HA Project Sponsor for the research study was Mr R K W Lung.

WS Atkins Consultants Ltd carried out the design of the remedial works at this site. The Main Contractor for the scheme was Edmund Nuttall Ltd and the piling works was subcontracted to Cementation Foundation Skanska Ltd. Particular thanks are due to Mr T Wright, Mr C Mobbs (WSA), Mr T Doyle (EN), and Mr M Harrington (CFS) for their co-operation during this research study.

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10 References


Abstract

Following extensive TRL research on the topic, a single row of bored piles spaced at intervals was selected by the HA Maintaining Agent for the remediation of a failed cutting slope on the A12 Colchester Bypass. This London Clay slope has had a history of failures and been previously reinstated, but failed again by deep-seated slippage in the underlying clay. A more permanent structural solution was therefore sought by using the piling technique to stabilise the slope.

The performance of the slope and piles was monitored both during construction and over the first two years in service. Instrumentation was installed to monitor the subsurface lateral movements of the ground and piles, the distribution of bending moment in the piles, and the pore water pressure regime in the remediated slope. This report describes the instrumentation and interprets the monitoring data. The implications of the findings are discussed with a view of improving the design guidance for future construction of a similar type.

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- 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)
- 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)
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