



# **Case history studies of soil berms used as temporary support to embedded retaining walls**

**Prepared for Quality Services (Civil Engineering), Department of the Environment, Transport and the Regions**

M R Easton and P Darley

First Published 1999  
ISSN 0968-4107  
Copyright Transport Research Laboratory 1999.

This report has been produced by the Transport Research Laboratory, under/as part of a Contract placed by the Department of the Environment, Transport and the Regions. Any views expressed are not necessarily those of the Department.

TRL is committed to optimising energy efficiency, reducing waste and promoting recycling and re-use. In support of these environmental goals, this report has been printed on recycled paper, comprising 100% post-consumer waste, manufactured using a TCF (totally chlorine free) process.

**Transport Research Foundation Group of Companies**

Transport Research Foundation (a company limited by guarantee) trading as Transport Research Laboratory. Registered in England, Number 3011746.

TRL Limited. Registered in England, Number 3142272.

Registered Offices: Old Wokingham Road, Crowthorne, Berkshire, RG45 6AU.

# CONTENTS

|  | Page |
|--|------|
| <b>Executive Summary</b>   | 1    |
| <b>1 Introduction</b>  | 3    |
| <b>2 Basis of back-analyses</b>                                  | 3    |
| <b>3 A4/A46 Batheaston–Swainswick Bypass</b>                     | 3    |
| 3.1 Description of the retaining walls                           | 3    |
| 3.2 Soil parameters  | 7    |
| 3.3 Measured performance   | 7    |
| 3.4 Comparison of measured movements and back-analyses           | 8    |
| <b>4 A406 East of Fallosen Way</b>                               | 8    |
| 4.1 Construction of the retaining wall                           | 8    |
| 4.2 Soil conditions  | 17   |
| 4.3 Measured performance   | 17   |
| 4.4 Comparison of measured movements and back-analyses           | 17   |
| <b>5 A40 Long Lane improvement</b>                               | 17   |
| 5.1 Construction of the retaining wall                           | 17   |
| 5.2 Soil properties  | 26   |
| 5.3 Measured performance   | 26   |
| 5.4 Comparison between measured performance<br>and back-analyses | 26   |
| <b>6 A50 Blythe Bridge to Queensway</b>                          | 26   |
| 6.1 Construction of the retaining wall                           | 26   |
| 6.2 Ground conditions  | 32   |
| 6.3 Measured performance   | 32   |
| 6.4 Comparison of measured movements and back-analyses           | 32   |
| <b>7 Discussion</b>  | 36   |
| <b>8 Conclusions</b>   | 37   |
| <b>9 Acknowledgements</b>  | 37   |
| <b>10 References</b>   | 37   |

|   |    |
|---|----|
| <b>Appendix A: Finite element analysis of A4/A46 Batheaston – Swainswick Bypass</b> | 38 |
| A1 Details of mesh  | 38 |
| A2 Material properties  | 38 |
| A2.1 Soil parameters  | 38 |
| A2.2 Structural components  | 40 |
| A3 <i>In situ</i> ground stresses and pore water pressures                          | 40 |
| A4 Construction sequence  | 40 |
| A4.1 Berms only   | 40 |
| A4.2 Horizontally propped wall  | 40 |
| <b>Appendix B: Finite element analyses of A406 East of Falloden Way</b>             | 40 |
| B1 Details of mesh  | 40 |
| B2 Material properties  | 40 |
| B2.1 Soil Parameters  | 40 |
| B2.2 Structural components  | 42 |
| B3 <i>In situ</i> ground stresses and pore water pressures                          | 42 |
| B4 Construction sequence  | 42 |
| B4.1 Berms only   | 42 |
| B4.2 Simulated raked props  | 42 |
| <b>Appendix C: Finite element analyses of A40 Long Lane improvement</b>             | 42 |
| C1 Details of mesh  | 42 |
| C2 Material properties  | 44 |
| C2.1 Soil parameters  | 44 |
| C2.2 Structural components  | 44 |
| C3 <i>In situ</i> ground stresses and pore water pressures                          | 44 |
| C4 Construction sequence  | 44 |
| C4.1 Raked props and berms  | 44 |
| C4.2 Berms only   | 45 |
| C4.3 Horizontally propped wall  | 45 |
| <b>Appendix D: Finite element analyses of A50 Blythe Bridge to Queensway</b>        | 45 |
| D1 Details of mesh  | 45 |
| D2 Material properties  | 45 |
| D2.1 Soil/rock parameters   | 45 |
| D2.2 Structural components  | 46 |
| D3 <i>In situ</i> ground stresses and pore water pressures                          | 47 |
| D4 Construction sequence  | 47 |
| D4.1 Berms only   | 47 |
| D4.1 Temporary horizontal props   | 47 |
| <b>Abstract</b>   | 50 |
| <b>Related publications</b>   | 50 |

## Executive Summary

---

Finite element modelling to investigate the effectiveness during the construction period of both soil berms and raked props, as temporary support systems for embedded retaining walls permanently propped at carriageway level, has been previously reported (Carder and Bennett, 1996). The report suggested that for walls founded in stiff over-consolidated clay, these methods of support were both feasible, and cost effective alternatives to temporary bracing over the entire carriageway width using steel props.

This study reviews the performance of the temporary support system at four completed retaining wall schemes. Soil berms were employed at three of these schemes, namely the A4/A46 Batheaston-Swainswick Bypass, the A406 East of Falloden Way, and the A50 Blythe Bridge to Queensway. At the other scheme, the A40 Long Lane Improvement close to Hillingdon Station, soil berms were used in conjunction with raked steel temporary props as the method of temporary support.

Where possible data from on site construction monitoring, construction sequences, and soil parameters were obtained from the various schemes. Using this data, a series of finite element back-analyses were undertaken to ascertain the likely wall movements, which would have been anticipated at each scheme. The aim of these analyses was to provide a prediction of behaviour as near to Class A (Lambe, 1973) as was possible, ie. by using the best fit soil parameters in conjunction with the actual construction sequence. Comparison was then made between these predicted movements and the actual measured movements. Numerical predictions of the magnitudes of lateral wall movement which would have been expected if horizontal steel props had been employed as temporary support were also made. This enabled some assessment to be made of the relative effectiveness of soil berms and horizontal props as methods of temporary support.

The code of practice for earth retaining structures BS8002 recommends that the lateral movement at the top of the structure should not exceed 0.5% of the retained height. The measured values obtained from all four sites fell well below this value. In a review of ground movements caused by embedded retaining wall construction Carder (1995) found that for schemes studied by TRL maximum values of between 0.2% and 0.3% of the retained height were observed depending on the stiffness of the temporary support system. The movements measured and predicted in this report are consistent with these values.

This study indicates that the use of soil berms and/or raked temporary props is a practical alternative to employing horizontal steel props and could result in considerable reductions in the cost of embedded retaining wall construction. In cases of uncertainty these support systems should be used in conjunction with the Observational Method.



## 1 Introduction

---

Finite element modelling investigating the effectiveness during the construction period of both soil berms and raked props as temporary support systems for embedded retaining walls permanently propped at carriageway level, has been previously reported (Carder and Bennett, 1996). The report suggested that for walls founded in stiff over-consolidated clay, these methods of support were both feasible, and cost effective alternatives to temporary bracing over the entire carriageway width using steel props.

This study reviews the performance of the temporary support system at four completed retaining wall schemes. Soil berms were employed at three of these schemes, namely the A4/A46 Batheaston-Swainswick Bypass, the A406 East of Falloden Way, and the A50 Blythe Bridge to Queensway. At the other scheme, the A40 Long Lane Improvement close to Hillingdon Station, soil berms were used in conjunction with raked steel temporary props as the method of temporary support.

Where possible data from on site construction monitoring, construction sequences, and soil parameters were obtained from these schemes. Using this data, a series of finite element back-analyses were undertaken to ascertain the likely wall movements which would have been anticipated at each scheme. The aim of these analyses was to provide a prediction of behaviour as near to Class A (Lambe, 1973) as was possible, ie. by using the best fit soil parameters in conjunction with the actual construction sequence. Comparison was then made between these predicted movements and the actual measured movements.

Numerical predictions of the magnitudes of lateral wall movement which would have been expected if horizontal steel props had been employed as temporary support were also made. This enabled some assessment of the relative effectiveness of soil berms and horizontal props as methods of temporary support.

## 2 Basis of back-analyses

---

The back-analyses were carried out using the finite element package SAGE CRISP. It must be noted that two dimensional plane strain analyses were used throughout. For this reason when modelling berm performance, predicted wall movements are likely to represent upper bound values as no account is taken of the additional support from the adjacent unexcavated berm or completed permanent carriageway prop. When modelling construction employing raked and horizontal temporary props, the use of plane strain analyses is more realistic.

In constructing the finite element meshes, careful consideration was given to modelling the various soil strata, the installation and removal of structural elements, and the sequential excavation of soil berms. The soil strata were considered as being elastic perfectly plastic materials obeying the Mohr-Coulomb yield criteria.

The stiffnesses of individual structural elements, such as temporary props, were calculated taking into account the distance between their centres along the embedded walls. Where the walls were constructed from bored piles or were

T-shaped diaphragm panels, they were modelled as rectangular elements with an equivalent flexural rigidity. Concrete parameters assumed no cracking at small strains, long term strength and additional effects of steel reinforcement.

More detailed descriptions of the finite element meshes, soil parameters and construction sequences for each site are given in Appendices A to D.

## 3 A4/A46 Batheaston–Swainswick Bypass

---

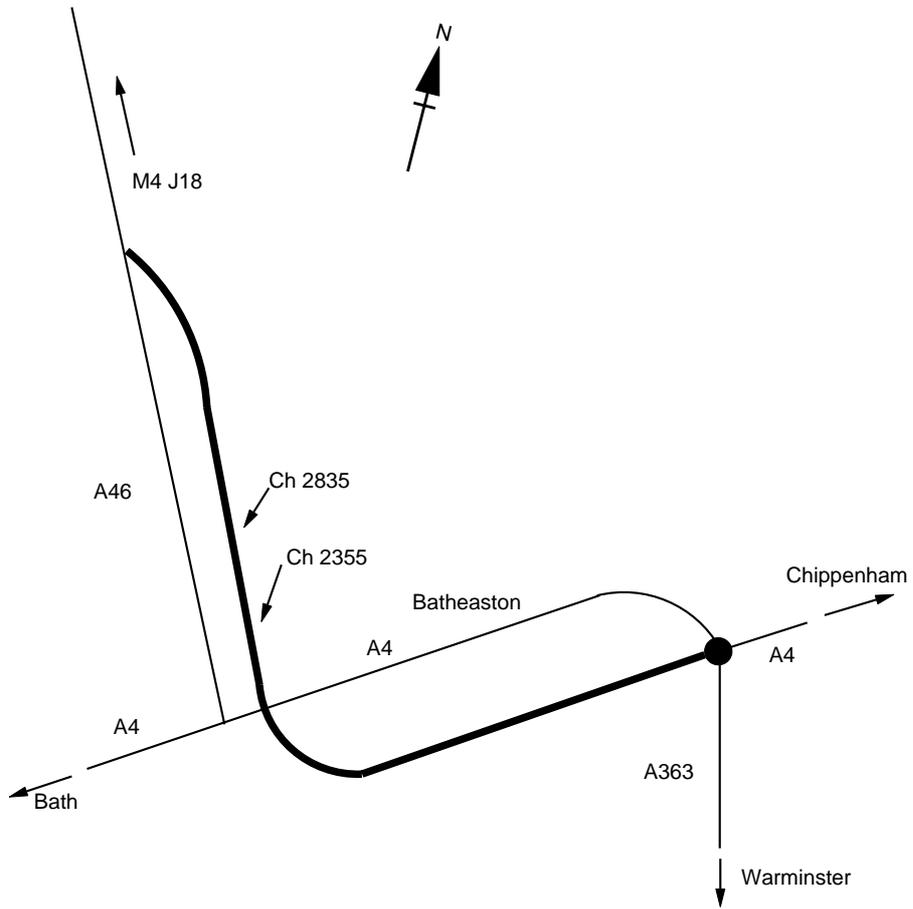
Construction of the A4/A46 Batheaston-Swainswick Bypass commenced in 1994 and the location of the site is shown in Figure 3.1. In order to minimise landtake, the bypass included a 813m long retained cutting constructed using diaphragm walling (Gosney et al, 1997). The construction sequence adopted utilised the Observational Method (Nicholson et al, 1997; Nicholson et al, 1998) employing controlled excavation of soil berms as temporary support to the wall rather than the temporary steel props originally envisaged. Data from the construction monitoring and site investigations were supplied by Ove Arup and Partners in an unpublished report to TRL, using information supplied by Amey Construction Ltd.

The measured wall movements during construction are described at chainages 2835 and 2355, Figure 3.1, although comparisons with finite element back-analyses were undertaken at chainage 2835 only.

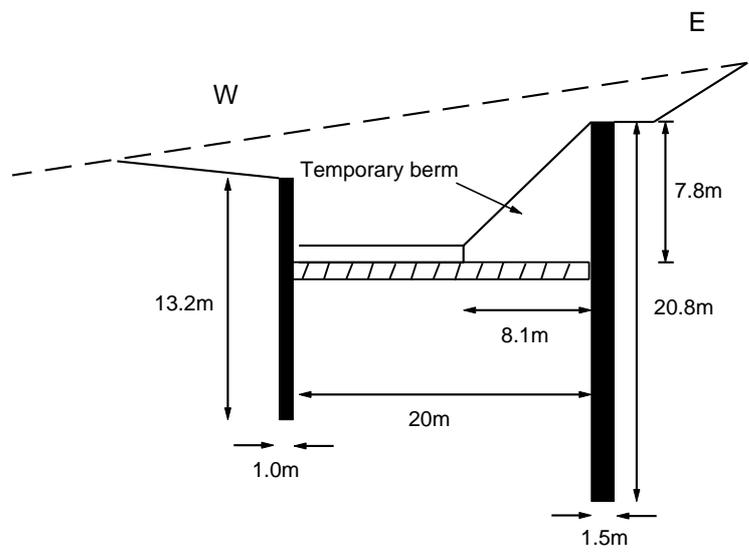
### 3.1 Description of the retaining walls

The diaphragm wall panels forming the east wall at chainage 2835 were 20.8m deep, 4.5m wide and 1.5m thick: those for the west wall were 11.7m deep, 4.5m wide and 1.0m thick. A cast *in situ* reinforced concrete capping beam was constructed on top of the wall panels. The upper level of the full width permanent reinforced concrete props originally proposed below the carriageway were to have been 6.3m below the top of the diaphragm panel H5. At Chainage 2800 the formation level of the permanent prop slab replacing the discrete props was constructed some 7.8m and 4.7m below the top of the capping beams on the east and west walls respectively. Cross-sections through the structure showing the alternative berm excavation and permanent strut design at chainages 2800 and 2850 are shown in Figure 3.2. The construction sequence at chainage 2800 is given in Figure 3.3. Two inclinometer tubes (IR15 and IR16) were cast at chainage 2835 into panels H5 (east wall) and G1 (west wall) respectively. Lateral movements of the capping beam were also obtained by standard surveying techniques.

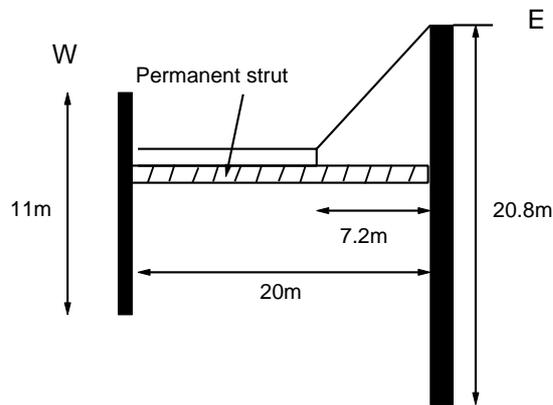
At chainage 2355 the diaphragm panels forming the east wall were 19.1m deep, 4.5m wide and 1.2m thick: those for the west wall were 21.8m deep, 4.5m wide and 1.2m thick. A cast *in situ* reinforced concrete capping beam was constructed on top of the diaphragm wall panels. Cross-sections through the structure at chainages 2315 and 2380 either side of the chosen chainage are given in Figure 3.4. As above, lateral movements of the capping beam were obtained by standard surveying techniques and from two inclinometer tubes (IR5 and IR6) cast into panels C22 (east wall) and D9 (west wall) respectively.



**Figure 3.1** Location of the site



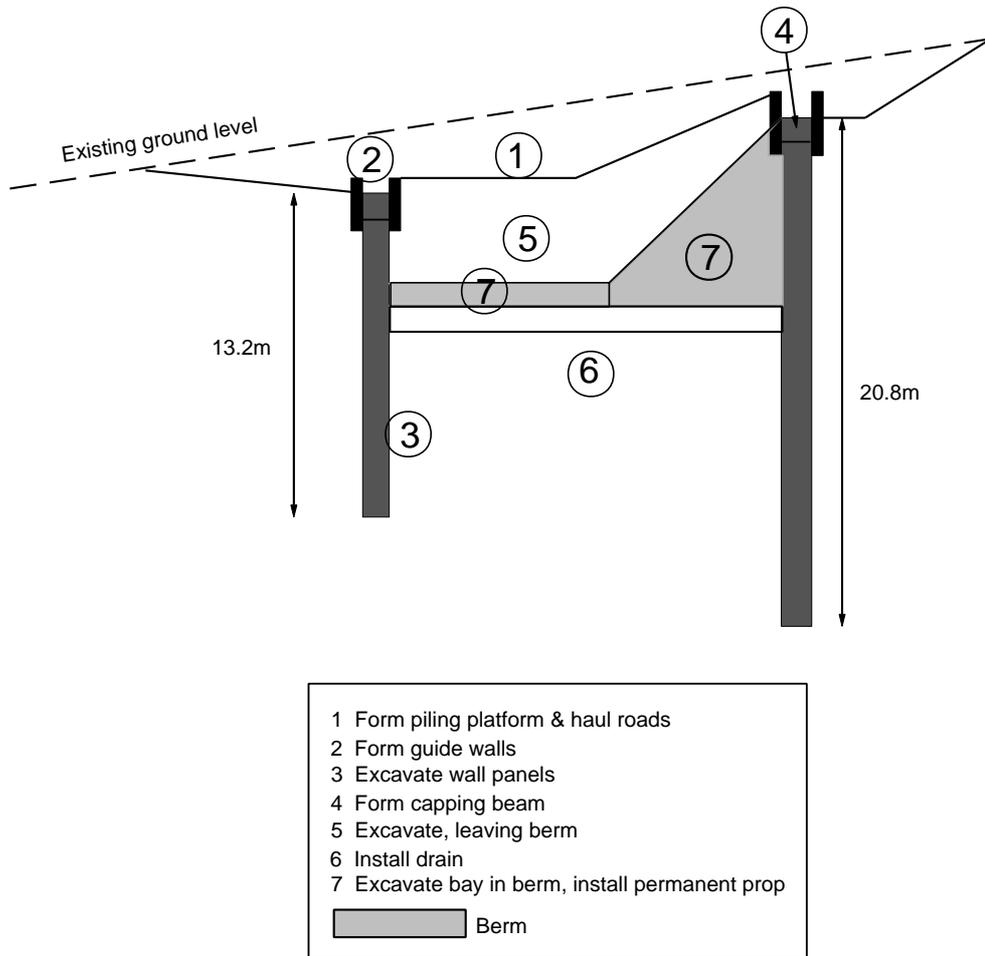
Chainage 2800 Design section



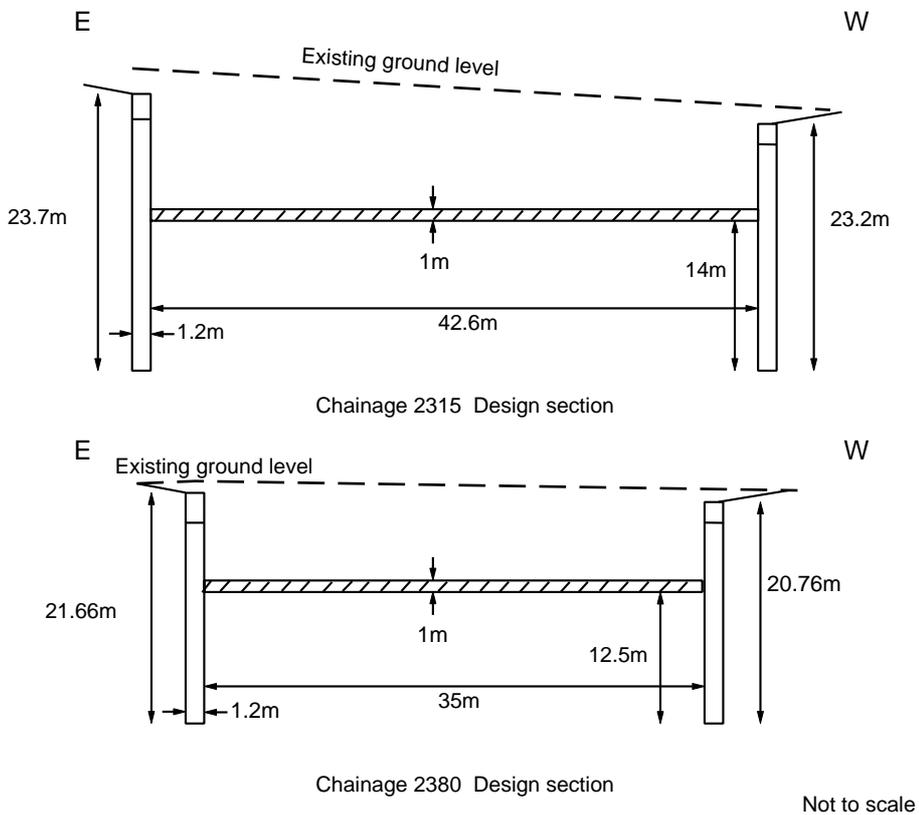
Chainage 2850 Design section

Not to scale

**Figure 3.2** Cross section at chainages 2800 and 2850



**Figure 3.3** Construction sequence at chainage 2800



**Figure 3.4** Cross sections at chainages 2315 and 2380

### 3.2 Soil parameters

At chainage 2835 the walls were founded in Lias Clay overlain by Midford sands, these strata are identified in the log from the site investigation borehole SB24 shown in Figure 3.5. (SB24 was situated approximately 15m east of the centreline at this chainage). The Midford sands comprised of fairly uniform medium dense yellow sandy and coarse silts, with some blocks of moderately strong to strong sandstone randomly distributed throughout the sands. The Lias Clay consisted mainly of grey silty micaceous clays with occasional clayey silts and thin beds of clayey limestone.

At chainage 2355 the soil comprised Lias Clay to the full height of both east and west walls. Here, the Lias Clay was described as fresh, fissured, medium grey, silty clay. The borehole log for borehole SB9 is shown in Figure 3.6. The soil parameters employed in the design of the retaining walls and stated in the Approval in Principle (AIP) documentation (Gibb, 1993) are given in Table 3.1.

Further detailed assessment of these parameters was undertaken to allow safe construction using the Observational Method. Structural (serviceability) design was based on 'most unfavourable' soil parameters and ultimate limit state was checked using both 'most probable' and 'most unfavourable' conditions with appropriate factors of safety on soil strength. This procedure and the soil parameters adopted are discussed in more detail by Nicholson et al, 1998.

Analyses of performance were also undertaken by Gourvenec et al (1996) who used a modulus for Midford Sand of  $E'=10+10z$  and Lias Clay of  $E'=12+12z$  (MN/m<sup>2</sup>), where  $z$  is the depth below ground level. In later analyses Gourvenec (1998) used a constant value of 12.24MN/m<sup>2</sup> for the Midford Sand and  $E'=80+7.2z$  MN/m<sup>2</sup> for the Lias Clay.

### 3.3 Measured performance

Monitoring of wall movements was required under the Observational Method and field data from two locations (chainages 2835 and 2355) were selected for the purposes of this study. At the first location (chainage 2835), the soil berms were excavated in 5m long bays. At chainage 2355 excavation was carried out over an entire 30m length as progressive excavation in shorter bays was found unnecessary. It is also worth noting that at chainage 2835 the original ground was sloping and the eastern wall was therefore significantly higher than the western wall of the retained cutting.

The lateral movements at the top of the retaining wall measured by inclinometer and by surveying at several stages of construction are summarised in Tables 3.2 and 3.3 for chainages 2835 and 2355 respectively. Typical lateral

**Table 3.2 Measured lateral movement of walls (Ch 2835)**

| Date    | East wall cumulative movement (mm) |           | West wall cumulative movement (mm) |           | Remarks                     |
|---------|------------------------------------|-----------|------------------------------------|-----------|-----------------------------|
|         | Inclinometer                       | Surveying | Inclinometer                       | Surveying |                             |
| 11.5.95 | 0                                  |           | 0                                  |           | Datum                       |
| 13.5.95 |                                    |           | -1.5                               |           |                             |
| 20.5.95 | 3                                  |           | -0.5                               |           |                             |
| 22.5.95 |                                    |           |                                    |           | Excavate to leave berm      |
| 24.5.95 | 5                                  | 2         | 3.5                                | 7         |                             |
| 25.5.95 |                                    |           |                                    |           |                             |
| 26.5.95 | 6                                  |           | 5.5                                |           |                             |
| 1.6.95  | 8.5                                |           | 5.5                                |           |                             |
| 5.6.65  | 8.5                                |           | 8.5                                |           |                             |
| 6.6.95  | 12.5                               | 12        |                                    |           | Excavation of berm complete |
| 7.6.95  | 12.5                               |           | 8.5                                | 10        |                             |
| 8.6.95  | 12.5                               |           | 9.5                                |           |                             |
| 10.6.95 | 13.5                               |           | 9.5                                |           |                             |
| 12.6.95 |                                    |           |                                    |           | Strut complete              |
| 13.6.95 | 13                                 |           | 9.5                                | 14        |                             |
| 14.6.95 | 16                                 | 14        | 9.5                                |           |                             |
| 23.6.95 |                                    |           | 9.5                                |           |                             |

*Movement towards the excavation is positive.*

movement profiles with depth obtained from inclinometer surveys assuming base fixity of the tubes are given in Figure 3.7. A comparison of the movements measured by inclinometer surveys and standard surveying techniques for both chainages are given in Figures 3.8 and 3.9.

The results for panel H5 in the east wall show good agreement between the inclinometer data and that obtained by surveying of the capping beam. This indicated only minor movement of the toe and overall movement at the top of the wall of about 15mm. The results for the shallower west wall at chainage 2835 showed smaller movements but poorer agreement between the two measurement techniques.

Figure 3.9 shows the results for both walls at chainage 2355, and it is noted that inexplicably large wall movements of the west wall of up to 35mm were recorded. The reason for the relatively large movements of the west wall has never been explained. In practice the movements exceeded the red trigger limit, and part of the retained soil was excavated from behind the wall and placed within the excavation forming a small berm. Small differences

**Table 3.1 Design parameters stated in the AIP**

| Strata              | $K_o$                | $c_u$ (kPa)         | $\phi'$ (deg) | $G$ (MPa)   | $E_u/c_u$ |
|---------------------|----------------------|---------------------|---------------|-------------|-----------|
| Midford sand        | 0.5                  | -                   | 32            | $4+4z$      | -         |
| Weathered Lias Clay | 2.4                  | $50+8.2z$           | 26            | $8.3+1.37z$ | 500       |
| Intact Lias Clay    | 2.6 reducing to 1.75 | $50+8.2z$ (max 214) | 27            | $8.3+1.3z$  | 500       |

$z$  = depth below ground level

**Table 3.3 Measured lateral movement of walls (Ch 2355)**

| Date     | East wall cumulative movement (mm) |           | West wall cumulative movement (mm) |           | Remarks                     |
|----------|------------------------------------|-----------|------------------------------------|-----------|-----------------------------|
|          | Inclinometer                       | Surveying | Inclinometer                       | Surveying |                             |
| 20.6.95  | 0                                  |           | 0                                  |           | Datum                       |
| 22.6.95  | 2                                  |           |                                    |           |                             |
| 23.6.95  | 4                                  | 4         | 2                                  |           |                             |
| 27.6.95  | 5                                  |           | 2                                  |           |                             |
| 30.6.95  | 5                                  |           | 3                                  | 3         |                             |
| 03.7.95  | 4                                  |           | 5.5                                |           |                             |
| 6.7.95   |                                    | 11        |                                    |           | Excavate to leave berm      |
| 11.7.95  |                                    |           |                                    |           | Excavation of berm complete |
| 14.7.95  |                                    |           | 19                                 | 18        |                             |
| 17.7.95  | 10                                 |           | 22                                 |           |                             |
| 19.7.95  | 12                                 |           | 22                                 |           |                             |
| 20.7.95  | 10                                 | 15        | 22                                 | 29        | Strut complete              |
| 26.7.95  | 12                                 |           | 28                                 |           |                             |
| 01.8.95  | 14                                 |           | 27                                 |           |                             |
| 15.8.95  | 14                                 |           | 27                                 |           |                             |
| 29.9.95  |                                    |           | 30                                 |           |                             |
| 10.11.95 | 30                                 |           | 35                                 |           |                             |

Note: Movement towards the excavation is positive.

between the two measuring techniques of the east wall, chainage 2355m, suggest possible movement at the toe of the wall.

### 3.4 Comparison of measured movements and back-analyses

The predicted horizontal movements of the retaining wall from a finite element consolidation analysis using the original design parameters are shown in Figure 3.10. The predicted movements are compared with the measured lateral movement of the top of the wall for the actual construction sequence using berms in Figure 3.10a. In this case, predicted values were approximately three times higher than were measured for two reasons. Firstly the original soil parameters were worst case to ensure safe design and construction and secondly the analysis was carried out in plane strain whereas berm excavation actually took place in 5m long bays. The predicted movements in Figure 3.10a are therefore expected to form an upper bound.

Figure 3.11a shows results from a similar analysis but employing best estimated soil parameters as reported by Gourvenec (1998). In this case the measured and predicted movements were in good agreement with values of about 15mm. The analysis predicted little, if any, movement of the toe of the wall. Measured performance during construction validated this prediction as good correlation existed between movements obtained by surveying and inclinometer studies, suggesting base fixity of the wall panel (Table 3.2).

Figures 3.10b and 3.11b show the lateral movements predicted with both sets of soil parameters if horizontal props had been employed as temporary support rather than using soil berms. In both cases the horizontal props were effective at controlling top of wall movement during construction, although movement occurred on their release. The range of predicted movements of the top of the wall is large for the different soil parameters; from 15mm to 75mm in the case where soil berms were modelled, and 8mm to 25mm when props were employed.

For best estimated soil parameters, predicted movements of the top of the wall when using soil berms and horizontal props (ie. 15mm and 8mm respectively) were well within the movement limit of 0.5% of the retained height suggested in BS8002 (British Standards Institution, 1994). The use of soil berms in conjunction with the Observational Method which were adopted at this site therefore provided an economic form of construction.

Further details of the finite element analyses are given in Appendix A.

## 4 A406 East of Falloden Way

The scheme forms part of the upgrading of the A406 North Circular Road and runs from east of Falloden Way to Finchley High Road in north London. For part of its length the road runs in cut-and-cover tunnel, taking the A406 underneath East End Road, near St Marylebone Cemetery. Unpublished data from the construction monitoring and site investigations were supplied to TRL by Edmund Nuttall Ltd.

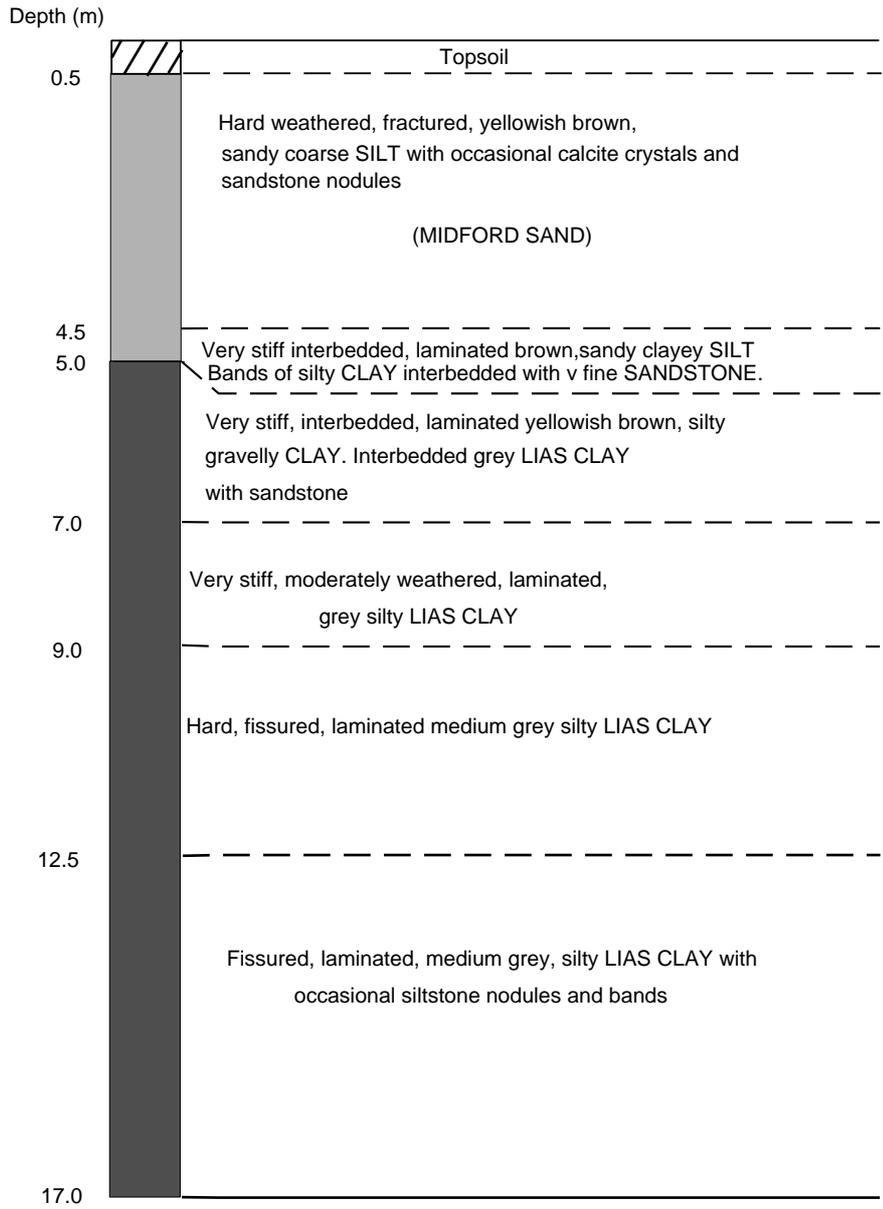
Close to the western end of the tunnel, ramps and stairs were constructed to provide pedestrian access to bus stops on both sides of the A406. At this section the retaining walls were formed from 'T' shaped diaphragm panels. A location plan of the site is given in Figure 4.1. The dimensions of the structure and the ground conditions are summarised in Figure 4.2.

### 4.1 Construction of the retaining wall

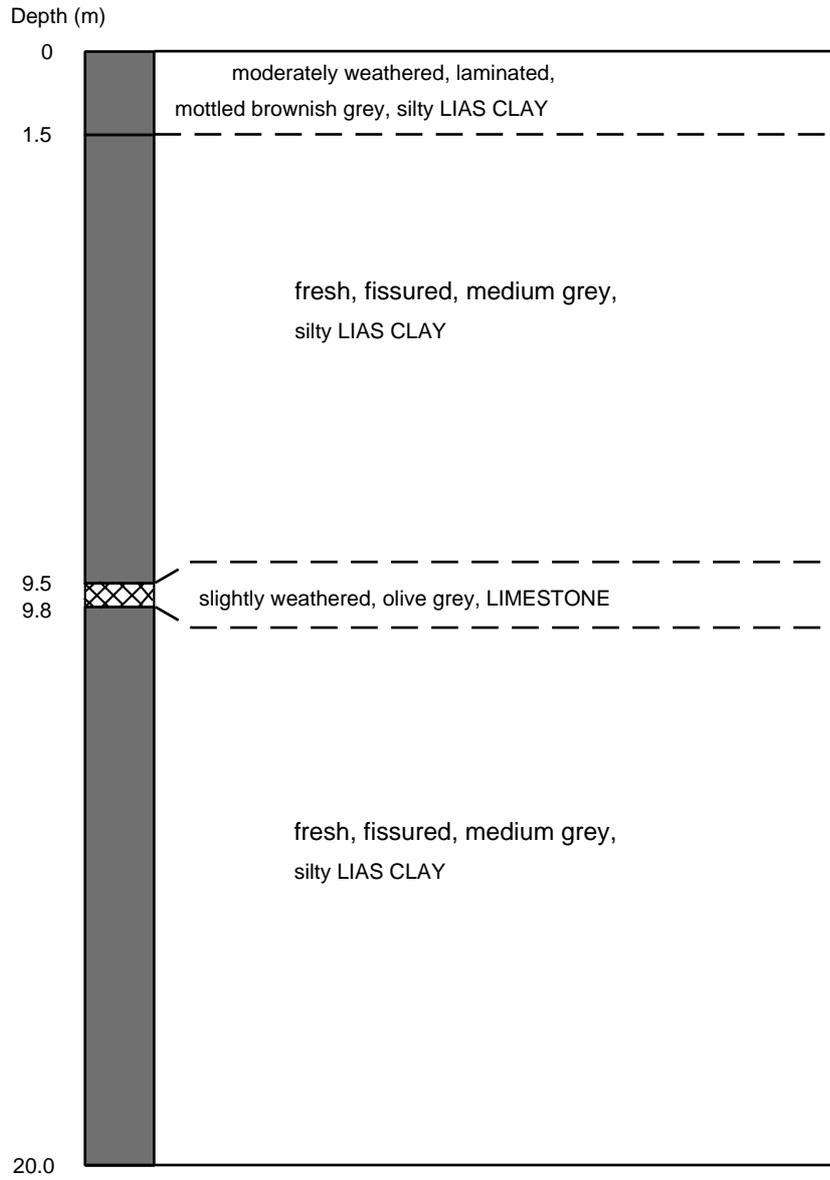
Wall movements obtained during excavation of the ground in front of the south wall indicated that instead of the temporary steel props envisaged in the original design, soil berms should be used as temporary support to the north wall during excavation for, and construction of, the permanent prop slab below the carriageway.

The north wall was constructed from 'T' shaped reinforced concrete diaphragm wall panels which were 4m wide and 1m thick with an additional 2.5m counterfort as shown in Figure 4.2a. The depth of the panel at chainage 630 which was selected for this study was 26.3m.

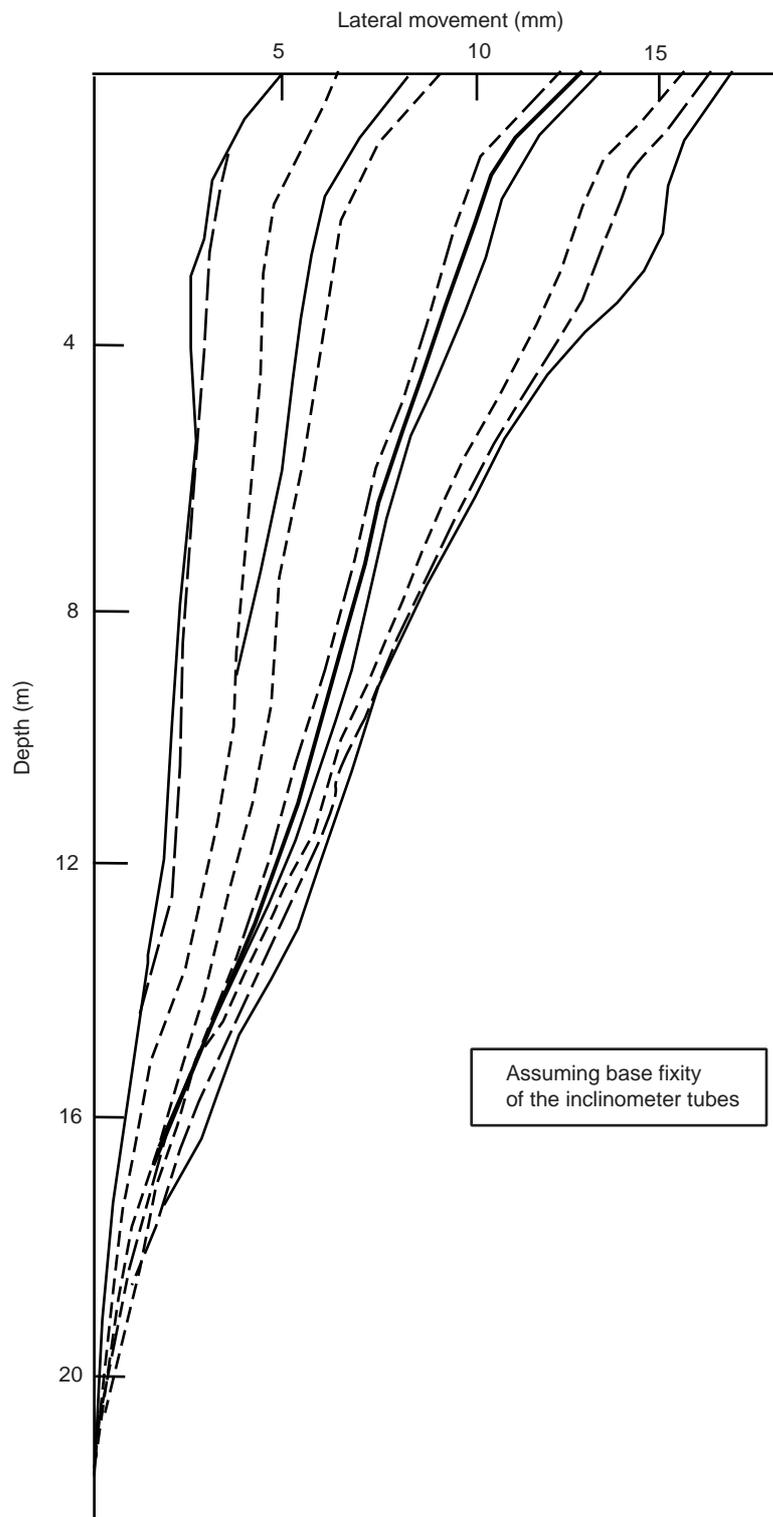
A reinforced concrete slab constructed beneath carriageway level as permanent prop. This prop incorporated thirty two concrete tension piles of 900mm diameter which restrained the vertical heave of the clay below carriageway level (Figure 4.2b).



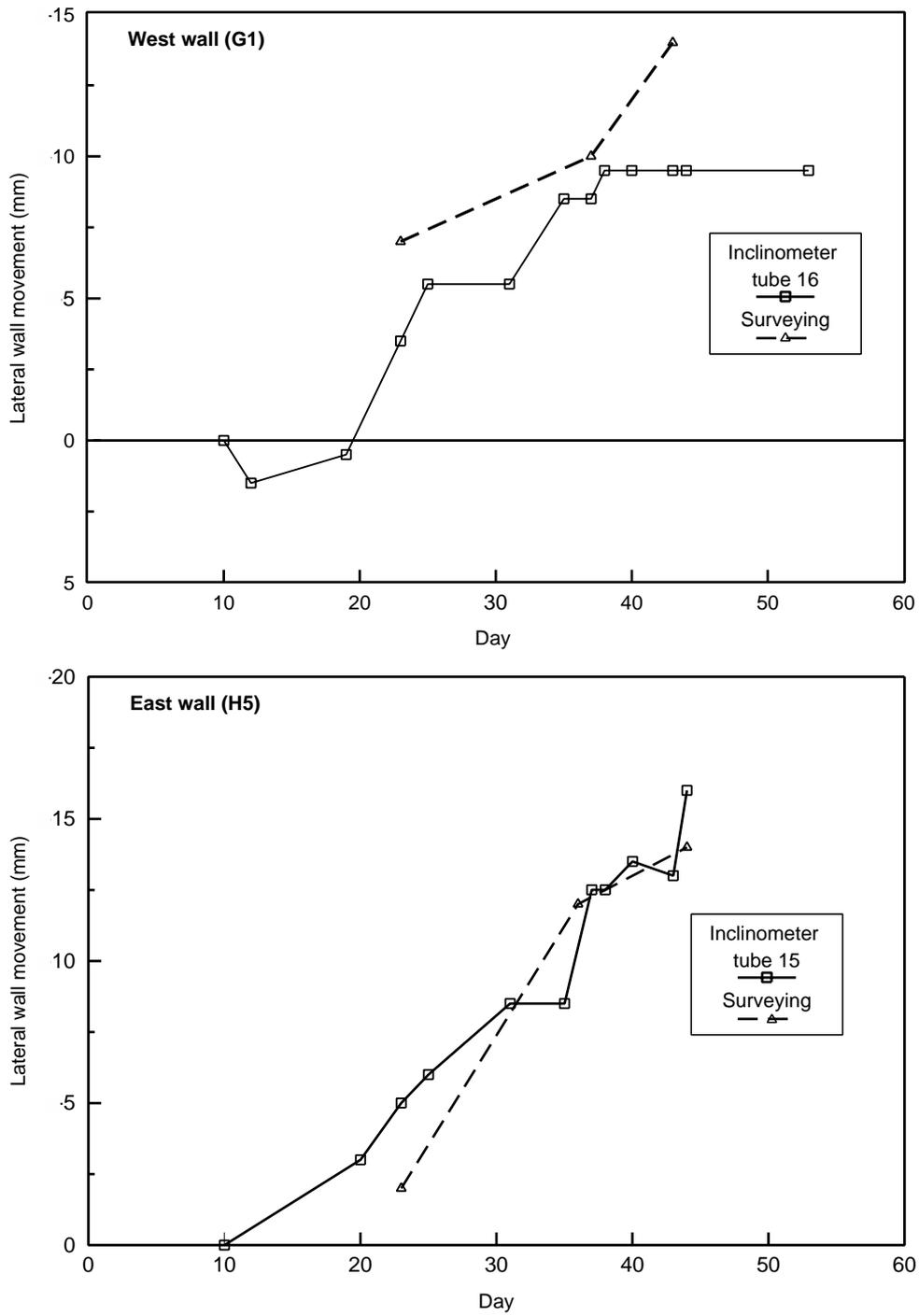
**Figure 3.5** Summary of borehole log SB24



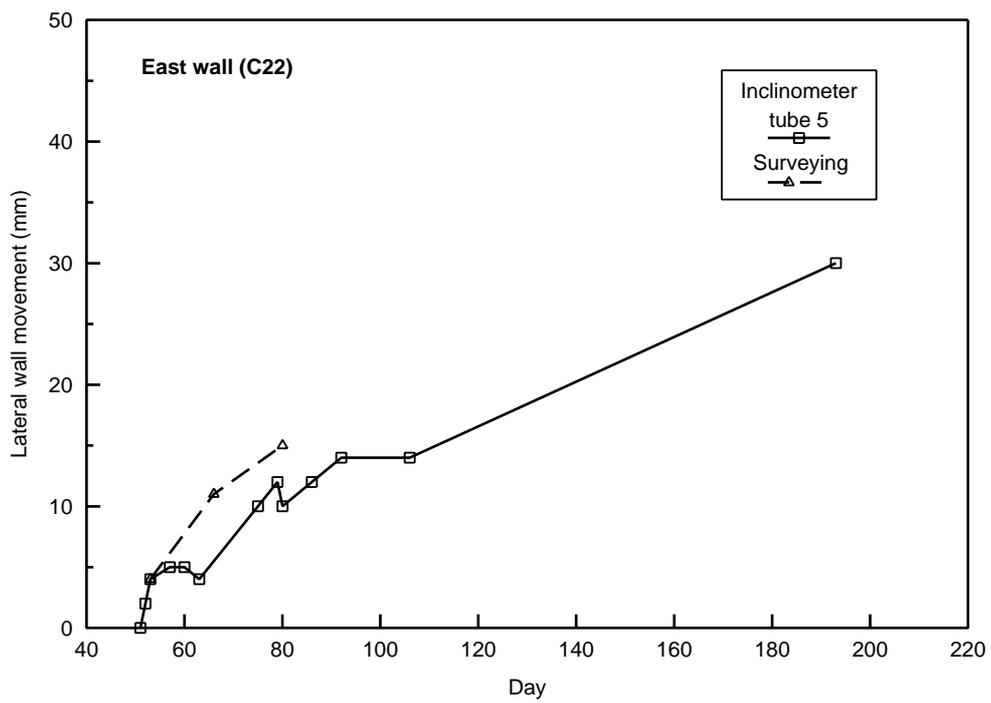
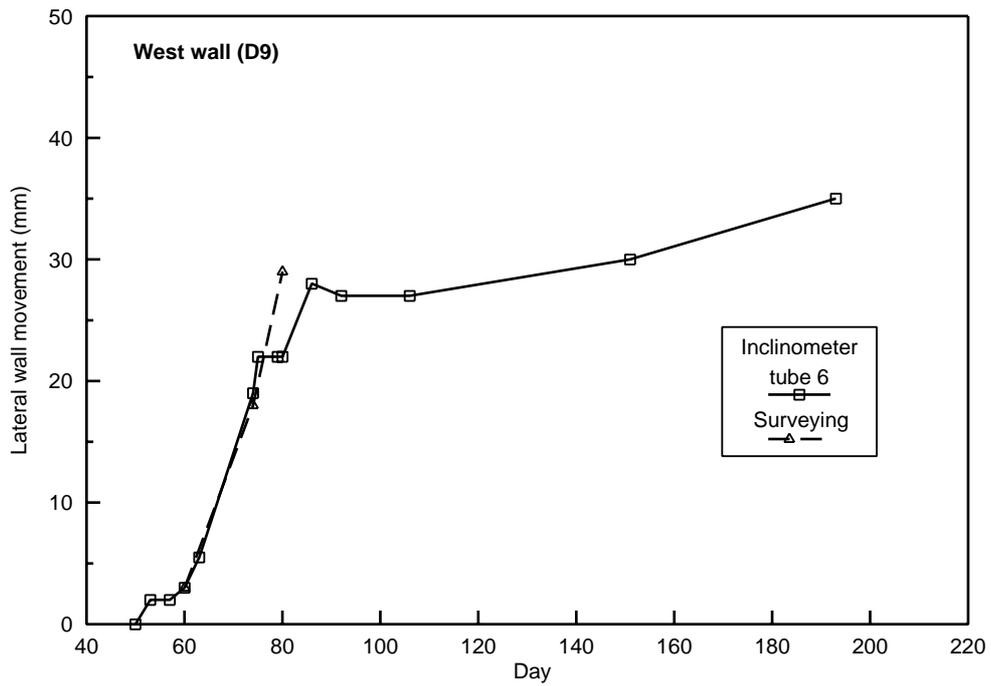
**Figure 3.6** Borehole log SB9



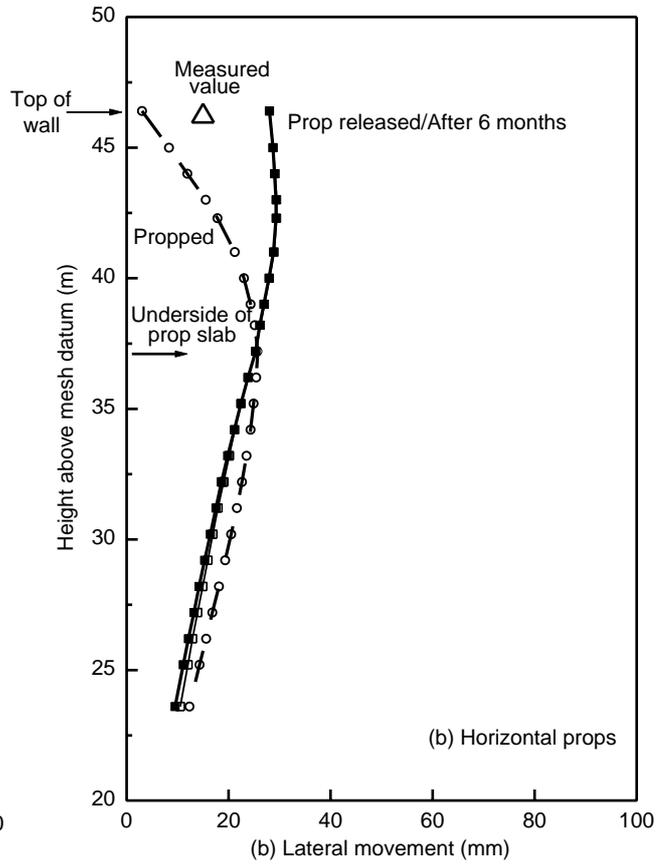
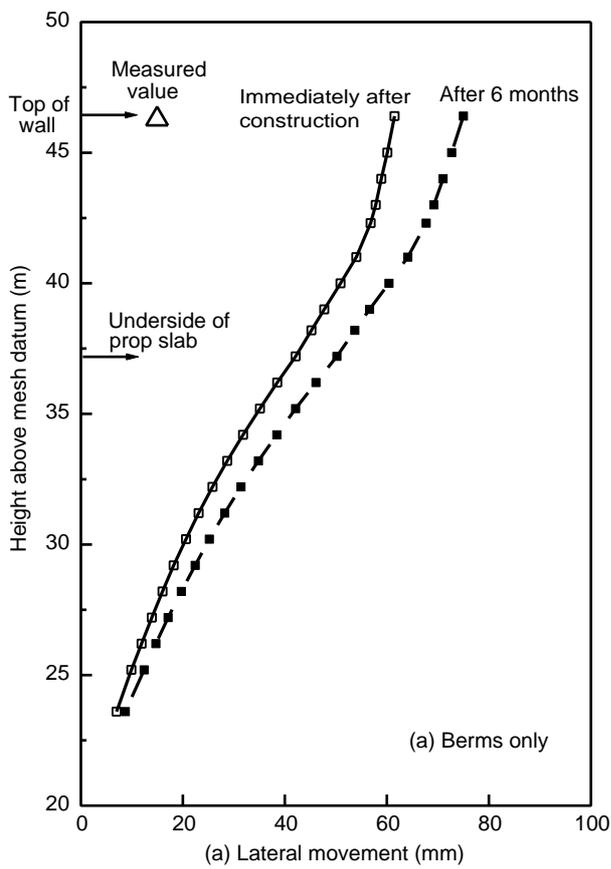
**Figure 3.7** Typical lateral movement profiles



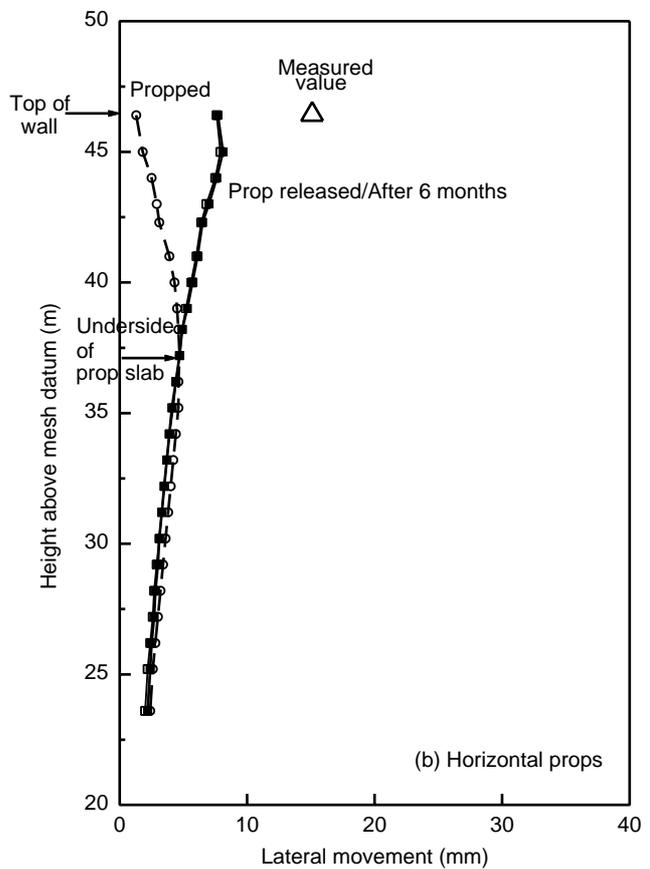
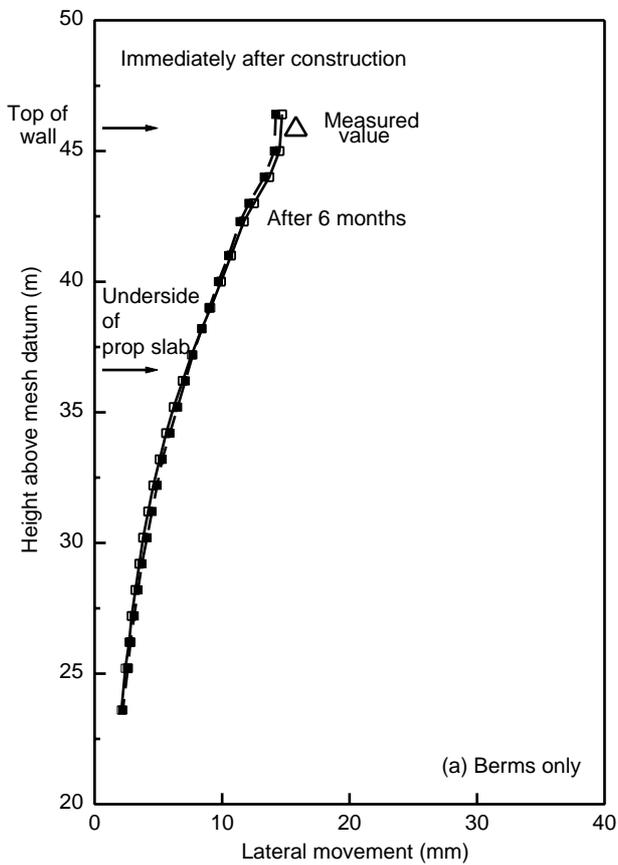
**Figure 3.8** Lateral wall movements at capping beam for change 2835



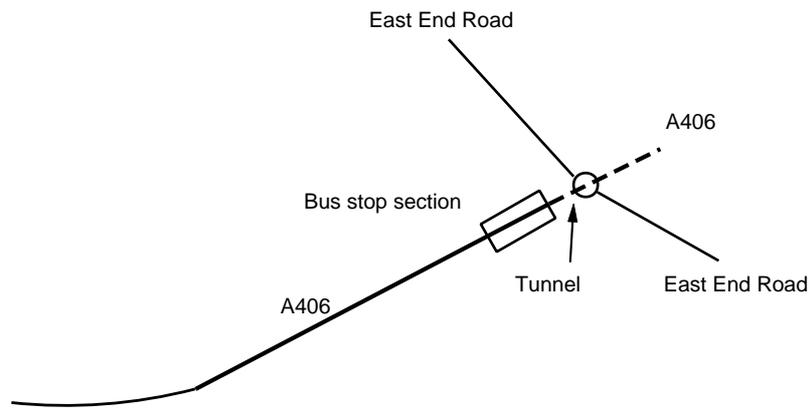
**Figure 3.9** Lateral wall movements at capping beam for chainage 2355



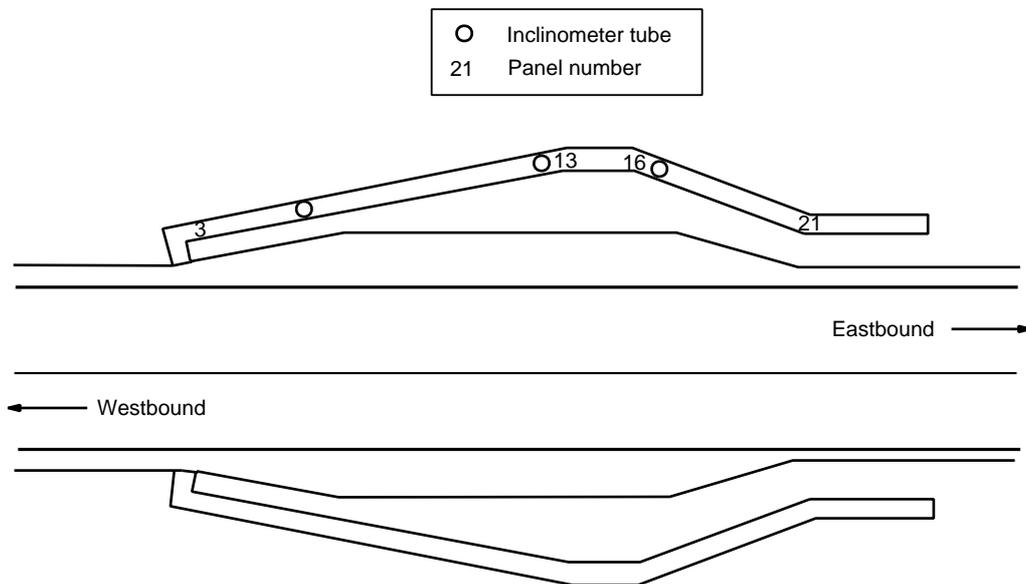
**Figure 3.10** Predicted horizontal movement of retaining wall using original soil parameters



**Figure 3.11** Horizontal movement of retaining wall using best estimated soil parameters



**Figure 4.1a** Location of site



**Figure 4.1b** Plan of bus stop section

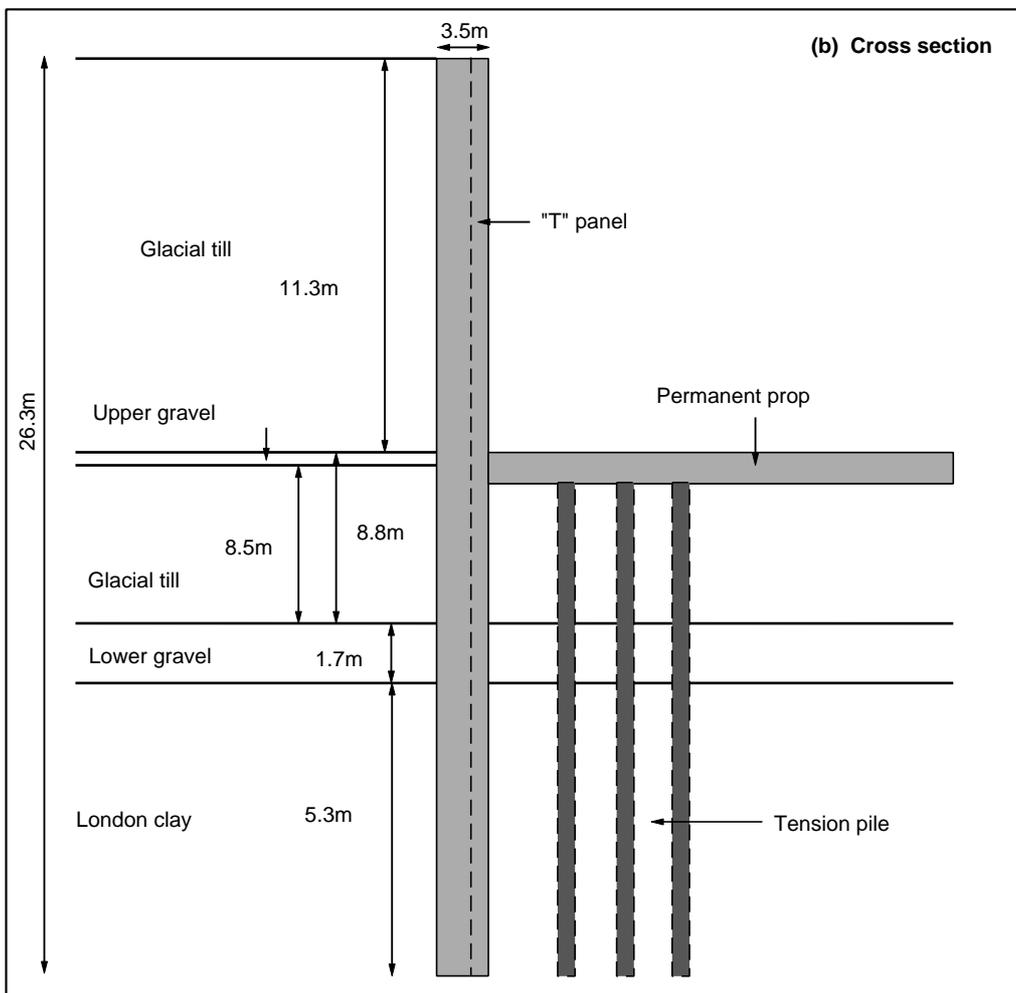
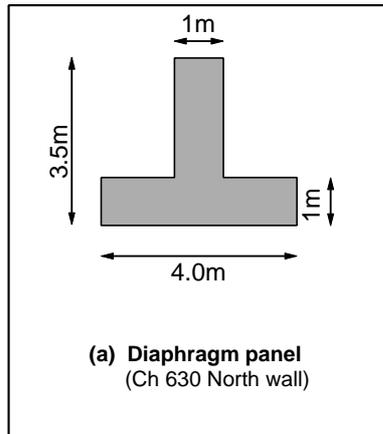


Figure 4.2 Wall dimensions and soil layers

## 4.2 Soil conditions

The ground conditions comprised some 20m of Glacial Till overlying the London Clay in which the toe of the retaining wall was founded. Gravel layers existed at two levels, the lower layer marking the boundary between Glacial Till and London Clay, with the upper layer being some 11m below ground level. Figure 4.3 shows the natural moisture contents, Atterberg limits and plasticity index obtained from samples taken from seven boreholes located between chainages 480 and 850 during the site investigations in the area. The variation with depth of the SPT 'N' values obtained from the same boreholes is shown in Figure 4.4. Values of undrained shear strength for the upper 25 metres of ground are given in Figure 4.5.

Figure 4.6 shows the 'most probable' ground water distribution which identified that a perched water table existed at this site.

## 4.3 Measured performance

The soil berms supporting the north wall were excavated following the schedule given in Figure 4.7. Excavation took place during July 1996 beginning at the eastern end of the wall (panel 21) and progressing westwards.

Figure 4.8 shows the horizontal movements measured on the top of the north wall capping beam on panels 8 to 17, those close to chainage 630, between May 1996 and October 1996. These movements were obtained by standard surveying methods with a survey point located on the top of the panels. The results show that the measured movements along this length of wall varied from a minimum of about 7mm to a maximum of 13mm. In some cases, movements were recorded on survey stations when no bulk excavation was taking place; these movements could generally be attributed to localised construction activities which are not detailed in this report.

The range of lateral movements measured at the top of the three inclinometer tubes installed in the retaining wall are shown in Figure 4.9. The horizontal movements of the tops of the tubes ranged between 5mm to 15mm and were therefore in reasonable agreement with the range of movements measured by standard surveying.

## 4.4 Comparison of measured movements and back-analyses

Figure 4.10 shows the lateral movement profiles with depth obtained from the finite element analyses. The measured horizontal movements of the top of the wall during construction, for panels 8 to 17, are also shown in Figure 4.10a. It must be noted that the finite element analysis used was a two dimensional analysis and hence the supporting effect of adjacent sections of soil berm still in place was not modelled. The calculated movements will therefore represent an upper bound of values as in the analysis the complete berm was removed instantaneously, whereas in practice excavation of the soil berm took several weeks to progress from one end of the wall to the other (Figure 4.7). On this basis reasonable correlation was obtained between with the predicted movements being a few millimetres higher than those measured when using berms for temporary support (Figure 4.10a).

Results from the comparative analysis using temporary props are given in Figure 4.10b. In this case the steel props were modelled as being raked rather than horizontal as the distance between the north and south walls was such as to render the use of horizontal props impractical. Generally movements of about 13mm were predicted when using props as opposed to the 17mm predicted for soil berms. This difference is small and would be reduced still further if the soil berm analysis had been three dimensional.

Both analyses showed small movements of the toe of the wall of between 5mm and 6mm. The reasonable agreement between the surveyed movements at the top of the wall and the top of the inclinometer tubes does however suggest that only minor toe movements occurred.

Further details of the finite element analyses are given in Appendix B.

## 5 A40 Long Lane improvement

---

As part of the A40 Long Lane improvement scheme an embedded retaining wall with a permanent stabilising base was constructed close to Hillingdon station. Temporary steel raked props were installed to support the contiguous bored pile retaining wall during excavation of a temporary berm and subsequent construction of the permanent reinforced concrete prop slab below the carriageway. A plan showing the location of the site is given in Figure 5.1.

During 1992 a section of the retaining wall was monitored by TRL for the London Regional Office of the Department of Transport. The results of this monitoring were described by Carder and Brookes (1992).

### 5.1 Construction of the retaining wall

The wall was constructed using cast *in situ* reinforced concrete bored piles of 1200mm diameter and installed at 1300mm centres. The pile length at the location being considered was about 23m, the bottom of the permanent stabilising base being some 11m below the top of the capping beam. The stabilising base extended 6m out from the wall and was 1m thick. A section through the wall is shown in Figure 5.2.

Bulk excavation in front of the wall was completed in several stages, commencing at the eastern end of the wall near the Hillingdon Underground station and progressing westwards. First stage excavation was down to near final road level along the centre of the underpass, but leaving a temporary berm against the wall.

A thrust beam was then installed at the foot of the berm before installing the raked temporary props. The props comprised spiral-weld steel tubes, each 13m long, 762mm in diameter, with a nominal wall thickness of 14.2mm giving a cross-sectional area of 33,360mm<sup>2</sup>. The lower ends of the props were fitted with spreader beams to accommodate the hydraulic jacks, used for pre-stressing, and packing plates. The upper ends of the raked props were fixed onto the capping beam. In the instrumented area the props were deployed at alternate nominal spacings of 5m and 7m and were set at a mean angle of 23.5° to the horizontal. After positioning, each prop was pre-stressed by jacking to a nominal load of 259 kN.

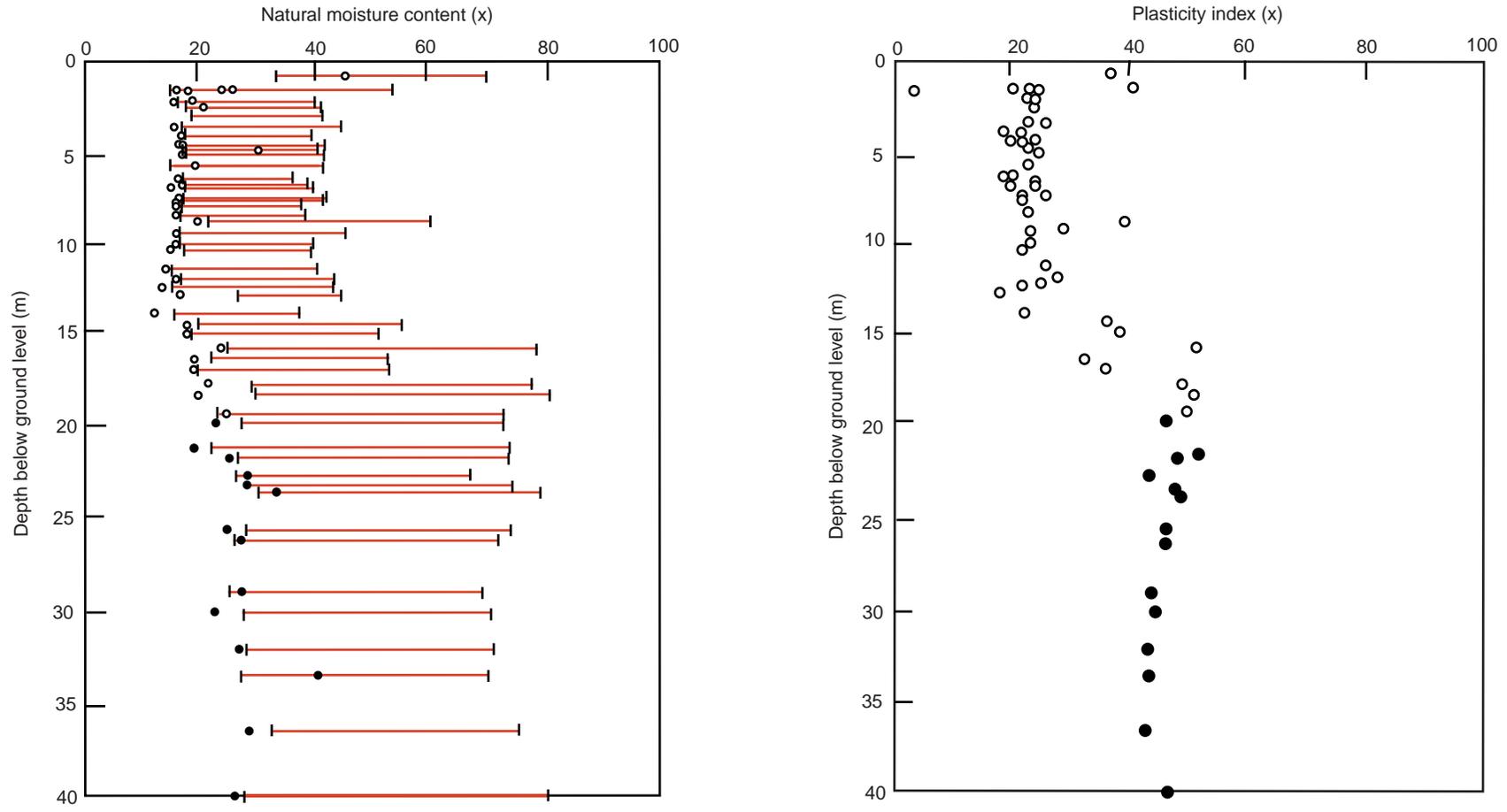


Figure 4.3 Moisture contents and Atterberg Limits

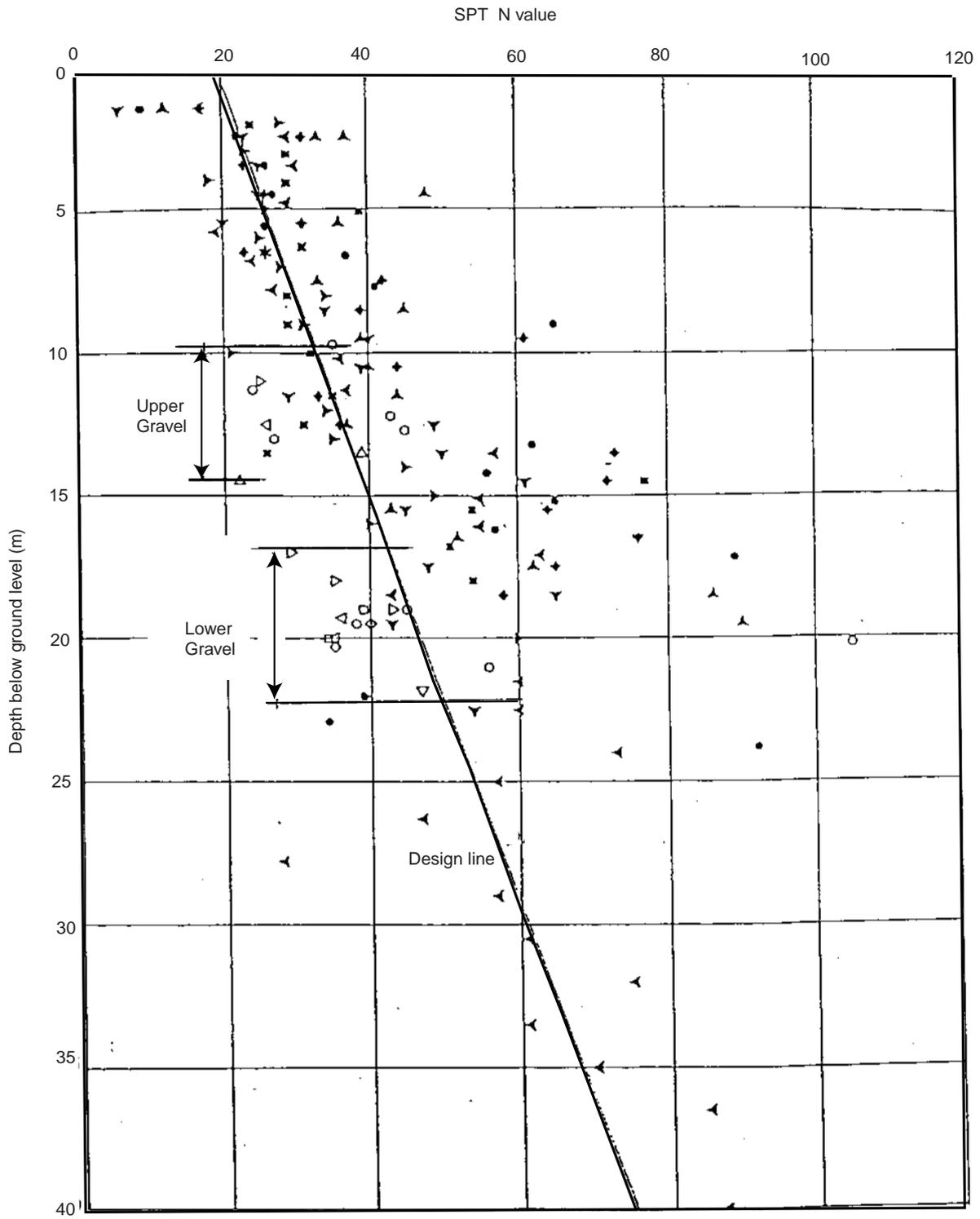
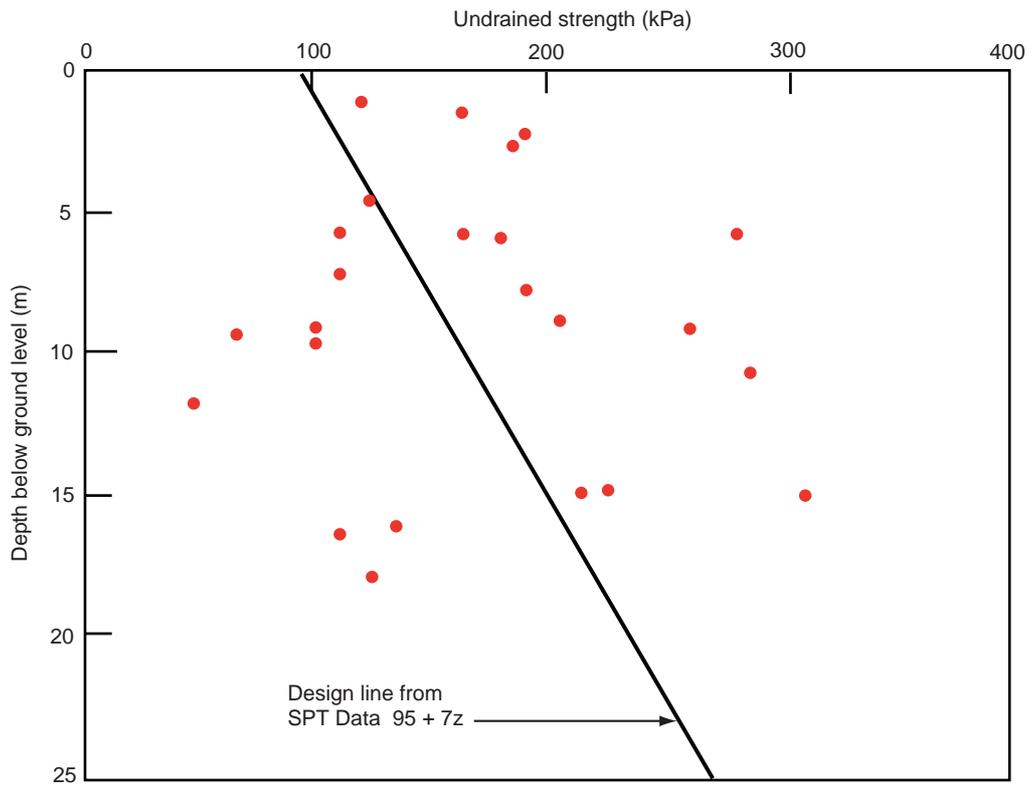
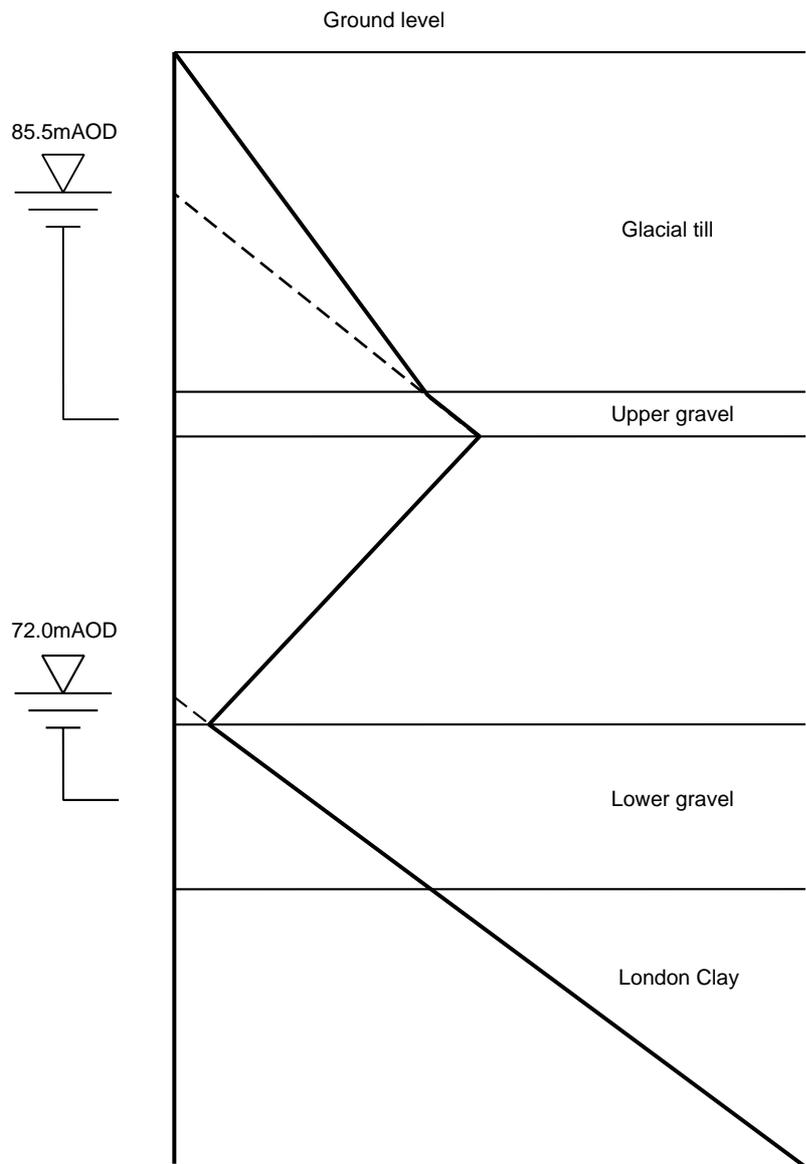


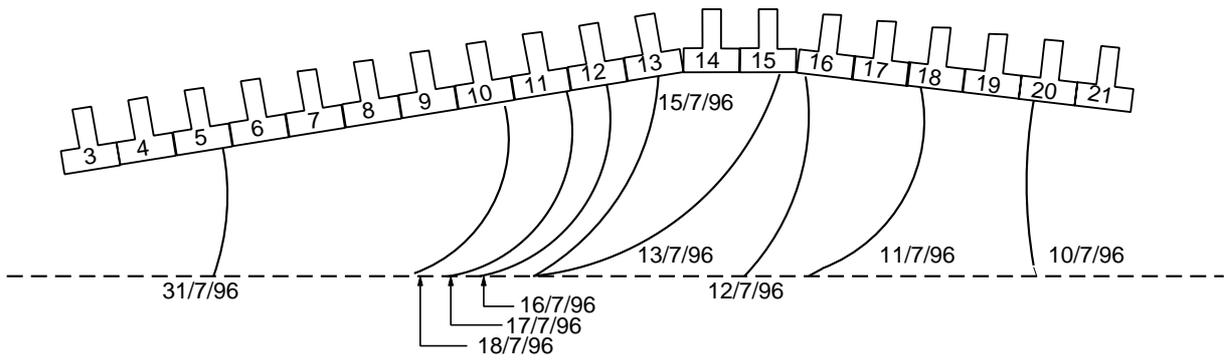
Figure 4.4 SPT values



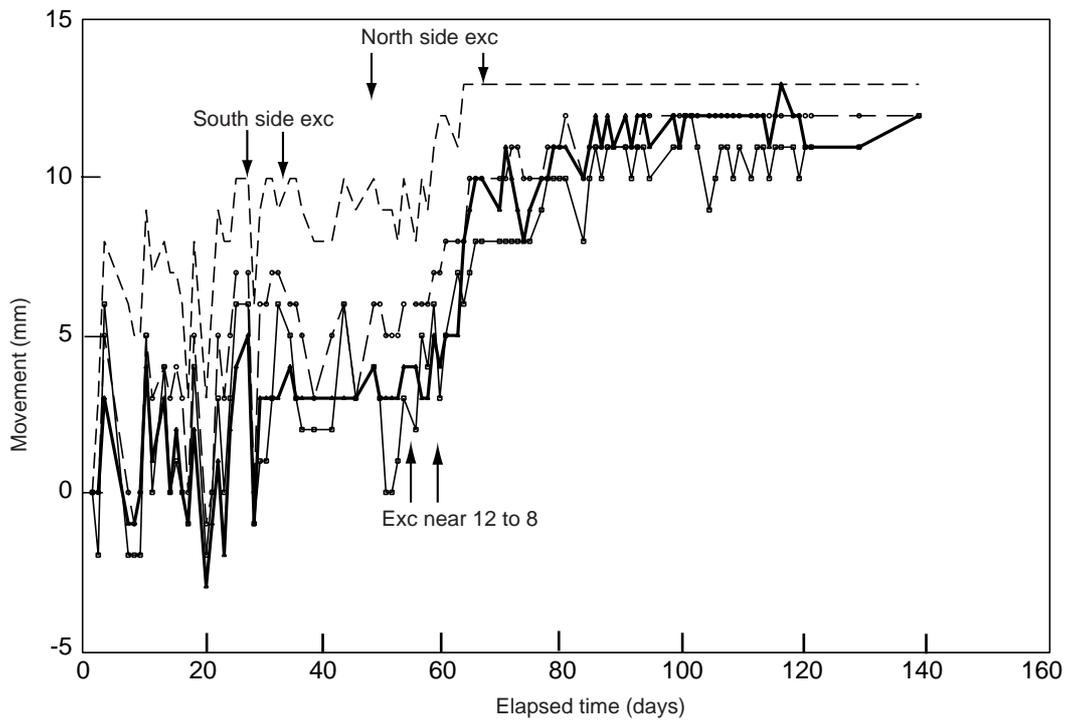
**Figure 4.5** Undrained shear strength



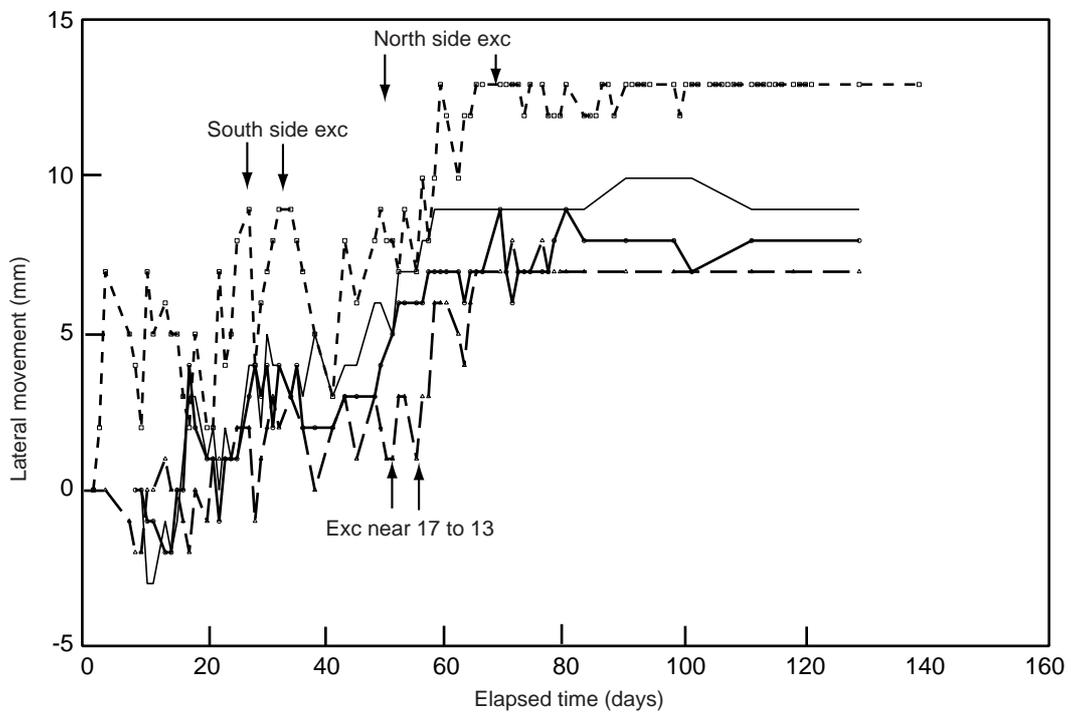
**Figure 4.6** Most probable groundwater pressure distribution



**Figure 4.7** Excavation schedule for north wall

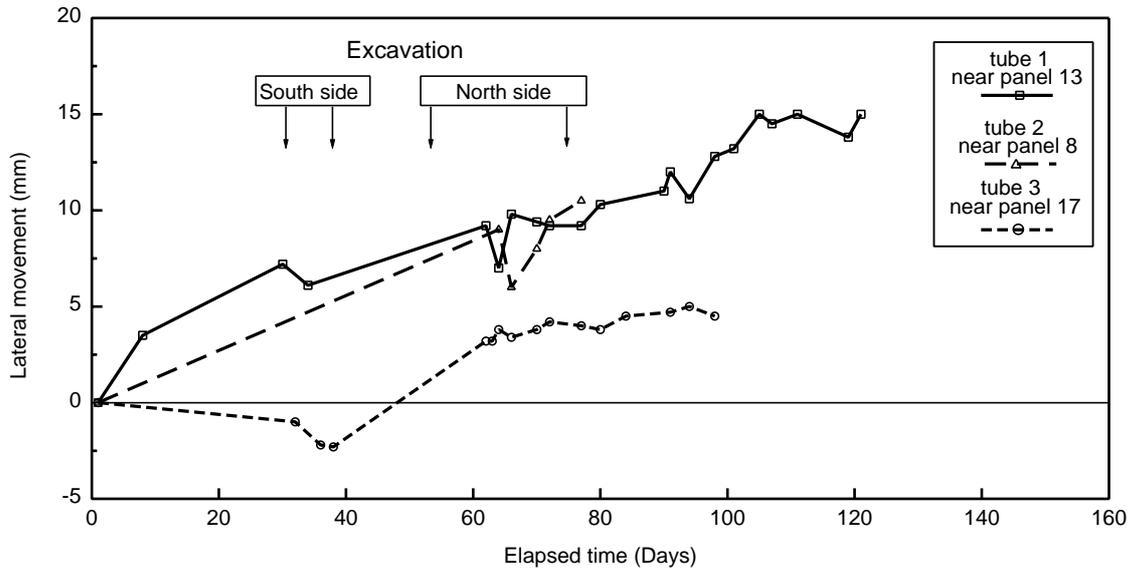


(a) Panels 8 to 12

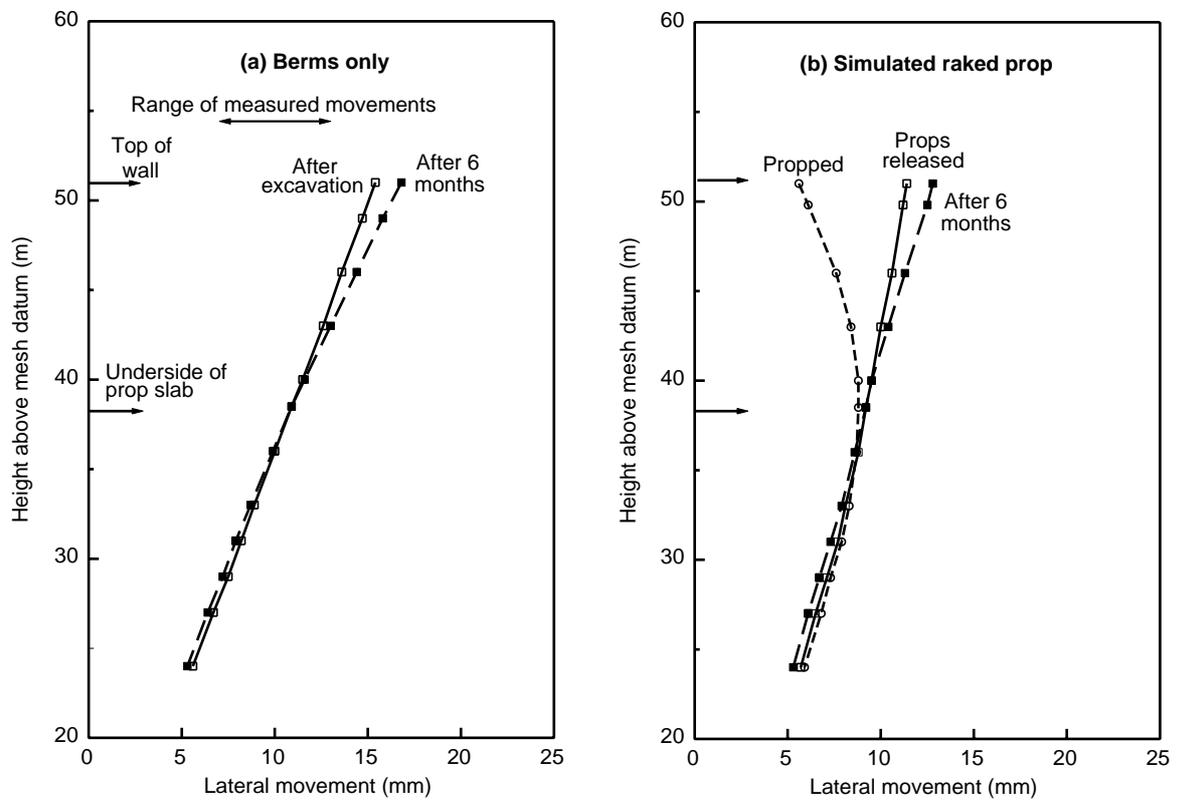


(b) Panels 13 to 17

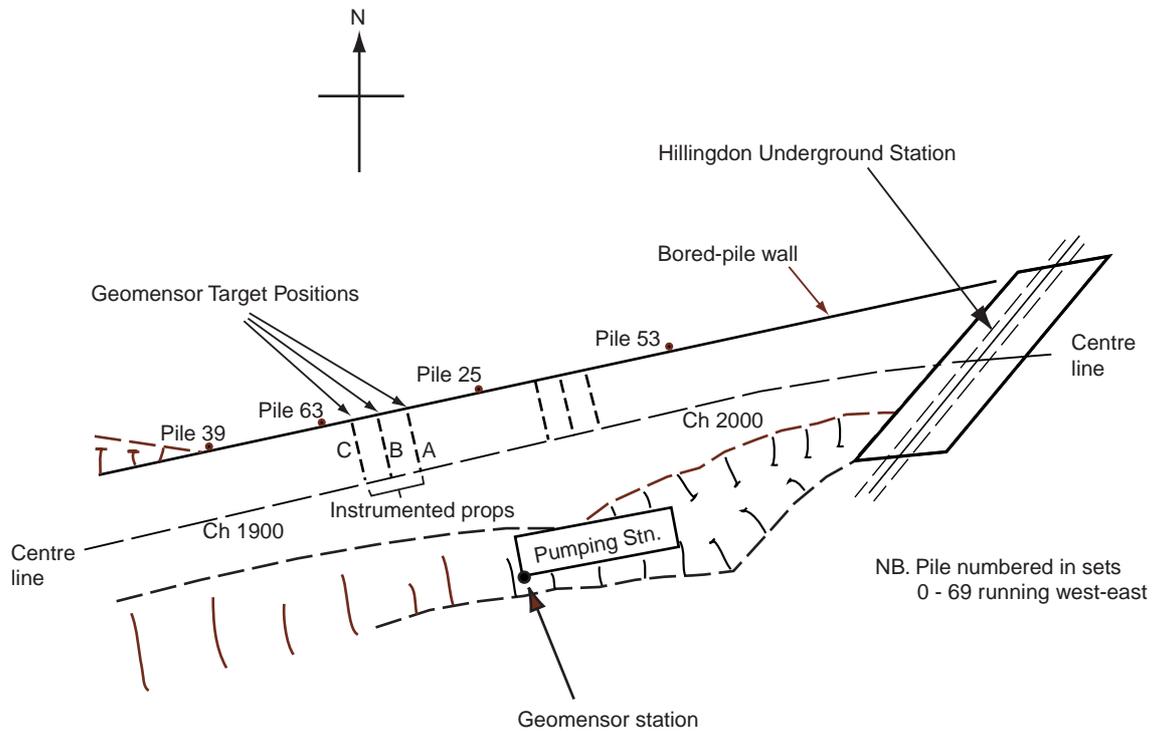
**Figure 4.8** Lateral movements at the top of the wall (surveying)



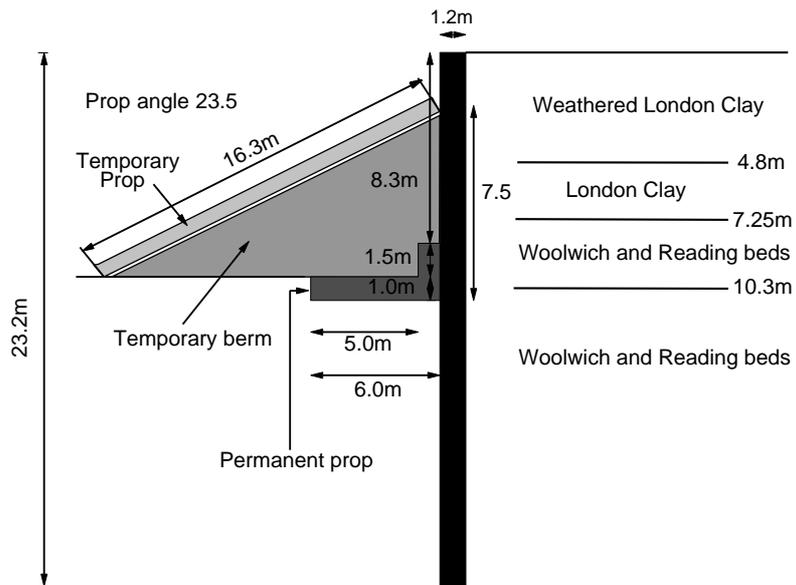
**Figure 4.9** Lateral movement of the tops of three inclinometer tubes



**Figure 4.10** Predicted horizontal movement of retaining wall



**Figure 5.1** Plan of the site



**Figure 5.2** Section of north wall A40 Long Lane

Following installation of the temporary props, the soil berm beneath them was excavated and installation of the permanent stabilising base followed. Temporary prop A (Figure 5.1) remained in place for 41 days, whilst props B and C were in place for 46 and 47 days respectively.

## 5.2 Soil properties

Figure 5.2 also summarises the soil strata encountered in the site investigation borehole closest to the instrumented area. The soil layers comprised weathered and intact London Clay down to a depth of about 7m overlying Woolwich and Reading beds. The Woolwich and Reading beds were described as varying from firm to stiff grey fissured clay with fine sand and silt to stiff brown purplish silty clay. The soil profile from the nearest borehole together with the *in situ* moisture contents, and undrained shear strengths obtained are given in Figure 5.3. Plasticity data obtained from clay samples at three depths in this borehole are given in Table 5.1.

**Table 5.1 Plasticity data**

| Soil type               | Depth (m) | LL (%) | PL (%) | PI (%) |
|-------------------------|-----------|--------|--------|--------|
| Weathered London Clay   | 4.6       | 69     | 22     | 47     |
| Unweathered London Clay | 5.3       | 59     | 21     | 38     |
|                         | 6.3       | 65     | 22     | 43     |

## 5.3 Measured performance

Measurements were made of the lateral movement of the top of the wall at prop locations A,B and C using a high precision electronic distance measurement system (Geomensor). These measurements were taken by inserting a target reflector into sockets installed in the top of the capping beam. The reference station for the Geomensor was sited on a section of the south wall forming a part of a pumping station as this was felt to be the most stable location available with suitable lines of site to the targets (Figure 5.1). Geomensor surveys were conducted twice weekly during the construction period and at less frequent intervals to monitor longer term wall movements. In addition to the movement measurements, the temporary props were also instrumented with strain gauges and thermocouples to monitor load and temperature changes: these measurements are reported by Carder and Brookes (1992).

The variation of the lateral movement of the top of the wall at prop B is shown in Figure 5.4. The data show that up until the prop was released lateral movement towards the excavation was about 6mm. Following release of the temporary props a further 3mm to 4mm movement occurred.

The long term variations in lateral movement of the wall capping beam for all three prop locations are shown in Figure 5.5. At each location the wall had moved by about 10mm after release of the raked temporary props. After prop release the wall continued to move towards the excavation, reaching peak values of between 11mm (prop B) and 14mm (prop A) during the winter of 1992/93. Towards the end of 1994 the wall again appeared to move a few millimetres towards the road but monitoring ceased at this time. When comparing the measured values with movements calculated from finite element analyses it

should be noted that measurements of lateral movement commenced after bulk excavation of the haul road (first stage excavation) had already taken place. Allowance is made for this when comparing measured and predicted wall movements in Section 5.4.

## 5.4 Comparison between measured performance and back-analyses

The lateral movement profiles with depth for the wall derived from the numerical analyses using different methods of temporary support are shown in Figure 5.6. The measured movements from the estimated datum at the end of first stage excavation (Section 5.3) are shown for the actual construction sequence using temporary raked props in Figure 5.6a. In this case, the calculated values are in good agreement with those obtained from field measurements. This analysis also suggested that the toe of the wall moved horizontally by about 5mm, unfortunately it was not possible to confirm this from the field measurements as no inclinometer tubes had been installed in the wall.

An analysis simulating construction using soil berms only and no raked props is shown in Figure 5.6b. As would be expected in this instance the top of the wall showed a small increase in lateral movement of about 4mm from the 26mm predicted when using raked props. On this basis it is possible that the raked props could have been omitted at this scheme provided that, during the progressive excavation of the berms and installation of the stabilising base, monitoring had taken place using the Observational Method.

Figure 5.6c shows the lateral movement profiles obtained if horizontal props spanning the underpass were modelled. It must be noted that this method of support would not have been practical or economic on this site because of the large width of the underpass, but nevertheless the comparative study is of interest. Lateral movements of 24mm were calculated 6 months after the end of construction and these were marginally less than that predicted using raked props, but the differences were not significant.

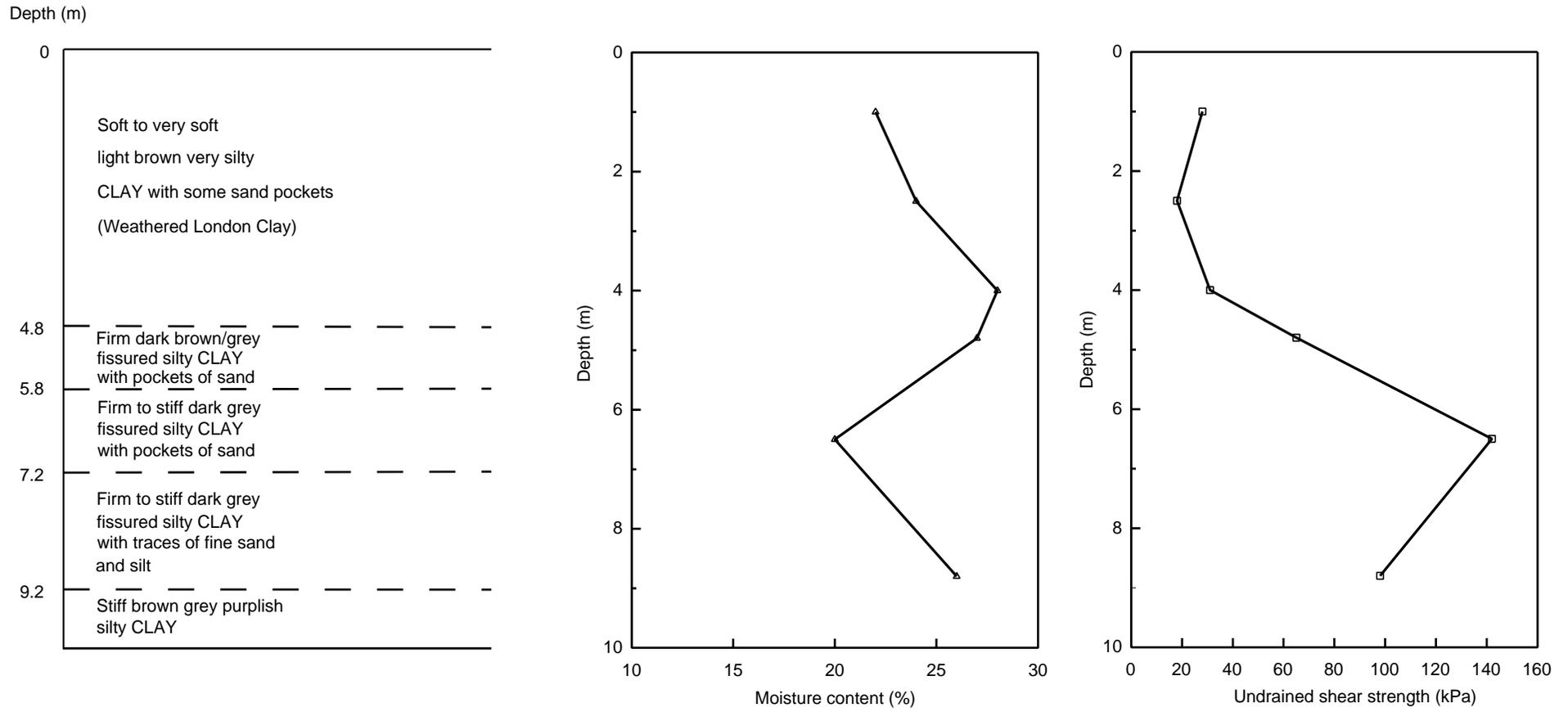
A more detailed description of the mesh, the soil parameters and the construction sequences used in the finite element analyses is given in Appendix C.

## 6 A50 Blythe Bridge to Queensway

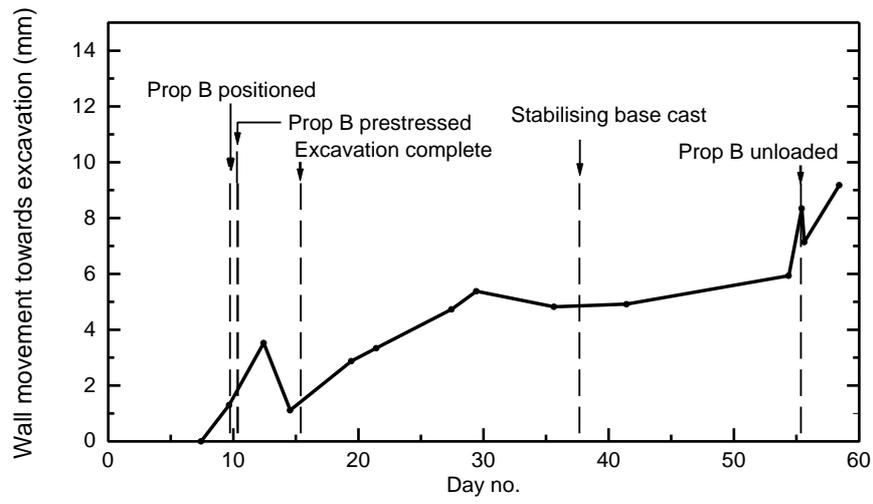
The construction scheme forms part of the new realignment of the A50, a dual two lane road running through the south eastern outskirts of Stoke on Trent. The road passes very close to several industrial premises which were deemed to be sensitive to any significant ground movements. A plan showing the location of the site is given in Figure 6.1.

### 6.1 Construction of the retaining wall

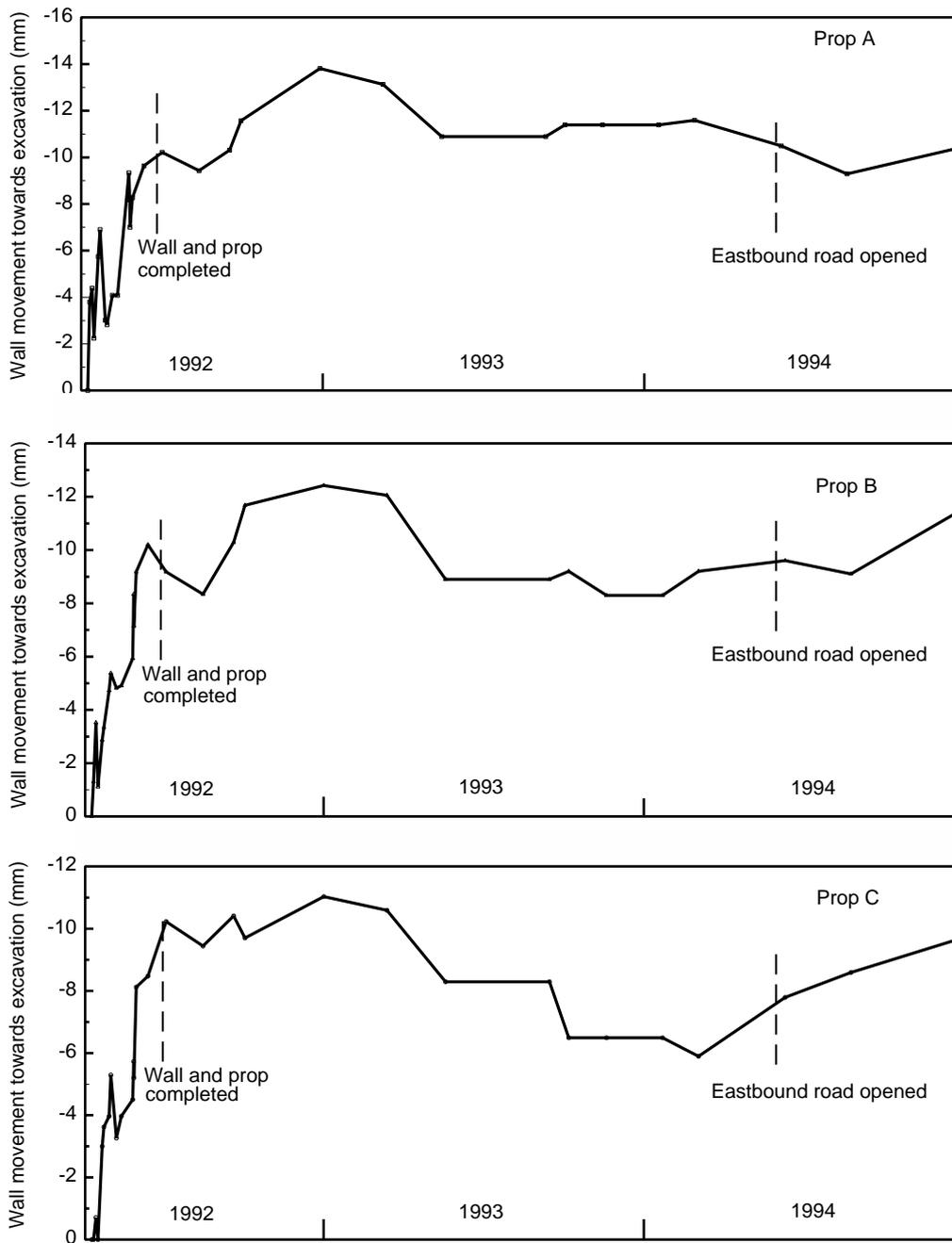
A contiguous bored pile retaining wall formed the north wall of a reinforced concrete trough some 25m wide. The other side of the trough comprised a conventional reinforced concrete cantilever retaining wall. The two



**Figure 5.3** Soil strata, moisture content, and undrained shear strength



**Figure 5.4** Top of wall movement at prop B



**Figure 5.5** Long term wall movements

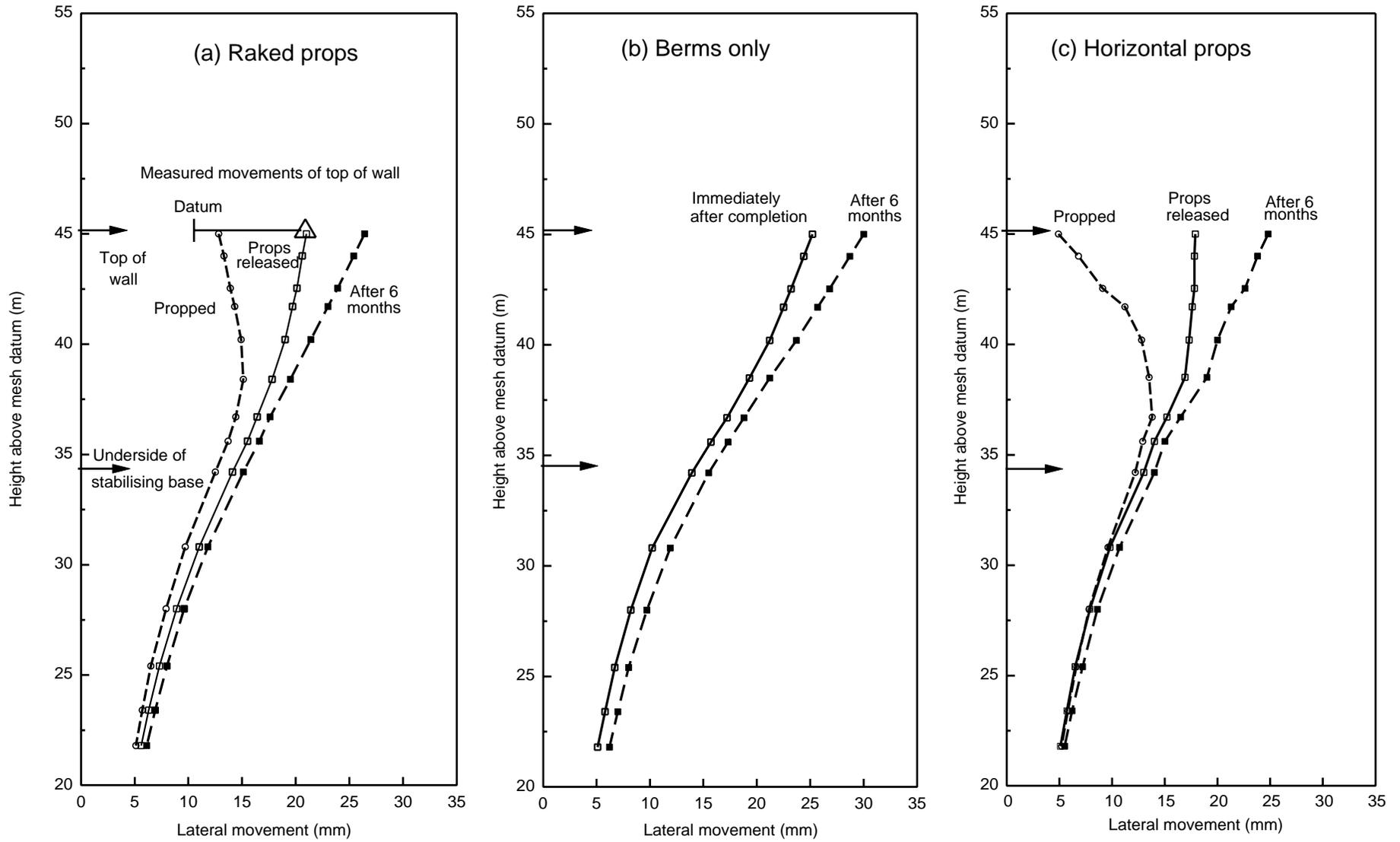


Figure 5.6 Predicted horizontal movement of retaining wall

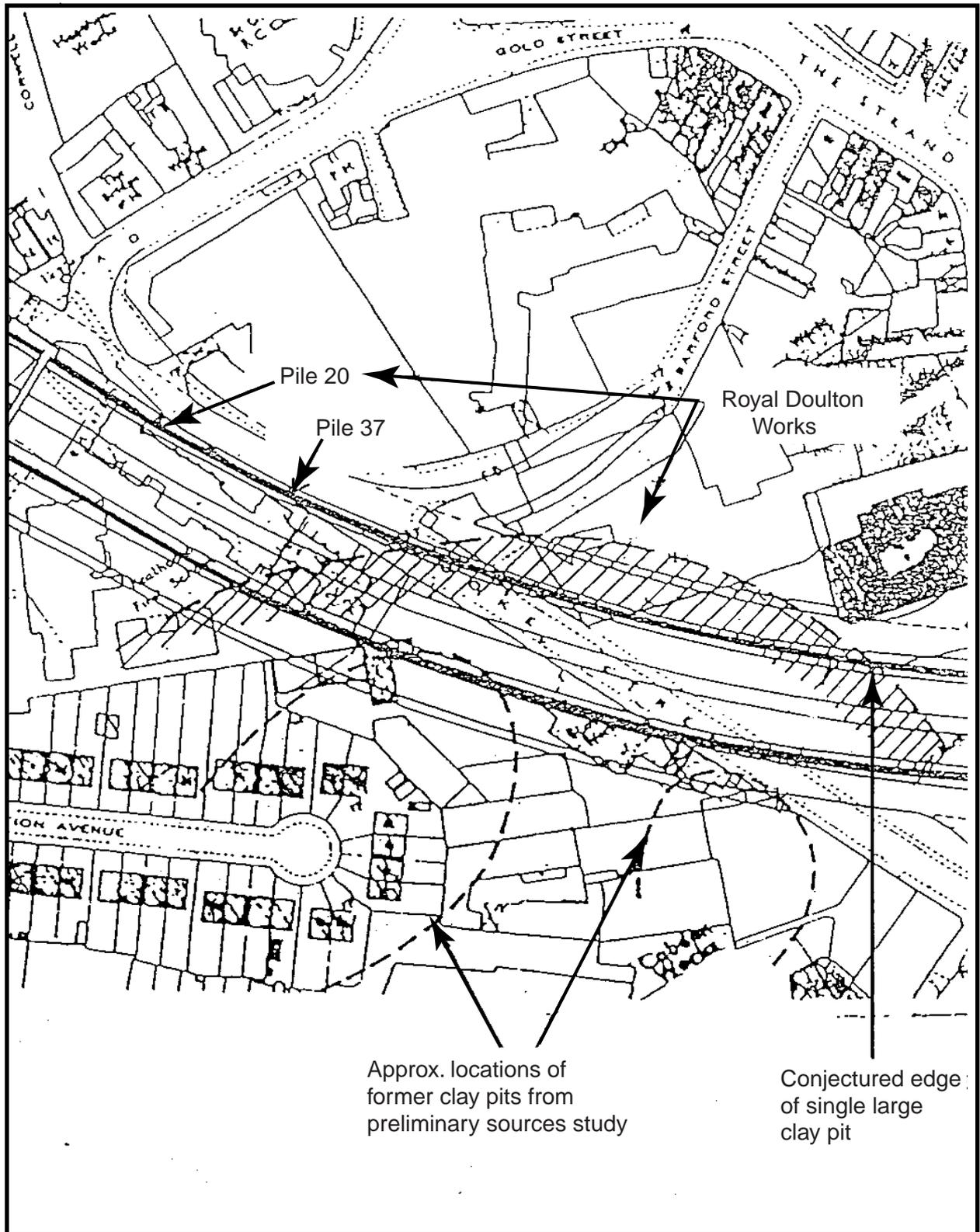


Figure 6.1 Plan of the site

walls were propped apart by a reinforced concrete slab on which the carriageway was constructed.

The contiguous bored pile wall consisted of 1500mm diameter piles at 1650mm centres, the gap between the piles was in-filled with no fines concrete. A reinforced concrete capping beam was cast along the top of the piles. A cross section through the wall is given in Figure 6.2.

## 6.2 Ground conditions

Generally the wall was founded in Middle Coal Measures which dipped towards the south west and south south west at angles between 10° and 20°. Weak and polished bedding planes were present in the strata. Former clay pits were also evident during the site investigation for the scheme.

For the purpose of this report, two instrumented locations (ie. piles 37 and 20 of the north wall) are considered and the respective ground strata and their depths are shown in Figure 6.3. The ground conditions near pile 37 are shown in Figure 6.3a and mainly comprised sandstone, with underlying layers of silt, hard sandstone and mustone, with a 1.5m thick layer of glacial till overlying them. The ground water level was assumed to be about 10m below ground surface although a perched water table also existed in the made ground.

At the first location, near pile 20 (Figure 6.3b), the strata mainly comprised mudstone and sandstone, with these strata being overlaid by a 1.5m thick layer of glacial material. The ground water level was similar.

The generalised soil/rock properties for the scheme are given in Table 6.1.

The lateral pressure behind the wall may be considered to be a linear distribution with depth. The rock pressure coefficient for a retained rock mass containing potential failure planes is analogous to the earth pressure coefficient for soils. The rock pressure coefficient corresponds to the limiting state of equilibrium and its value depends on the orientation of joints and bedding planes and the shear strength along them. Details of the equivalent horizontal earth pressures used in the back-analyses are given in Appendix D.

## 6.3 Measured performance

Lateral movement data was obtained from the construction monitoring of piles 20 and 37 which were 18.5m and 17.7m deep respectively. The movements were measured by carrying out inclinometer surveys on access tubes installed near vertically in each pile. From the surveys which were carried out at the main stages of construction, the lateral movement of the top of the wall was calculated assuming base fixity of the piles.

Figure 6.4 shows the profiles of lateral movement with depth obtained from the inclinometer tube in pile 20 for the period during which excavation of the ground in front of the wall took place. At this location the supporting berm was removed in bays about 7m long and the permanent prop slab constructed. Lateral movements of the wall were checked before excavation of the subsequent bays. Prior to berm excavation less than 2mm of lateral movement due to bulk excavation of area 1 (Figure 6.2) had been observed at the top of the wall. Lateral movements increased to about 10mm after excavation of the supporting berm close to the pile was completed.

The lateral movements observed for pile 37 are shown in Figure 6.5. At this location the supporting berm was not removed in bays as at pile 20, but was excavated continuously. In this case the top of the wall moved about 6mm due to bulk excavation and this movement increased to about 14mm following excavation of the berms.

## 6.4 Comparison of measured movements and back-analyses

The predicted lateral movements of the wall at pile 37 are shown in Figure 6.6. The soil parameters used are given in Appendix D, which also includes further details of the finite element analysis. The analysis where berms were used for temporary support showed a lateral movement at the top of the wall of about 11mm. This is in reasonably good agreement with the measured values obtained from the inclinometer surveys, which gave a movement of about 14mm after excavation. The analysis also predicted a lateral movement of the toe of about 3mm, but there were no measured values with which to compare this prediction.

**Table 6.1 Generalised design values for soil/rock properties**

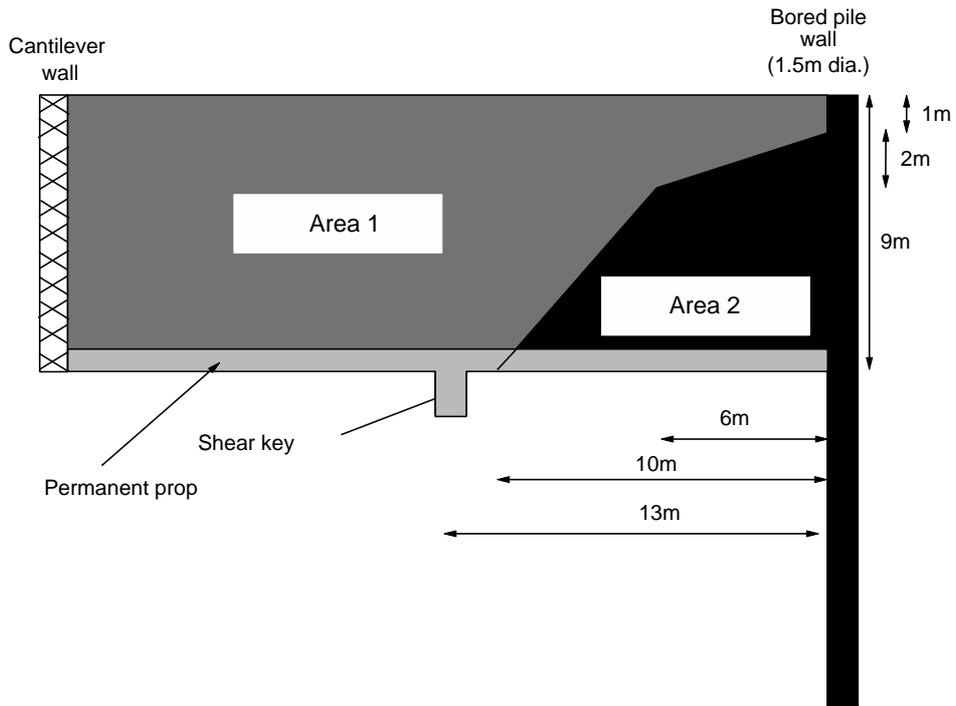
| Stratum                        | N (blows) | $c_u$ (kN/m <sup>2</sup> ) | $c'$ (kN/m <sup>2</sup> ) | $\phi'$ (°) | Horizontal           | Subgrade             |
|--------------------------------|-----------|----------------------------|---------------------------|-------------|----------------------|----------------------|
|                                |           |                            |                           |             | subgrade             | constant             |
|                                |           |                            |                           |             | reaction             | (MN/m <sup>2</sup> ) |
|                                |           |                            |                           |             | (MN/m <sup>2</sup> ) | (MN/m <sup>2</sup> ) |
|                                |           |                            |                           |             | ++                   | ++                   |
| Made ground                    | 11        | 25                         | 0                         | 25          | -                    | 1.50                 |
| Glacial                        | 12        | 172                        | 0 <sup>+</sup>            | 25          | -                    | 1.50                 |
| Middle Coal Measures (1Va/1Vb) | 100       | -                          | 0                         | 30          | 31.5                 | -                    |
| Middle Coal Measures (111)*    | 175       | -                          | -                         | -           | 105                  | -                    |
| Middle Coal Measures (111)**   | -         | -                          | -                         | -           | 420                  | -                    |

+ Less conservative value from site measurements of 8kN/m<sup>2</sup>

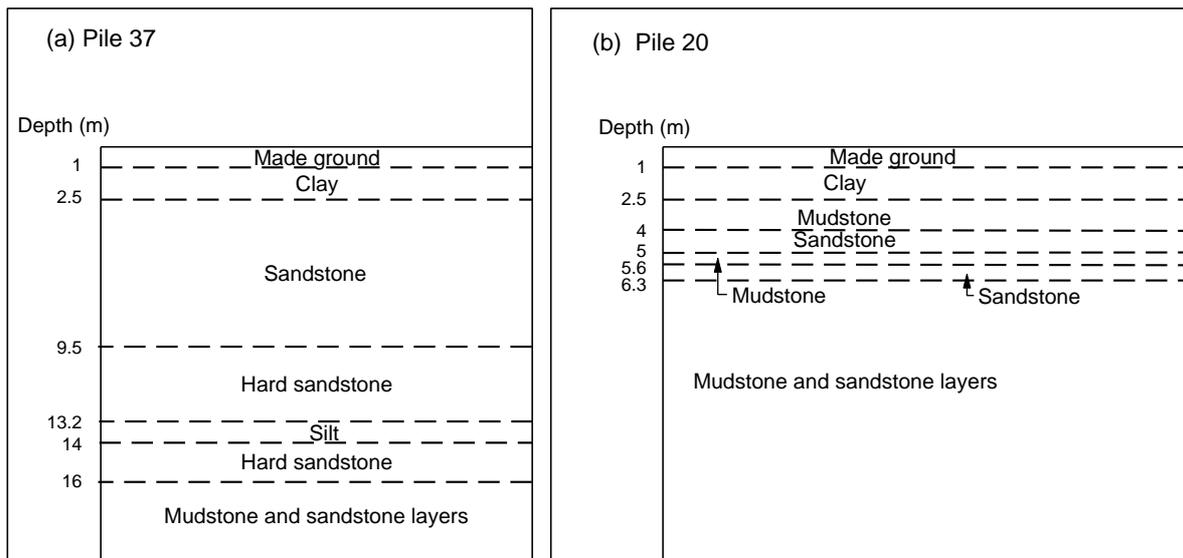
++ As defined by Terzaghi (1955)

\* Unconfined compressive strength of 4MN/m<sup>2</sup> (mudstone & weak siltstone)

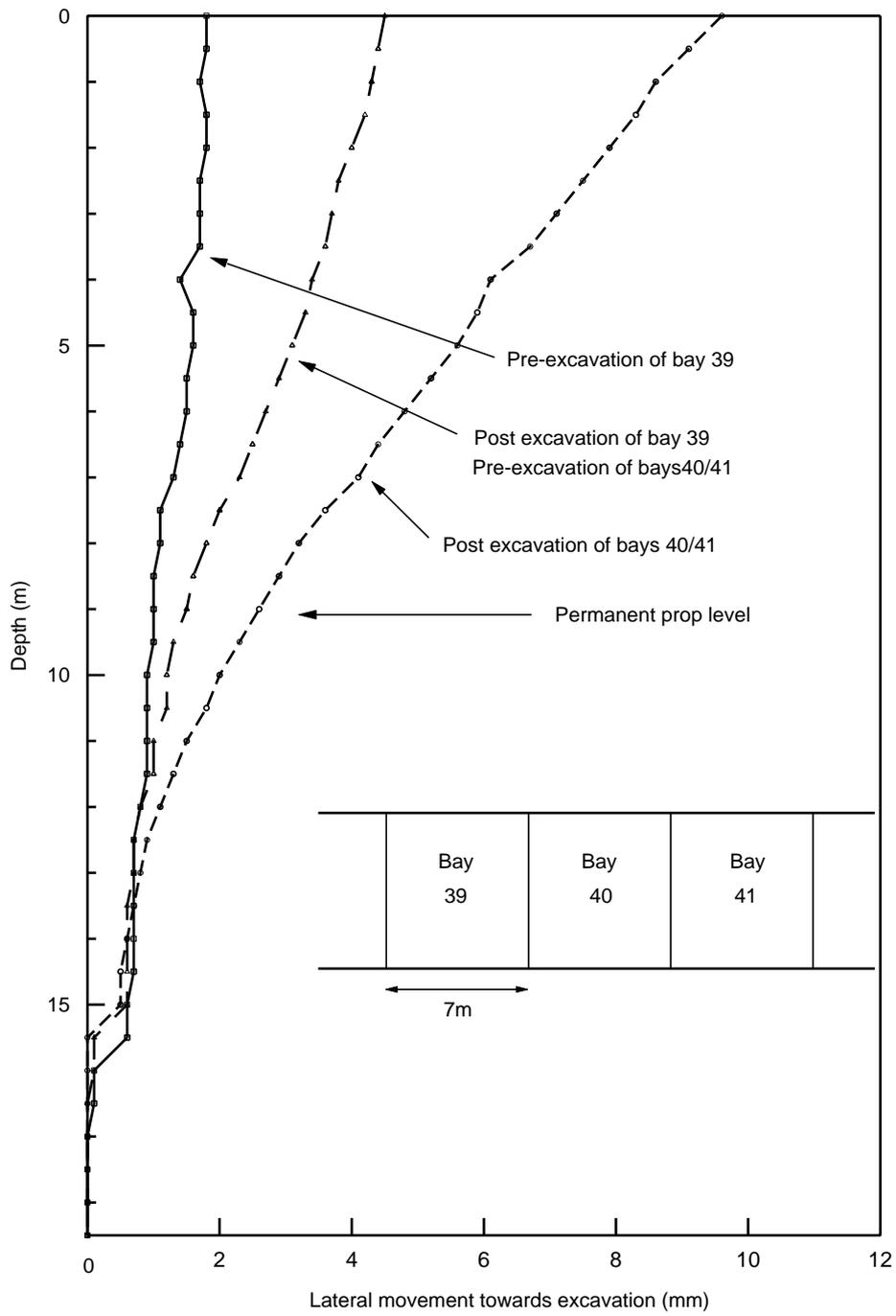
\*\* Unconfined compressive strength of 10MN/m<sup>2</sup> (moderately weak to moderately strong siltstone & sandstone)



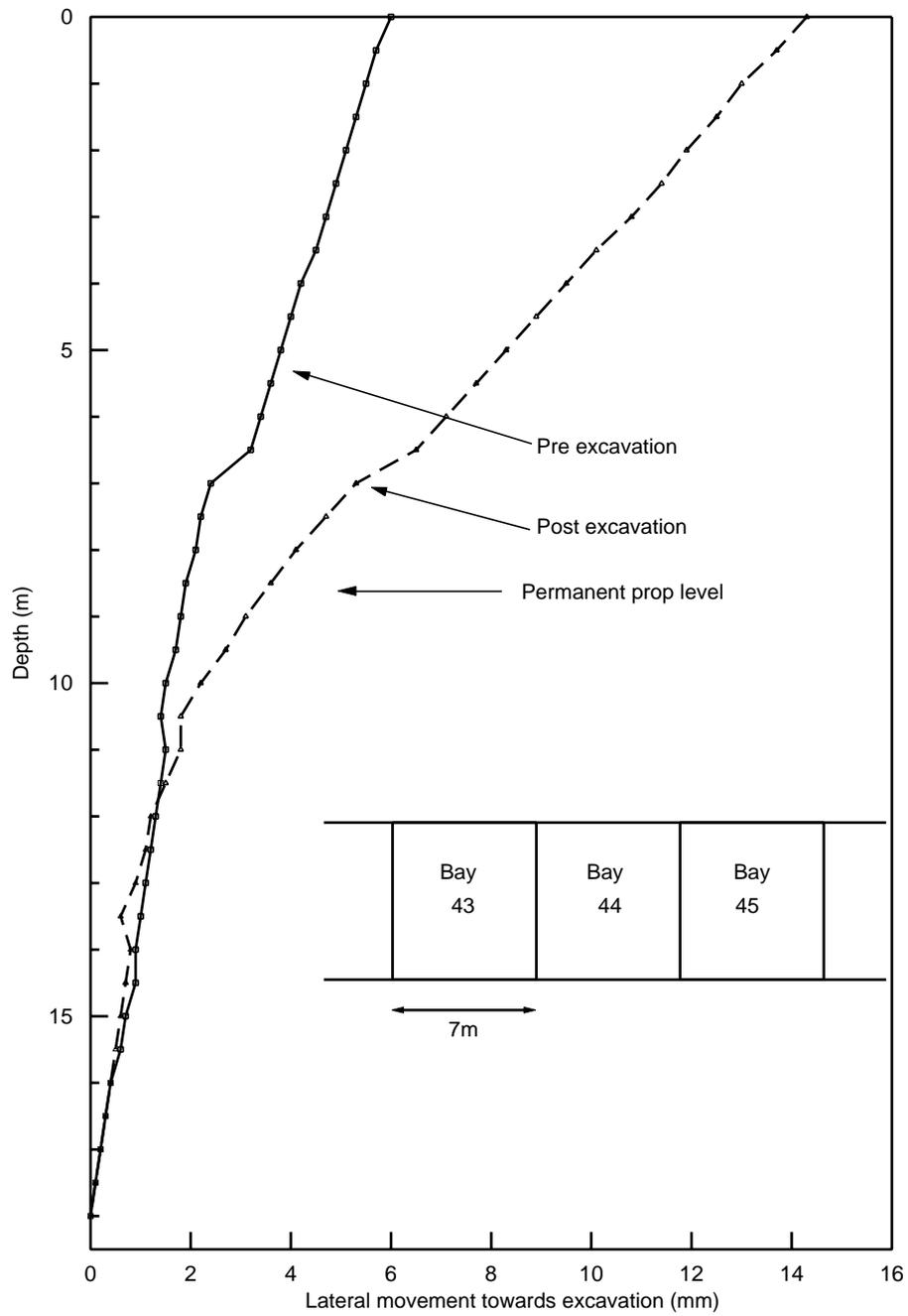
**Figure 6.2** Schematic section and berm dimensions



**Figure 6.3** Soil strata at two pile locations



**Figure 6.4** Measured lateral movements of pile 20 (bay40) during excavation



**Figure 6.5** Measured lateral movements of pile 37 (bay44) during excavation

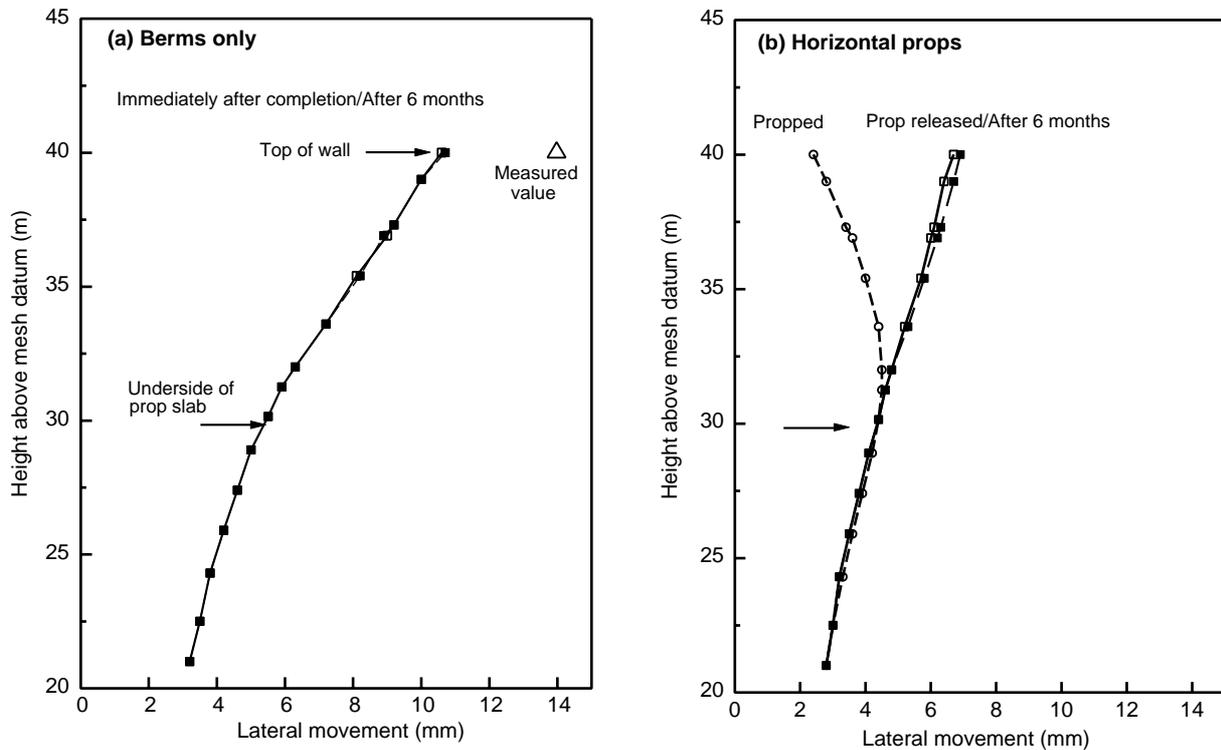


Figure 6.6 Predicted horizontal movement of retaining wall

Where horizontal props were modelled, lateral movements of the top of the wall were about 6mm after release of the props as shown in Figure 6.6. These values were slightly lower than those predicted when using soil berms, although predicted movements were small in both cases.

## 7 Discussion

The report describes the measured performance of embedded retaining walls at four different sites. At three sites soil berms were employed as temporary support, whilst at the fourth site raked steel props were used.

Finite element analyses have been carried for a section of embedded retaining wall at each site. The analyses were two dimensional in nature and hence the sequential removal of berms and/or temporary raked props along the length of wall could not be modelled. The lateral movements should therefore be considered to be an upper bound as support from adjacent unexcavated berm and already cast sections of permanent propping present in the field could not be modelled in the analysis.

Table 7.1 summarises the measured and predicted movements of the top of the walls expressed as a percentage of the retained height of each structure. The code of practice for earth retaining structures BS8002 recommends that the lateral movement at the top of the structure should not exceed 0.5% of the retained height. The measured values obtained from all four sites fall well below this value. Of the predicted lateral movements only the value obtained for Batheaston-Swainswick Bypass using the original design soil parameters exceeds this value, confirming the highly conservative parameters

Table 7.1 Wall movements as a percentage of retained height

| Site                                | Measured                      | Lateral movement at top of wall<br>(% of retained height) |             |                  |
|-------------------------------------|-------------------------------|---|-------------|------------------|
|                                     |                               | Predicted after 6 months                                  |             |                  |
|                                     |                               | Soil berms  | Raked props | Horizontal props |
| A4/A46 Batheaston-Swainswick ByPass | 0.17<br>(berms)               | 0.81*<br>0.15+  | -           | 0.33*<br>0.08+   |
| A406 East of Falloeden Way          | 0.04 to 0.11<br>(berms)       | 0.15  | 0.11        | -                |
| A40 Long Lane Improvement           | 0.19<br>(berms & raked props) | 0.28  | 0.24        | 0.23             |
| A50 Blythe Bridge to Queensway      | 0.14<br>(berms)               | 0.11  | -           | 0.07             |

\* Original design soil parameters (ie upper bound movements)  
+ Best estimate soil parameters

employed. Predictions based on best estimate soil parameters were considered more realistic for this site. In a review of ground movements caused by embedded retaining wall construction Carder (1995) found that for schemes studied by TRL maximum values of between 0.2% and 0.3% of the retained height were observed depending on the stiffness of the temporary propping system. The movements measured and predicted in this report are consistent with these values.

In general the values predicted by the back-analyses modelling the actual construction methods employed on site were in reasonably good agreement with those observed in the field. The predicted values from analyses modelling the use of horizontal temporary props showed smaller lateral movements than those for soil berms or raked props but the differences were not large.

## 8 Conclusions

---

Construction data have been obtained from four road schemes where either soil berms or raked steel props were used as temporary support to embedded retaining walls. The measured lateral movements of the walls were compared with those predicted from plane strain finite element analyses carried out using SAGE CRISP:

- 1 For all the schemes studied the measured lateral movement of the top of the walls was significantly less than the value of 0.5% of the retained height suggested in BS8002. Generally lateral movements were less than the 0.3% of retained height established by Carder (1995) from a literature review of a number of retaining wall schemes.
- 2 The lateral movements predicted using best estimates of the soil parameters and modelling the actual construction sequence were also below the value of 0.3% of the retained height and in reasonable agreement with the measured values.
- 3 The predicted lateral movements obtained where raked or horizontal props were modelled were smaller than for soil berms but the differences were not great.
- 4 This study indicates that the use of soil berms and/or raked temporary props is a practical alternative to employing horizontal steel props and could result in considerable reductions in the cost of embedded retaining wall construction. In cases of uncertainty these temporary support systems should be used in conjunction with the Observational Method. Uncertainties may exist where softening of the ground is likely because of high water levels or where permeable or soft soil layers exist. Both of these factors may affect the efficiency of soil berm behaviour.
- 5 For all of the schemes studied the soil berms/raked props gave sufficient support to the embedded walls, no additional measures were required. The movement monitoring under the Observational Method enabled construction to proceed with confidence.

## 9 Acknowledgements

---

The work described in this report forms part of the research programme of the Civil Engineering Resource Centre of TRL and was funded by Quality Services (Civil Engineering) of the Highways Agency. The HA Project Manager was Dr D I Bush. The authors were assisted in the study by Dr D R Carder.

The authors wish to acknowledge the cooperation of the various HA Project Managers for each scheme in both

assisting TRL in the identification of suitable sites and subsequently allowing TRL to approach the parties involved.

The invaluable cooperation and assistance given by the following organisations and individuals are gratefully acknowledged: Ove Arup and Partners (D P Nicholson, Che-Ming Tse and T Chapman), Sir Alexander Gibb and Partners, Amey Construction (G D Cunningham and G Gellatly), Edmund Nuttall Ltd (S Deeble and J Hancock), Gifford Graham and Partners, Scott Wilson Kirkpatrick and Partners (R Quinn and D Cragge), Taylor Woodrow Foundation Engineering, WSP Graham, Mott MacDonald and Partners, Fairclough Civil Engineering Ltd, and Southampton University (W Powrie and S Gourvenec).

## 10 References

---

- British Standards Institution (1994).** *Code of practice for earth retaining structures BS8002*. British Standards Institution, London.
- Brookes A H and Carder D R (1996a).** *Behaviour of the diaphragm walls of a cut-and-cover tunnel constructed in boulder clay at Finchley*. TRL Report TRL187. Transport Research Laboratory, Crowthorne.
- Brookes A H and Carder D R (1996b).** *Behaviour of a cantilever contiguous bored pile wall in boulder clay at Finchley*. TRL Report TRL244. Transport Research Laboratory, Crowthorne.
- Burland J B and Kalra J C (1986).** *Queen Elizabeth II conference centre: geotechnical aspects*. Proceedings of the Institution of Civil Engineers, Part 1, Vol 80, pp1479-1503.
- Carder D R (1995).** *Ground movements caused by different embedded retaining wall construction techniques*. TRL Report TRL172. Transport Research Laboratory, Crowthorne.
- Carder D R and Bennett S N (1996).** *The effectiveness of berms and raked props as temporary support to retaining walls*. TRL Report TRL213. Transport Research Laboratory, Crowthorne.
- Carder D R and Brookes A (1992).** *Discussion to Session 5. Retaining structures* (Ed. Clayton CRI), pp498-501. Thomas Telford, London.
- Gannon J A, Masterton G G T, Wallace W A and Muirwood D (1996).** *Piled foundations in weak rock*. Construction Industry Research and Information Association, RP509.
- Gibb (1993).** *A4/A46 Batheaston/Swainswick Bypass*. Approval in Principle.
- Gosney R F C, Gellatly G, Kerr A and Petch K A (1997).** *A4/A46 Batheaston/Swainswick Bypass*. Proceedings of the Institution of Civil Engineers, Civil Engineering, August, pp101-110.

**Gourvenec S (1998).** *Chapter 5: Numerical analysis of case study.* PhD thesis, Southampton University.

**Gourvenec S, Powrie W, Lacy M and Stevenson M (1996).** *Observation of diaphragm wall movements in Lias Clay during construction of the A4/A46 bypass in Bath, Avon.* Geotechnical Aspects of Underground Construction in Soft Ground (Eds. Mair and Taylor). Balkema, Rotterdam.

**Lambe T W (1973).** *Thirteenth Rankine Lecture: Predictions in soil engineering.* Geotechnique, Vol 23, No 2, pp149-202.

**Nicholson D, Tse C-M and Penny C (1997).** *The Observational Method in ground engineering: principles and applications.* CIRIA Funders Report CP/49. Construction Industry Research and Information Association, London.

**Nicholson D, Cunningham G D, Armstrong A and Boothman (1998).** *Value achieved using the Observational Method on the retained cutting at the Batheaston/Swainswick bypass.* Paper on the Observational Method at Batheaston. No publication data.

**Ove Arup & Partners (1995).** *A406 North Circular Road - East of Falloden Way to Finchley High Road Improvement.* Feasibility report for the removal of temporary props for the east end road ramps and stairs. Ove Arup & Partners.

**Ove Arup & Partners (1997).** *Batheaston Bypass data.* Factual report on instrumentation results at CH2835 and CH2355. Ove Arup & Partners.

**Selvadurai A P S (1979).** *Elastic analysis of soil-foundation interaction.* Developments in Geotechnical Engineering, Vol 17, pp422-423.

**Symons I F (1992).** *Design of embedded retaining walls in stiff clays.* TRL Research Report TRL359. Transport Research Laboratory, Crowthorne.

**Watson G V R and Carder D R (1994).** *Comparison of measured and computed performance of a propped bored pile retaining wall at Walthamstow.* Proceedings of the Institution of Civil Engineers, Geotechnical Engineering, Vol 107, pp127-133.

## Appendix A: Finite element analysis of A4/A46 Batheaston–Swainswick Bypass

### A1 Details of mesh

Diagrammatic representation of the finite element meshes employed in these analyses are shown in Figures A1 and A2. During the construction phase berms were only employed as temporary support to the 1.5m thick east wall. For this reason, the modelling assumed no lateral ground movement on the centreline between the walls of the retained cutting. In reality the axis of symmetry did not correspond to the excavation centre line due to the topography of the site. This possible source of modelling error is confirmed as non-critical via the SAFE computational analysis documented by Nicholson et al, 1998.

The general geometry of the site also proved difficult to model due to its sloping profile. For convenience, a surcharge in the form of a varying distributed load (VDL) representing the overburden of sloped soil strata was applied to the surface nodes. This load ranged from zero at the capping beam to 100kN/m<sup>2</sup> at the edge of the mesh. It is to be noted that such methods model total stress and will not accurately depict sloping ground water tables and relative increases in water pressures. A subsequent check analysis without this distributed load actually demonstrated that its effect was small in any event.

### A2 Material properties

#### A2.1 Soil parameters

The first set of soil parameters adopted for analysis was stated in the Approval In Principle (AIP) by the Engineer (Gibb, 1993) and reproduced in report form to the Transport Research Laboratory by Ove Arup & Partners (1997). These design based parameters give a worst case scenario or an upper limit of predicted movement.

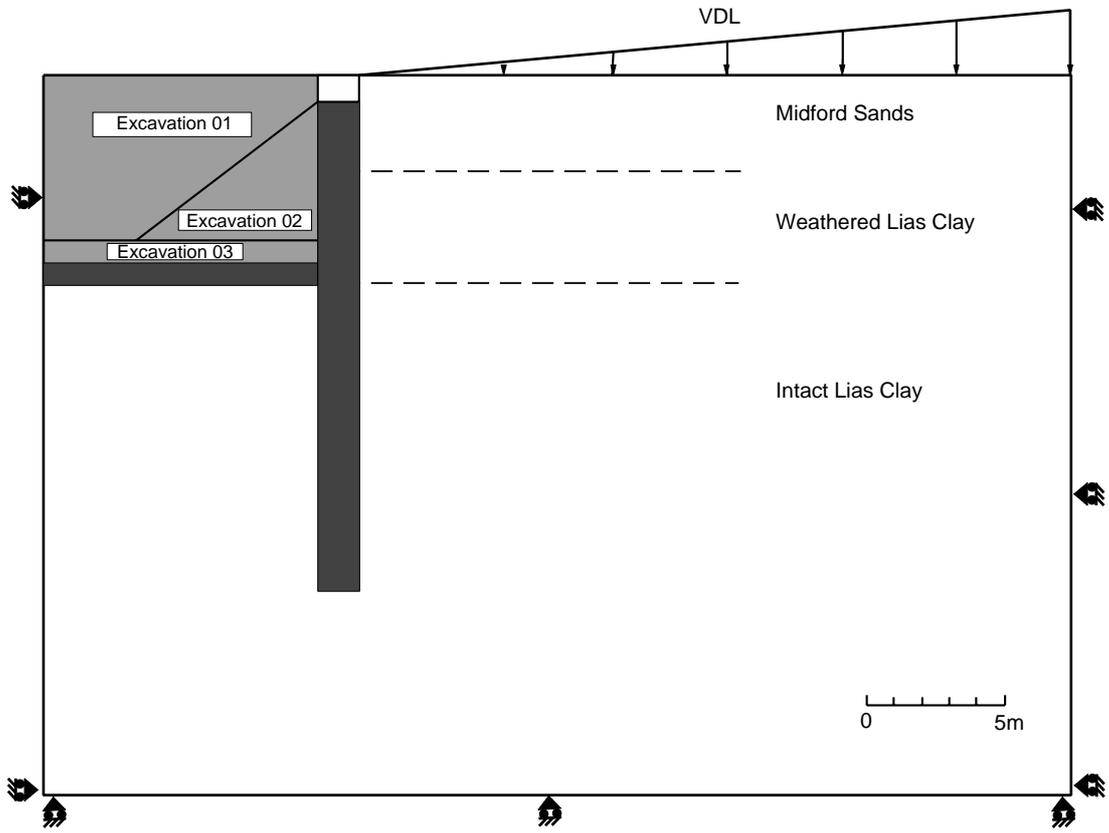
The second set of data was taken from a 3D analysis of the same site, using the CRISP finite element package, carried out by Gourvenec (1998). The data employed was more of a best estimate, with the clay strata assumed to be generally stiffer than stated in the AIP.

These and all other soil parameters are summarised in Table A1.

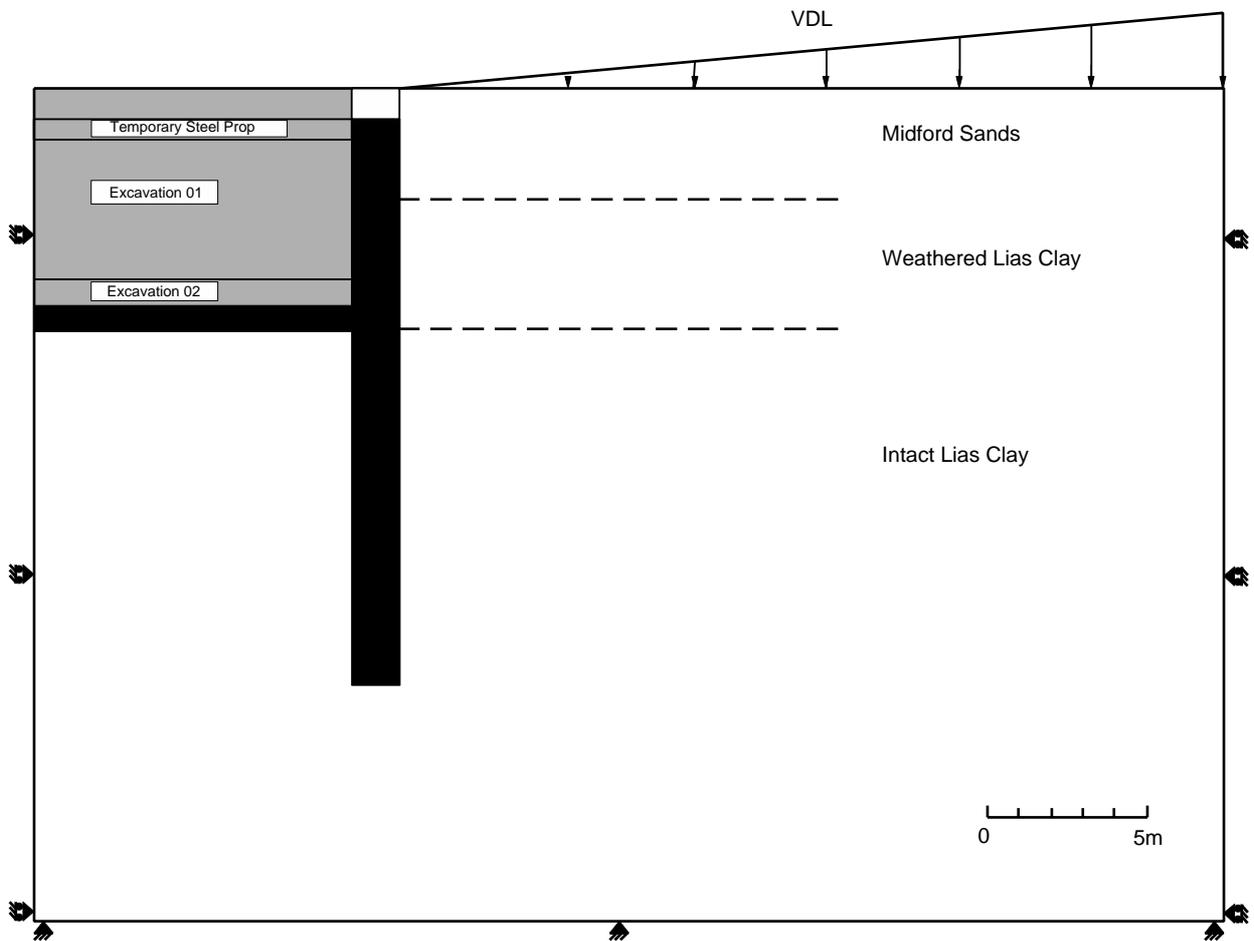
**Table A1 Soil parameters used in the analysis**

| Soil type                             | $c'$ (kN/m <sup>2</sup> ) | $\phi'$ (°) | $E'$ (MN/m <sup>2</sup> ) | $\nu$ | $\gamma_{bulk}$ (kN/m <sup>3</sup> ) | $k_h$ (m/sec)        | $k_v$ (m/sec)         |
|---------------------------------------|---------------------------|-------------|---------------------------|-------|--------------------------------------|----------------------|-----------------------|
| <b>(a) Original design parameters</b> |                           |             |                           |       |                                      |                      |                       |
| Midford Sands                         | 0                         | 32          | 10+10z                    | 0.32  | 20                                   | 1 10 <sup>-6</sup>   | 1 10 <sup>-6</sup>    |
| Weathered Lias Clay                   | 1                         | 26          | 22.6+3.7z                 | 0.36  | 20                                   | 2.8 10 <sup>-9</sup> | 2.8 10 <sup>-10</sup> |
| Intact Lias Clay                      | 5+0.25z                   | 27          | 22.5+3.5z                 | 0.35  | 20                                   | 2.8 10 <sup>-9</sup> | 2.8 10 <sup>-10</sup> |
| <b>(b) Best fit parameters</b>        |                           |             |                           |       |                                      |                      |                       |
| Midford Sands                         | 0                         | 35          | 12.2                      | 0.2   | 20                                   | 1 10 <sup>-6</sup>   | 1 10 <sup>-6</sup>    |
| Lias Clay*                            | 1                         | 26          | 80+7.2z                   | 0.2   | 20                                   | 2.8 10 <sup>-9</sup> | 2.8 10 <sup>-10</sup> |

\* No distinction made between weathered and intact properties.



**Figure A1** Diagrammatic representation of finite element mesh, berms as support



**Figure A2** Diagrammatic representation of finite element mesh, props as support

### A2.2 Structural components

Three structural components were considered, these being the 1.5m thick reinforced concrete diaphragm wall panels, the reinforced concrete prop slab and the horizontal temporary steel props.

As the reinforced concrete retaining wall was effectively a simple slab, unlike bored pile and T-shaped panels, the flexural rigidity of the wall was correctly represented by using its elastic modulus of  $30 \times 10^3 \text{ MN/m}^2$  in the analysis. The same value was also applied to the reinforced concrete prop slab at formation level.

The temporary steel props were included in some of the analyses to compare with the performance when using soil berms: temporary props were not employed in the site construction sequence. Because the props were modelled as being installed at 5m centres, the stiffness of one prop was used to calculate the equivalent stiffness per metre run of wall.

These values are summarised along with other structural properties data in Table A2.

**Table A2 Structural component data**

| Structural component              | $E$<br>( $\text{MN/m}^2$ ) |        |      | $G$<br>( $\text{MN/m}^2$ ) |        |      | $\gamma_{\text{bulk}}$<br>( $\text{kN/m}^3$ ) |
|-----------------------------------|----------------------------|--------|------|----------------------------|--------|------|---|
| Diaphragm wall panel (1.5m thick) | 30                         | $10^3$ | 0.15 | 13                         | $10^3$ | 20   |   |
| Permanent prop slab               | 30                         | $10^3$ | 0.15 | 13                         | $10^3$ | 20   |   |
| Temporary steel prop              | 1.39                       | $10^3$ | 0.15 | 0.65                       | $10^3$ | 1.95 |   |

### A3 In situ ground stresses and pore water pressures

The two data sets provided drastically varying *in situ* conditions based on the different  $K_0$  profiles which can be seen in Figure A3(b). Both sources consider the Midford Sands to have a  $K_0$  value of approximately 0.5, but contrast greatly on consideration of the Lias Clay stratum. The original design data, which can be considered to give a worst case scenario for safe design, gives a peak value of  $K_0$  of approximately 2.5 decreasing to 1.75 at depth, whereas the best estimate has a peak value of around 1.7 decreasing to about 1.1 at a depth of 50m below ground level.

The *in situ* stresses are represented as shown in Figure A3(a).

### A4 Construction sequence

The construction sequence on site entailed an initial stage of ground preparation, installation of the diaphragm walls, followed by excavation between the retaining walls to leave a temporary berm supporting the 1.5m thick east retaining wall. The subsequent removal of the berm profile and installation of the permanent prop slab was mirrored as closely as possible although, as the analysis was carried out in plane strain, movement predictions are expected to be an upper bound.

Where temporary steel props were modelled typical construction sequences were assumed. This involved assuming the duration of the construction periods and the temporary prop design to ensure a representative case was established.

### A4.1 Berms only

- Preparation of ground profile - 6 months.
- Installation of diaphragm wall.
- Excavation to leave temporary berm (excavation 01 in Figure A1) - 14 days.
- Excavation of soil berm (excavation 02) - 7 days.
- Excavation to formation level (excavation 03) - 1 day.
- Installation of reinforced concrete prop slab - 7 days.
- Period of consolidation - 6 months.

### A4.2 Horizontally propped wall

- Preparation of ground profile - 6 months.
- Installation of diaphragm wall.
- Installation of temporary steel prop - 15 days.
- Bulk excavation (excavation 01 in Figure A2) - 14 days.
- Excavation to formation level (excavation 02) - 1 day.
- Installation of reinforced concrete prop slab - 7 days.
- Removal of temporary steel prop - 5 days.
- Period of consolidation - 6 months.

## Appendix B: Finite element analyses of A406 East of Falloden Way

### B1 Details of mesh

The finite element mesh was designed allowing for the various soil strata, installation and removal of structural elements and the sequential excavation of soil berms. Information regarding the dimensions and construction sequences was obtained from the site records provided by Edmund Nuttall Ltd who also supplied a feasibility report by Ove Arup and Partners (1995). Diagrammatic representation of the mesh is shown in Figure B1.

### B2 Material properties

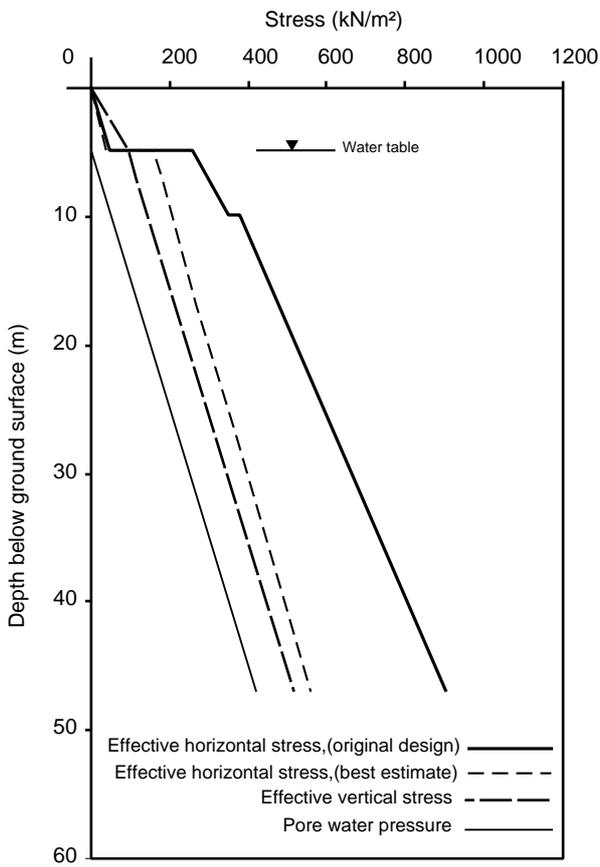
#### B2.1 Soil Parameters

Best fit strength parameters were used for the Glacial Till and London Clay of  $c'=6\text{kN/m}^2$  and  $\phi'=26^\circ$  and  $c'=20\text{kN/m}^2$  and  $\phi'=25^\circ$  respectively. Values of  $c'=0$  and  $\phi'=36^\circ$  were assumed for both the Upper and Lower Gravels.

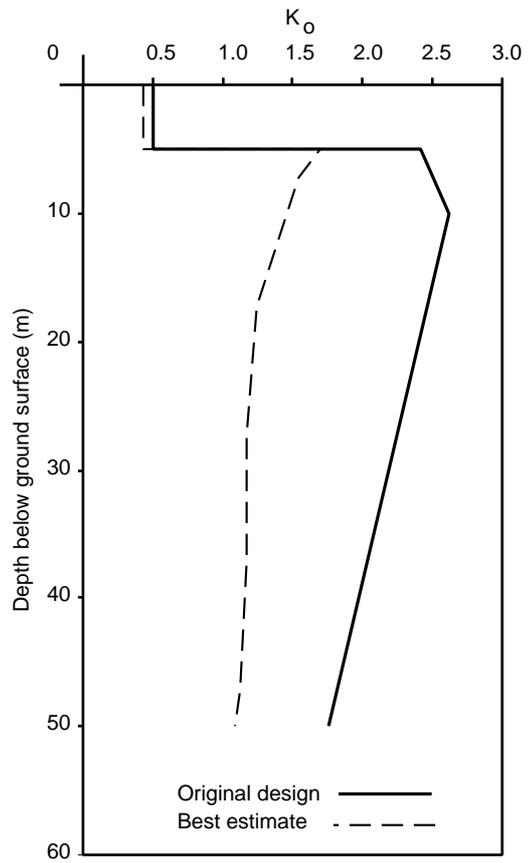
The variation of modulus with depth for both the Glacial Till and the London Clay was assumed as  $E'=114 + 7.9z \text{ MN/m}^2$  ( $E'$  being the drained elastic modulus and  $z$  being the depth in metres). This equation was based on back analysis of TRL measurement data on the performance of cantilever and multi-propped walls at nearby locations on the same site (Brookes and Carder, 1996a and b). The back analysis was carried out by Ove Arup and Partners who found that  $E'$  was approximately equal to  $1500c_u$ .

For the Glacial Till and London Clay, horizontal and vertical permeabilities were taken as  $5 \times 10^{-10}$  and  $1 \times 10^{-10} \text{ m/sec}$  respectively. The clays were considered to be slightly more permeable horizontally because of the presence of sand lenses. A typical permeability of  $1 \times 10^{-6} \text{ m/sec}$  was assumed for the gravel layers.

Other soil parameters are as given in Table B1.

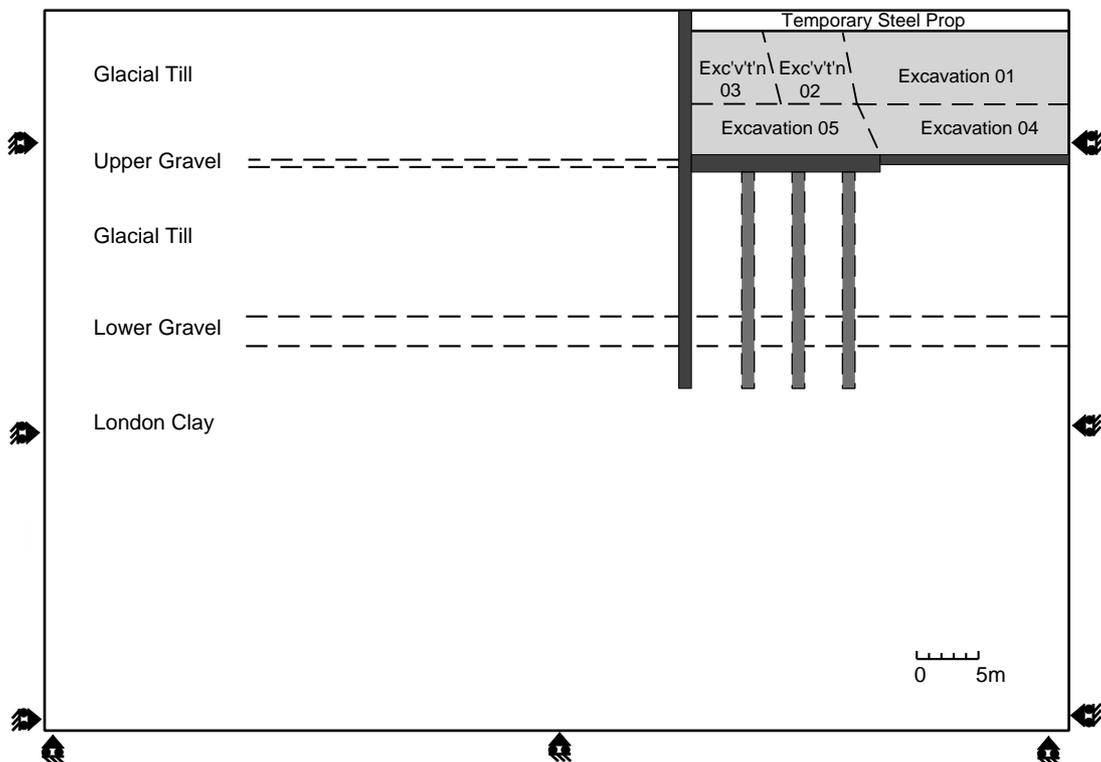


(a) In situ ground stresses and pore water pressures



(b)  $K_0$  values, variation with depth

**Figure A3** *In situ* ground conditions



**Figure B1** Diagrammatic representation of finite element mesh

**Table B1 Soil parameters used in the analysis**

| Soil type    | $c'$ (kN/m <sup>2</sup> ) | $\phi'$ (°) | $E'$ (MN/m <sup>2</sup> ) | $\nu$ | $\gamma_{bulk}$ (kN/m <sup>3</sup> ) | $k_h$ (m/sec)       | $k_v$ (m/sec)       |
|--------------|---------------------------|-------------|---------------------------|-------|--------------------------------------|---------------------|---------------------|
| Glacial Till | 6                         | 26          | 114+7.9z                  | 0.3   | 20                                   | 5 10 <sup>-10</sup> | 1 10 <sup>-10</sup> |
| Upper Gravel | 0                         | 36          | 60                        | 0.3   | 20                                   | 1 10 <sup>-6</sup>  | 1 10 <sup>-6</sup>  |
| Lower Gravel | 0                         | 36          | 80                        | 0.3   | 20                                   | 1 10 <sup>-6</sup>  | 1 10 <sup>-6</sup>  |
| London Clay  | 20                        | 25          | 114+7.9z                  | 0.3   | 20                                   | 5 10 <sup>-10</sup> | 1 10 <sup>-10</sup> |

### B2.2 Structural components

In the analyses, four main structural elements were used to represent the T-shaped diaphragm walls, the permanent prop slab, the piles installed beneath the carriageway to minimise heave, and the temporary props where applicable.

The wall comprised T-shaped diaphragm panels which were 26.3m deep with a 4m by 1m front section with a 2.5m by 1m counterfort. This was modelled using a 1m thick rectangular wall with an equivalent stiffness of 0.57 10<sup>6</sup>MN/m<sup>2</sup> per metre run of wall. This value was calculated assuming a concrete stiffness of 30 10<sup>3</sup>MN/m<sup>2</sup>.

The permanent prop slab was constructed with a reduced thicknesses towards the centre of the underpass, this is modelled in the finite element mesh. The slab stiffness was taken to be 30 10<sup>3</sup> MN/m<sup>2</sup>, and the connection between the diaphragm wall and the slab considered as monolithic.

Bored piles of nominal 1m diameter were installed at approximately 5m centres beneath the prop slab to reduce heave. These piles were modelled as a composite material with stiffness and permeability properties varying with depth and were determined according to the relative pile-soil spacings.

If an alternative temporary support had been required, raked props would have been the most likely choice due to the width of the excavation. For ease of analysis this was simulated by horizontal steel temporary props with a reduction in their stiffness by the cosine of the angle of inclination to the horizontal. These horizontal steel temporary props were assumed to be 1.2m in diameter with a wall thickness of 16mm and placed at 5m centres and were modelled as 1.2m rectangular elements with equivalent stiffness of 2 10<sup>3</sup> MN/m<sup>2</sup>.

### B3 In situ ground stresses and pore water pressures

The assumed *in situ* ground stress and pore water pressure distributions with depth are shown in Figure B2. The values of K (ratio of horizontal to vertical effective stress) used in the analysis were between 1 and 1.5 for the Glacial Till and 1.5 for the underlying London Clay. A value of  $K_o$  of 0.4 was calculated using the relation  $K_o = 1 - \sin\phi'$  and a  $\phi'$  of 36° for the gravel bands.

Perched water tables existed at this site because of the presence of the two permeable gravel layers and the distribution of pore water pressure used in the analysis is shown in Figure B2.

### B4 Construction sequence

Two construction sequences were modelled in this particular analysis. The first being a direct interpretation of

the actual construction programme and the second being an idealised representation of the construction sequence had temporary raked props been used. Time periods for each construction phase have been determined from site records, and where required typical time periods have been assumed (ie. installation of simulated raked props).

#### B4.1 Berms only

- Initial installation of retaining wall.
- Progressive excavation of soil berms to 3.5m above formation (excavations 01, 02 and 03 in Figure B1) - 51 days.
- Installation of prop slab piles - 22 days.
- Progressive excavation to formation level (excavations 04 and 05) - 7 days.
- Installation of the permanent prop slab at carriageway level - 3 days.
- Period of consolidation - 6 months.

#### B4.2 Simulated raked props

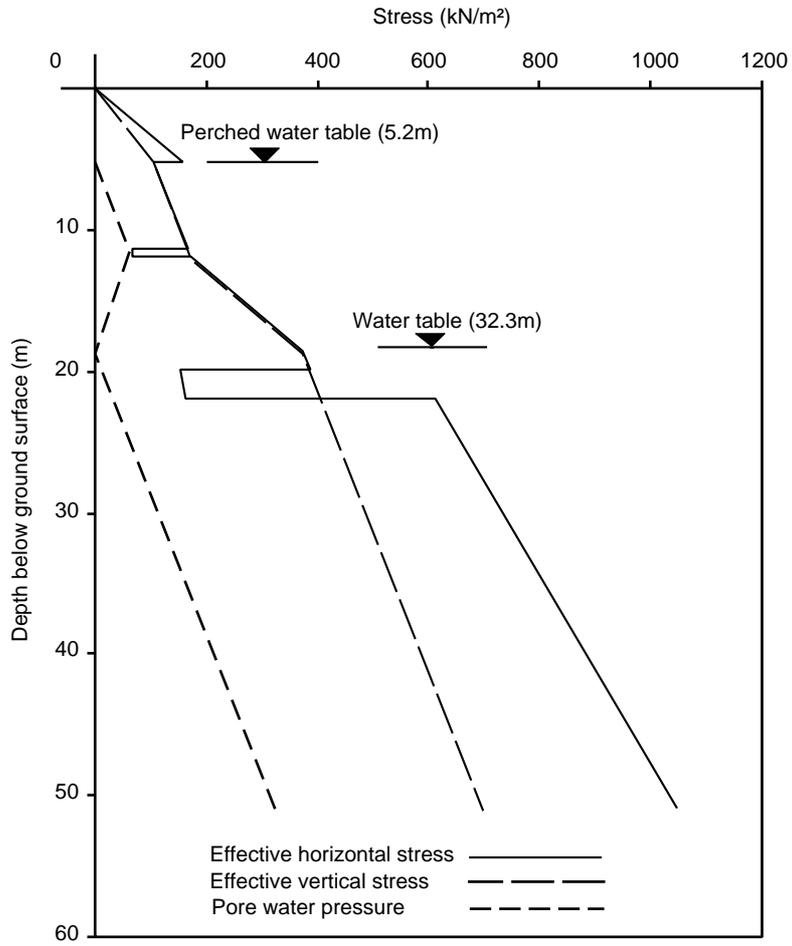
- Initial installation of retaining wall.
- Installation of simulated temporary raked props - 1 day.
- Bulk excavation to 3.5m above formation (excavations 01, 02 and 03 in Figure B1) - 51 days.
- Installation of prop slab piles - 22 days.
- Excavation to formation level (excavations 04 and 05) - 7 days.
- Installation of the permanent prop slab at carriageway level - 3 days.
- Removal of simulated raked props - 3 days.
- Period of consolidation - 6 months.

## Appendix C: Finite element analyses of A40 Long Lane improvement

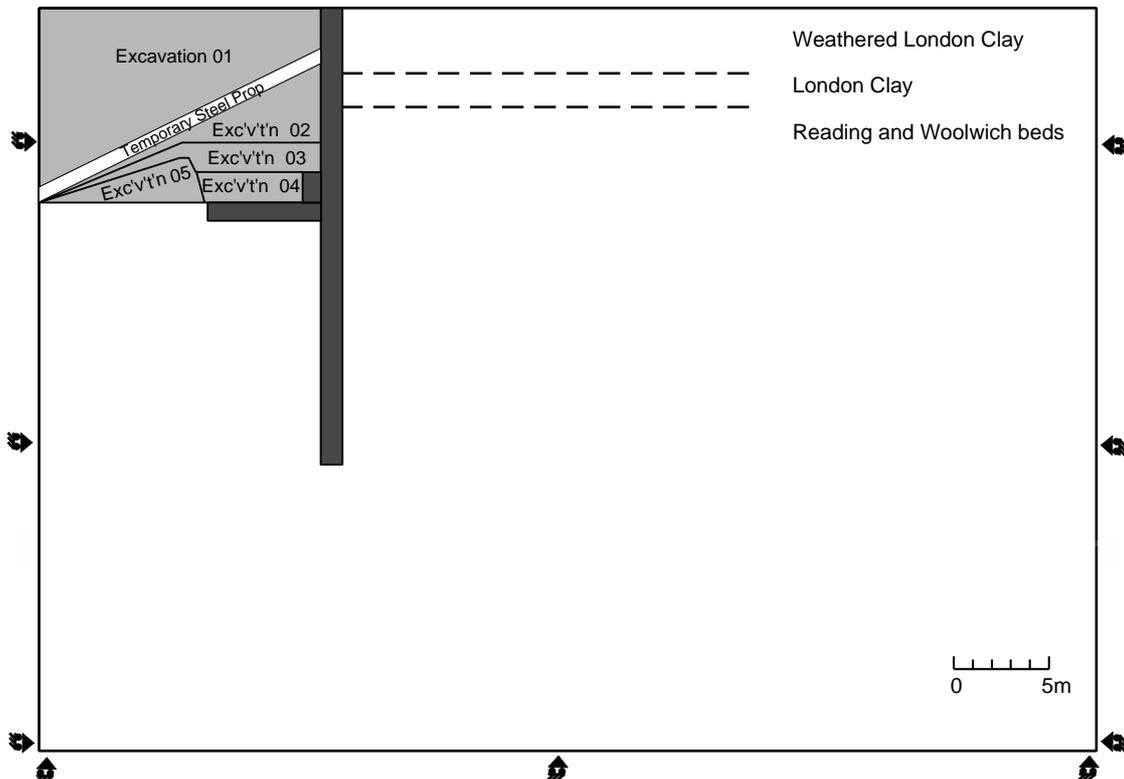
### C1 Details of mesh

The finite element package used to perform the analysis was SAGE CRISP. The soil strata were considered as being elastic perfectly plastic materials obeying the Mohr-Coulomb yield criterion, with consolidating elements used to model the construction timetable.

Diagrammatic representation of the finite element mesh employed in this analysis is shown in Figure C1. The mesh was designed to accommodate the temporary raked props and the various stages of excavation before and after its



**Figure B2** *In situ* ground stresses and pore water pressures



**Figure C1** Diagrammatic representation of finite element mesh

installation which occurred on site. The lower horizontal boundary was completely fixed in both directions, whereas the vertical edge boundaries were fixed only in the horizontal direction. These particular fixities assume no lateral movement of the thrust beam against which the raked props react. Three different horizontal layers were considered to allow for the three different soil strata.

## C2 Material properties

### C2.1 Soil parameters

Soil parameters chosen for this analysis were taken from site data provided by Mott MacDonald and Taywood Foundations by way of borehole logs.

The variation of modulus with depth for all three strata was assumed as  $E' = 32 + 8.4z$  MN/m<sup>2</sup>, where  $z$  is the depth in metres below the clay surface. This equation follows the upper bound for horizontal modulus recommended by Burland and Kalra (1986) and used with success for predicting wall movements at other London Clay sites (Watson and Carder, 1994).

For the London Clay and the Reading and Woolwich beds the value of  $c'$  was taken to be constant with depth, at a value of 20 kN/m<sup>2</sup>. However the value of  $c'$  for the weathered London Clay was considered as increasing linearly from near zero, due to surface fissuring, to the value of 20kN/m<sup>2</sup> where the clay became unweathered. The resulting equation for the weathered clay was  $c' = 4.16z$  kN/m<sup>2</sup>.

These and the various other soil parameters are tabulated in Table C1.

### C2.2 Structural components

Three structural components existed in this particular analysis. These were the reinforced concrete bored pile wall and the stabilising base slab, and the raked temporary steel props. The 1200mm diameter bored piles at 1300mm centres were modelled as a 1200mm thick rectangular wall of unit length and the equivalent plane strain stiffness calculated so that the wall had the same flexural rigidity. The same problem did not arise with the stabilising base which was continuous in nature and a typical value for reinforced concrete was adopted.

The temporary steel props were 762mm in diameter with a wall thickness of 14.2mm spaced at an average 6m centres. The equivalent axial stiffness and bulk density of the rectangular prop elements were therefore calculated on this basis.

The properties of the structural components are summarised in Table C2.

### C3 In situ ground stresses and pore water pressures

The water table has been assumed to be 1m below the existing ground surface and increasing hydrostatically with

**Table C2 Structural component data**

| Structural component                          | $E$<br>(MN/m <sup>2</sup> ) |                 |      | $G$<br>(MN/m <sup>2</sup> ) |                 |      | $\gamma_{bulk}$<br>(kN/m <sup>3</sup> ) |
|---|-----------------------------|-----------------|------|-----------------------------|-----------------|------|---|
| Bored pile wall                               | 16.3                        | 10 <sup>3</sup> | 0.15 | 7.09                        | 10 <sup>3</sup> | 22.6 |   |
| Permanent stabilising base                    | 30                          | 10 <sup>3</sup> | 0.15 | 13                          | 10 <sup>3</sup> | 22.1 |   |
| Temporary steel props (horizontal* and raked) | 1.5                         | 10 <sup>3</sup> | 0.15 | 0.65                        | 10 <sup>3</sup> | 0.56 |   |

\* The effects of using horizontal props instead of raked props and berms was also investigated.

depth. A typical value for  $K_0$ , the ratio of *in situ* horizontal to vertical effective stress, for London Clay has been taken to be 2 (Symons, 1992). The variation of the *in situ* stresses and pore water pressures with depth are shown in Figure C2.

## C4 Construction sequence

The original construction sequence on site entailed excavation between the retaining walls leaving 1:1.75 berms to the underside of the capping beam whereupon temporary raked props were installed at an average of 6m centres. The remaining soil berm beneath the raked props was then excavated to formation level, the reinforced concrete stabilising base installed and the temporary props removed after a reasonable period of curing. More details on the construction sequence are given in section C4.1.

For comparative purposes, the movements if berms only (ie. no raked props) had been used for the temporary support of the wall were also predicted. The effect if the raked props had been horizontal rather than vertical was also separately investigated. The construction sequences arbitrarily adopted for these methods are described in sections C4.2 and C4.3.

### C4.1 Raked props and berms

The sequence of events detailed below model the actual construction procedure employed on site:

- Installation of bored pile retaining wall.
- Excavate leaving 1:1.75 berms to underside of the capping beam (excavation 01 in Figure C1) - 10 days.
- Installation of temporary raked props at 23° to the horizontal - 5 days.
- Progressive excavation of soil berms (excavations 02, 03 and 04) - 27 days.
- Installation of stabilising base - 5 days.
- Removal of raked props - 14 days.
- Period of consolidation - 6 months.

**Table C1 Soil parameters used in the analysis**

| Soil type                 | $c'$ (kN/m <sup>2</sup> ) | $\phi'$ (°) | $E'$ (MN/m <sup>2</sup> ) | '   | $\gamma_{bulk}$ (kN/m <sup>3</sup> ) | $k_h$ (m/sec)       | $k_v$ (m/sec)       |
|---------------------------|---------------------------|-------------|---------------------------|-----|--------------------------------------|---------------------|---------------------|
| Weathered London Clay     | 4.16z                     | 22          | 32+8.4z                   | 0.3 | 20                                   | 1 10 <sup>-9</sup>  | 1 10 <sup>-10</sup> |
| London Clay               | 20                        | 22          | 32+8.4z                   | 0.3 | 21                                   | 5 10 <sup>-10</sup> | 1 10 <sup>-10</sup> |
| Reading and Woolwich beds | 20                        | 22          | 32+8.4z                   | 0.3 | 22                                   | 5 10 <sup>-10</sup> | 1 10 <sup>-10</sup> |

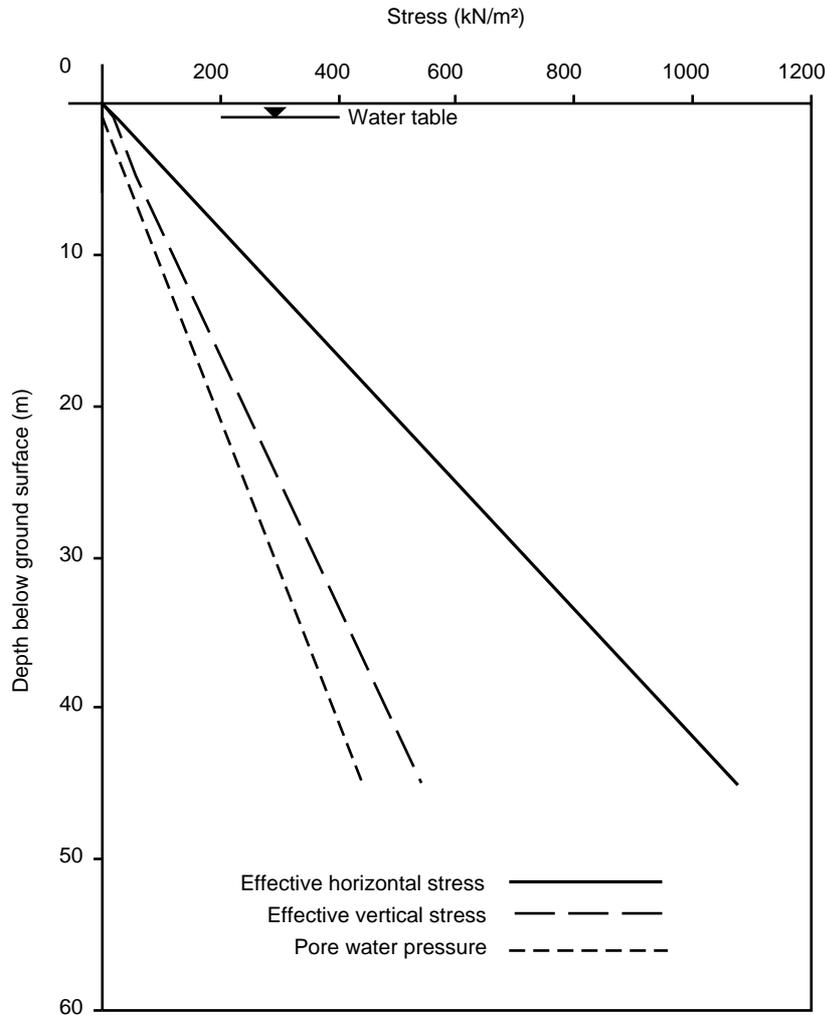


Figure C2 *In situ* ground stresses and pore water pressures

#### C4.2 Berms only

This sequence provides predicted movements if soil berms had been used as the sole method of lateral support to the retaining wall during both bulk excavation and construction of the stabilising base at formation level:

- Installation of bored pile retaining wall.
- Excavation leaving 1:1.75 berms to underside of the capping beam (excavation 01 in Figure C1) - 40 days.
- Progressive excavation of soil berms (excavations 02, 03 and 04) - 40 days.
- Installation of stabilising base - 5 days.
- Period of consolidation - 6 months.

#### C4.3 Horizontally propped wall

Horizontal props were included in the analysis in order to provide comparative results to those obtained from raked propping. It should be noted that this was not a viable site option due to the width of the underpass:

- Installation of bored pile retaining wall.
- Installation of temporary horizontal steel prop - 5 days.
- Bulk excavation to formation level (excavations 01, 02, 03 and 04 in Figure C1) - 30 days.
- Installation of stabilising base - 5 days.

e Removal of horizontal props - 2 days.

f Period of consolidation - 6 months.

## Appendix D: Finite element analyses of A50 Blythe Bridge to Queensway

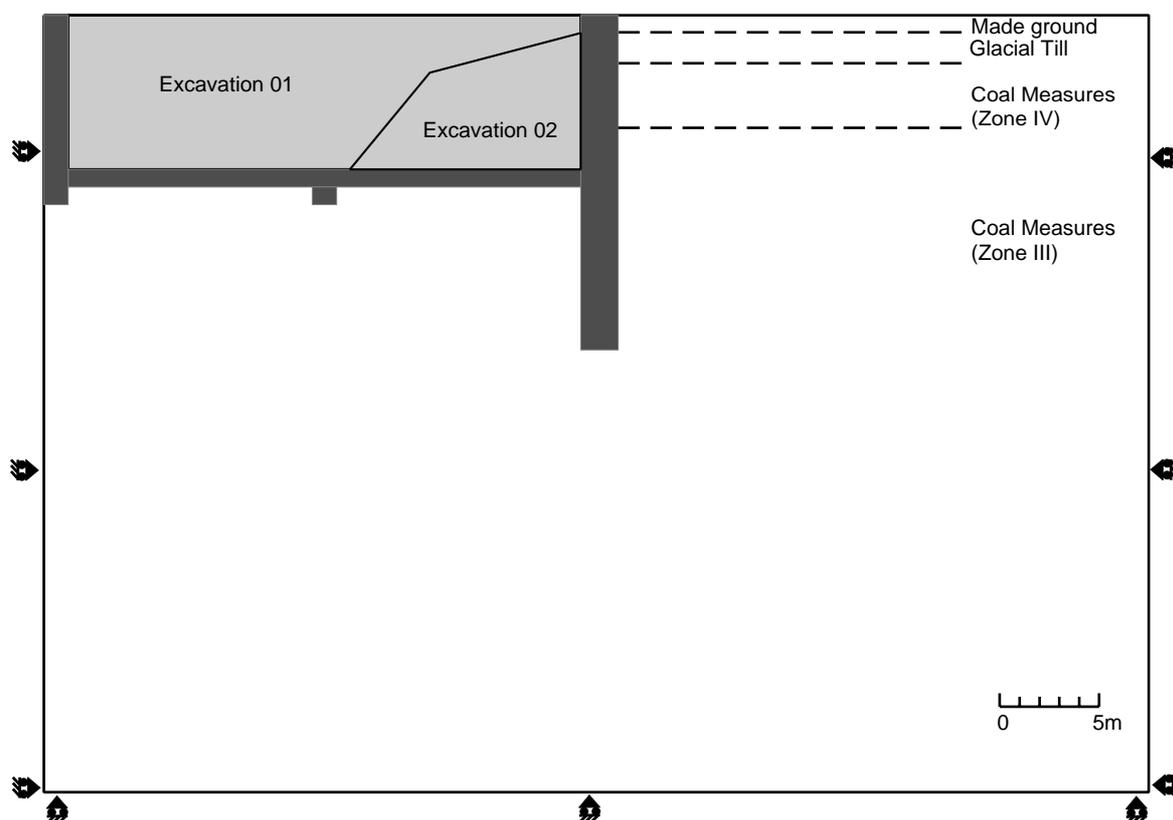
### D1 Details of mesh

The finite element mesh was designed to allow for the various soil strata, installation of the structural elements and the sequential excavation of the soil berms. A diagrammatic representation of the mesh is shown in Figure D1.

### D2 Material properties

#### D2.1 Soil/rock parameters

The retaining structure was constructed predominately in Middle Coal Measures, ranging from zones III to IV. The soil profile in the instrumented area is summarised in Figure D1. Due to the lack of site investigation data, the parameters employed have been based upon various sources, including original design notes and past publications referencing similar such strata.



**Figure D1** Diagrammatic representation of finite element mesh

The made ground parameters were taken from design notes which used a  $c'$  of zero and a  $\phi'$  of  $25^\circ$ : the  $c_u$  from laboratory tests was  $25\text{kN/m}^2$ . The elastic modulus of the made ground was then calculated as  $12.5\text{MN/m}^2$  using the relation  $E'$  is equal to  $500c_u$  (Selvadurai, 1979); this value was considered to be constant with depth.

Glacial Till parameters were established from the *in situ* and laboratory tests. Values for  $c_u$  of  $172\text{kN/m}^2$  were related to elastic modulus by the relationship for firm to stiff clays of  $E'$  is equal to  $1000c_u$  (Selvadurai, 1979). Soil strength parameters used were  $c' = 8\text{kN/m}^2$  and  $\phi' = 25^\circ$ , these being obtained from site sources. The variation in elastic modulus with depth of the Glacial Till was unavailable and a typical value was employed which varied linearly with depth at a rate of  $7.9\text{MN/m}^2$  per metre.

As previously indicated the Coal Measures were documented as being two different weathered zones, ie. zones III and IV. From correlations between site data and previous published work (Gannon et al, 1996), the elastic modulus was considered to be in the range of 120 to  $585\text{MN/m}^2$ . For the purpose of these analyses, the elastic

moduli for the two zones (III and IV) were taken as  $585\text{MN/m}^2$  (maximum value) and  $352\text{MN/m}^2$  (average value) respectively, with no variation as the depth increased. The Coal Measures were assumed to have zero cohesion value.

Other soil/rock parameters are as given in Table D1.

### D2.2 Structural components

In the analyses, the three main structural elements were the 1.5m diameter bored piles, the reinforced concrete prop slab with a horizontal shear key and the temporary steel prop. These were represented in the model as follows.

The contiguous bored piles were modelled as a rectangular element (1.5m  $\times$  1m) with equivalent stiffness of  $16.1 \times 10^3\text{MN/m}^2$  to give the same flexural rigidity. The permanent prop slab and shear key were continuous and the plane strain stiffness was therefore taken as  $30 \times 10^3\text{MN/m}^2$  which is typical for reinforced concrete.

For comparison, horizontal temporary steel props were included in one of the analyses to investigate the magnitude of the wall movements if this method of temporary support had been used. For this purpose,

**Table D1** Soil/rock parameters used in the analysis

| Soil type           | $c'$ (kN/m <sup>2</sup> ) | $\phi'$ (°) | $E'$ (MN/m <sup>2</sup> ) | $\nu$ | $\gamma_{bulk}$ (kN/m <sup>3</sup> ) | $k_h$ (m/sec)       | $k_v$ (m/sec)       |
|---------------------|---------------------------|-------------|---------------------------|-------|--------------------------------------|---------------------|---------------------|
| Made ground         | 0                         | 25          | 12.5                      | 0.37  | 18                                   | $1 \times 10^{-7}$  | $1 \times 10^{-7}$  |
| Glacial Till        | 8                         | 25          | $172 + 7.9z$              | 0.3   | 20                                   | $5 \times 10^{-10}$ | $1 \times 10^{-10}$ |
| Coal Measures (IV)  | 0                         | 35          | 352                       | 0.33  | 20                                   | $5 \times 10^{-7}$  | $5 \times 10^{-7}$  |
| Coal Measures (III) | 0                         | 35          | 585                       | 0.33  | 20                                   | $5 \times 10^{-7}$  | $5 \times 10^{-7}$  |

cylindrical hollow steel props were assumed to have been used at 5m centres and typical values of stiffness and density were adopted. The values employed were  $1.39 \times 10^3 \text{MN/m}^2$  and  $0.51 \text{kN/m}^3$  for the equivalent stiffness and density respectively.

### D3 *In situ* ground stresses and pore water pressures

The assumed *in situ* ground stress and pore water pressure distributions with depth are shown in Figure D2. The values of  $K_0$  (ratio of *in situ* horizontal to vertical effective stress) used in the analyses were between 1 and 1.5 for the Coal Measures and 0.57 for the made ground and Glacial Till. Diagrammatic representation showing variation of  $K_0$  with depth is shown in Figure D3.

From the original site investigation, the ground water table within the Coal Measures was established as being at about 10m depth below the existing ground level. A perched water table was also identified within the made ground. The distribution of pore water pressure adopted for the analyses is shown in Figure D2.

**Table D2 Structural component data**

| Structural component  | $E$<br>( $\text{MN/m}^2$ ) |        |      | $G$<br>( $\text{MN/m}^2$ ) |        |      | $\gamma_{\text{bulk}}$<br>( $\text{kN/m}^3$ ) |
|-----------------------|----------------------------|--------|------|----------------------------|--------|------|---|
| Bored pile wall       | 16.1                       | $10^3$ | 0.15 | 7.0                        | $10^3$ | 22.6 |   |
| Prop slab             | 30                         | $10^3$ | 0.15 | 13                         | $10^3$ | 22.1 |   |
| Temporary steel props | 1.39                       | $10^3$ | 0.15 | 0.6                        | $10^3$ | 0.51 |   |

### D4 Construction sequence

The 1500mm bored piles forming the wall were installed during the period between October 1994 and the following January 1995. The capping beam on top of the bored piles was completed in February 1995.

Bulk excavation of Area 01, including the excavation for the shear key (Figure D1), was undertaken between March and April 1995. The resulting construction stage time was based around this two month period. Bulk excavation of Area 02 took place between the end of June and mid September 1995, with a total of eleven berms being removed in this period. The final construction phase of installing the prop slab at formation level over lapped the Area 02 excavation stage, running from the end of July to mid October 1995. The average time delay between the excavation of the temporary berms and the completion of their corresponding prop slabs was approximately one month.

One other construction case was modelled, this being the inclusion of temporary steel props in place of the soil berms. The construction sequences adopted were then very similar with the main variation being bulk excavation in one stage for the propped case.

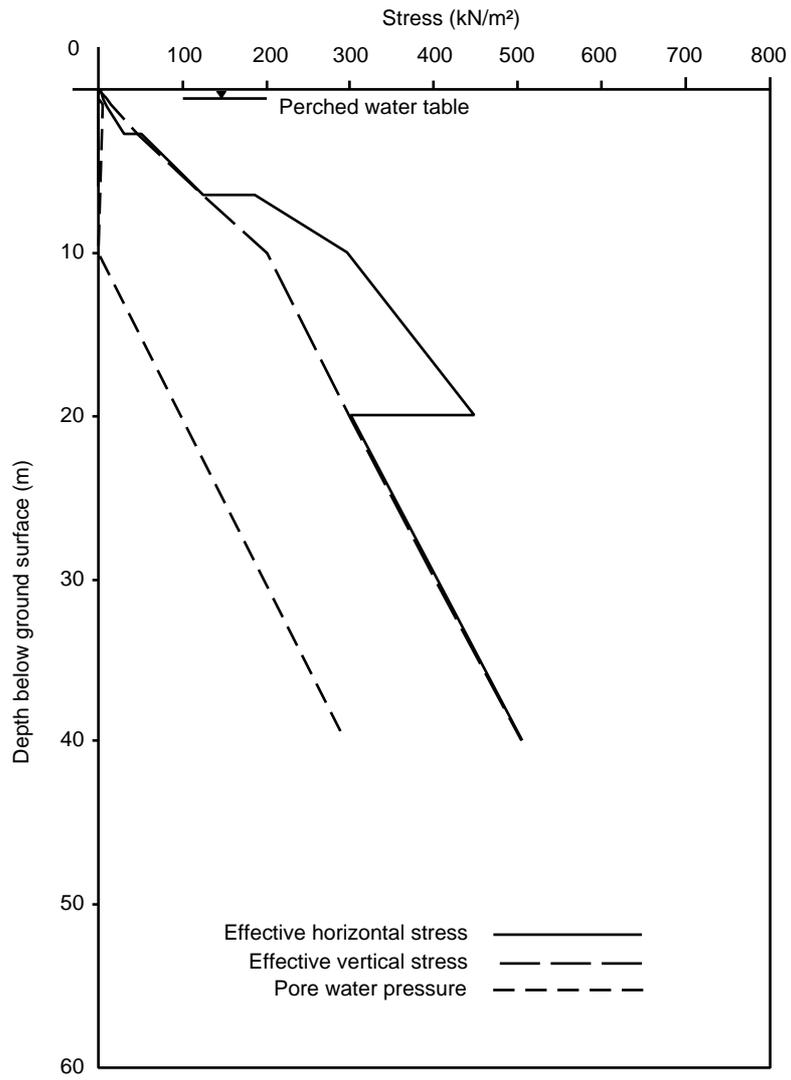
#### D4.1 Berms only

The sequence detailed below shows the actual time periods of each construction step modelled in the finite element analysis to the nearest whole day:

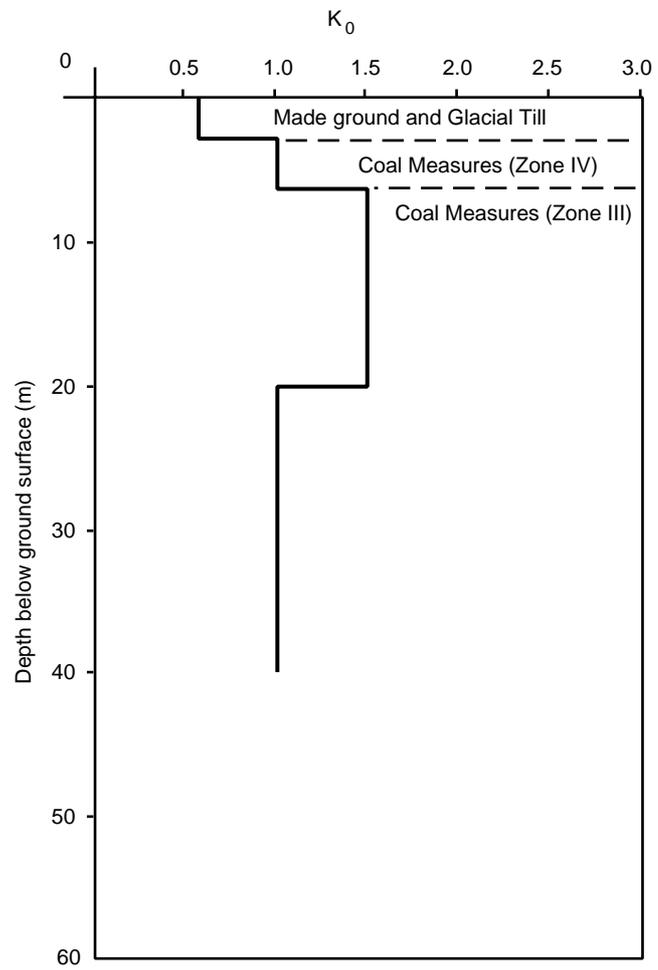
- Installation of bored pile retaining wall.
- Excavation to leave temporary berm (excavation 01 in Figure D1) - 30 days.
- Excavation of soil berm (excavation 02) - 2 days.
- Installation of permanent prop slab - 12 days.
- Period of consolidation - 6 months.

#### D4.1 Temporary horizontal props

- Installation of bored pile retaining wall.
- Installation of temporary horizontal prop - 10 days.
- Bulk excavation (excavations 01 and 02 in Figure D1) - 30 days.
- Installation of permanent prop slab - 12 days.
- Removal of temporary horizontal prop - 10 days.
- Period of consolidation - 6 months.



**Figure D2** *In situ* ground stresses and pore water pressures



**Figure D3** Variation of  $K_0$  with depth

## Abstract

---

This study reviews the performance of the temporary support systems used at four completed embedded retaining wall schemes. Soil berms were employed at three of these schemes and, at the other scheme, soil berms were used in conjunction with raked steel temporary props. The measured wall movements in each case are compared with those calculated by back-analysis using a finite element model.

The effectiveness of the construction method is assessed from the numerical analyses by comparing predicted movements using soil berms (and raked props where appropriate) with those predicted if the alternative method of using horizontal propping near the top of the wall had been used.

## Related publications

---

- TRL381 *The long term performance of embedded retaining walls* by D R Carder and P Darley. 1998 (price £35, code H)
- TRL320 *A comparison of embedded and conventional retaining wall design using Eurocode 7 and existing UK design methods* by D R Carder. 1998 (price £25, code E)
- TRL244 *Behaviour of a cantilever contiguous bored piled wall in boulder clay at Finchley* by A H Brookes and D R Carder . 1997 (price £25, code E)
- TRL213 *The effectiveness of berms and raked props as temporary support to retaining walls* by D R Carder and S N Bennett. 1996 (price £25, code E)
- TRL187 *Behaviour of the diaphragm walls of a cut-and-cover tunnel constructed in boulder clay at Finchley* by A H Brookes and D R Carder. 1996 (price £35, code H)
- TRL172 *Ground movements caused by different embedded retaining wall construction techniques* by D R Carder. 1995 (price £25, code E)
- TRL128 *Doubly-propped embedded retaining walls in clay* by D J Richards and W Powrie. 1995 (price £35, code H)
- RR359 *Design of embedded retaining walls in stiff clays* by I F Symons. 1993 (price £35, code H)
- RR116 *A parametric study of the stability of embedded cantilever retaining walls* by I F Symons and H Kotera. 1987 (price £20, code B)

Prices current at June 1999

For further details of these and all other TRL publications, telephone Publication Sales on 01344 770783 or 770784, or visit TRL on the Internet at <http://www.trl.co.uk>.